UNITED STATES
NUCLEAR REGULATORY COMMISSION
WASHINGTON, D. C. 20555
August 3, 1990

Docket No. 40-2061 Board Notification 90-06

MEMORANDUM FOR: Atomic Safety and Licensing Appeal Board and All Parties

FROM: Charles J. Haughney, Chief
Fuel Cycle Safety Branch
Division of Industrial and Medical Nuclear Safety
Office of Nuclear Material Safety and Safeguards

SUBJECT: NEW INFORMATION POTENTIALLY RELEVANT AND MATERIAL TO BOARD PROCEEDING IN THE MATTER OF KERR-MCGEE CHEMICAL CORPORATION, WEST CHICAGO RARE EARTHS FACILITY, DOCKET 40-2061-ML

In conformance with the Commission's policy on notification of Licensing Boards, Appeal Boards, or Commission of new, relevant, and material information, this memorandum calls attention to the documents listed below.

The enclosed memorandum, dated August 1, 1990, provides a summary of the July 25, 1990, telephone conversation with Kerr McGee on erosion protection design of the West Chicago disposal cell (Enclosure 1). The enclosed letter, dated July 31, 1990 (Enclosure 2), provides information to supplement the report on erosion evaluation for the disposal cell that was forwarded by Board Notification 90-05. The additional information was requested by the July 25, 1990 telephone conversation. These documents are being brought to the attention of the Board as information which may be relevant and material to issues pending before the Board.

Jerry J. Swift for
Charles J. Haughney, Chief
Fuel Cycle Safety Branch
Division of Industrial and Medical Nuclear Safety
Office of Nuclear Material Safety and Safeguards

Enclosures:
1. Memorandum to CJHaughney from JSwift dated 8/1/90
2. Letter to CJHaughney from Kerr-McGee dated 7/31/90

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1/1

Board Notification 90-06 dated August 3, 1990
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Adjudicatory File
Atomic Safety and Licensing Board
U.S. Nuclear Regulatory Commission
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Atomic Safety and Licensing Appeal
Panel (5)
U.S. Nuclear Regulatory Commission
Washington, DC 20555

Atomic Safety and Licensing
Board Panel (1)*
U.S. Nuclear Regulatory Commission
Washington, D C 20555

Enclosure 1

UNITED STATES
NUCLEAR REGULATORY COMMISSION
WASHINGTON, D. C. 20555

AUG 01 1990

MEMORANDUM FOR: Charles J. Haughney, Chief
Fuel Cycle Safety Branch
Division of Industrial and
Medical Nuclear Safety

FROM: Jerry J. Swift, Section Leader
Advanced Fuel and Special
Facilities Section
Fuel Cycle Safety Branch

SUBJECT: CONFERENCE CALL OF JULY 25, 1990, WITH KERR-MCGEE ON EROSION PROTECTION DESIGN OF THE WEST CHICAGO DISPOSAL CELL

The following is a record of the July 25, 1990 conferance call from the Nuclear Regulatory Commission (NRC) to James L. Grant & Associates, authors of the Kerr-McGee report entitled, "Erosion Evaluation West Chicago Disposal Cell," dated July 23, 1990. The Kerr-McGee report presents the results of analyses of potential erosion of the disposal cell proposed as a part of the closure of the Kerr-McGee Rare Earths Facility in West Chicago, Illinois. The purpose of the conference call was to request additional information needed to complete NRC's review of the report. A list of persons who participated in the conference call is enclosed.

During the discussions, Mr. Terry (Ted) L. Johnson of NRC's Division of Low-Level Waste Management and Decommissioning identified the specific areas of the report that require additional supporting documentation. These areas involve Sections 3, 4, 5, and 6 of the report and are outlined below.

Section 3 - Sizing of Erosion Barrier Materials

The stone size necessary to prevent erosion of the intrusion barrier was determined using flow rate estimates. Figure 5 of Section 3 shows a stone size (D-50 value) of 9.5 to 10.0 inches for a unit discharge rate of 9 cfs/ft at a 20% slope. It is unclear why a D-50 value of 9.0 inches was used for the stone size, as indicated on page 20 of the report.

Section 4 - Diversion Ditch and Toe of Slope Protection

Additional information regarding the diversion channel and toe is required for staff to base a conclusion on the adequate protection of these areas. Information including cross sections of critical areas, calculations and any basis for the decision on rock size should be included in the report.

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Section 5 - Offsite Flooding

The report states that the flood flow rate value was derived by using Regulatory Guide 1.59 and was calculated to be about 61,000 cfs. The report should show how Regulatory Guide 1.59 was used in arriving at this value. If drainage area data presented in the guide was used to estimate the flood flow rate for an 18 square mile drainage area, then any extrapolation used should be presented in the report.

Slopes used to determine the flow velocities and depth in the channel should be shown in the report as well as calculations used in deriving the values.

Section 6 - Sedimentation Basin Failure

It is stated in this section that the berm that forms the side of the basin adjacent to the disposal cell will be protected from erosion by the same riprap protection provided in the drainage ditches around the disposal cell. This section lacks sufficient information to support this conclusion. Additional information is needed or a justification should be provided as to why this issue is not critical to the analysis.

Mr. Grant, of James L. Grant & Associates, responded in a positive way to each area discussed and stated that he will either provide the NRC with the additional information requested or will show adequate justification as to why the information is not needed. At the conclusion of the meeting it was agreed that Mr. Grant would "informally" send NRC the majority of the requested information that was readily available to him. A follow-up package addressing all the open items will formally be submitted to the NRC by Kerr-McGee as a supplement to the report. The purpose of an earlier informal submittal by Mr. Grant is to allow NRC staff to continue their review of the report. NRC should expect the early informal package by July 26, 1990 and the formal package by the week of July 30, 1990.

Jerry J. Swift, Section Leader
Advanced Fuel and Special
Facilities Section
Fuel Cycle Safety Branch

Enclosure:
1. List of participants

ENCLOSURE 1

July 25, 1990

CONFERENCE CALL PARTICIPANTS

Erosion Protection Design, West Chicago

NAME	AFFILIATION
Jerry J. Swift	NRC
Keith K. McDaniel	NRC
Merri L. Horn	NRC
Ted L. Johnson	NRC
Robert L. Fonner	NRC
Patricia Jehle	NRC
Bernard M. Bordenick	NRC
Paul H. Lohaus	NRC
Scott Munson	KM-KMCC
James L. Grant	KMCC/Grant Assoc.
Richard A. Meserve	KMCC/Covington & Burling
Joseph G. Klinger	Il. Dept of Nuclear Safety
Ellen Zisook	Karaganis & White/West Chicago
Jack Russell	EPA

Enclosure 2

KERR-McGEE CHEMICAL CORPORATION

KERR-McGEE CENTER • OKLAHOMA CITY, OKLAHOMA 73125

July 31, 1990

Mr. Charles J. Haughney, Chief
Fuel Cycle Safety Branch
Division of Industrial and Medical
Nuclear Safety, NMSS
U.S. Nuclear Regulatory Commission
Washington, D.C. 20555

RE: Docket: 40-2061
License: STA-583
License Amendment Request for Authorization for On-Site Receipt and Disposal of Off-Site Contaminated Soils - Supplemental Information

Dear Mr. Haughney:

During a conference call on July 25, 1990, NRC staff requested that Kerr-McGee Chemical Corporation ("Kerr-McGee") submit certain information to supplement the report filed on July 23, 1990, concerning the erosion evaluation of the West Chicago disposal cell. This letter is to provide the requested information on behalf of Kerr-McGee:

1. Staff requested information to justify the selection of rock with a D_{50} of 9 inches for the intrusion barrier along the side slope of the disposal cell. Kerr-McGee's aim was to select a rock size that would be adequate to withstand the upper-bound estimate of the unit flow in a hypothetical gully along the side slope of the cell. As noted in the July 23 report (page 19), an upper bound estimate for this flow is about 8.8 cfs/ft. The relation set out in Figure 4.3 of Development of Design Criteria by Riprap Testing in Flumes, NUREG/CR-4651 (1987), shows that the median stone diameter to withstand a flow of this magnitude is nearly nine inches. A copy of Figure 4.3 is attached as Appendix 1.

2. Staff requested that Kerr-McGee submit the calculations underlying section 4 of Kerr-McGee's July 23 submittal. That section concerns the design of the riprap for the diversion ditches so that the cell can withstand storm flows that are significantly in excess of the probable maximum flood from the drainage areas upstream of the Kerr-McGee site. The calculations from the files of Kerr-McGee's consultants are attached as Appendix 2.

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3. Staff requested that Kerr-McGee provide a cross-section showing the toe of the pile and its relationship to the diversion ditches. A conceptual design is attached as Appendix 3. The dimensions for the riprap depth shown in the sketch are appropriate for the diversion ditches along the western and northern sides of the cell; Kerr-McGee will install the riprap surrounding the cell so that the thickness of the riprap is at least twice the appropriate median stone diameter.

4. Staff requested that Kerr-McGee submit the calculations underlying section 5 of Kerr-McGee's July 23 submittal. That section concerns the adequacy of the cell design to withstand off-site flooding in Kress Creek that is significantly in excess of the probable maximum flood. The requested calculations are attached as Appendix 4.

5. Staff requested that Kerr-McGee submit information in support of the assertion in section 6 of the July 23 submittal that the riprap protection is adequate to prevent the release of radiological material in the event of failure of the sedimentation basin. The requested information is attached as Appendix 5. The riprap that protects the diversion ditch forms the side of the sedimentation basin adjacent to the disposal cell. As is shown in the appendix, that riprap is adequate to assure that a failure of the sedimentation basin will not threaten to erode the cell.

It is Kerr-McGee's understanding that the costs of construction of the cell are not substantially affected by the various commitments that Kerr-McGee has made in connection with its July 23 submittal. Indeed, the commitments do not alter the fundamental design of the cell in any material respect. These are the type of specifications that Kerr-McGee anticipated developing, subject to staff review, in the course of the detailed engineering work that will precede the commencement of cell construction.

Because of the limited time available to respond to the staff's requests, formal narratives to describe the calculations have not been prepared. Rather, we have submitted relevant material from the files of Kerr-McGee's consultants. Although we believe this material is understandable, Kerr-McGee and its consultants stand ready to answer any further questions you may have concerning the erosion report or this submittal.

Very truly yours,

John C. Stauter

J.C. Stauter, Vice President
Environmental Services

JCS:gw

Enclosure

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Summary of Calculations I-1 through I-9

Several calculations contributed to the assembly of section 4: "Diversion Ditch and Toe of Slope Protection."

Planimeter measurement and calculation I-1 show the overall contributing drainage area to be 307 acres, 265 of which are offsite.

The contributing drainage areas for the westward and eastward drainage paths are produced in calculation I-6. As shown, about 94% of the drainage area yields westward flow around the disposal cell.

A copy of the upstream drainage map is included in calculation I-6, while the longest cross-basin flow path, 4800 feet, is generated in calculation I-3.

The slope of the off-site drainage basin is found to be .50% in calculation I-2, while the natural ground slope on-site is documented as 1% in calculation I-8.

Cross-sectional profiles of the drainage ditch were generated in calculation I-5. These cross-sections were then used to determine flows in channel and overbank ditch regions in calculation I-7. Flow data is included in the attachments to calculation I-7.

Calculation I-7 also gives depth vs. discharge and velocity vs. discharge curves, which can together be used to relate velocity and discharge to given depths.

The time of concentration for a 6500 cfs design discharge (recommended by NRC during a July 13 meeting) is shown to be about 41 minutes in calculation I-9, and the westward channel drainage time is an additional 3+ minutes.

For a previously analyzed design discharge of 920 cfs, total time of concentration of 1.7 hours is yielded in calculation I-3. This calculation also provides shear stress and median riprap size information.

The peak PMF flow rate for the Kress Creek drainage area (shown in calculation I-4) was estimated using NRC Reg. Guide 1.59.

JAMES L. GRANT AND ASSOCIATES, INC.
CALCULATION SUMMARY SHEET

PROJECT NO: 809307

TASK NO: 001

CALCULATION NUMBER: I-1

DATE: 6/24/90

DESCRIPTION OF CALCULATION:

USE OF A PLANIMETER TO:

1) CHECK ACCURACY OF THE LARGER SCALE FIG. 2-2 MAP VS. SMALL SCALE USGS MAP
2) MEASURE DRAINAGE BASIN AREA FROM FIG 2-2
3) CALCULATE SITE AREA BY SUBTRACTING AREAS
4) MEASURE SITE AREA ON FIG 2-2 DIRECTLY

REFERENCES USED:

1) U.S.G.S. WEST CHICAGO QUADRANGLE MAP (TOPOGRAPHIC)
2) FIGURE 2-2 ("OFF-SITE MONITORING WELLS"), KERR-MCGEE WEST CHICAGO PROJECT

CALCULATION MADE BY: [signature] DATE: 6/24/90

CALCULATION CHECKED BY: [signature] DATE: 6/25/90

CALCULATION APPROVED BY: James L. Grant DATE: 7-9-90

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FROM U.S.G.S. W. CHICAGO QUAD MAP:

MEASUREMENT OF FULL (SITE-INCLUDED) BASIN:

1st	.235
2nd	.233
3rd	.233

AVG = .234 v.u. ✓

now from previous planimeter calibration,

.07 v.u. = 1 in²

and map scale is

1 in = 2,000 ft ✓

so area of full basin is

$$.234 \text{ v.u.} \left(\frac{1 \text{ in}^2}{.07 \text{ v.u.}}\right)\left(\frac{2000 \text{ ft}}{\text{in}}\right)^2 = \frac{13.4 \text{ million ft}^2}{43,560 \frac{\text{ft}^2}{\text{acre}}}$$

A = 308 acres ✓

TO CHECK ACCURACY OF THE LARGER SCALE MAP, RE-MEASURE AREA FROM FIG. 2.2 (OFFSITE MONITORING WELLS) MAP

MEASUREMENT OF FULL (SITE INCLUDED) BASIN

1st	.936
2nd	.933
3rd	.937

AVG = .935

as before, .070 v.u. = 1 in²

this time, map scale is

1 in = 1,000 ft ✓

AREA OF FULL BASIN IS

$$.935 \text{ v.u.} \left(\frac{1 \text{ in}^2}{.07 \text{ v.u.}}\right)\left(\frac{1,000 \text{ ft}}{\text{in}}\right)^2 \left(\frac{\text{acre}}{43,560 \text{ ft}^2}\right) =$$

A = 307 acres ✓

FROM FIG 2.2 MAP

MEASUREMENT OF BASIN ONLY: (taken into account railroad ditch)

1ST .804

2ND .803

3RD .806

AVG = .804 v.u. ✓

$$\left(.804\ v.u.\right)\left(\frac{1 in^2}{.070\ v.u.}\right)\left(\frac{1000 ft}{in}\right)^2\left(\frac{acre}{43,650\ ft^2}\right)$$

BASIN AREA = 264 acres ✓

SITE AREA = FULL AREA − BASIN AREA

= 307 acres − 264 acres

SITE AREA = 43 acres ✓

CHECK BY MEASURING SITE DIRECTLY WITH PLANIMETER (FROM FIG 2-2)

1ST .127

2ND .126

3RD .136

AVG .130 v.u. ✓

THIS CHECKS OUT WITH SECTION 9.5.6 OF KERR-McGEE ENGINEERING REPORT (42.4 ac.)

$$SITE\ AREA = \left(.130\ vu\right)\left(\frac{1 in^2}{.07\ vu}\right)\left(\frac{1000 ft}{in}\right)^2\left(\frac{acre}{43,560 ft^2}\right) = 42.5\ acres$$

BECAUSE LARGER AREAS (SUCH AS THE FULL AREA AND BASIN AREA) ARE MEASURED MORE ACCURATELY BY THE PLANIMETER, WE MIGHT EXPECT SOME ERROR IN DIRECTLY MEASURING SITE

JAMES L. GRANT AND ASSOCIATES, INC.
CALCULATION SUMMARY SHEET

PROJECT NO: B097307

TASK NO: 001

CALCULATION NUMBER: I-2

DATE: 6/24/90

DESCRIPTION OF CALCULATION:

MEASURE SLOPE OF BASIN BY:
DRAWING ~~[illegible]~~ LINES ⊥ TO CONTOURS FROM 3 DIFFERENT HIGH POINTS IN THE BASIN TO A LOWER POINT ON SITE. MEASURE LENGTHS OF THESE LINES, AND READ ELEVATION DROP FROM CONTOUR MAP. DIVIDE LATTER BY FORMER TO GET SLOPE.

REFERENCES USED:

USGS WEST CHICAGO QUADRANGLE TOPO MAP

CALCULATION MADE BY: John H. Schreiber DATE: 6/24/90

CALCULATION CHECKED BY: David G. Schrecker DATE: 6/25/90

CALCULATION APPROVED BY: James L. Grant DATE: 7-9-90

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JOB NO. 809307 SHEET 2 OF 2
JOB NAME WEST CHICAGO CONSULTING
BY JHS DATE 6/24/90
CHECKED BY DCS DATE 6/25/90

LINE ①:

from map, elevation drop = 770 ft − 750 ft = 20 ft ✓

from scaled measurement of line = 3300 ft ✓

SLOPE = $\frac{20}{3300}$ = .61% ✓

LINE ②:

from map, elevation drop = 770 − 750 ft = 20 ft ✓

scaled measurement of line = 3400 ft ✓

SLOPE = $\frac{20}{3400}$ = .59% ✓

LINE ③

from map, elevation drop = 760 − 750 ft = 10 ft ✓

scaled measurement of line = 3500 ft ✓

SLOPE = $\frac{10}{3500}$ = .29% ✓

AVG SLOPE = $\frac{.61 + .59 + .29}{3}$ = .50% = AVG SLOPE ✓

GRANT, SCHREIBER & ASSOCIATES
CALCULATION SUMMARY SHEET

JECT NO: 809307

< NO: 001

ULATION NUMBER: I-3

: 6-27-90

RIPTION OF CALCULATION:

Determine potential erosion at toe of slope and
version ditches around the disposal cell

ENCES USED:

ee, R.H., "A Guide to Hydrologic Analysis Using SCS Methods", tice Hall, Inc., 1982.

"Application of Probable Maximum Precipitation Estimates, United States of the 105th Meridian," Hydrometeorological Report No. 52, 1987

Project No. 809307 Calculations I-1 and I-2. 6-24-90

Corporation "Kerr-McGEE Chemical Corporation Rare Earths Facility ion of Resistance to Erosion," Final Report, Prepared for EPA Region V D. 68D90130, Deliver Order No. 3, May 4, 1990.

TION MADE BY: David L. Schreiber DATE: 6-27-90

TION CHECKED BY: Kevin S. Rouch DATE: 7-6-90

TION APPROVED BY: [signature] DATE: 7-9-90

6 Attachments

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JAMES L. GRANT & ASSOCIATES, INC. • geotechnical engineering • management • computer science 8301 E. PRENTICE AVE., SUITE 402 ENGLEWOOD, COLORADO, 80111 (303) 779-0576	JOB NO. 809307 SHEET 3 OF 10 JOB NAME West Chicago Erosion BY DCS DATE 6-23-90 CHECKED BY KSR DATE 6-28-90

Determine potential erosion at toe of slope and diversion ditches around disposal cell.

Riprap requirements, if any, will be determined using the U.S.N.R.C. Staff Technical Position (STP), May 1990.

The design flow will be the PMF. Contributing drainage areas, slopes, etc. to the ditches will be determined from USGS quadrangle maps and other available maps and plans.

Delineate offsite drainage area that may contribute runoff as inflow to the diversion ditches during a PMF

The site is located in a developed area of West Chicago. As such, there are existing storm drainage systems, both surface and subsurface. However, the design criteria for these systems is the 100-year flood, not the PMF. Thus, no credit for these systems is taken in estimating the PMF flows into the diversion ditches. This is a conservative assumption.

The KMCC Rare Earths Facility and its upgradient drainage area are included on USGS quadrangle maps (7.5 minute) for West Chicago and Naperville, Illinois. These maps provide topographic contours at 10-ft elevation intervals, with some information at 5-ft intervals.

Using these maps the contributing drainage area was delineated by drawing a line around the topographic high points upgradient of the site. The contributing upgradient drainage area is basically part of a divide that directs surface drainage either to Kress Creek (west and south of the site) or to the West Branch DuPage River (east of the site). The upgradient drainage area and the site itself slopes gently to the southwest towards Kress Creek.

The upgradient drainage area is bounded on the west by the Elgin, Joliet, & Eastern Railroad bed, on the north by the Chicago and North Western Railroad bed, and on the east by Neltnor Blvd. During a PMF this area will drain to the southwest across the site and around the disposal cell into the detention/sedimentation pond in the southwest corner of the site. The contributing drainage area, including the site was determined using a planimeter (see Calculation No. I-1, 6/29/90)

A_T = 307 acres ✓

The site area itself is comprised of 32.9 acres (the Disposal Cell and the South 300 ft of the Intermediate site) and 9.5 acres (the Factory site and the northern portion of the Intermediate site). Thus, site area is

$A_s = 42.4$ acres ✓

Therefore, the offsite drainage area that may contribute runoff as inflow to the diversion ditches during a PMF is

$A_O = A_T - A_S$

$= 307 - 42.4$

$A_O = 265$ acres ✓

Determine the peak runoff rate from the total drainage area, including offsite area, as well as the site.

Use the Rational Method, since this is a small area and we are taking no credit for the existing storm drainage system.

$Q = CIA$

where

Q = Peak runoff rate (cfs)
C = Runoff coefficient
I = Average rainfall intensity (in/hr) lasting for a critical period of time, t_c
t_c = Time of concentration
A = Drainage area (acres)

Estimate time of concentration from the remotest point in the basin to the diversion ditch, t_c.

Assume that the runoff will be flowing across this small basin as shallow concentrated flow (since the length of travel is much greater than 300 ft, which is applicable to sheet flow).

See Fig. 3 from NUS report (Attachment No. 1), which is taken from McEwen, 1982, Fig. 8 (p. 22)

From Calculation No. I-2 (6-24-90), the average basin slope is

$S = 0.005$ ft/ft (or 0.5%) ✓

For paved area $V_1 = 1.40$ ft/sec ✓, $A_1 \approx 20\%$
For grassed waterway $V_2 = 1.05$ ft/sec ✓, $A_2 \approx 30\%$
For nearly bare $V_3 = 0.71$ ft/sec ✓, $A_3 \approx 20\%$
For woodland $V_4 = 0.35$ ft/sec ✓, $A_4 \approx 30\%$

Derive composite velocity based upon percentage of area of basin that is comprised of the four above land uses.

$$V_1 \frac{A_1}{A} + V_2 \frac{A_2}{A} + V_3 \frac{A_3}{A} + V_4 \frac{A_4}{A} = V$$

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JOB NO. 909307 SHEET 5 OF 10
JOB NAME West Chicago Erosion
BY DCS DATE 6-25-90
CHECKED BY KSD DATE 6-28-90

$1.4(0.2) + 1.05(0.3) + 0.71(0.2) + 0.35(0.3) = V$
$0.28 + 0.315 + 0.142 + 0.105$ ✓
$V = 0.84$ ft/sec

from McEwen (Attachment No. 1), eq 11

$t_{c_o} = L/V$

where
L is the longest flow path length from the USGS drainage basin map. the length L from the most remote point to the diversion ditch is

$L = 3300 + 1000 + 500$
$L = 4800$ ft ✓

where 3300 ft is length of Line 1 (Calc. No. I-2, 6-24-90), 1000 ft is length beyond Line 1 to basin edge, and 500 ft is length from diversion ditch to Line 1

thus

$t_{c_o} = \frac{4800 \text{ ft}}{0.84 \frac{\text{ft}}{\text{sec}} (3600 \frac{\text{sec}}{\text{hr}})}$ ✓

$t_{c_o} = 1.6$ hr ✓

Estimate time of concentration through diversion ditch to detention pond
Assume, for conservatism, that the velocity in the channel is the recommended maximum permissible velocity of (see NRCS Report, p. 14) ✓
$V_d = 5.0$ ft/sec ✓

thus, the travel time through the ditch is

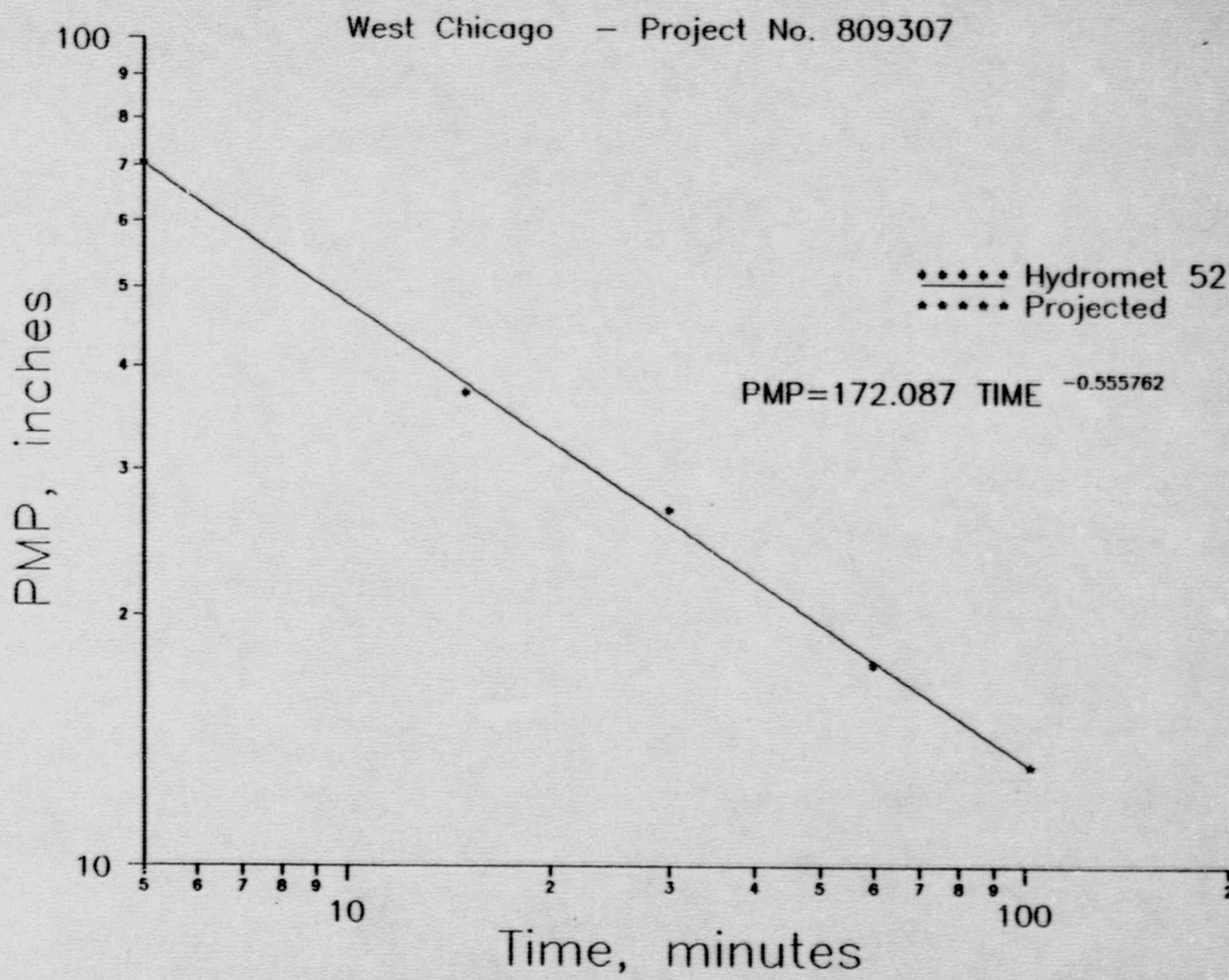
$t_{c_d} = L_d/V_d$

$= \frac{1200 \text{ ft}}{(5.0 \frac{\text{ft}}{\text{sec}})(3600 \frac{\text{sec}}{\text{hr}})}$

$t_{c_d} = 0.1$ hr ✓

Therefore, the total time of concentration through the overall basin is

$t_c = t_{c_o} + t_{c_d}$
$= 1.6 + 0.1$ ✓
$t_c = 1.7$ hr ✓



West Chicago - Project No. 809307
100
9
8
7
6
5
4
3
2
10
PMP, inches
Hydromet 52
Projected
PMP=172.087 TIME −0.555762
5
6
7
8
9
10
2
3
4
5
6
7
8
9
100
2
Time, minutes

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JOB NO. 809307 SHEET 8 OF 10
JOB NAME West Chicago Erosion
BY DCS DATE 6-27-90
CHECKED BY KSR DATE 7-6-90

Determine peak runoff rate from the 307-ac basin

$$Q = CIA$$
$$= 0.39\,(7.7\ \tfrac{in}{hr})(307\ ac)$$
$$\underline{Q = 920\ cfs}\ ✓$$

Determine rip-rap size if needed for this peak runoff rate. Assume uniform flow in diversion channels, thus Manning's Equation applies:

$$Q = \frac{1.49}{n} A R^{2/3} S^{1/2}$$

where

n = roughness coefficient
A = wetted cross-sectional area
R = hydraulic radius
S = energy slope ≈ bottom slope

From Chow, p21 (Attachment No 4), for a triangular channel

$$A = zy^2\ ✓$$
$$R = \frac{zy}{2\sqrt{1+z^2}}\ ✓$$

where z = horizontal component of side slope = 5 ✓
(from Engineering Report, Section 2.5.6.1)
y = water depth

thus,

$$920 = \frac{1.49}{n}(5y^2)\left[\frac{5y}{2\sqrt{1+(5)^2}}\right]^{2/3}(0.01)^{1/2}$$

$$\frac{920\,n}{1.49(5)(0.1)} = y^2\left[\frac{5y}{2\sqrt{26}}\right]^{2/3}$$

$$1235\,n = y^2 (0.49)^{2/3} y^{2/3}$$

$$1986\,n = y^{8/3}$$

$$\underline{y = 17.25\,(n)^{.375}}\ ✓$$

Based upon USGS Water Supply Paper No. 1849, pp. 136-137 (Attachment No 5), assume

$$n = 0.045$$

then $\underline{y = 5.4\ ft}$ ✓

check on sensitivity of n. If n = 0.03 (pp. 29-33, USGS WSP 1849)

$\underline{y = 4.6\ ft}$ ✓

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Compute the actual shear stress produced using the Safety Factor Method (Final NRC Staff Technical Position on Design of Erosion Protection Covers, May 1990), Appendix D, pp D-12 thru D-15 (Attachment No. 6)

$$\tau = w y S$$

where

τ = shear stress
w = unit weight of water
y = normal depth
S = bottom slope

Use a water depth of 5.0 ft

$$\tau = 62.4 \text{ lb/ft}^3 (5.0 \text{ ft})(0.01)$$

$$\tau = 3.12 \text{ lb/ft}^2$$

Compute the required rock size from the NRC Staff Technical Position

$$\tau = a(w_s - w_w) D_{50}$$

where

a = 0.04
w_s = unit weight of stone, say 165 lb/ft³
w_w = unit weight of water
D_{50} = stone size

Thus,

$$\tau = 0.04(165 - 62.4) D_{50}$$

$$\tau = 4.1 D_{50}$$

or

$$D_{50} = \tau / 4.1$$

$$= \frac{3.12}{4.1}$$

$$D_{50} = 0.76 \text{ ft, or say}$$

$$D_{50} = 9 \text{ inch stone}$$

This will protect the diversion channels from eroding during the local PMF

The Engineering Report, Section 9, Sketch No. 2&3 indicates a stone size in the channel section of 4-inches.

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JOB NAME West Chicago Erosion
BY DCS DATE 6-27-90
CHECKED BY KSR DATE 7-6-90

Check the stone size required to withstand erosion from a Standard Project Flood (SPF)

An SPF is generally considered to be about 40-60 percent the magnitude of a PMF, thus, for 50%

$Q_{spf} \cong 460$ cfs ✓

Determine rip-rap size needed for the SPF peak flow. From p. 7 of these calcs (uniform flow)

$$\frac{Q n}{1.49 (S)(0.1)} = (0.48)^{2/3} y^{8/3}$$

$$\frac{460 n}{1.49 (0.5)(0.62)} = y^{8/3}$$

$$994 n = y^{8/3}$$

$$y = 13.3 n^{.375}$$ ✓

Since flow is less than SPF, the rip-rap will be smaller in size, thus use

$$n = 0.03$$

then

$$y = 13.3 (0.03)^{0.375}$$

$$y = 3.6 \text{ ft}$$ ✓

Compute shear stress produced

$$\tau = \omega y S$$

$$= 62.4 (3.6)(0.01)$$

$$\tau = 2.25 \text{ lb/ft}^2$$ ✓

Compute rock size required

$$\tau = k(\omega_s - \omega) D_{50}$$

$$\tau = 4.1 D_{50}$$

$$D_{50} = \frac{2.25}{4.1}$$

$$D_{50} = 0.55 \text{ ft}$$ ✓

$$D_{50} \cong 6.6 \text{ inches}$$ ✓

A GUIDE TO HYDROLOGIC ANALYSIS USING SCS METHODS

RICHARD H. McCUEN
University of Maryland

Attachment I

PRENTICE-HALL, INC., Englewood Cliffs, New Jersey 07632
1982

The Upland Method

Essentially the same input is required for estimating t_c with the upland, or velocity, method as with the lag method. The velocity method has an intermediate step in which the velocity is estimated with the land use and the slope; Fig. 8 is used to estimate the velocity in feet per second (fps). The time-of-concentration equals the ratio of the hydraulic flow length to the velocity:

$$t_c = \frac{\ell}{V} \tag{11}$$

If ℓ is measured in feet and V in fps, then the value resulting from Eq. 11 must be divided by 3600 in order to convert t_c from seconds to hours.

Adjustment for Urbanization

The curve number appears not to adequately reflect the effect of the soil cover complex in urban areas on the runoff potential. For composite land use areas where urban land uses provide a more efficient flow pattern than pervious land uses, Eq. 9 overestimates lag. TR-55 provides two figures that can be used to adjust the lag computed by Eq. 9 for the percentages of the hydraulic length that is modified (PHLM) and the impervious areas (PIMP); the lag adjustment factors are used independently, and therefore, both values can be applied on the same project. The lag factor (LF) for each adjustment can be computed using values from the following Equation:

$$LF = 1 - PRCT(-0.006789 + 0.000335\,CN - 0.0000004298\,CN^2 - 0.00000002185\,CN^3)$$

in which CN is the curve number for future land use conditions and PRCT is either the percent hydraulic length modified (PHLM) or the percent impervious area (PIMP). The lag computed from Eq. 9 is then multiplied by the lag factor from the above equation. If both adjustments are necessary, then the equations are used for both modifications and two lag factors are applied to the lag computed from Eq. 9.

TR-55 provides two important guidelines for use of these adjustment factors:

1. Since the lag factors are used only with future-condition curve numbers, the lag factors cannot be used to directly compute the decrease in lag from present conditions.

2. When only peak discharges are to be computed using the TR-55 methods, lag does not have to be computed; therefore, these lag factor adjustments are not necessary.

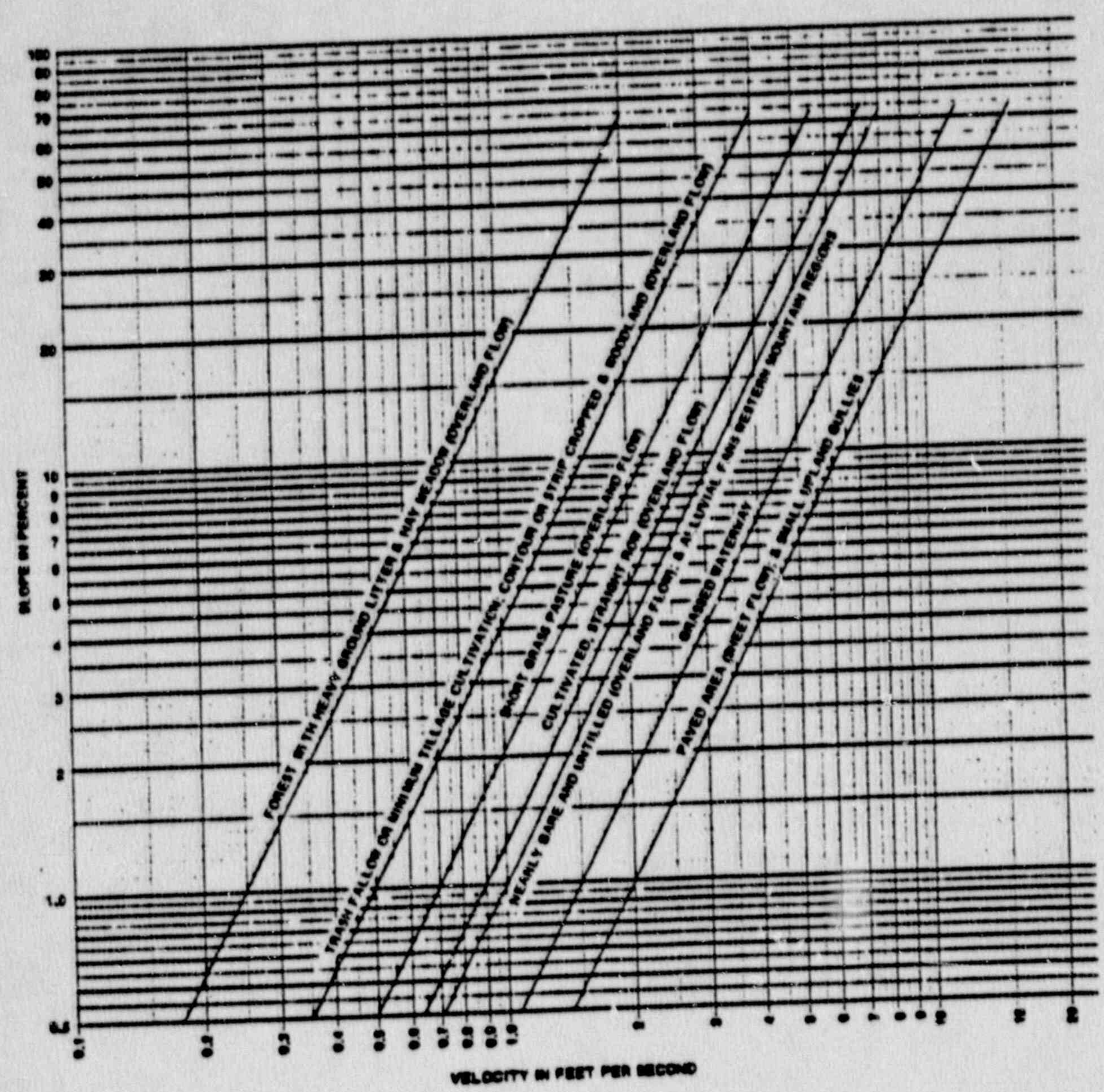


Figure 8. Velocities for upland method of estimating T_c

From McCuen

NATIONAL OCEANIC AND ATMOSPHERIC ADMINISTRATION

CORPS OF ENGINEERS

PB83-118166

NOAA HYDROMETEOROLOGICAL REPORT NO. 52

Application of Probable Maximum Precipitation Estimates - United States East of the 105th Meridian

Attachment II

Prepared by
E.M. Hansen, L.C. Schreiner & J.F. Miller
Hydrometeorological Branch
Office of Hydrology
National Weather Service

WASHINGTON, D.C.
August 1982

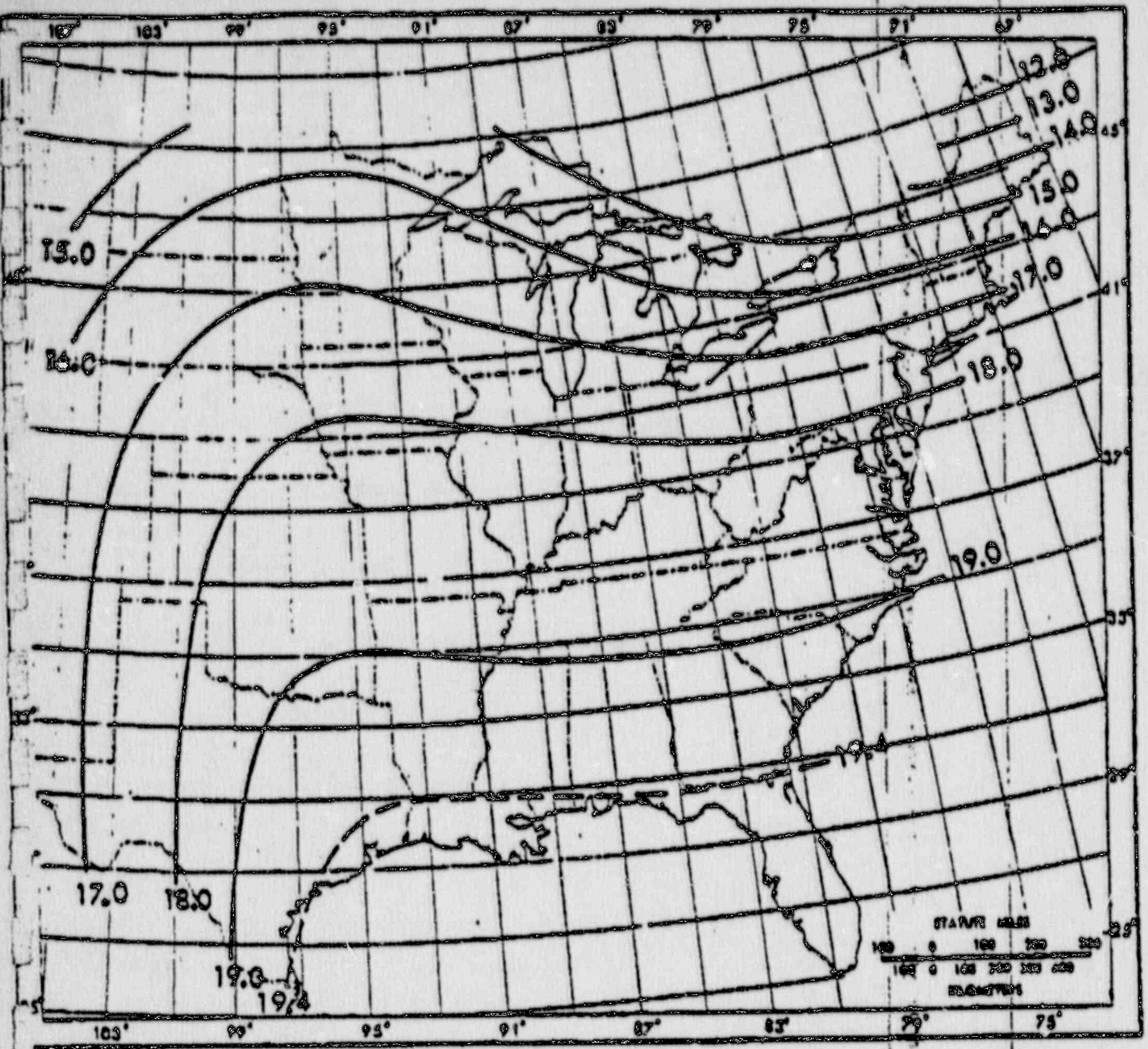


Figure 24.—1-hr 1-mi² PMP analysis based on figure 23 and 6-hr 10-mi² precipitation from HMR No. 51.

transposition limits. Comparison of this 18.3-in. value with the 1-hr 1-mi² PMP on figure 24 shows a difference of 0.6 in. We consider this a reasonable velopment of a moisture maximized transposed amount.

3.3 Depth-area ratios

Preparation of 1-hr PMP values over the range of area sizes of interest -quired development of depth-area reduction ratios. A primary basis for such duction ratios is the list in table 19 of 12 extreme storms (those noted by asterisks) for which point or 1-mi² data are available at 1 hr. A problem with the data from these 12 storms is the limited area of most storms. Nearly 60 rcent have an areal extent of less than 340 mi², while one fourth of them

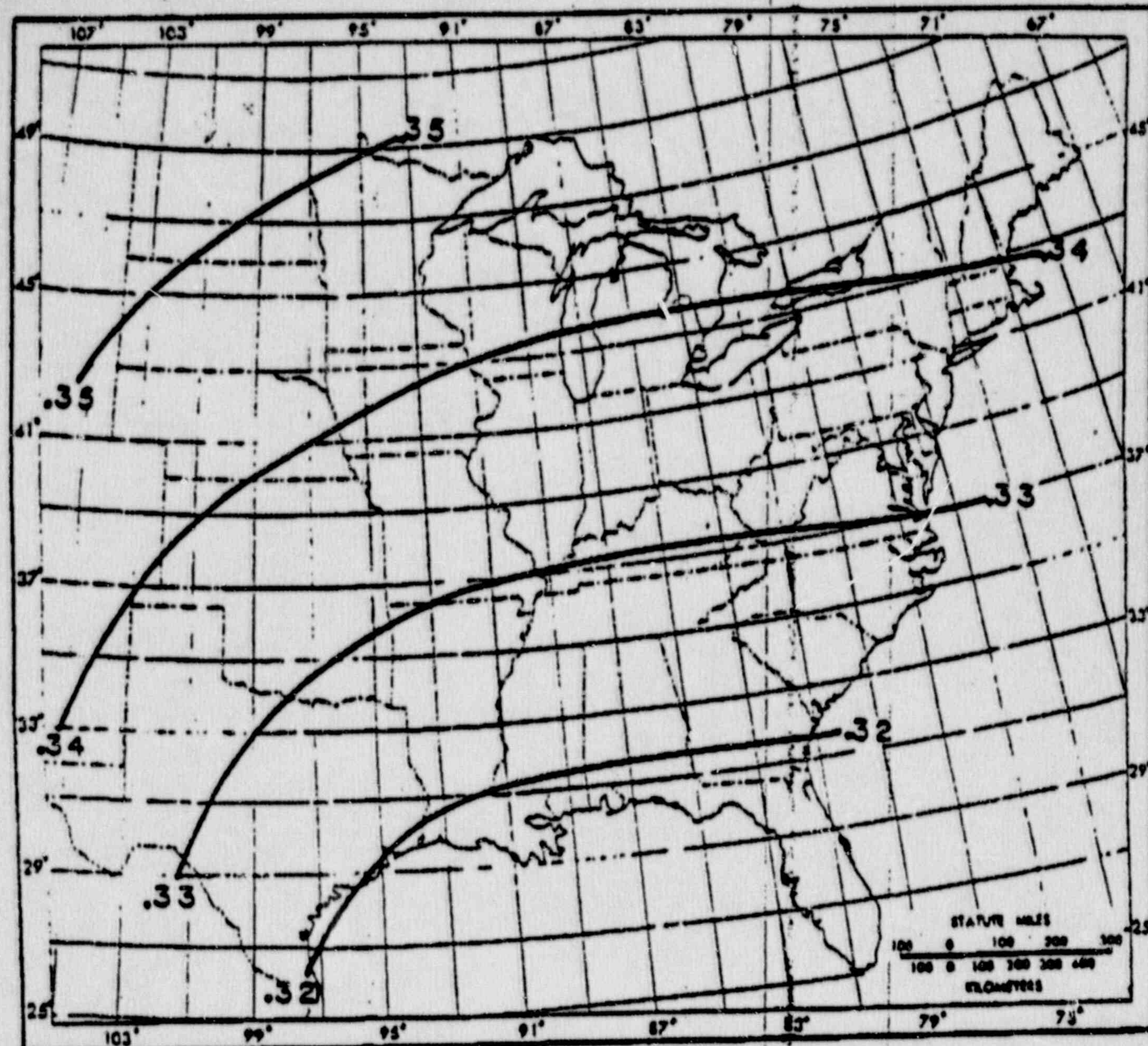
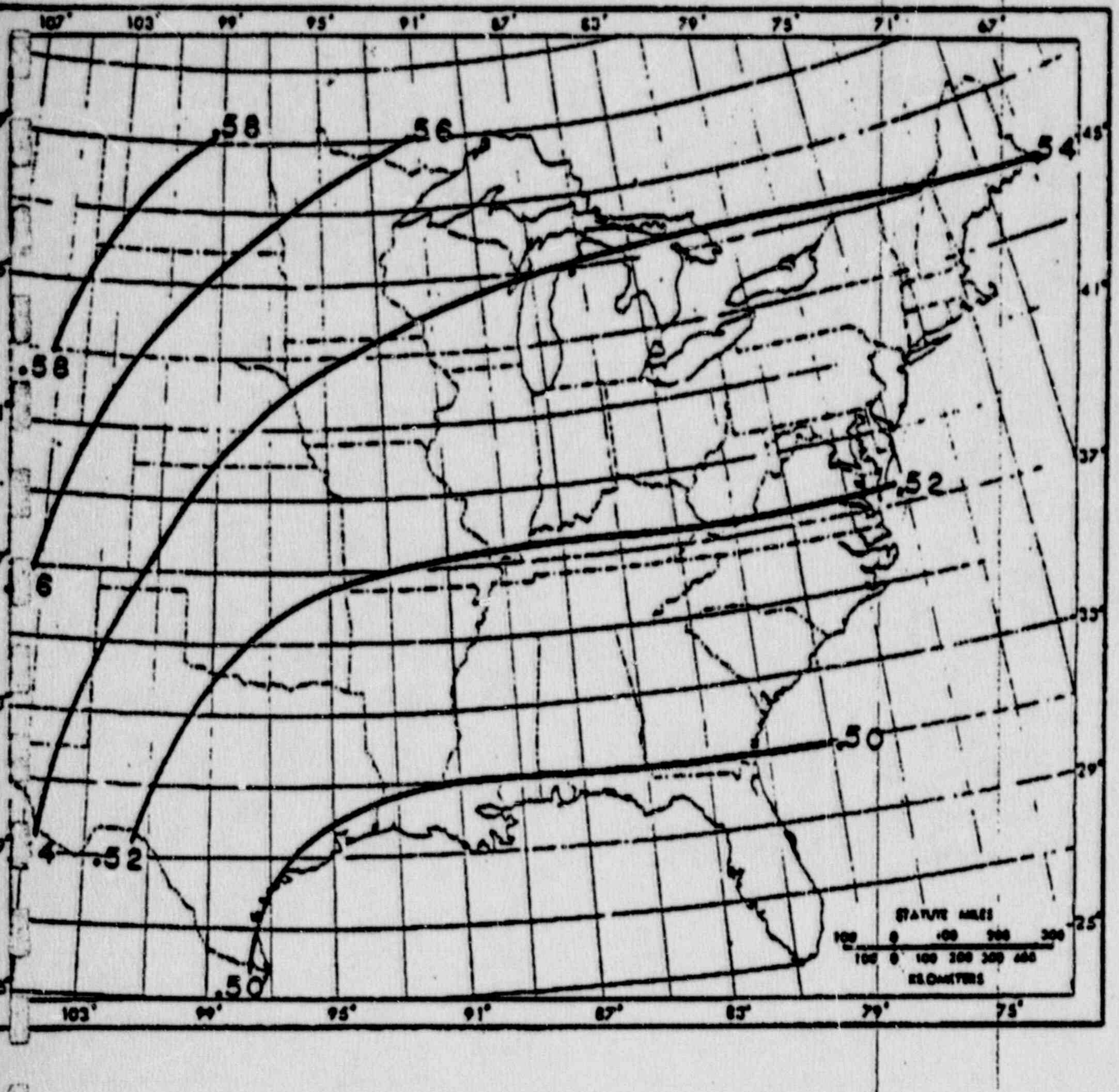


Figure 36.—Ratio analysis of 5- to 60-min precipitation used to obtain 5-min PMP. (Applicable to area sizes < 200 mi^2.)



igure 37.—Ratio analysis of 15- to 60-min precipitation used to obtain 15-min PMP. (Applicable to area sizes < 200 mi^2.)

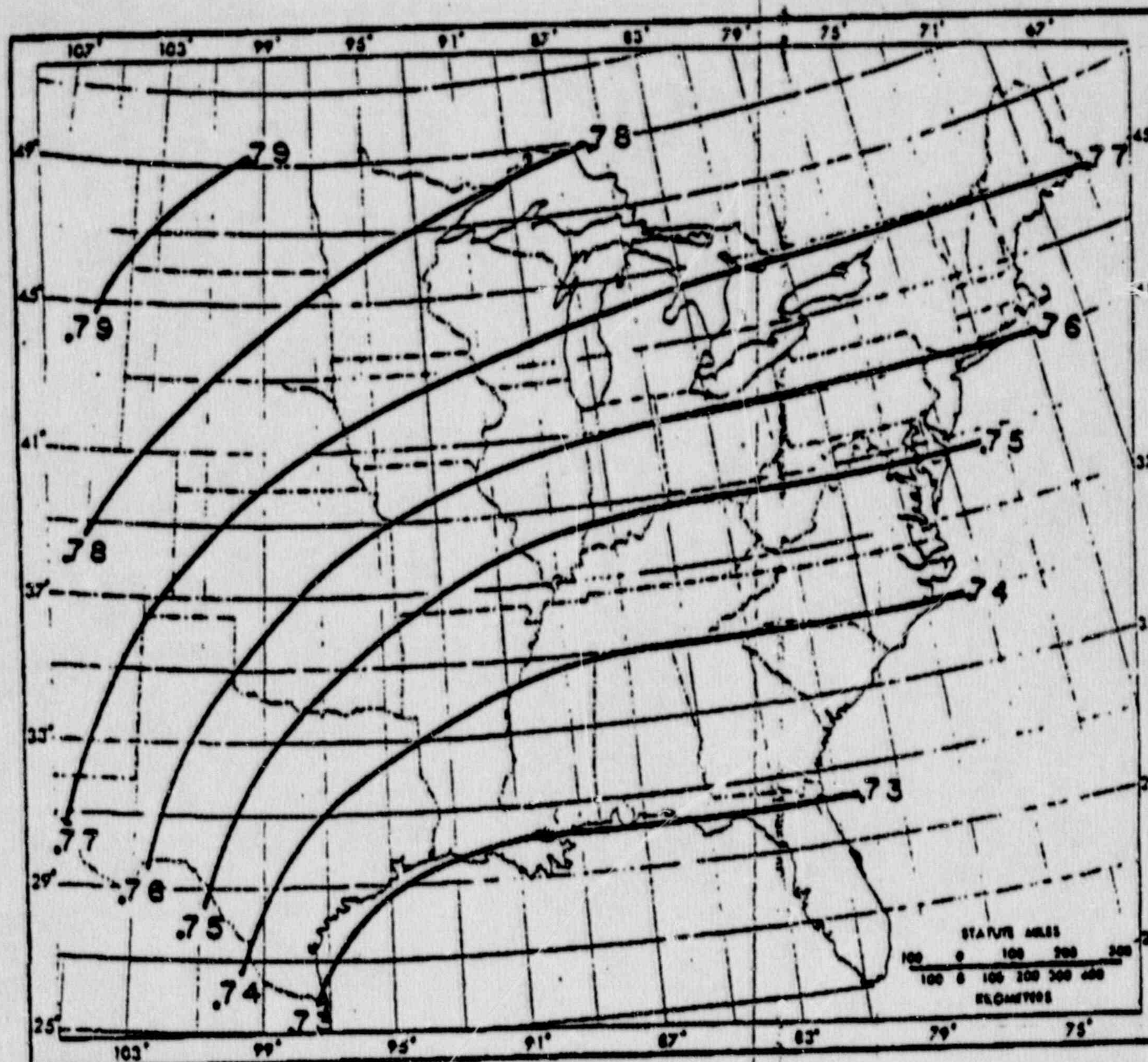


Figure 38.—Ratio analysis of 30- to 60-min precipitation used to obtain 30-min PMP. (Applicable to area sizes < 200 mi^2.)

ENGINEERING HYDROLOGY
PRINCIPLES AND PRACTICES

VICTOR MIGUEL PONCE
San Diego State University

Attachment III

PRENTICE HALL, Englewood Cliffs, New Jersey 07632

1989

TABLE 4-1(a) AVERAGE RUNOFF COEFFICIENTS FOR URBAN AREAS: 5-Y AND 10-Y DESIGN FREQUENCY

Description of Area	Runoff Coefficients
Business	
Downtown areas	0.70 to 0.95
Neighborhood areas	0.50 to 0.70
Residential	
Single-family areas	0.30 to 0.50
Multiple units, detached	0.40 to 0.60
Multiple units, attached	0.60 to 0.75
Residential (suburban)	0.25 to 0.40
Apartment-dwelling areas	0.50 to 0.70
Industrial	
Light areas	0.50 to 0.80
Heavy areas	0.60 to 0.90
Parks, cemeteries	0.10 to 0.25
Playgrounds	0.10 to 0.25
Railroad yard areas	0.20 to 0.40
Unimproved areas	0.10 to 0.30

Character of Surface	Runoff Coefficients
Streets	
Asphaltic	0.70 to 0.95
Concrete	0.80 to 0.95
Brick	0.70 to 0.85
Drives and walks	0.70 to 0.85
Roofs	0.75 to 0.95
Lawns, sandy soil	
Flat (2 percent)	0.05 to 0.10
Average (2 to 7 percent)	0.10 to 0.15
Steep (7 percent)	0.15 to 0.20
Lawns, heavy soil	
Flat (2 percent)	0.13 to 0.17
Average (2 to 7 percent)	0.18 to 0.22
Steep (7 percent)	0.25 to 0.35

Source: Design and Construction of Sanitary and Storm Sewers. ASCE Manual of Engineering Practice, no. 37, 1960.

From Ponce

and, therefore, of rainfall frequency. Higher values of runoff coefficient are applicable for higher values of rainfall intensity and return period. A typical C versus I curve is shown in Fig. 4-1 [5]. Alternate ways of expressing the variation of runoff coefficient with rainfall frequency are shown in Figs. 4-2 and 4-3 [6, 22].

With runoff coefficient, rainfall intensity, and catchment area determined, the peak discharge is calculated by Eq. 4-1. The apparent simplicity of the procedure, however, is misleading. For one thing, there is a range of possible runoff coefficients for each surface condition. Therefore, the chosen C value is usually based on additional field information or designer's experience. The effect of frequency and/or antecedent moisture condition needs to be evaluated carefully. Furthermore, there is no absolutely certainty that the calculated time of concentration (and therefore, the rainfall duration) is correct or even that it remains constant throughout the range of possi-

Figure 4-1 Variatio
with rainfall intensit

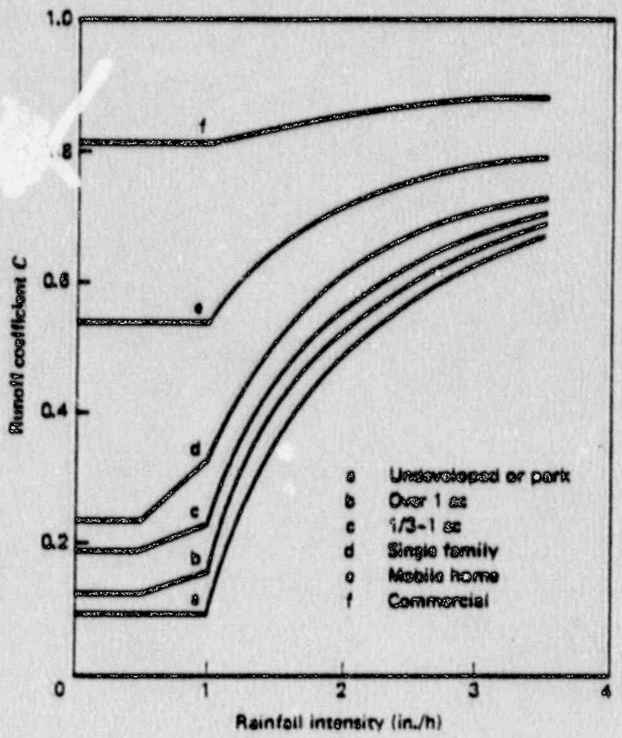
From Ponce

TABLE 4-1(b) AVERAGE RUNOFF COEFFICIENTS FOR RURAL AREAS

Topography and Vegetation	Soil Texture		
	Open Sandy Loam	Clay and Silt Loam	Tight Clay
Woodland[1]			
Flat	0.10	0.30	0.40
Rolling	0.25	0.35	0.50
Hilly	0.30	0.50	0.60
Pasture			
Flat	0.10	0.30	0.40
Rolling	0.16	0.36	0.55
Hilly	0.22	0.42	0.60
Cultivated Land			
Flat	0.30	0.50	0.60
Rolling	0.40	0.60	0.70
Hilly	0.52	0.72	0.82

[1]*Note:* Flat (0–5% slope); rolling (5–10%); hilly (10–30%).

Source: Schwab, R. J. et al. (1971). *Elementary Soil and Water Engineering*. 2d. ed. New York: John Wiley.



... Variation of runoff coefficient ... intensity [5].

Sewers.

coefficient are applicable ...
typical C versus I curve ...
at ... of runoff coefficient ...
2].

ent area determined, th...
p... ity of the procedure ...
ossible runoff coefficient ...
is usually based on add...
of ... equency and/or a...
F... thermore, there is ...
(and therefore the ra...
ughout the range of p...

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OPEN-CHANNEL HYDRAULICS

VEN TE CHOW, Ph.D.

Professor of Hydraulic Engineering
University of Illinois

Attachment IV

McGRAW-HILL BOOK COMPANY, INC.

New York Toronto London

1959

, but the change in elevation is
: flowing partly full is a covered
stalled to drain water through
The open-flow tunnel is a com-
onvey water through a hill or any

ll with unvarying cross section
rismatic channel. Otherwise, the
, trough spillway having variable
ifically indicated, the channels

book refers to the cross section of
of the flow. A vertical channel
l passing through the lowest or
or horizontal channels, therefore,
channel section.

very irregular, usually varying
n approximate trapezoid. For
channel may consist of a main
s and one or more side channel

ed with sections of regular geo-
etric shapes that are in common
ape for channels with unlined
or stability. The rectangle and
oid. Since the rectangle has
nnels built of stable materials,
timber. The triangular section
gutters, and laboratory works.
ts and culverts of small and
an approximation of sections of
The round-cornered rectangle
round-bottom triangle is an
usually created by excavation

e circle are frequently used in
gh for a man to enter. These
ing to their form; they may be

e intersection of the sides with the
formula $z = T/4y$.
and parabolic sections of higher order:
omputed from the side slope assumed

TABLE 2-1. GEOMETRIC ELEMENTS OF CHANNEL SECTIONS

Section	Area A	Wetted perimeter P	Hydraulic radius R	Top width T	Hydraulic depth D	Section factor $Z = A\sqrt{D}$
Rectangle	by	$b + 2y$	$\frac{by}{b+2y}$	b	y	$by^{1.5}$
Trapezoid	$(b+zy)y$	$b + 2y\sqrt{1+z^2}$	$\frac{(b+zy)y}{b+2y\sqrt{1+z^2}}$	$b + 2zy$	$\frac{(b+zy)y}{b+2zy}$	$\frac{[(b+zy)y]^{1.5}}{\sqrt{b+2zy}}$
Triangle	zy^2	$2y\sqrt{1+z^2}$	$\frac{zy}{2\sqrt{1+z^2}}$	$2zy$	$\frac{1}{2}y$	$\frac{\sqrt{2}}{2}zy^{2.5}$
Circle	$\frac{1}{8}(\theta - \sin\theta)d_0^2$	$\frac{1}{2}\theta d_0$	$\frac{1}{4}\left(1 - \frac{\sin\theta}{\theta}\right)d_0$	$(\sin\frac{1}{2}\theta)d_0$ or $2\sqrt{y(d_0-y)}$	$\frac{1}{8}\left(\frac{\theta - \sin\theta}{\sin\frac{1}{2}\theta}\right)d_0$	$\frac{\sqrt{2}}{32}\frac{(\theta-\sin\theta)^{1.5}}{(\sin\frac{1}{2}\theta)^{0.5}}d_0^{2.5}$
Parabola	$\frac{2}{3}Ty$	$T + \frac{8}{3}\frac{y^2}{T}$ *	$\frac{2T^2y}{3T^2+8y^2}$ *	$\frac{3}{2}\frac{A}{y}$	$\frac{2}{3}y$	$\frac{2}{9}\sqrt{6}\,Ty^{1.5}$
Round-cornered rectangle ($y > r$)	$\left(\frac{\pi}{2} - 2\right)r^2 + (b+2r)y$	$(\pi - 2)r + b + 2y$	$\frac{(\pi/2 - 2)r^2 + (b+2r)y}{(\pi-2)r + b + 2y}$	$b + 2r$	$\frac{(\pi/2-2)r^2}{b+2r} + y$	$\frac{[(\pi/2-2)r^2 + (b+2r)y]^{1.5}}{\sqrt{b+2r}}$
Round-bottomed triangle	$\frac{T^2}{4z} - \frac{r^2}{z}(1 - z\cot^{-1}z)$	$\frac{T}{z}\sqrt{1+z^2} - \frac{2r}{z}(1 - z\cot^{-1}z)$	$\frac{A}{P}$	$2[z(y-r) + r\sqrt{1+z^2}]$	$\frac{A}{T}$	$A\sqrt{\frac{A}{T}}$

* Satisfactory approximation for the interval $0 < x \leq 1$, where $x = 4y/T$. When $x > 1$, use the exact expression $P = (T/2)[\sqrt{1+x^2} + 1/x \ln(x + \sqrt{1+x^2})]$.

Roughness Characteristics of Natural Channels

By HARRY H. BARNES, Jr.

GEOLOGICAL SURVEY WATER-SUPPLY PAPER 1849

Color photographs and descriptive data for 50 stream channels for which roughness coefficients have been determined

Attachment V

UNITED STATES GOVERNMENT PRINTING OFFICE, WASHINGTON : 1967

No. 47 downstream along right bank from above section 1,
Columbia River at The Dalles, Oreg.

No. 38 upstream along left bank from section 4,
Columbia River at The Dalles, Oreg.

[illegible]4–[illegible]7. Columbia River at The Dalles, Oreg.

Gage location.—Lat 45°36′10″, long 121°10′40″, in NW¼ sec. 3, T. 1 N., R. 13 E., at upstream end of Port of The Dalles dock at The Dalles, 3.2 miles downstream from The Dalles Dam and at mile 189.3. Section 4 is at cableway 9 miles upstream from gage.

Drainage area.—237,000 sq mi, approximately.

Date of flood.—May 31, 1948.

Gage height.—154.56 ft at gage; 171.44 ft at section 1.

Peak discharge.—1,000,000 cfs.

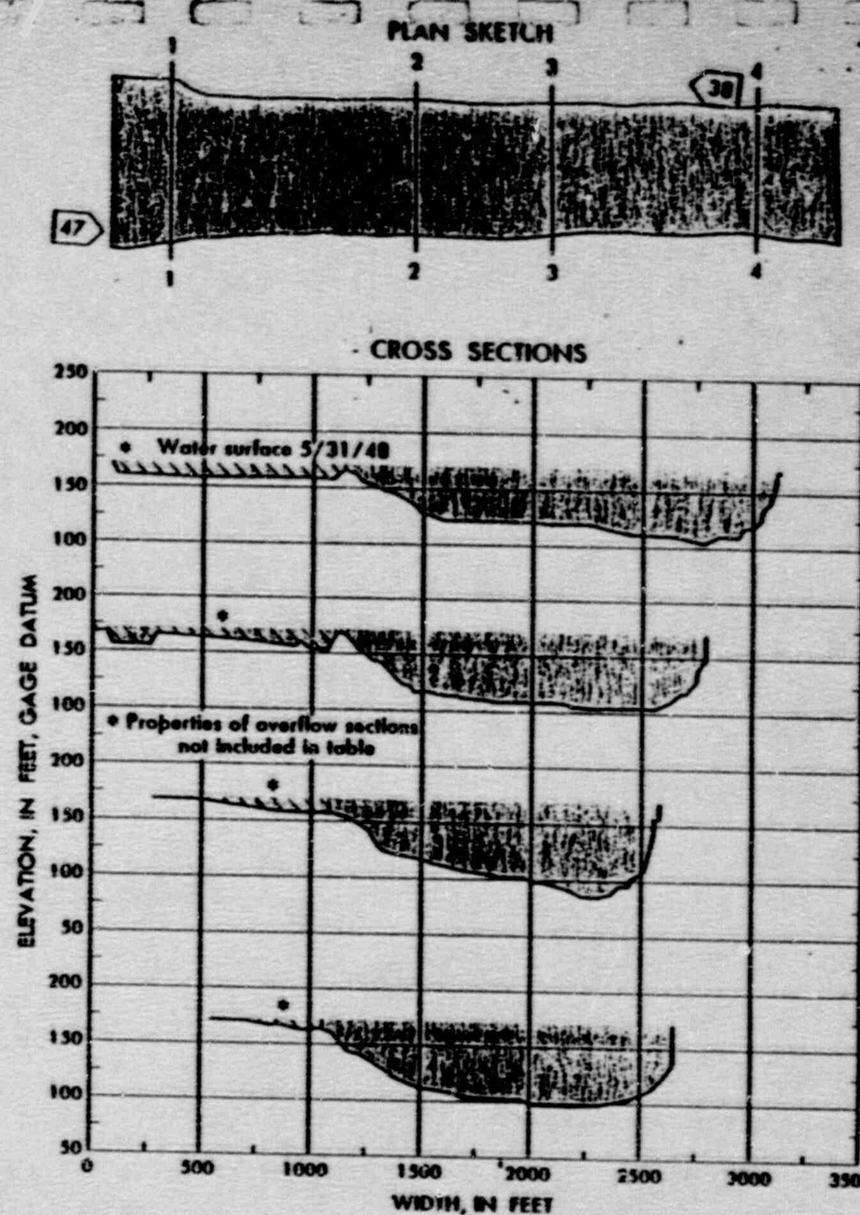
Computed roughness coefficient.—Manning $n=0.030$.

Description of channel.—Left bank consists of sand, gravel, and boulders, and has light cover of brush in some places. Right bank is formed by severely scalloped basalt cliffs. The bed material is fairly well graded from 18-inch boulders along the left bank down to sand at the base of the right bank.

Reach properties

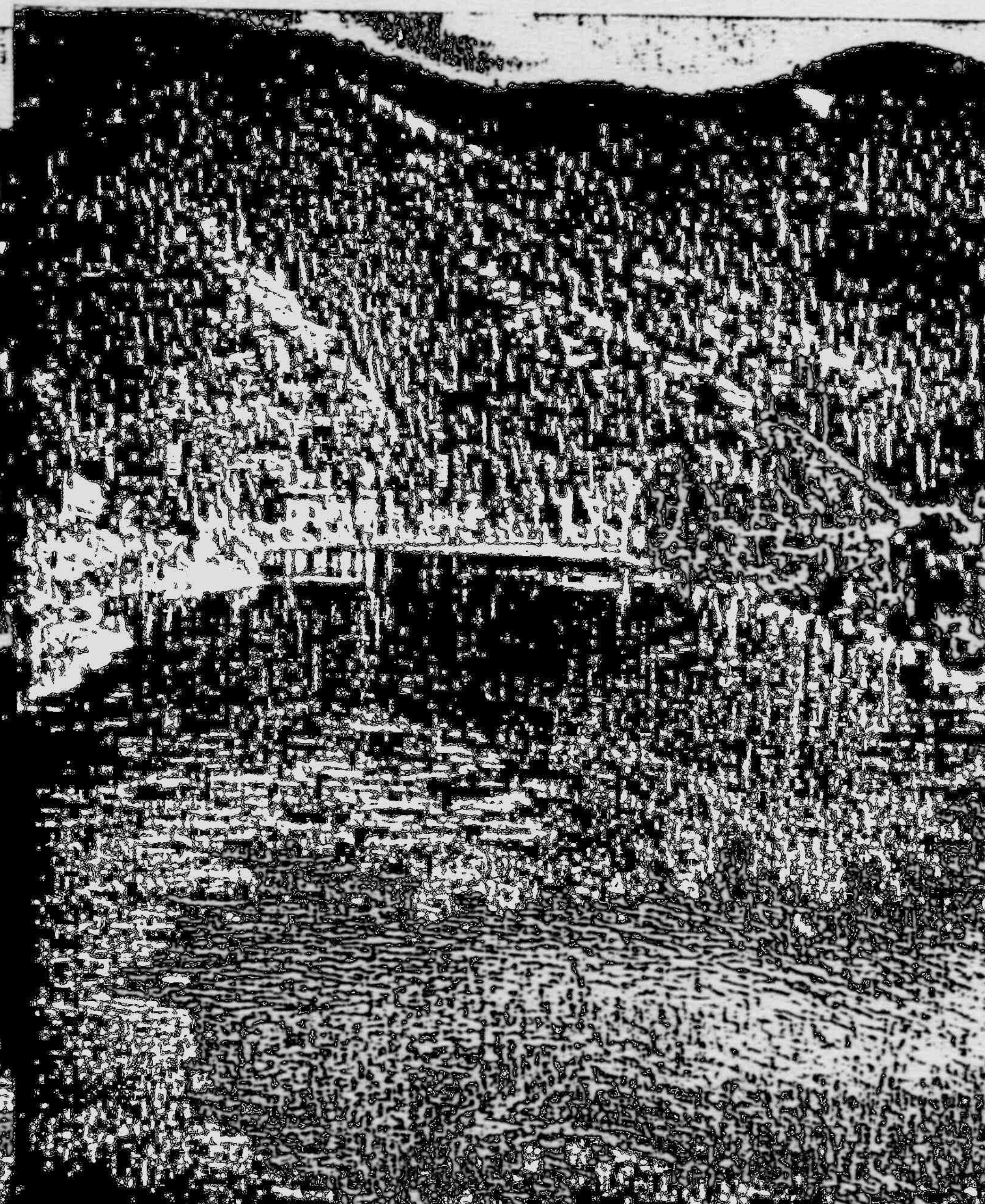
Section	Area (sq ft)	Top width (ft)	Mean depth (ft)	Hydraulic radius (ft)	Mean velocity (ft per sec)	Length (ft) between sections	Fall (ft) between sections
1..........	94,650	1,980	47.80	47.28	10.57	....	....
2..........	90,810	1,640	55.37	54.10	11.01	2,003	0.50
3..........	89,040	1,485	59.96	57.60	11.23	778	.20
4..........	91,890	1,600	57.43	55.81	10.88	1,394	.41

Notes.—



Plan sketch and cross sections, Columbia River at The Dalles, Oreg.

No. 18 downstream along left bank from above section 3, Clark Fork above Missoula, Mont.



No. 19 downstream through reach from bridge 400 ft above section 1, Clark Fork above Missoula, Mont.

No. 769 downstream from section 3, Provo River near Hailstone, Utah.

No. 770 upstream from section 3, Provo River near Hailstone, Utah.

DRAFT

FINAL
STAFF TECHNICAL POSITION
DESIGN OF EROSION PROTECTION COVERS FOR STABILIZATION OF URANIUM MILL TAILINGS SITES

U. S. Nuclear Regulatory Commission

May, 1990

Attachment VI

DRAFT

DRAFT

rainfall within a given storm event, the magnitude of historic flood events can provide some guidance in the selection of design parameters. For example, a flood with a magnitude of 2630 cfs occurred on a 200-acre drainage basin in southwestern Utah (Crippen and Bue, 1977, see Ref. D11). It can be seen that very high values of rainfall intensity and very low values of infiltration were necessary to produce such a flood.

Step 4. Calculate Riprap Size Required

a. Assume a trial rock size D_{50}.

b. Calculate Manning's 'n' value using either (1) the method discussed by Abt, et al. (1987, see Ref. D4), if the channel slope is steep and the depth of flow is small relative to the assumed D_{50} or (2) using the U. S. Army Corps of Engineers method (USCOE, 1970, Plate 4, see Ref. D5), if the slope is mild and the depth of flow is large, relative to the assumed D_{50}.

c. Calculate normal depth using Manning's equation (Chow, 1959, see Ref. D6) if the channel cross-section and slope are uniform. Otherwise, a standard-step backwater model, such as HEC-2 (USCOE, see Ref. D7) should be used to determine flow depths and velocities.

d. Compute the peak shear stress produced in the channel. The peak shear stress for a typical V-shaped or trapezoidal channel will be produced at the point where the depth of flow is the greatest. This depth should be used for design and should be used to compute shear stress.

e. Compute the rock size necessary to resist the computed shear stress. Return to (a) if the computed D_{50} is significantly different from the assumed D_{50}.

DRAFT

3.3 Recommendations

Recommendations for each design area are discussed in the design procedures. As stated, the rock in the channels should be designed for the peak shear stress (rather than the average shear stress) produced. Manning's 'n' values should be determined based on the relative depth of flow in the channel.

In many cases where natural gullies discharge into diversion ditches, it may be necessary to assess the potential for possible clogging of the ditch due to sediment and debris. Particularly where the inflow slopes are greater than the ditch slopes, it is possible that the natural gully will be capable of moving material that the diversion ditch cannot flush out. If the larger material cannot be flushed by the ditch flows, the capacity of the ditch may be compromised, resulting in possible overtopping of the ditch. The following recommendations should be followed in such cases.

1. Diversion ditches should be designed to be self-cleaning.

2. If a ditch cannot be designed to be self-cleaning, it should be designed to contain the sediment/debris that will be deposited in the ditch during the design life. Justification may also be provided to show that there is little or no debris/sediment to be transported. It may also be possible to show that the configuration of the deposits in the ditch will have no adverse effects on either the flow capacity or the stability of the ditch.

3.4 Example of Procedure Application

A 15-foot wide trapezoidal channel with 1V on 5H side slopes will be constructed on a 5 percent slope and will carry a discharge of 1000 cfs. Determine the riprap requirements.

Step 1. Assume a trial D_{50} equal to 2.0 feet (24 inches).

DRAFT.

Step 2. Compute Manning's 'n' value.

Since the slope is relatively steep, the flow depth is likely to be small relative to the riprap size. Therefore, the 'n' value should be computed in accordance with the recommendations of Abt, et al. (1987, see Ref. D4).

Using the equation from Ref. D4:

$$n = 0.0456\ (24 \times 0.05)^{.159}$$

$$n = 0.047$$

Step 3. Determine normal depth (y).

By trial and error for the trapezoidal channel, with

$n = .047$; $Q = 1000$ cfs; and $S = 0.05$,

$y = 3.0$ feet

Step 4. Compute the actual shear stress produced.

Using the Safety Factors Method or the simple equation, $t = WyS$, which closely approximates the Safety Factors Equation for computing shear stress,

$$t = (62.4)\ (3.0)\ (0.05) = 9.36\ \text{lb/ft}^2$$

Step 5. Compute the required rock size.

Using an equation of the U. S. Army Corps of Engineers (USCOE, 1970, see Ref. D5),

DRAFT

$t = a(W_s - W_w)(D_{50})$ where:

$a = 0.04$
W_s = unit weight of rock, in lb/ft^3, and
W_w = unit weight of water = 62.4 lb/ft^3

Based on an assumed stone weight of 165 pounds per cubic foot,

$t = 4.1\ D_{50}$.

The required size is calculated to be:

$D_{50} = t / 4.1$

$D_{50} = 9.36 / 4.1 = 2.3$ feet

Since the required rock size (2.3 feet) is greater than the rock size assumed (2.0 feet), another iteration with a larger D_{50} will be necessary.

3.5 Limitations

The procedures just discussed may require several iterations before an agreement can be reached between the assumed and computed rock size. In some cases where the slope is very steep and discharges are very large, a balance may never be able to be reached, indicating that the slope or discharge is so great that riprap protection cannot be feasibly provided. For very steep slopes, use of the Stephenson Method, discussed previously, may be considered in sizing riprap.

4. RIPRAP DESIGN FOR APRONS AND DIVERSION CHANNEL OUTLETS

GRANT, SCHREIBER & ASSOCIATES
CALCULATION SUMMARY SHEET

PROJECT NO: 809307

TASK NO: 1

CALCULATION NUMBER: I-5

DATE: 7-17-90

DESCRIPTION OF CALCULATION:

GENERATION OF CROSSECTIONAL PROFILE PLOTS OF W. CHICAGO SITE DRAINAGE DITCH AT 4 LOCATIONS VIA MEASUREMENT, CONVERSION SPREADSHEET ENTRY, AND GRAPH GENERATION

REFERENCES USED:

1) (USNRC) KERR-MCGEE CHEM. CORP. "WEST CHICAGO PROJECT", ENGINEERING REPORT, FEBRUARY 1986, BINDER 9 OF 20, SKETCHES SECTION, WASTE STABILIZATION SITE, CIVIL COMPLETED CELL PLAN (ATTACHMENT I)

CALCULATION MADE BY: [signature] DATE: 7-17-90

CALCULATION CHECKED BY: Kevin S. [signature] DATE: 7-18-90

CALCULATION APPROVED BY: [signature] DATE: 7-18-90

1 ATTACHMENT ? PAGES

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BY JHS DATE 7-17-90
CHECKED BY KSR DATE 7-18-90

OBJECTIVE: OBTAIN CROSSECTIONAL PROFILES FOR THE SITE DRAINAGE DITCH AT 4 DIFFERENT LOCATIONS.

METHOD: DRAW CROSSECTION LINES ⊥ TO DRAINAGE DITCH ON N, S, E, AND W SIDES OF SITE.

EXTEND THESE LINES UNTIL AN EQUAL ELEVATION IS PRESENT AT EACH ENDPOINT (ie: ONE SIDE OR THE OTHER IS A HIGH POINT ON THE CONTOUR MAP)

MARK POINTS AT KNOWN ELEVATIONS, ENTER DATA INTO SPREADSHEET PROGRAM, AND PRODUCE GRAPHS REPRESENTING CROSSECTION PROFILES

SOLUTION:

EAST SIDE DATA:

POINT	X1 (cm)	X2 (ft)	X2/X1	ELEVATION (ft)
1	0	0	—	750
2	.3	15	50	747
2A	.5	26.5	53	749
3	1.5	80	53.33	746
4	1.95	105	53.85	747
5	2.8	152	54.29	748
6	2.9	157	54.14	749
7	3.2	168	52.5	750

AVG = 53.02

CONVERSION FACTOR OF 1 cm (MEASURED) = 53 ft (ACTUAL) WILL BE USED IN THE FOLLOWING DATA SETS FOR S, N, AND W SIDE DITCHES.

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SOUTH SIDE DATA:

POINT	X1 (cm)	X2 (ft) (= 53X1)	ELEVATION (ft)
1	0	0	745
2	.95	51	737
3	1.15	62	739
4	1.42	75	745

NORTH SIDE DATA:

POINT	X1 (cm)	X2 (ft) (53X1)	ELEVATION (ft)
1	0	0	750
2	.5	26.5	745
2A	.7	37.1	742
3	1.5	79.5	741
4	1.7	90.1	740
5	1.85	98.1	739.9
6	1.95	103	740
7	2.1	111	741
8	2.85	151	742
9	3.6	191	743
10	3.95	209	744
11	4.45	236	745
12	6.65	352	747
13	6.75	358	748
14	6.8	360	749
15	7.1	376	750

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WEST SIDE DATA:

NOTE: THIS DATA WAS OBTAINED BY MEASUREMENT VIA AN ENGINEER'S SCALE (40 SCALE) AND THEN CONVERTED TO ACTUAL FT. USING THE CONVERSION FACTOR:

1" = 34.09'

POINT	X1 (40 SCALE)	X2 (ft) (= 34.09 X1)	ELEVATION (ft)
1	0	0	750 ✓
2	.75	25.6 ✓	745 ✓
3	1.5	51.1 ✓	740 ✓
4	2.35	80.1 ✓	734 ✓
5	2.5	85.2 ✓	734 ✓
6	3	102.3 ✓	737 ✓
7	3.35	114.2 ✓	735 ✓
8	3.6	122.7 ✓	735 ✓
9	4	136.4 ✓	740 ✓
10	4.3	146.6 ✓	745 ✓

EDGE POINTS OF DITCH (points 2 and 10)

THE 4 PROFILES (E, S, N, AND W SIDES) ARE ATTACHED.

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JOB NO. 809307 SHEET 5 OF 10
JOB NAME W CHICAGO CONSULTING
BY JHS DATE 7-17-90
CHECKED BY KSR DATE 7-18-90

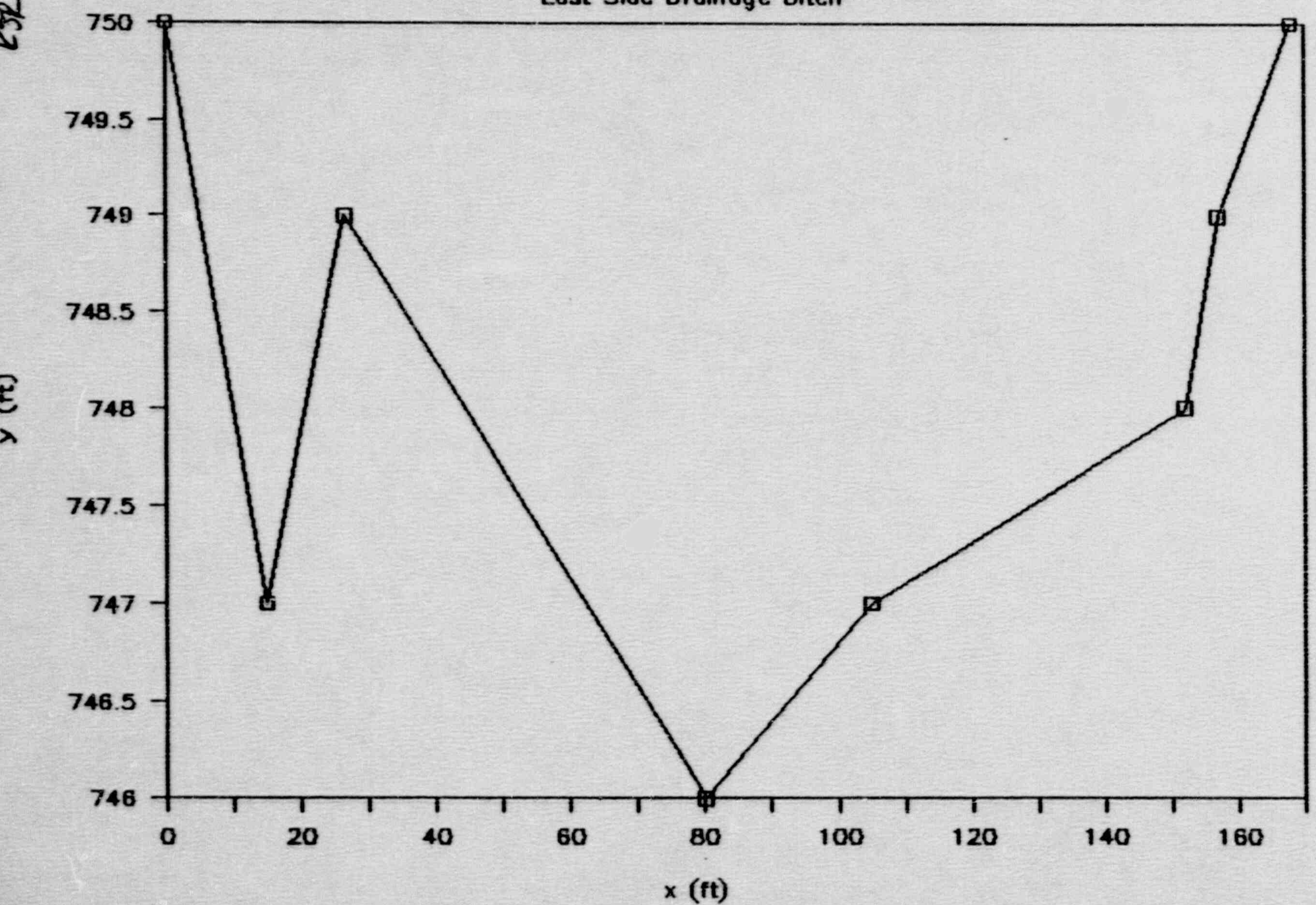
SE SIDE DATA :

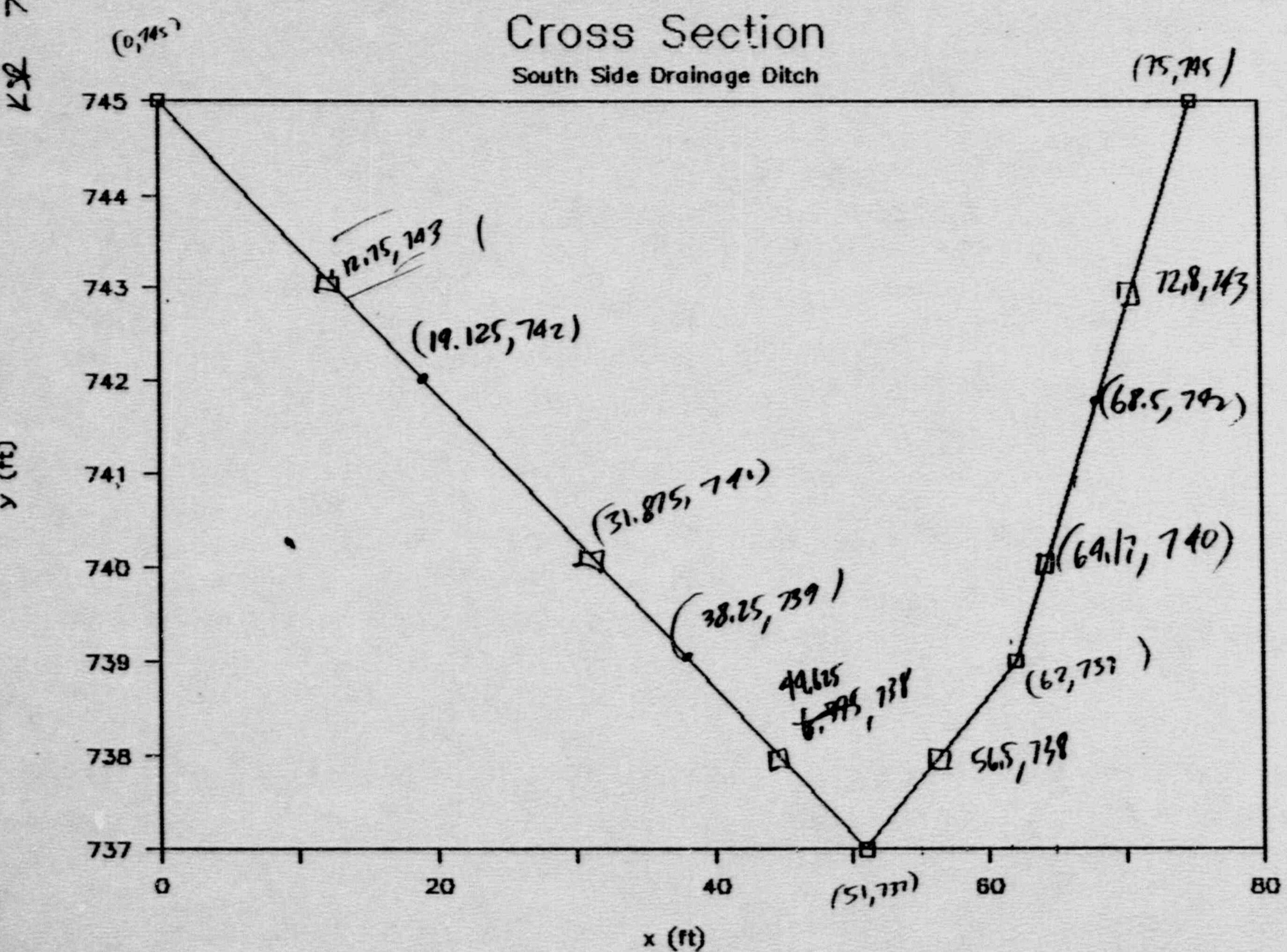
POINT	X1 (in)	X2 (ft) = 53X1	ELEVATION (ft)
1	0	0	745
2	.4	21.2	741
3	.6	31.8	743
4	.75	39.8	740
5	1.05	55.7	740
6	1.25	66.3	740
7	1.4	74.2	740
8	1.65	87.5	740
9	1.95	103.4	745

THE SE PROFILE, GENERATED FROM THESE DATA IS ATTACHED

Cross Section

East Side Drainage Ditch





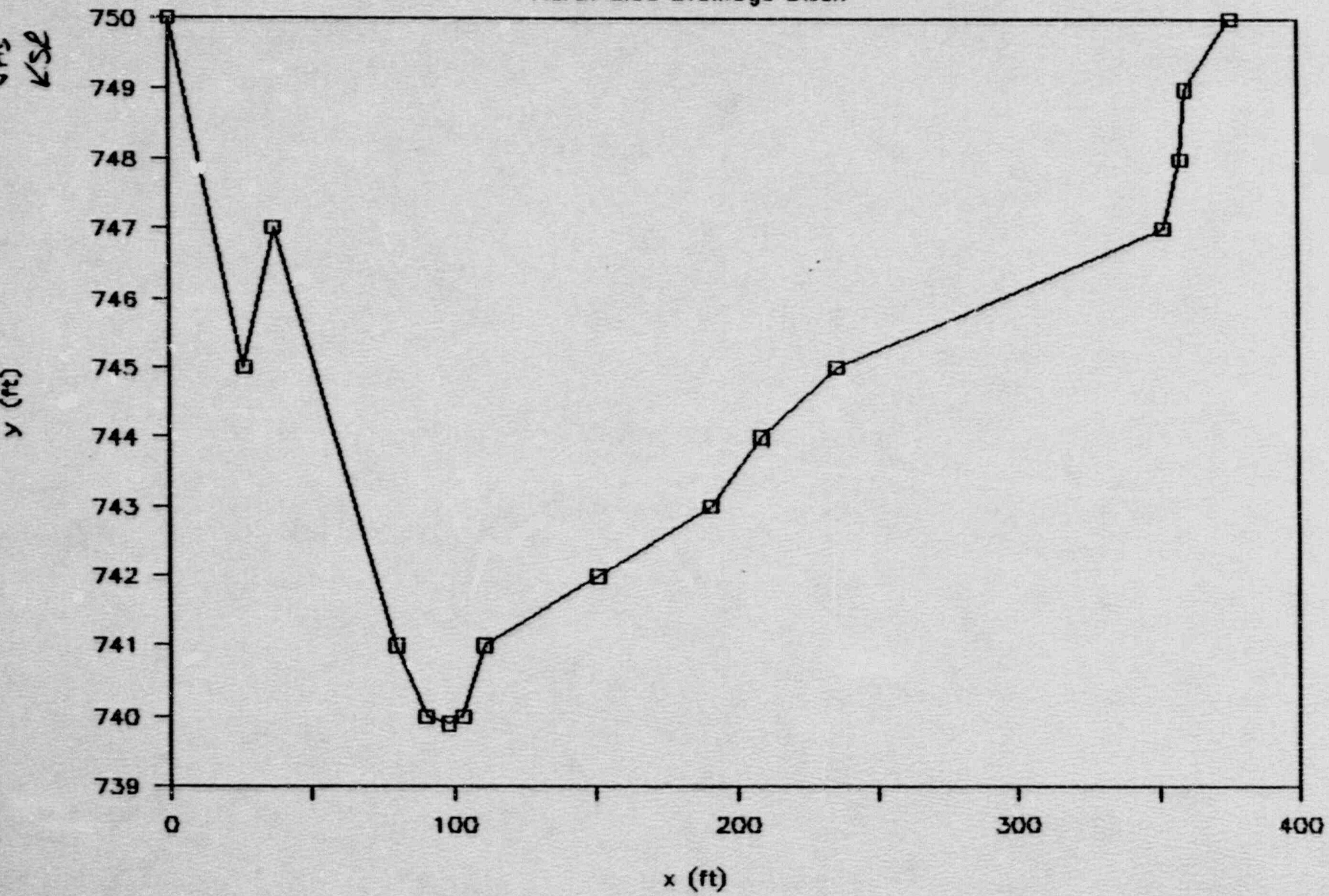
8C1507 7of
JHS 7-18-9?
KS? 7-18-9?

W. CHICAGO CONSULTING
JHS 7-17-9?

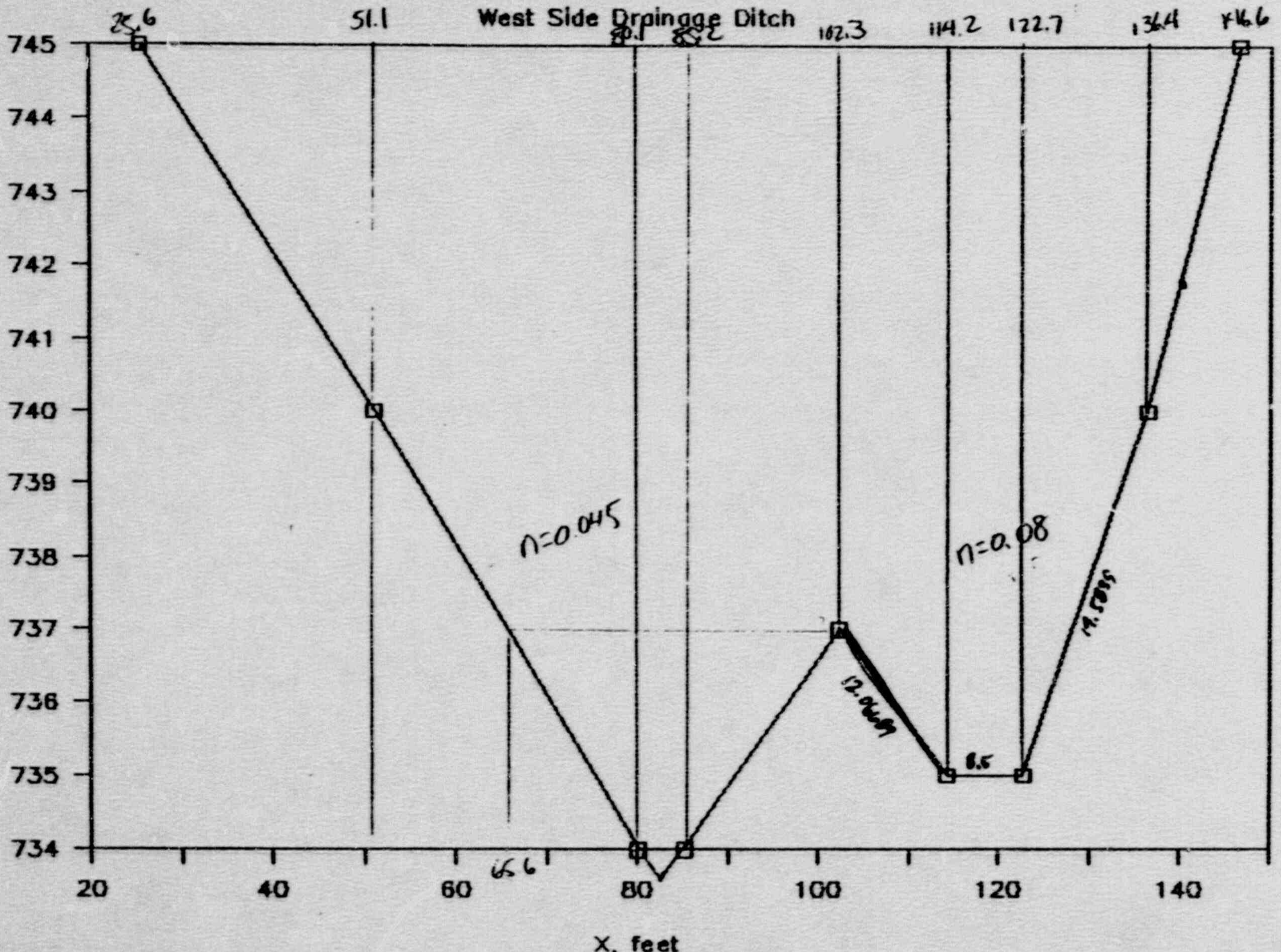
004207 8 of 10
JHS 7-18-90
KSL 7-18-90

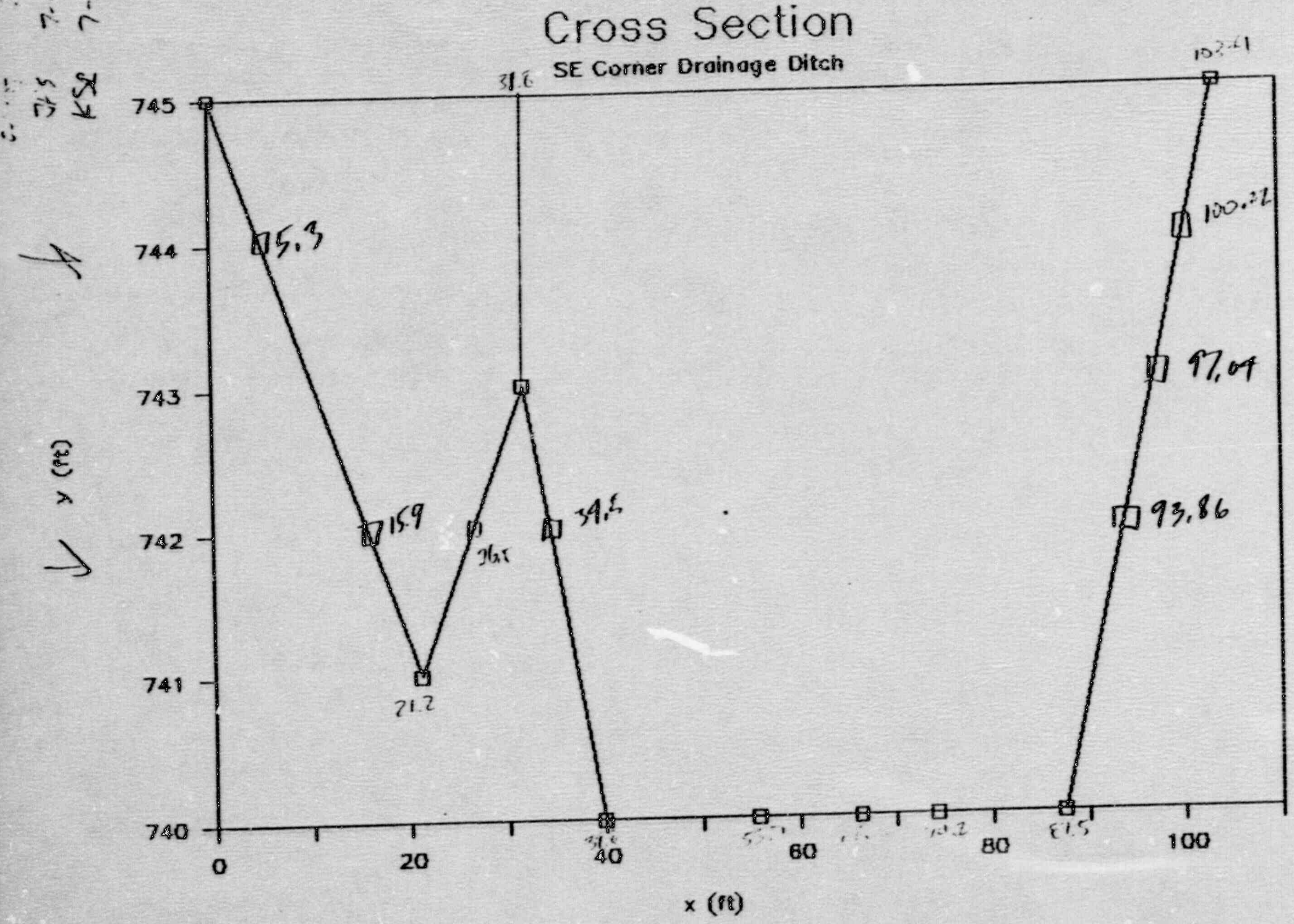
Cross Section

North Side Drainage Ditch



Cross Section





Cross Section
SE Corner Drainage Ditch
745
744
743
742
741
740
y (ft)
0
20
40
60
80
100
x (ft)

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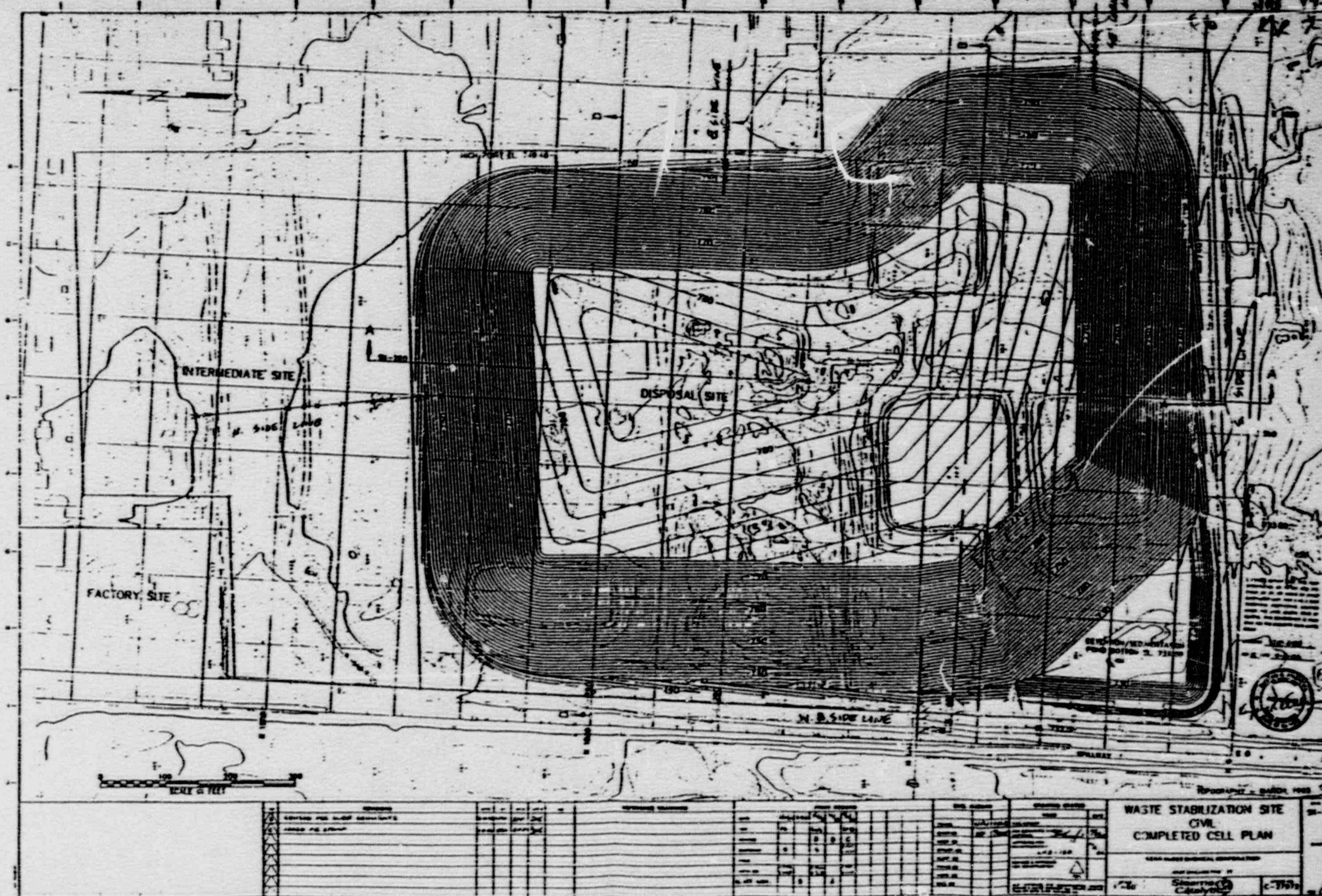
JOB NO. 809307 SHEET 1 OF 2
JOB NAME W. CHICAGO CONSULTING
BY JHS DATE 7-12-90
CHECKED BY KSR DATE 7-18-90

ATTACHMENT I

WASTE STABILIZATION SITE
CIVIL
COMPLETED CELL PLAN


INTERMEDIATE SITE
DISPOSAL SITE
FACTORY SITE
N. B. SIDE LINE
SPILLWAY
SCALE IN FEET
WASTE STABILIZATION SITE
CIVIL
COMPLETED CELL PLAN

GRANT, SCHREIBER & ASSOCIATES

CALCULATION SUMMARY SHEET

PROJECT NO: [redacted] 809307

TASK NO: 01

CALCULATION NUMBER: I-6

DATE: 7-18-90

DESCRIPTION OF CALCULATION:

Determine drainage areas for north/west and east/south diversion ditches.

REFERENCES USED:

1. Kerr-McGee Chemical Corp, "West Chicago Project Engineering Report", Volume IX, Drawing SK-258 (site map) (attached)
2. USGS West Chicago Quadrangle map (part attached)

CALCULATION MADE BY: Kevin S. Rauch DATE: 7-18-90

...LATION CHECKED BY: John H. Schreiber DATE: 7-18-90

...LATION APPROVED BY: David J. Schreiber DATE: 7-18-90

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OBJECTIVE: Determine drainage areas for east side and west side drainage ditches.

SOLUTION:

The site was divided into three areas as shown on the attached site map.

The area of the east side was determined by planimetering the area between the property line and the drainage divide (as shown on the attached map) plus the area east of the property line, shown on the USGS West Chicago Quadrangle (attached) (note, this area was approximated by an equivalent rectangle).

PLANIMETER:

3 readings 2727
2737
2744

Avg 2736 ✓, or 27.36 in² ✓

Map Scale: 2.2 in = 300 ft or 1 in = 136.36 ft ✓

$$Area_1 = 27.36 in^2 \left(\frac{136.36 ft}{1 in}\right)^2 = 508,733 ft^2 = 11.7 ac ✓$$

$$Area_2 = (0.125 in)(0.375 in) \times \frac{(2000)^2 ft^2}{in^2} = 187,500 ft^2 = 4.3 ac ✓$$

East area = 11.7 + 4.3 = **16 acres** ✓

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JOB NAME WEST CHICAGO
BY KSR DATE 7-8-90
CHECKED BY JHS DATE 7-11-90

The area in the southwest corner (including the pond) was determined by planimeter

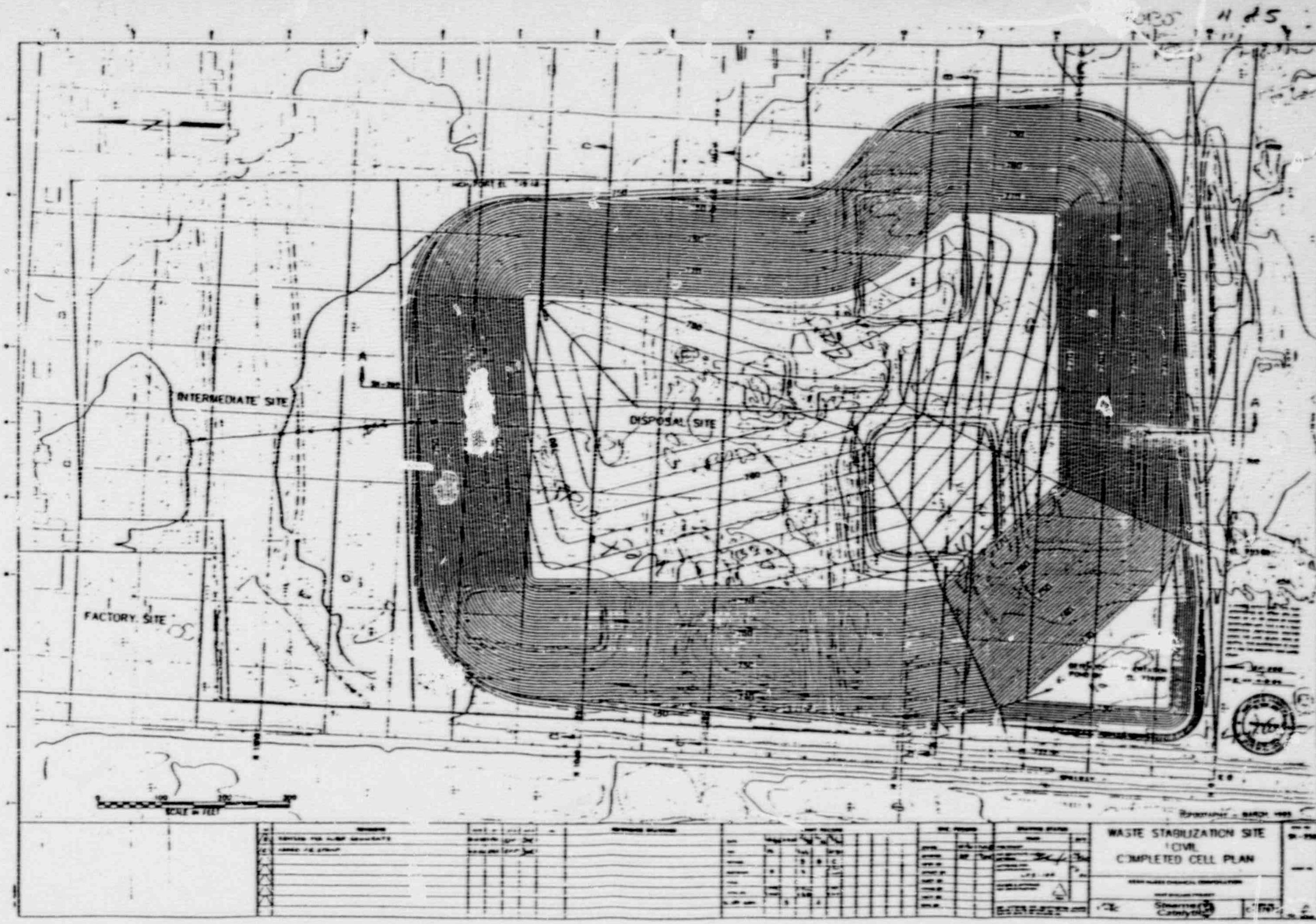
3 readings 997
985
997
Avg 993 ✓ or 9.93 in²

Map scale: 1" = 136.36'

S.W. Area = 9.93 in² $\frac{(136.36\text{ ft})^2}{\text{in}^2}$ = 184,644 ft² = 4 acres ✓

The west side area was determined by subtracting the east and S.W. corner areas from the total basin area of 307 acres (from Calc No. I-1).

West Area = 307 − 16 − 4 = **287 acres** ✓


INTERMEDIATE SITE
DISPOSAL SITE
FACTORY SITE
SCALE IN FEET
WASTE STABILIZATION SITE
CIVIL
COMPLETED CELL PLAN

Ingalton
Prince Crossing
Wheaton College Academy
Glen Oak Cem
Spring Lakes
Du Page
Branch
GENEVA
WASHINGTON
MAIN
3 TRACKS
Calvary Cem
Radio Towers
WEST CHICAGO
Jr High Sch
Gary Sch
Water Tank
Sewage Disposal
WEST DU PAGE PARK
DU PAGE
RIVER
Creek
EASTERN
BRANCH
ROAD
W I N F I E L D
West Chicago Quadrangle

GRANT, SCHREIBER & ASSOCIATES
CALCULATION SUMMARY SHEET

PROJECT NO: B09307

TASK NO: 01

CALCULATION NUMBER: I-7

DATE: 7-18-90

DESCRIPTION OF CALCULATION:

GENERATION OF VELOCITY VS. DISCHARGE AND DEPTH VS. DISCHARGE CURVES FOR 3 CROSSSECTIONS OF DRAINAGE DITCH AT WEST CHICAGO SITE.

REFERENCES USED:

B09307 TASK 1 CALCULATION I-5

CALCULATION MADE BY: [signature] DATE: 7-18-90

CALCULATION CHECKED BY: Kevin S. Powel DATE: 7-24-90

CALCULATION APPROVED BY: [signature] DATE: 7-18-90

Page 1 of 9

4 ATTACHMENTS

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JOB NO. 80307 SHEET 2 OF 9
JOB NAME W.C. CONSULTING
BY JHS DATE 7-18-90
CHECKED BY KSR DATE 7-18-90

OBJECTIVE : GENERATE FIGURES DEPICTING CHANNEL VELOCITY VS. TOTAL DISCHARGE AND NORMAL CHANNEL DEPTH VS. TOTAL DISCHARGE FOR 3 CROSSECTIONS AT W. CHICAGO SITE : SE CORNER, SOUTH SIDE, AND WEST SIDE. (THESE LOCATIONS ARE INDICATED ON PAGE 6).

METHOD : USING DATA FROM CALCULATION I-5, A SPREADSHEET WAS DEVELOPED TO CALCULATE THE FOLLOWING QUANTITIES :

1) CROSSECTIONAL PROFILE AREA FOR SEVERAL DITCH/WATER DEPTHS (FOR CHANNEL AND OVERBANK)
2) WETTED PERIMETER FOR CORRESPONDING DEPTHS
3) VELOCITY THROUGH THE DITCH AT CORRESPONDING DEPTHS
4) DISCHARGE THROUGH THE DITCH AT CORRESPONDING DEPTHS

GRAPHS COULD THEN BE GENERATED VIA LOTUS AND HARVARD GRAPHICS

SOLUTION :

1) AREA CALCULATION

COORDINATES WERE TAKEN DIRECTLY FROM THE CROSSECTION PROFILE PLOTS IN CALCULATION I-5. ADDITIONAL POINTS TO FILL OUT THE FINAL CURVES WERE LINEARLY INTERPOLATED FROM THESE PLOTS.

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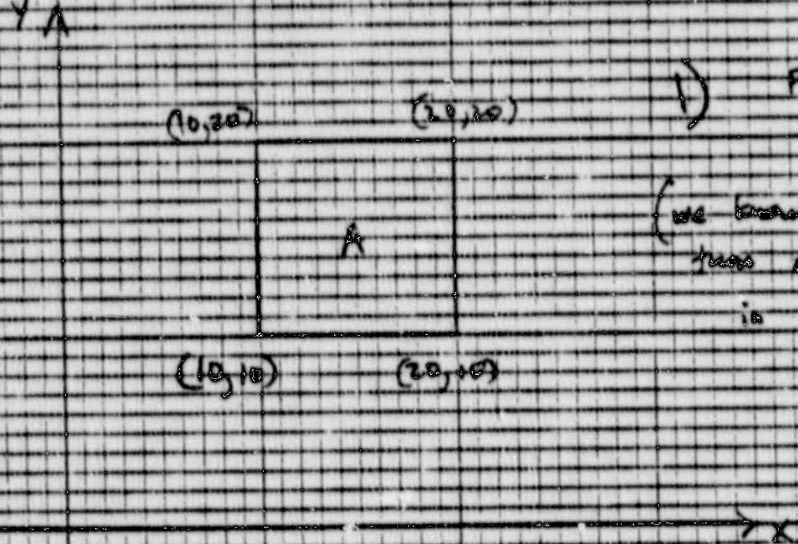
AS SHOWN IN THE FOLLOWING SPREADSHEET PAGES, THE AREAS FOR SEVERAL DEPTHS WERE TABULATED AND SUMMARIZED AT EACH CROSSECTION.

THE DOUBLE AREA COORDINATE METHOD WORKS LIKE THIS:



1) PICK COORDINATES OFF OF PLOT

(we know the area should turn out to be 100 sq. units in this case.)

2) TABULATE, REPEATING THE FIRST VALUE AT THE LAST

3) CROSS-MULTIPLY IN BOTH DIRECTIONS AND TABULATE.

X	Y
10	10
20	10
20	20
10	20
10	10

+ AREA	− AREA
100	200
400	200
400	200
100	200
1000	800

1000 − 800 = 200

$\frac{200}{2} = 100$ which checks out.

4) ADD AREA COLUMNS, SUBTRACT −AREA FROM +AREA, AND DIVIDE BY 2.

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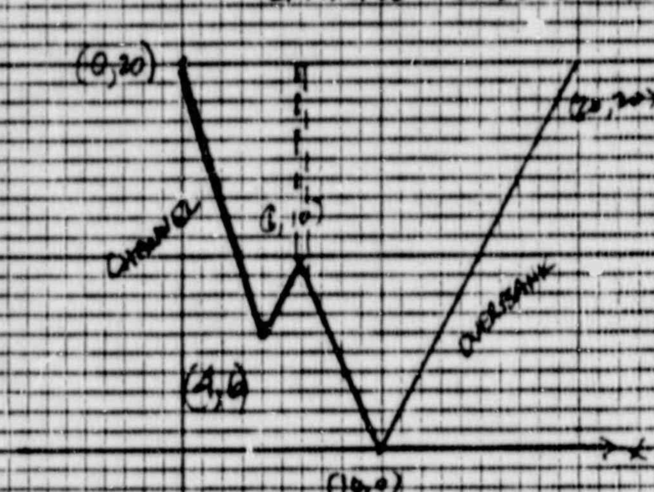
2) WETTED PERIMETER CALCULATION

THE PYTHAGOREAN THEOREM FROM GEOMETRY WAS USED TO CALCULATE WETTED PERIMETER FOR EACH CASE.

EXAMPLE CALCULATION:



DASHED LINES ARE NOT WETTED PERIMETER LINES.

CHANNEL:

$$P = \sqrt{(0-4)^2 + (20-6)^2} + \sqrt{(4-6)^2 + (6-10)^2}$$

OVERBANK

$$P = \sqrt{(6-10)^2 + (10-0)^2} + \sqrt{(10-20)^2 + (0-20)^2}$$

3) VELOCITY CAN NOW BE CALCULATED USING MANNING'S FORMULA.

$$V = \frac{1.49}{n} R^{2/3} S^{1/2}$$

WHERE R = HYDRAULIC RADIUS = $\frac{\text{AREA}}{\text{WETTED PERIMETER}} = \frac{A}{P}$

S = SLOPE = 1% ON-SITE *

AND n = .045 FOR CHANNEL ROCK, n = .03 FOR OVERBANK GRASS

4) DISCHARGE (Q) FOLLOWS FROM VELOCITY AND AREA

$$Q = VA$$

* SEE KMCC ENGINEERING REPT: VOL 9, PG 9-14

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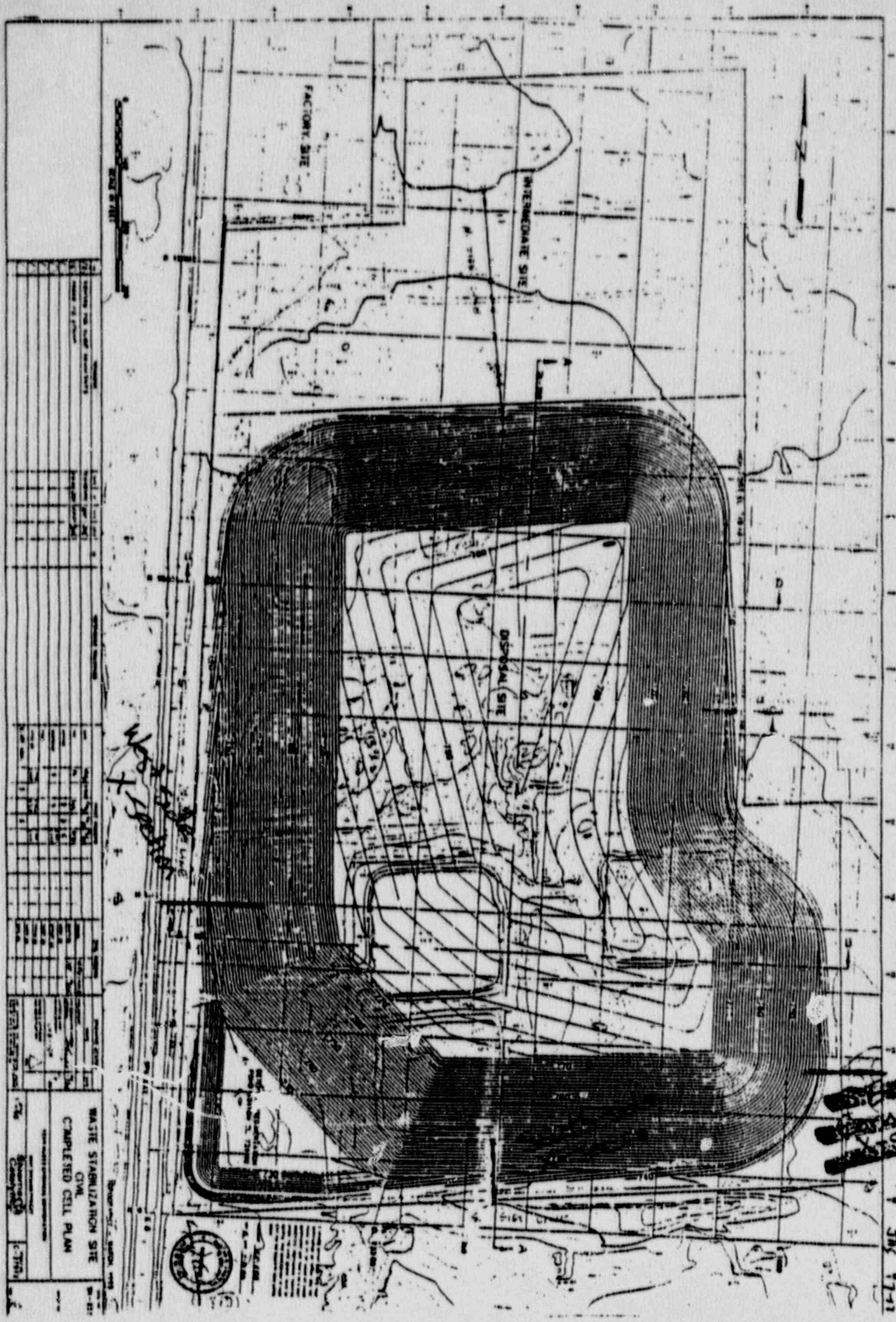
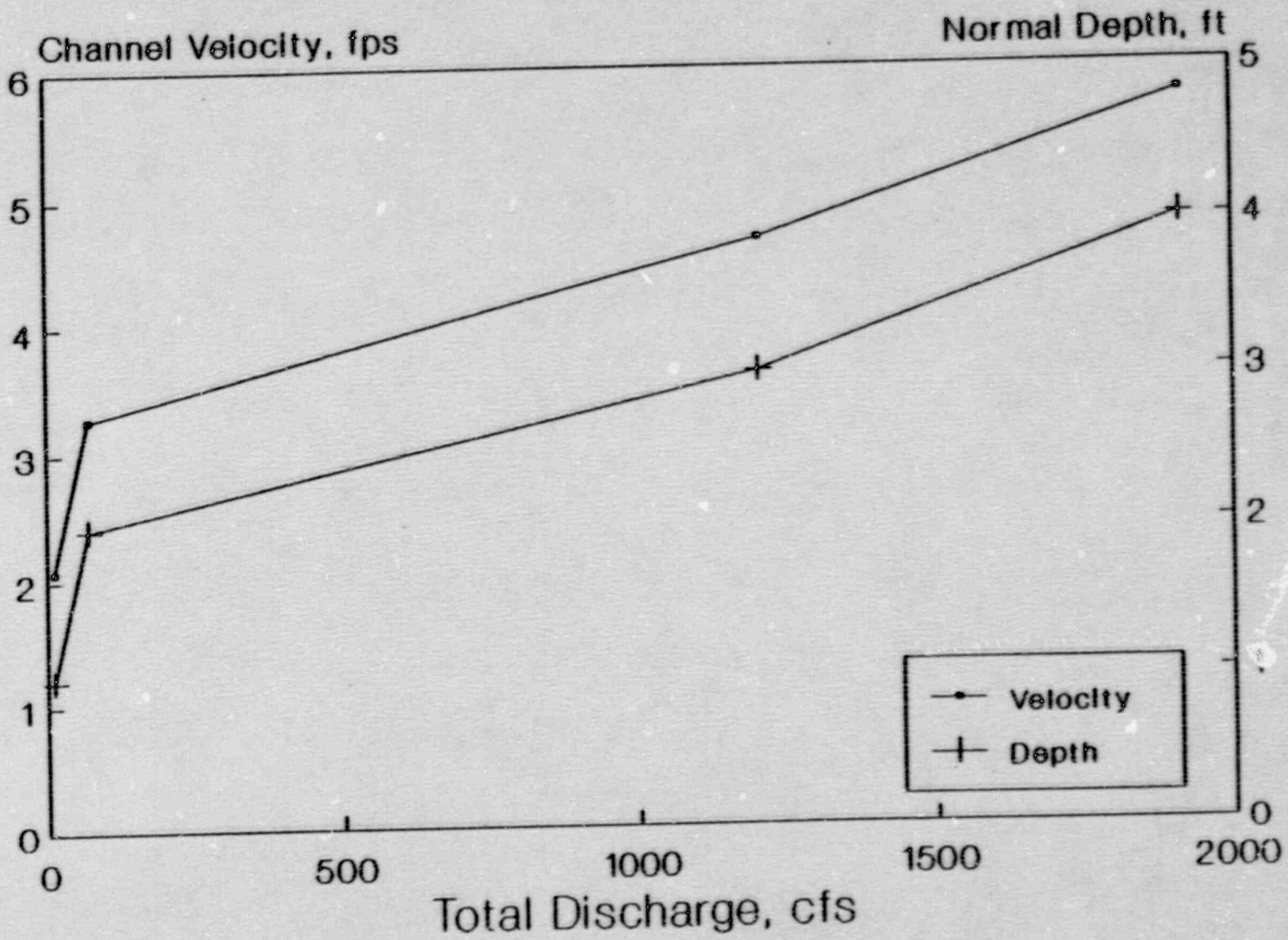
DATA AND RESULTS FOR EACH OF THE 3 CROSSECTIONS ARE ATTACHED.

NOTE THAT FOR THE SOUTH SIDE CROSSECTION, THERE IS NO DEFINED OVERBANK. IT IS CONSIDERED TO BE "ALL-CHANNEL"

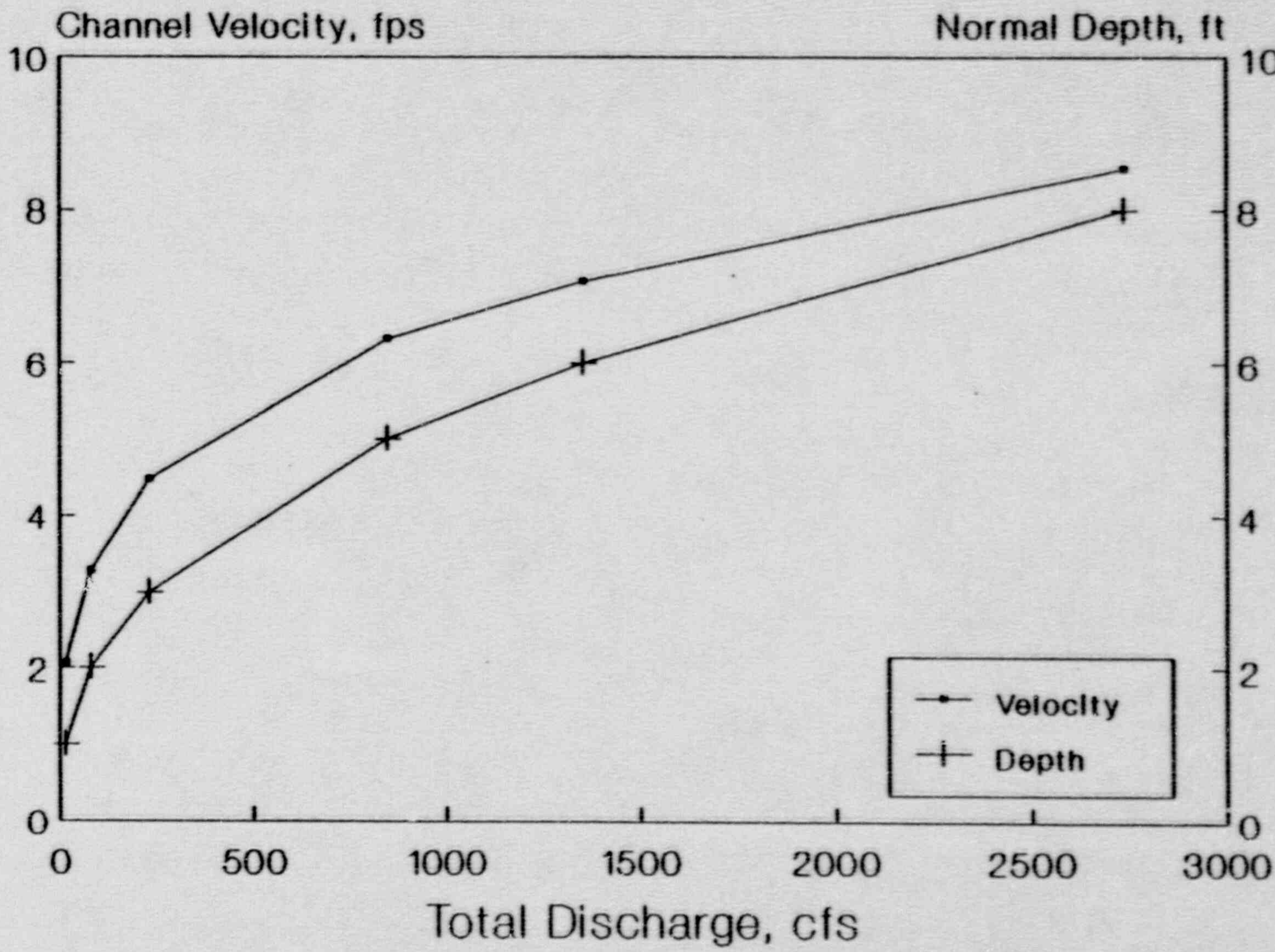
JHB 7-18-90
KSR 7-18-90



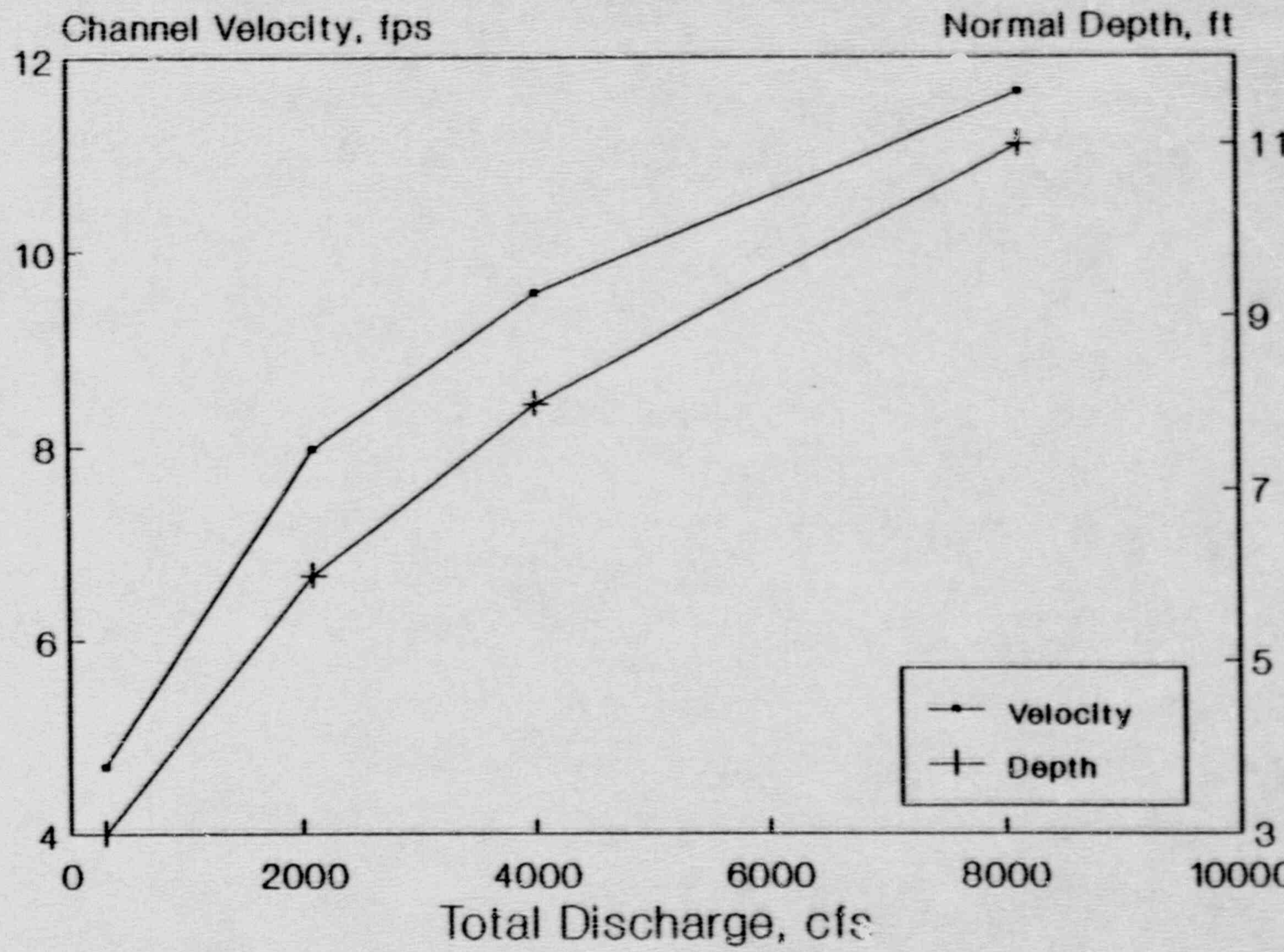
Southeast Corner Drainage Ditch



South Side Drainage Ditch



West Side Drainage Ditch



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JOB NO. 809307 SHEET 1 OF 4
JOB NAME W.C. CONSULTING
BY JHS DATE 7-18-90
CHECKED BY KSR DATE 7-24-90

ATTACHMENT I

SE CORNER DATA

Southeast Corner Cross Section

X	Y	
0	745	Channel
21.2	741	
31.8	743	---------
39.8	740	
55.7	740	Overbank
66.3	740	
74.2	740	
87.5	740	
103.4	745	

Determine area by double area coordinate method.

Water Surface Elevation [redacted]

Channel

X	Y	+ Area	_ Area	Wetted Perimeter
0	745	0	15794	21.57405
21.2	741	15751.6	23563.8	10.78702
31.8	743	23691	23627.4	
31.8	745	23691	0	
0	745			
		63133.6	62985.2	[redacted]

[redacted]

Overbank

X	Y	+ Area	_ Area	Wetted Perimeter
31.8	745	23627.4	23691	
31.8	743	23532	29571.4	8.544003
39.8	740	29452	41218	15.9
55.7	740	41218	49062	10.6
66.3	740	49062	54908	7.9
74.2	740	54908	64750	13.3
87.5	740	65187.5	76516	16.66763
103.4	745	77033	23691	
31.8	745			
		364019.9	363407.4	[redacted]

[redacted]

Water Surface Elevation [redacted]

Channel

+	_	Wetted

Overbank

+	_	Wetted

DCM 701 Attachment I

2 of 9

JHS - 7-18-90

KSR 7-24-91

X	Y	Area	Area	Perimeter
5.3	744	3927.3	15772.8	16.18054
21.2	741	15751.6	23563.8	10.78702
31.8	743	23659.2	23627.6	
31.8	744	23659.2	3943.2	
5.3	744			
		66997.3	66907.2	[illegible]

[illegible]

X	Y	Area	Area	Perimeter
31.8	744	23627.4	23659.2	
31.8	743	23532	29571.4	8.544003
39.8	740	29452	41218	15.9
55.7	740	41218	49062	10.6
66.3	740	49062	54908	7.9
74.2	740	54908	64750	13.3
87.5	740	65100	74148	13.31502
100.2	744	74548.8	23659.2	
31.8	744			
		361448.2	360975.8	[illegible]

Area [illegible] 236.2

Water Surface Elevation [illegible] 743 cfs

-------------------Channel -------------------

X	Y	+ Area	_ Area	Wetted Perimeter
10.6	743	7854.6	15751.6	10.78702
21.2	741	15751.6	23563.8	10.78702
31.8	743	23627.4	7875.8	
10.6	743	0	0	
		47233.6	47191.2	[illegible]

[illegible]

[illegible]

Water Surface Elevation [illegible] 742 ft

-------------------Channel -------------------

No Overbank flow at [illegible]

ATTACHMENT I

JHS 9-19-90

KSK 9-24-90

X	Y	+ Area	- Area	Wetted Perimeter
15.9	742	11781.9	15730.6	5.393514
21.2	741	15730.6	19636.5	5.393514
26.5	742	19663	11797.8	
15.9	742	0	0	
		47175.3	47164.7	1[redacted]

[redacted]

[redacted]

[redacted]

Slope = 0.01
n = 0.065

Slope = 0.01
n = 0.08

Channel: Elevation (ft)	Depth (ft)	Area (sq. ft)	Perim. (ft)	Velocity (ft/s)	Discharge (cfs)	Overbank: Area (sq. ft)	Perim. (ft)	Velocity (ft/s)	Discharge (cfs)	Total Discharge (cfs)
745	[redacted]	74.20	32.36	[redacted]	427	306.25	72.91	6.85	1685	[redacted]
744	[redacted]	45.05	26.97	[redacted]	210	236.20	69.56	6.21	994	[redacted]
743	[redacted]	21.20	21.57	[redacted]	69	0.00	0.00	0.00	0	[redacted]
742	[redacted]	5.30	10.79	[redacted]	11	0.00	0.00	0.00	0	[redacted]

ATTACHMENT I JHG 7-19-90
KSP 7-26-90

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JOB NO. 801307 SHEET 1 OF 5
JOB NAME W.C. CONSULTING
BY JHS DATE 7-18-90
CHECKED BY KSR DATE 7-24-90

ATTACHMENT II
SOUTH SIDE DATA

South Side Cross Section

X	Y
0	745
51	737
62	739
75	745

*This section considered to be all channel

Determine area by double area coordinate method.

Water Surface Elevation [redacted]

----------Channel----------				
X	Y	+ Area	_ Area	[redacted]
0	745	0	37995	51.62363
51	737	37689	45694	11.18033
62	739	46190	55625	14.31782
75	745	55875	0	
0	745	0	0	
		139754	139116	[redacted]

[redacted]

Water Surface Elevation [redacted]

----------Channel----------				
X	Y	+ Area	_ Area	[redacted]

2 of 5

7-18-90 JHS

7-24-90 KSR

X	Y	+ Area	− Area	Wetted Perimeter
38	739	28006	37689	13.15294
51	737	37689	45694	11.18033
62	739	45818	28082	
38	739	0	0	
		111513	111465	[illegible]

[redacted]

Water Surface Elevation [illegible]

X	Y	+ Area	− Area	[illegible]
19	742	14003	37842	[illegible]

X	Y	+ Area	− Area	Wetted Perimeter
51	737	37689	45694	11.18033
62	739	46004	50621.5	7.158910
68.5	742	50827	14098	
19	742	0	0	
		148523	148255.5	[illegible]

[redacted] 133.75

Water Surface Elevation [illegible] ft

------------------Channel ------------------

X	Y	+ Area	− Area	Wetted Perimeter
12.8	743	9433.6	37893	38.66833
51	737	37689	45694	11.18033
62	739	46066	53799.2	11.51694

72.8	743	54090.4	9510.4	
12.8	743	0	0	
		147279	146896.6	[redacted]

[redacted]

[redacted]

Channel				
		+	_	[redacted]
X	Y	Area	Area	[redacted]
31.9	740	23510.3	37740	19.33416
51	737	37689	45694	11.18033
62	739	45880	47443.8	2.416609
64.2	740	47508	23606	
31.9	740	0	0	
		154587.3	154483.8	[redacted]

[redacted]

Water Surface Elevation [redacted]

Channel				
		+	_	[redacted]
X	Y	Area	Area	[redacted]
44.6	738	32870.2	37638	6.477653
51	737	37638	41640.5	5.590169
56.5	738	41697	32914.8	
44.6	738	0	0	
		112205.2	112193.3	[redacted]

BMZ07 4 of 5
JHS 7-18-90
KSR 7-24-90

[redacted] i

South Side Drainage Ditch

Slope = 0.0[illegible]

n = 0.0[illegible]5

	---------	Channel	-----------------		
Elevation (ft)	Depth (ft)	Area (sq. ft)	Perim. (ft)	[redacted]	[redacted]
745	[redacted]	320.00	77.12	[redacted]	[redacted]
743	[redacted]	191.2	61.37	[redacted]	[redacted]
742	[redacted]	133.75	50.73	[redacted]	[redacted]
740	[redacted]	51.75	32.93	[redacted]	[redacted]
739	[redacted]	24.00	24.33	[redacted]	[redacted]
738	[redacted]	5.95	12.07	[redacted]	[redacted]

809307 5 of 5

JHB 7-18-90

KSR 7-24-90

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JOB NO. 809307 SHEET 1 OF 4
JOB NAME WC Consulting
BY JHS DATE 7-18-90
CHECKED BY KSR DATE 7-24-90

ATTACHMENT III

WEST SIDE DATA

809307 2 of 4
JHS 7-18-90
KSR 7-24-90

West Side Cross Section

X	Y	
0	750	
25.6	745	
51.1	740	
80.1	734	Channel
85.2	734	
102.3	737	---------
114.2	735	
122.7	735	Overbank
136.4	740	
146.6	745	

Determine area by double area coordinate method.

Water Surface Elevation = 745 ft

Channel X	Channel Y	Channel + Area	Channel − Area	Channel Wetted Perimeter	Overbank X	Overbank Y	Overbank + Area	Overbank − Area	Overbank Wetted Perimeter
25.6	745	18944	38069.5	25.98557	102.3	745	75395.1	76213.5	
51.1	740	37507.4	59274	29.61418	102.3	737	75190.5	84165.4	12.06689
80.1	734	5879[illegible]	62536.8	5.1	114.2	735	83937	90184.5	8.5
85.2	734	62792.4	75088.2	17.36116	122.7	735	90798	100254	14.58389
102.3	737	76213.5	75395.1		136.4	740	101618	108484	11.35957
102.3	745	76213.5	19072		146.6	745	109217	76213.5	
25.6	745				102.3	745			
		330464.2	329435.6	78.06092			536155.6	53551[illegible]	66.51036
		Area=	514.3				Area=	320.35	

Water Surface Elevation = 740 ft

Channel X	Channel Y	Channel + Area	Channel − Area	Channel Wetted Perimeter	Overbank X	Overbank Y	Overbank + Area	Overbank − Area	Overbank Wetted Perimeter
51.1	740	37507.4	59274	29.61418	102.3	740	75395.1	75702	
80.1	734	58793.4	62536.8	5.1	102.3	737	75190.5	84165.4	~~[illegible]~~
85.2	734	62792.4	75088.2	17.36116	114.2	735	83937	90184.5	12.06689
102.3	737	75702	75395.1		122.7	735	90798	100254	8.5
102.3	740	75702	37814		136.4	740	100936	75702	14.5839
51.1	740				102.3	740			
		310497.2	310108.1	52.07534			426256.6	426007.9	~~22.56689~~
		Area=	194.55				Area=	124.35	35.15078

Water Surface Elevation = 742 ft

Channel X	Channel Y	Channel + Area	Channel − Area	Channel Wetted Perimeter	Overbank X	Overbank Y	Overbank + Area	Overbank − Area	Overbank Wetted Perimeter

809307 3 of 4
JHS 7-18-90
KSR 7-24-90

40.9	742	30266	37916.2	10.39422	102.3	742	75395.1	75906.6	

81.1	740	37507.4	50274	29.61418	102.3	737	75190.5	84165.4	12.06689
80.1	734	58793.4	62536.8	5.1	114.2	735	83937	90184.5	8.5
85.2	734	62792.4	75088.2	17.36116	122.7	735	90798	100254	14.58389
102.3	737	75906.6	75395.1		136.4	740	101208.8	103970	4.561797
102.3	742	75906.6	30147.8		140.5	742	104251	75906.6	
40.9	742	============	============	============	102.3	742	============	============	============
		341172.4	340551.1	62.46957			530786.4	530387.1	39.71258
		Area=	307.15				Area=	196.65	

Water Surface Elevation = 737 ft

----Channel----					----Overbank----			
X	Y	+ Area	− Area	Wetted Perimeter	X	Y	+ Area	− Area
65.6	737	48150.4	59033.7	14.40709				
80.1	734	58793.4	62536.8	5.1				
85.2	734	62792.4	75088.2	17.36116				
102.3	737	75395.1	48347.2					
65.6	737	============	============	============				
		245131.3	245005.9	37.26825				
		Area=	62.7					

→ see next page for summary.

Slope = 0.01 | Slope = 0.01

n = 0.045 | n = 0.08

	----Channel----				----Overbank----				Total
Elevation (ft)	Area (sq. ft)	Perim. (ft)	Velocity (ft/s)	Discharge (cfs)	Area (sq. ft)	Perim. (ft)	Velocity (ft/s)	Discharge (cfs)	Discharge (cfs)
745	514.3	78.06092	11.63667	5984.743	320.35	46.51036	6.742375	2159.919	8144.662
742	307.15	62.46957	9.574035	2940.665	196.65	39.71258	5.410949	1064.063	4004.728
740	194.55	52.07534	7.972106	1550.973	124.35	~~23.56689~~ 35.15078	~~5.644929~~ 4.324177	~~701.9470~~ 537.7115	~~2252.920~~ 2088.685
737	62.7	37.26825	4.683739	293.6704	0	0			293.6704

$$V = \frac{1.49}{n} R^{2/3} S^{1/2}$$

$$R = \frac{A}{P}$$

$$Q = VA$$

West Side Drainage Ditch

	Channel		Overbank
Slope =	0.01	Slope =	0.01
n =	0.045	n =	0.06

	Channel					Overbank				Total
Elevation (ft)	Depth (ft)	Area (sq. ft)	Perim. (ft)	Velocity (ft/s)	Discharge (cfs)	Area (sq. ft)	Perim. (ft)	Velocity (ft/s)	Discharge (cfs)	Discharge (cfs)
745	11	514.30	78.06	11.64	5985	320.35	46.51	6.74	2160	8145
742	8	307.15	62.47	9.57	2941	196.65	39.71	5.41	1064	4005
740	6	194.55	52.08	7.97	1551	124.35	35.15	4.32	538	2089
737	3	62.70	37.27	4.68	294	0.00	0.00			294

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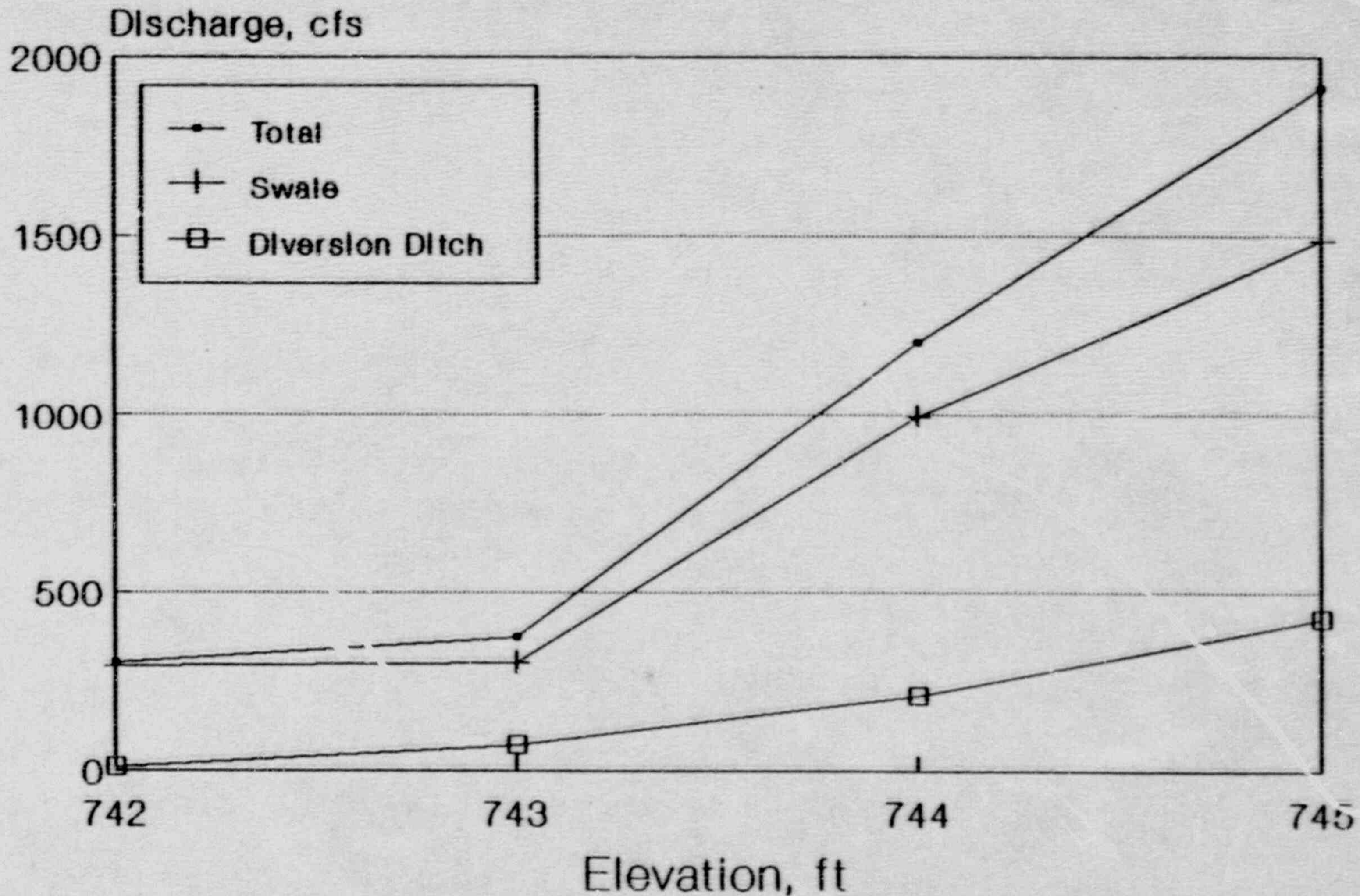
JOB NO. 809307 SHEET 1 OF 2
JOB NAME WL CONSULTING
BY JHB DATE 7-18-90
CHECKED BY KSR DATE 7-24-90

ATTACHMENT IV

RATING CURVE FOR SE CORNER
(DERIVED FROM SE CORNER DATA)

Southeast Corner Drainage Ditch
Rating Curve



GRANT, SCHREIBER & ASSOCIATES
CALCULATION SUMMARY SHEET.

PROJECT NO: 8-1385

TASK NO: 1

CALCULATION NUMBER: I-5

DATE: 7/25/90

DESCRIPTION OF CALCULATION:

JUSTIFICATION AND DOCUMENTATION OF NATURAL SITE SLOPE OF 1%.

REFERENCES USED:

1) KMCC W.C. PROJECT ENGINEERING RPT., VOL 9, PG 9-14

2) WASTE STABILIZATION SITE CIVIL COMPLETED CELL PLAN (WSSCCCP) MAP TAKEN FROM ABOVE REPORT

3) USGS W. CHICAGO QUAD MAP

CALCULATION MADE BY: John H. Schreiber DATE: 7/25/90

CALCULATION CHECKED BY: Kevin S. Couch DATE: 7-25-90

CALCULATION APPROVED BY: David H. Schreiber DATE: 7-25-90

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JOB NO. [illegible] SHEET ______ OF 5
JOB NAME [illegible]
BY [illegible] DATE [illegible]
CHECKED BY ESR DATE 7-25-90

OBJECTIVE: DOCUMENT THE 1% NOMINAL SLOPE VALUE USED IN PREVIOUS W. CHICAGO SITE CALCULATIONS

METHOD: 1) LAY OUT 3 LINES ⊥ TO CONTOURS ON EAST SIDE OF SITE (NEAR RAILROAD TRACK) ON THE WSSCCP MAP (ATTACHED)

2) LAY OUT A LINE ⊥ TO CONTOURS ACROSS THE SITE ON THE USGS W.C. QUAD MAP

3) MEASURE "RISE" AS DIFFERENCE IN CONTOUR ELEVATIONS

4) MEASURE "RUN" AS SCALED LINE LENGTH

5) $SLOPE = \frac{RISE}{RUN}$ FOR EACH OF THE 4 LINES

6) AVERAGE THE 4 VALUES

7) CHECK THIS AGAINST "1% NOMINAL GRADE" GIVEN ON PAGE 9-14 OF RMEC REPORT

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JOB NO. [illegible] SHEET 3 OF 5
JOB NAME [illegible]
BY [illegible] DATE [illegible]
CHECKED BY KSR DATE 7-25-90

SOLUTION:
FROM "WASTE STABILIZATION SITE CIVIL COMPLETED CELL PLAN" (ATTACHED) SCALE: 1" = 136'

LINE ①
RISE = 745' − 730' = 15'
RUN = 6.7" (136'/in) = 911.2'
SLOPE = 1.6 % ✓

LINE ②
RISE = 740' − 730' = 10'
RUN = 5.55' (136'/in) = 754.8'
SLOPE = 1.3% ✓

LINE ③
RISE = 740' − 735' = 5'
RUN = 3.95" (136'/in) = 537.2'
SLOPE = .93% ✓

FROM USGS QUAD MAP (ONSITE, SCALE = 1" = 2000')

LINE ④
RISE = 740' − 730' = 10'
RUN = .6" (2000'/in) = 1200'
SLOPE = .83% ✓

AVG SLOPE FROM ALL 4 LINES = 1.16% ✓

WHICH AGREES WELL WITH THE "NOMINAL GRADE" OF 1% GIVEN ON PAGE 9-14 OF KMCC REPORT

KSB 7-25-90

While 10 CFR 40 and 40 CFR 192 are not specific in defining a design flood event that would satisfy longevity requirements, (i.e. - 1000 years) the use of a Probable Maximum Flood event (PMF) is a generally accepted approach. The PMF is defined as the largest flood that can reasonably be expected to occur at a given area, based on the simultaneous occurrence of critical meteorological and hydrologic conditions. A reduction of the PMF is commonly utilized when failure of the structure would present a low hazard. In this case, the structures under consideration are the drainage swales and retention ponds. Failure of these structures would cause only loss of the structures with little additional damage to property and with no loss of life envisioned.

9.5.6.1 <u>Drainage Swale</u>

The peak discharge in cubic feet per second (Q) to be handled by the drainage swale is determined by the Rational Method:

$$Q = CiA,$$

Where C = runoff coefficient, expressing the ratio of rate of runoff to rate of rainfall = 0.95

i = intensity of rainfall, in inches per hour, for the selected frequency (PMF-B) and for duration equal to the time of concentration (6 min) = 10.7 inches per hour

A = drainage area, in acres, tributary to the point under design (west swale at point it enters detention/sedimentation pond) = 20 acres

Q = (.95)(10.7)(20) = 203 CFS

The drainage swale is designed as a V-ditch with 5 to 1 side slopes, on a nominal grade of 1% from a high point near the northeast corner of the disposal cell to the detention/sedimentation pond at the southwest corner of the Disposal Site. The velocity in feet per second (V) and the required cross-sectional area in feet^2 (A) are computed by Manning's Equation:

$$Q = \underline{1.486}\ AR^{2/3}\ S^{1/2},$$

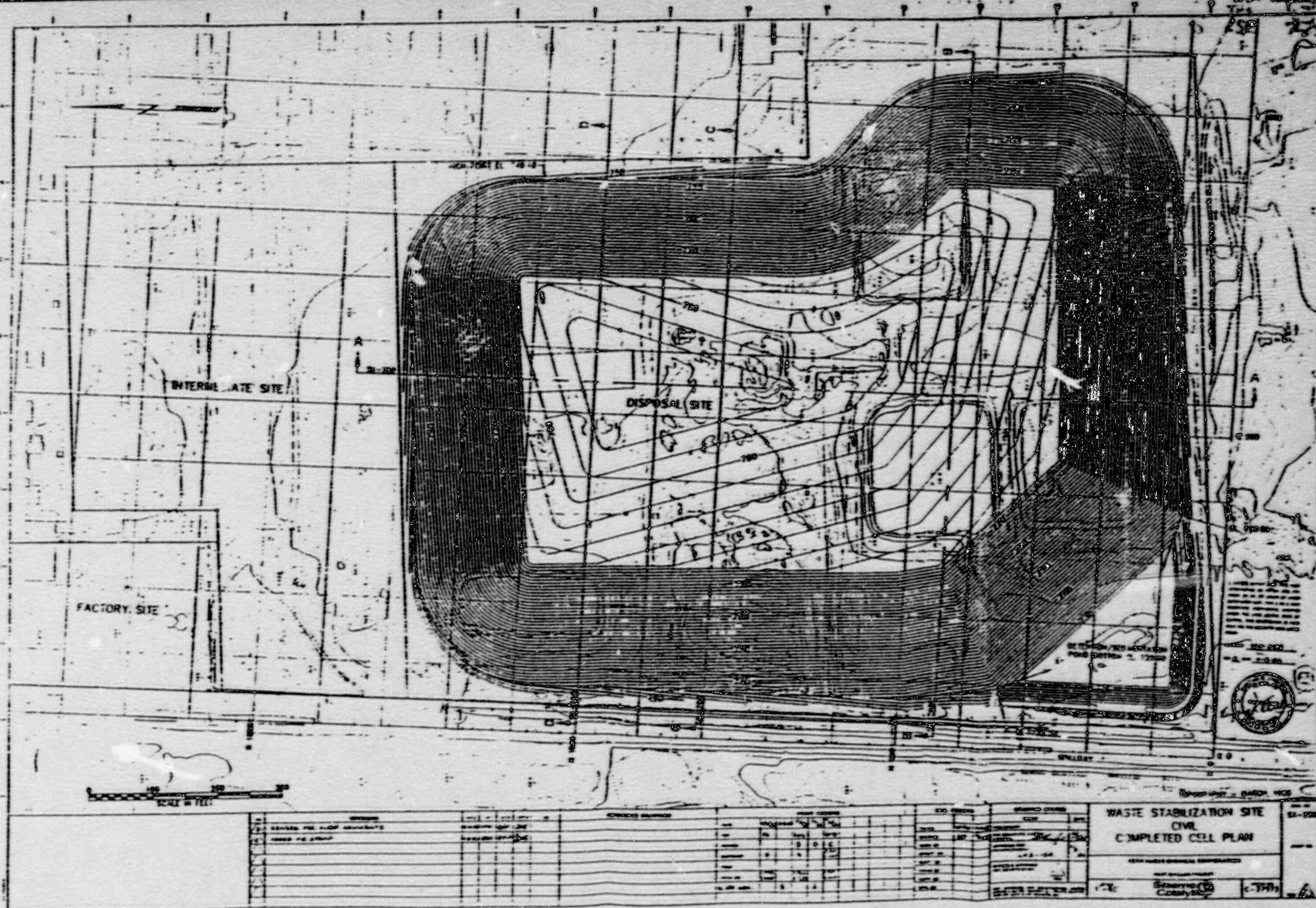
Where n = coefficient of roughness of channel = .025

R = hydraulic radius, in feet (area/wetted perimeter)

S = slope of channel, in percent

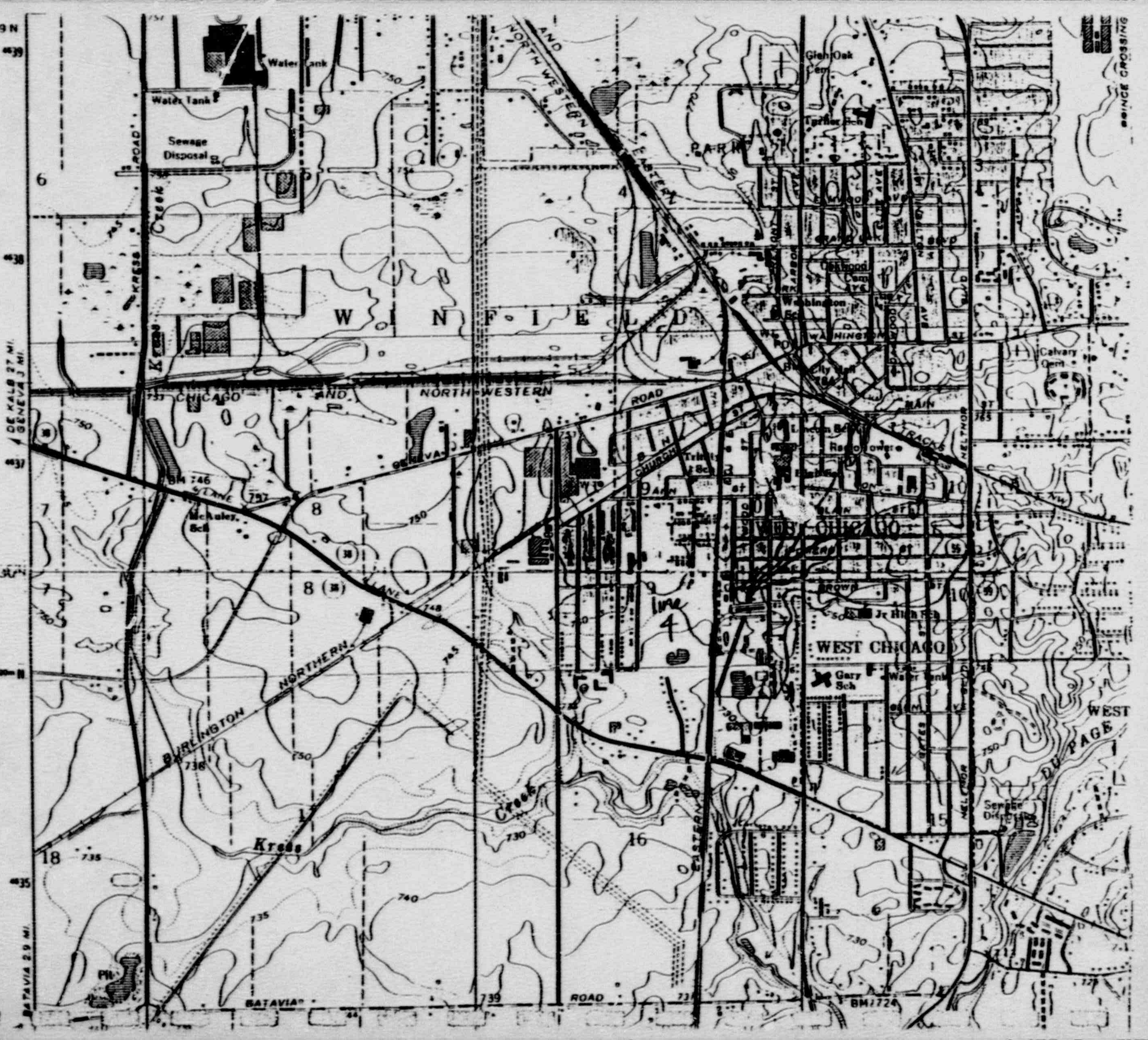
Q (cfs)	V (fps)	A (ft^2)	Depth of flow (ft)
203	6.8	31.3	2.5

Channel protection (riprap) is designed for the swale by Riprap Design with Safety Factors Method developed by Colorado State University for the Wyoming Highway Department:



INTERMEDIATE SITE
DISPOSAL SITE
FACTORY SITE
SPILLWAY
SCALE IN FEET
WASTE STABILIZATION SITE
CIVIL
COMPLETED CELL PLAN

USGS
WEST CHICAGO
QUADRANGLE



GRANT, SCHREIBER & ASSOCIATES
CALCULATION SUMMARY SHEET.

PROJECT NO: 809307

TASK NO: 1

CALCULATION NUMBER: I-9

DATE: 7/25/90

DESCRIPTION OF CALCULATION:

DETERMINE TIME OF CONCENTRATION OF RAINFALL TO CHANNEL + TIME IN CHANNEL TO POND

REFERENCES USED:

1) KIRPICH'S EQN FROM "METHODOLOGIES FOR EVALUATING LONG-TERM STABILIZATION DESIGNS OF URANIUM MILL TAILINGS IMPOUNDMENTS", NELSON, et al NUREG/CR-4620, ORNL/TM-10067 Pg 65, eqn (4-44)
2) CALCULATION I-2
3) CALCULATION I-3
4) CALCULATION I-6
5) WSSCCCP MAP

CALCULATION MADE BY: John H. Schreiber DATE: 7/25/90

CALCULATION CHECKED BY: Kevin S. Rauch DATE: 7-25-90

CALCULATION APPROVED BY: Edward H. Schreiber DATE: 7-25-90

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JOB NAME W.G. CONSULTING
BY JHS DATE 7/25/90
CHECKED BY KSR DATE 7-25-90

OBJECTIVE: ESTIMATE TIME OF CONCENTRATION t_c OF RAINFALL TO DRAINAGE CHANNEL AND ADD t_c TO TIME IN CHANNEL.

METHOD
1) USE KIRPICH'S FORMULA AND DATA FROM PREVIOUS CALCULATIONS TO ESTIMATE t_c
2) ESTIMATE TIME IN CHANNEL BY FINDING CHANNEL VELOCITY AT 6500 cfs DISCHARGE AND USING DISTANCE AROUND WEST SIDE CHANNEL
3) ADD RESULTS FROM 1) AND 2)

SOLUTION:

1) KIRPICH'S FORMULA:

$$t_c = .00013 \frac{L^{0.77}}{S^{0.385}}$$

where: L = length of basin in ft.
S = upgradient avg slope of basin
t_c = time of concentration in hrs

for this calculation,
L = 4800 ft (see pg 5 of calculation I-3)
S = .5 % (see justification in calculation I-2)

$$t_c = .00013 \frac{4800^{.77}}{.005^{.385}} = .683 \text{ hrs} = 40.97 \text{ min}$$

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JOB NO. 804307 SHEET 3 OF 5
JOB NAME W.G. CONSULTING
BY JWS DATE 7/25/90
CHECKED BY KSR DATE 7-25-90

2) ESTIMATE TIME IN CHANNEL

FROM WSSCCCP MAP (INCLUDED IN CALCULATION I-8) AND ATTACHED TO THIS CALC.

DISTANCE FROM (HIGH POINT) NE CORNER OF CHANNEL WESTWARD AND THEN SOUTHWARD TO SW CORNER IS

15.25"
15.10"
15.25"
15.10"

RECALL THAT $\frac{287}{307} = 93\%$ OF SITE AREA DRAINS WESTWARD. (calculation I-6)

$4\overline{)60.7"}$ = AVG = 15.2"

∴ IGNORE EASTWARD DRAINAGE SITUATION

SCALE 1" = 136'

DISTANCE = 2060'

OBTAIN VELOCITY FROM CALCULATION I-7 PAGE 9
AT DISCHARGE = 6500 cfs,
V = 10 fps

now $time = \frac{distance}{velocity} = \frac{2060\ ft}{10\ \frac{ft}{sec} \cdot 60\ \frac{sec}{min}} = 3.43\ min = t_2$

3) $t_{TOTAL} = t_c + t_{ch} = t_1 + t_2$

$= 40.97\ min + 3.43\ min$

$t_{TOTAL} = 44.40\ min = 44\ min, 24\ sec$

809307 4 of 5
WLC CONSULTING
JHS 7/25
KSR 7-25

that a conservative value of C be applied for PMF estimation since infiltration and storage comprise a low percentage of the runoff. Furthermore, the C values presented were derived for storms of 5-100 year frequencies. Therefore, less frequent, higher intensity storms will require the use of a higher C value (Chow, 1964). It is recommended that a runoff coefficient of 1.0 be used for PMF applications in very small watersheds since the effects of localized storage and infiltration will be small.

Table 4.5. Values of C for Use in Rational Formula.

Soil Type	Watershed Cover		
	Cultivated	Pasture	Woodlands
With above-average infiltration rates; usually sandy or gravelly	0.20	0.15	0.10
With average infiltration rates; no clay pans; loams and similar soils	0.40	0.35	0.30
With below-average infiltration rates; heavy clay soils or soils with a clay pan near the surface; shallow soils above impervious rock	0.50	0.45	0.40

Source: Chow, 1964.

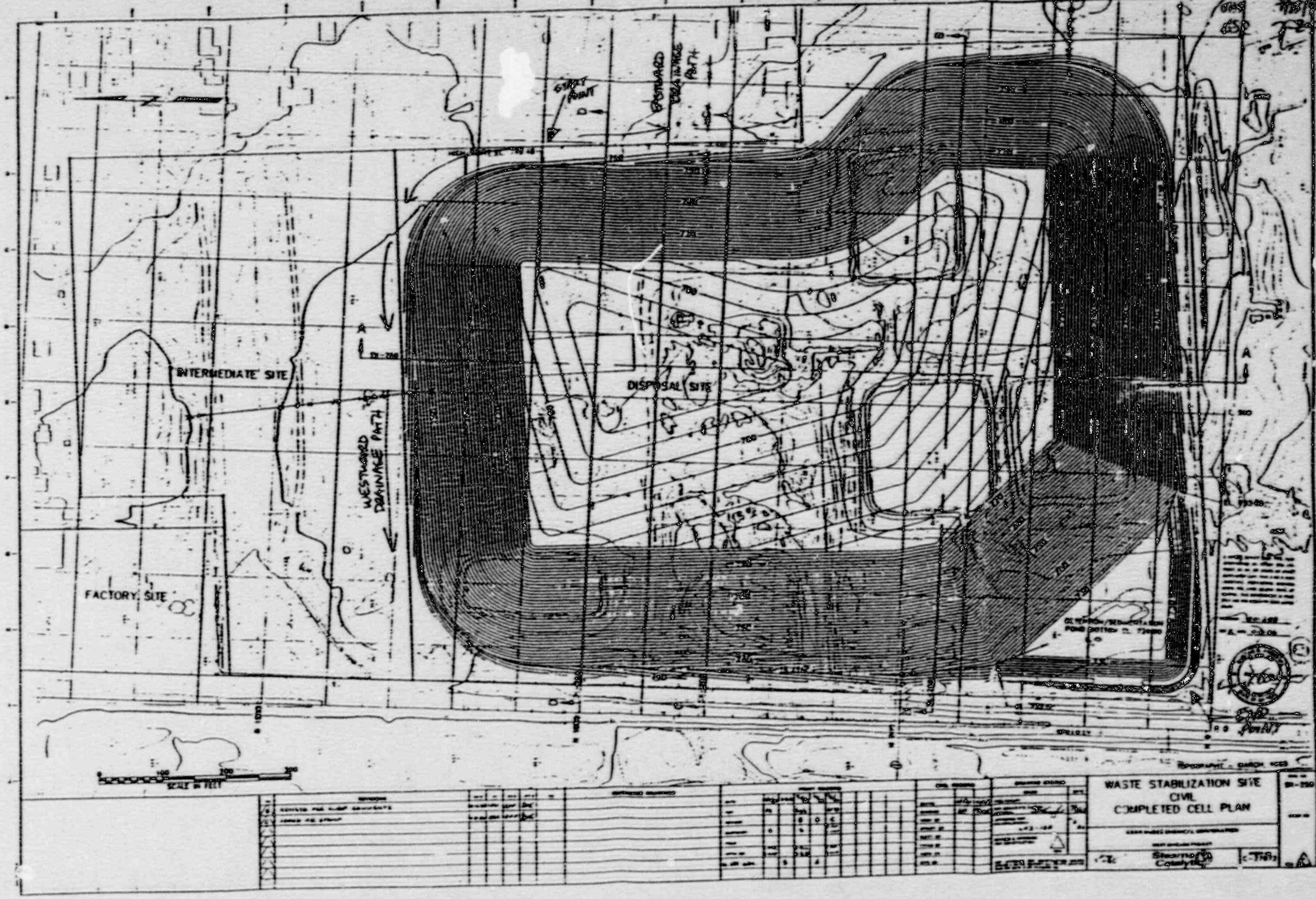
4.8.2 Rainfall Intensity

In order to determine the rainfall intensity, i, the time of concentration, t_c must be estimated. The time of concentration can be approximated by:

(a) Applying one of the many accepted empirical formulae such as

$$t_c = 0.00013 \frac{L^{0.77}}{S^{0.385}} \tag{4.44}$$

where L is the length of the basin in feet measured along the watercourse from the upper end of the watercourse to the drainage basin outlet and S is the average slope of the basin. Time of concentration is expressed in hours. This procedure is not applicable to rock covered slopes. This expression was



EASTWARD DRAINAGE PATH
START POINT
WESTWARD DRAINAGE PATH
INTERMEDIATE SITE
DISPOSAL SITE
FACTORY SITE
END POINT
SCALE IN FEET
WASTE STABILIZATION SITE
CIVIL
COMPLETED CELL PLAN

JAMES L. GRANT AND ASSOCIATES, INC.

CALCULATION SUMMARY SHEET

PROJECT NO: 809307

CALCULATION NUMBER:

DATE: 7-20-90

DESCRIPTION OF CALCULATION:

Calculation of Riprap Size for Q=6500 cfs in Western diversion ditch

REFERENCES USED:

NUREG/CR-4620

Draft STP - Design of Erosion Protection - (May, 1990)

CALCULATION MADE BY: [signature] DATE: 7-20-90

CALCULATION CHECKED BY: DCS DATE: 7-20-90

CALCULATION APPROVED BY: [signature] DATE: 7/20/90

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(303) 779-0576

JOB NAME West Chicago
BY JCG DATE 7-20-90
CHECKED BY DLS DATE 7-20-90

Calculating Riprap Size for
6,500 Cfs in western Diversion
Channel

1 - Depth of flow is about 10' (see Attached rating curve)

2 - Shear stress $\tau = \gamma d S = 6.24$

3 - $d_{50} = \dfrac{6.24}{.04(\gamma_s - \gamma_w)}$

$\gamma_s = 165 \text{ \#/ft}^3$

$d_{50} = 1.5 \text{ feet} = 18''$

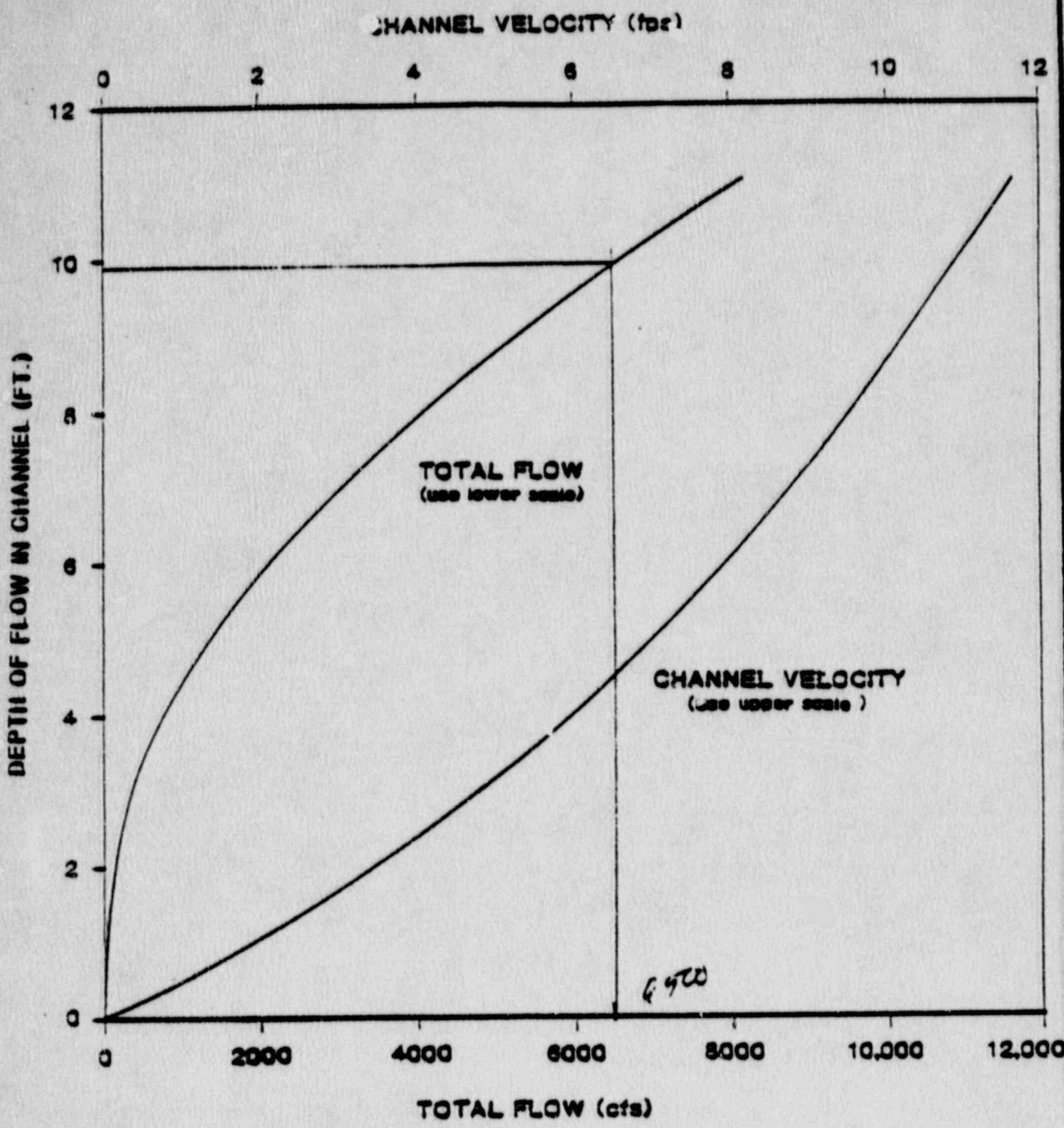

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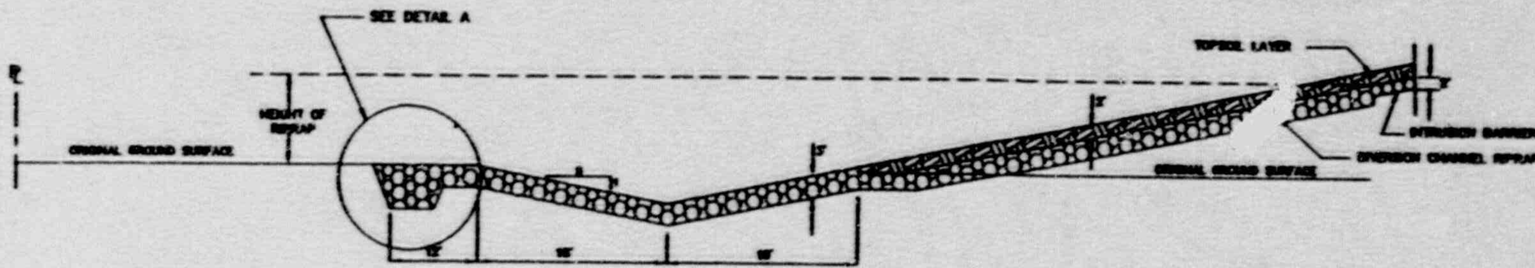
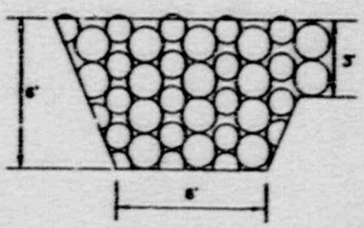
FIGURE 7

DIVERSION DITCH FLOW AND VELOCITY





DETAIL A
RIPRAP TERMINATION DETAIL

809307

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CONCEPTUAL DIVERSION CHANNEL
RIPRAP PLACEMENT
WEST CHICAGO FACILITY

GRANT, SCHREIBER & ASSOCIATES
CALCULATION SUMMARY SHEET

PROJECT NO: 809307

TASK NO: 01

CALCULATION NUMBER: I-4

DATE: 7-16-90

DESCRIPTION OF CALCULATION:

Determine peak flood flow (PMF) for Kress Creek drainage using NRC Regulatory Guide 1.59.

REFERENCES USED:

1) U.S. NRC, "Design Basis Floods for Nuclear Power Plants," Regulatory Guide 1.59 Revision 2, August 1977.
2) USGS West Chicago Quadrangle topographic map.
3) Crippen, J.R & C. D. Bue, "Maximum Flood Flows in the Conterminous United States," USGS Water Supply Paper 1887, 1977.

CALCULATION MADE BY: Kevin S. Rauch DATE: 7-16-90

CALCULATION CHECKED BY: John H. Schreiber DATE: 7-17-90

CALCULATION APPROVED BY: David H. Schreiber DATE: 7-18-90

2 Attachments

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JOB NAME West Chicago Erosion
BY KSR DATE 7-16-90
CHECKED BY JHS DATE 7-17-90

OBJECTIVE: Determine the PMF flow for the Kress Creek drainage using Figures B.2 through B.7 of NRC Regulatory Guide 1.59.

SOLUTION:

From the USGS West Chicago Quadrangle, the site location is approximately

Latitude 41° 5[illegible]0" ✓

Longitude 88° 12' 30" ✓

From Figures B.2 through B.7 of Ref 1 (see Attachment I) for the location above, the PMF flows for the given areas below are:

Area (mi.²)	PMF (cfs)
100	108,000 ✓
500	265,000 ✓
1,000	310,000 ✓
5,000	600,000 ✓
10,000	770,000 ✓
20,000	940,000 ✓

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The above data are shown plotted on a logarithmic scale on the next two pages.

Two procedures were used to extrapolate the data to estimate the PMF flow for the 18 mi^2 Kress Creek drainage

First, a power curve was fit to the data, since they plot along a relatively straight line, as shown on page 4. The equation of the power curve is

$$PMF = 19{,}096\ Area^{0.401}$$

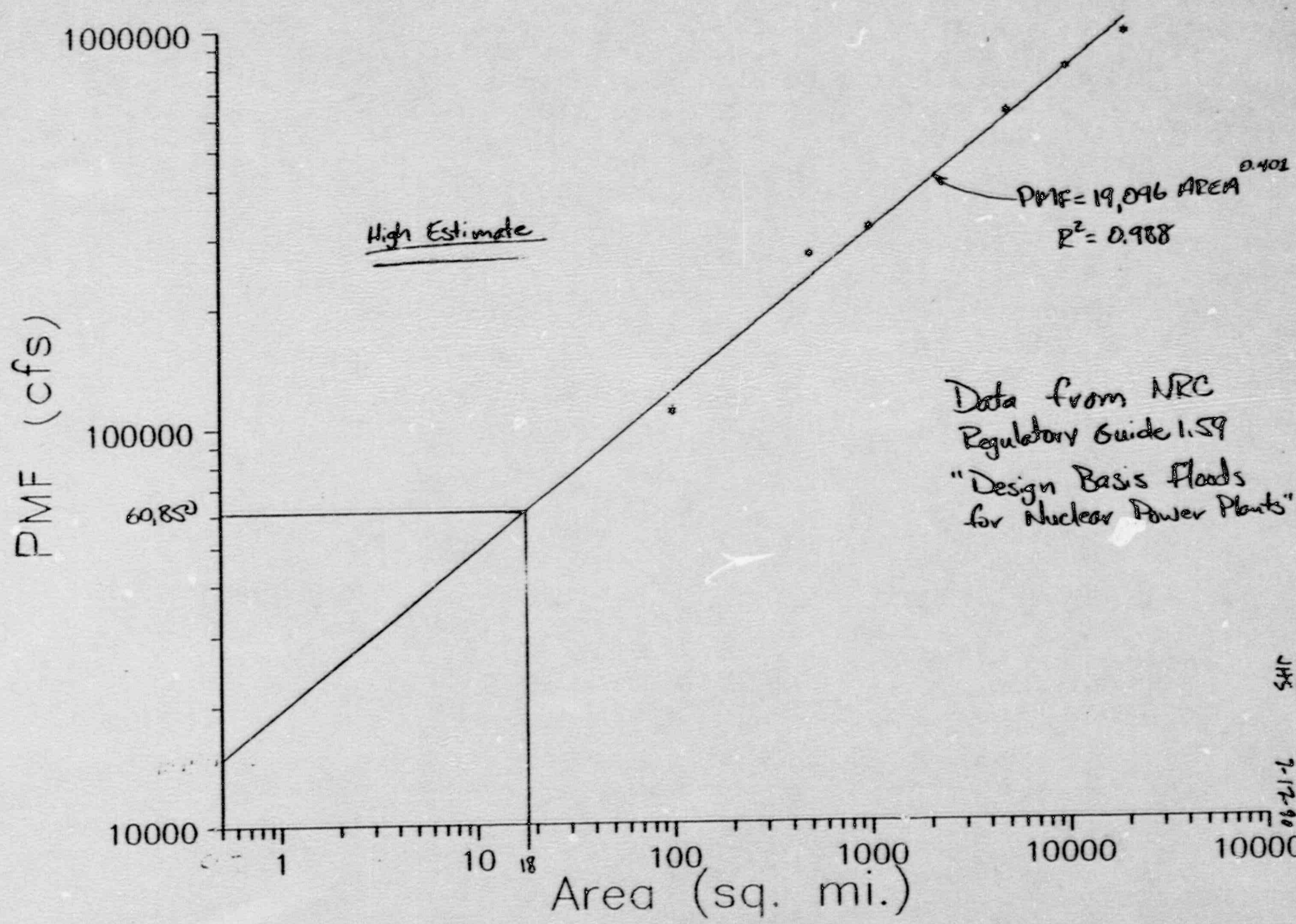
Using this equation results in the following PMF flow

Area (mi^2)	PMF (cfs)
18	60850

This curve is considered to be a high estimate

The second estimate involved plotting an "eyeball" curve through the points and extrapolating a line that increased in slope as area decreased, as shown on page 5. This was considered to be a low estimate of the PMF flow. The corresponding PMF value

AREA (mi^2)	PMF (cfs)
18	42,000



West Chicago Erosion
KSR 7-16-90
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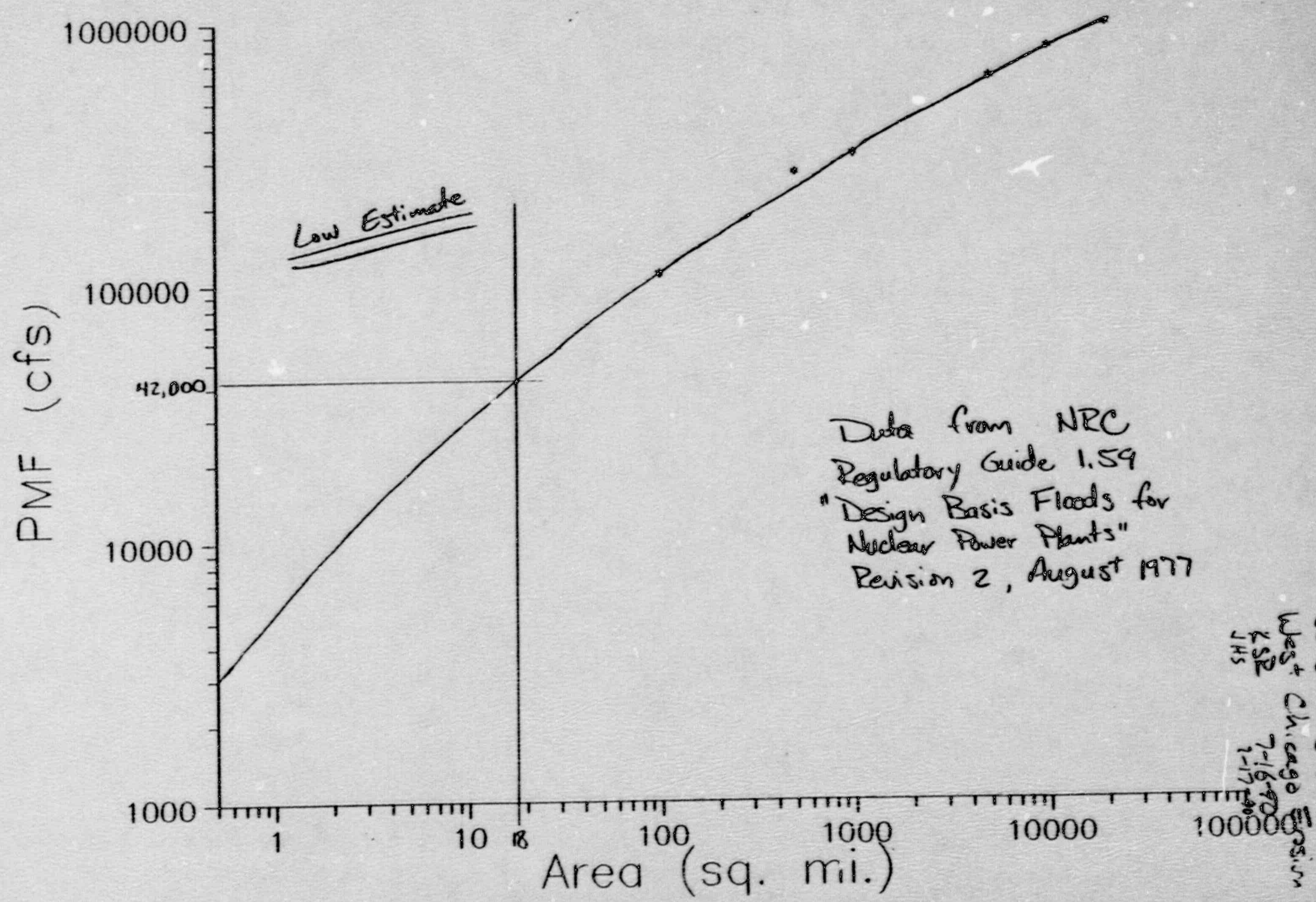

PMF (cfs)
1000000
100000
42,000
10000
1000
Area (sq. mi.)
1
10
18
100
1000
10000
100000
Low Estimate
Data from NRC
Regulatory Guide 1.59
"Design Basis Floods for
Nuclear Power Plants"
Revision 2, August 1977
West Chicago
KSR 7-16-90
JHS 7-17-90

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For Comparative purposes, the PMF value from Crippen and Bue (see Attachment II) were determined

Area (mi.²)	PMF (cfs)
18	68,000 ✓

From Fig. 8 – Region 6.

These values are in between the high and low estimates determined from NRC 1.59, and appear reasonable. ✓

Revision 2
August 1977

U.S. NUCLEAR REGULATORY COMMISSION

REGULATORY GUIDE

OFFICE OF STANDARDS DEVELOPMENT

REGULATORY GUIDE 1.59

DESIGN BASIS FLOODS FOR NUCLEAR POWER PLANTS

Attachment I

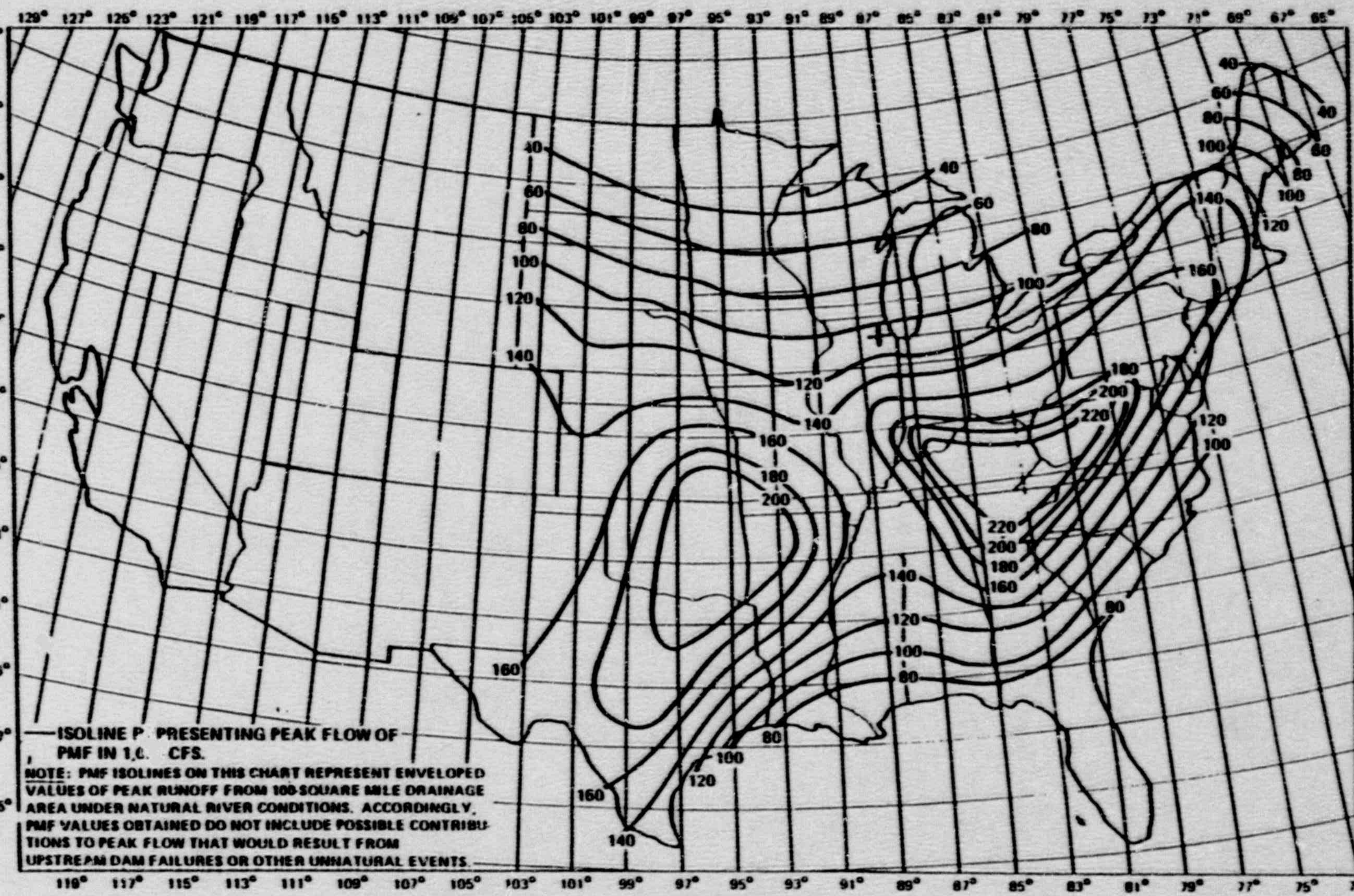

FIGURE B.2 PROBABLE MAXIMUM FLOOD (ENVELOPING PMF ISOLINES) FOR 100 SQUARE MILES

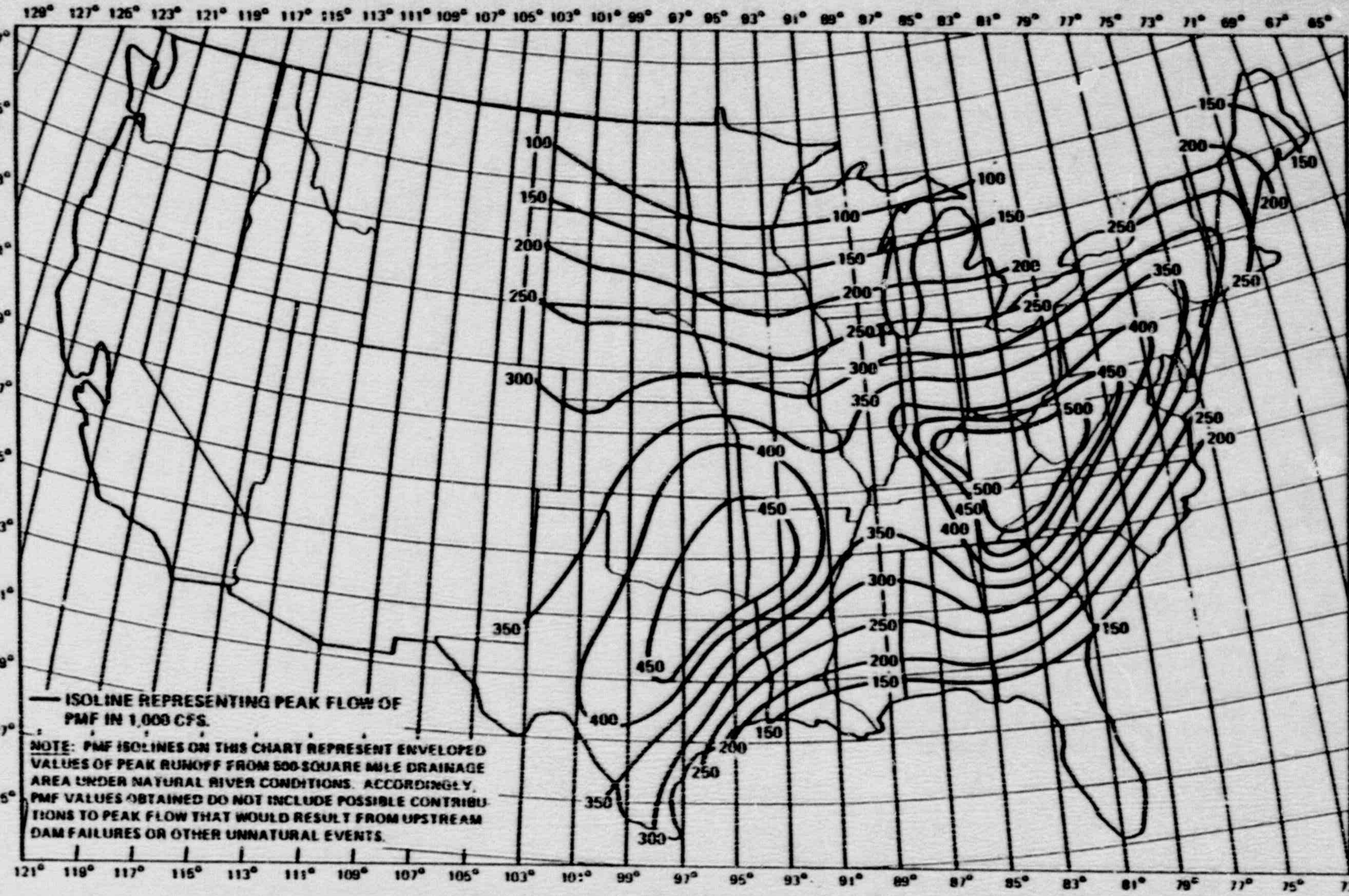

FIGURE B.3 PROBABLE MAXIMUM FLOOD (ENVELOPING PMF ISOLINES) FOR 500 SQUARE MILES

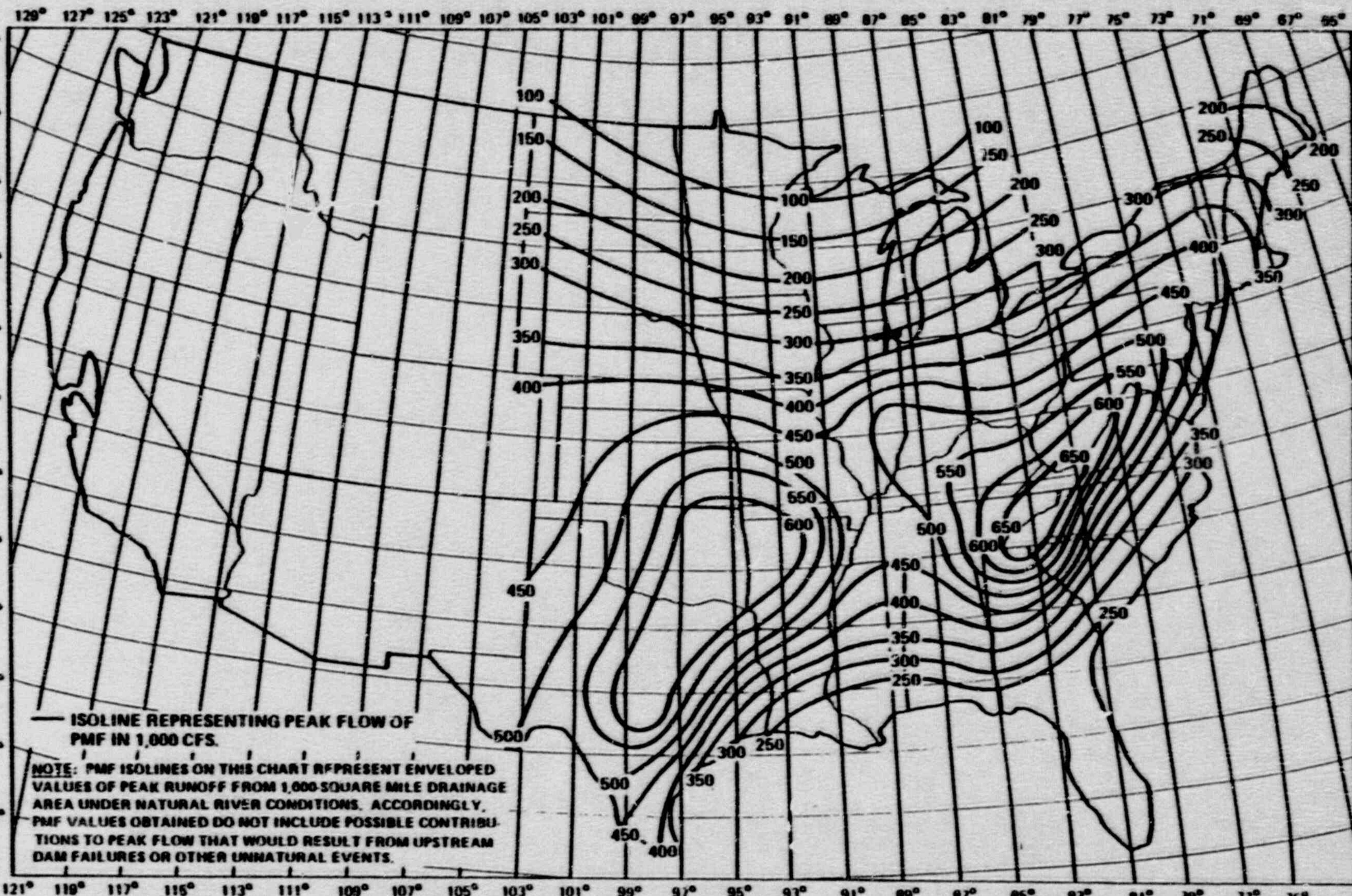

FIGURE B.4 PROBABLE MAXIMUM FLOOD (ENVELOPING PMF ISOLINES) FOR 1,000 SQUARE MILES

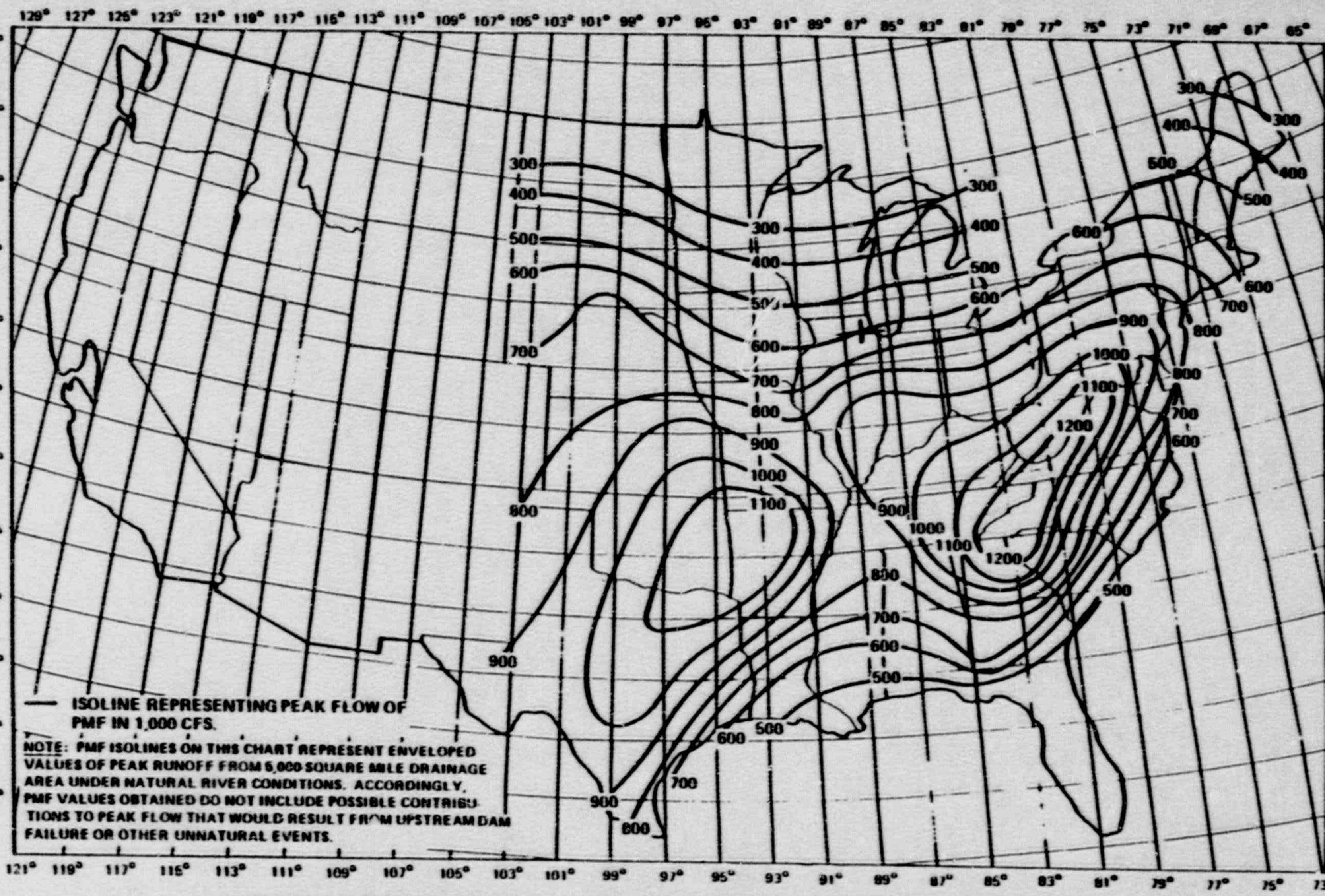

FIGURE B.5 PROBABLE MAXIMUM FLOOD (ENVELOPING PMF ISOLINES) FOR 5,000 SQUARE MILES

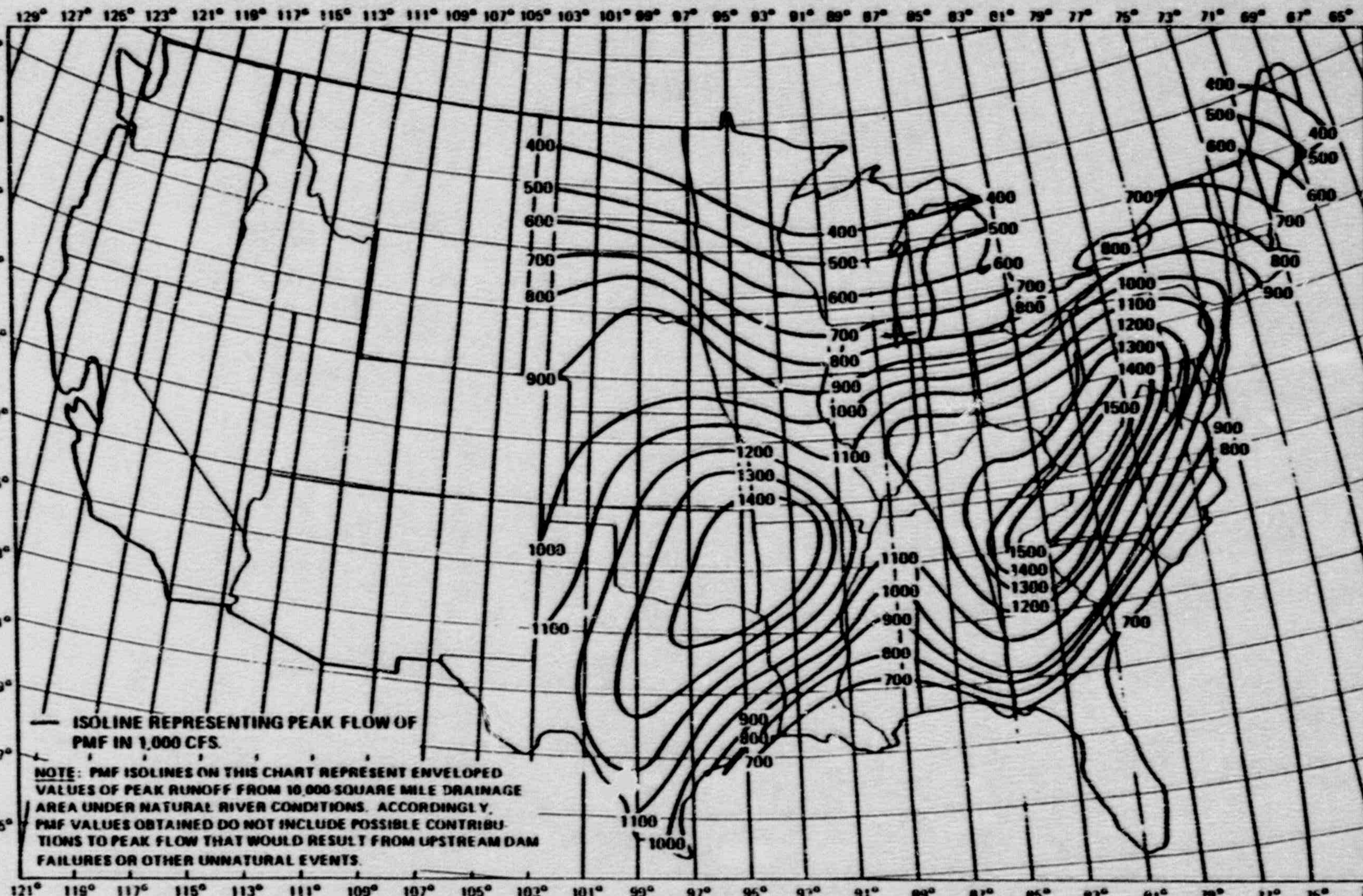

FIGURE B.6 PROBABLE MAXIMUM FLOOD (ENVELOPING PMF ISOLINES) FOR 10,000 SQUARE MILES

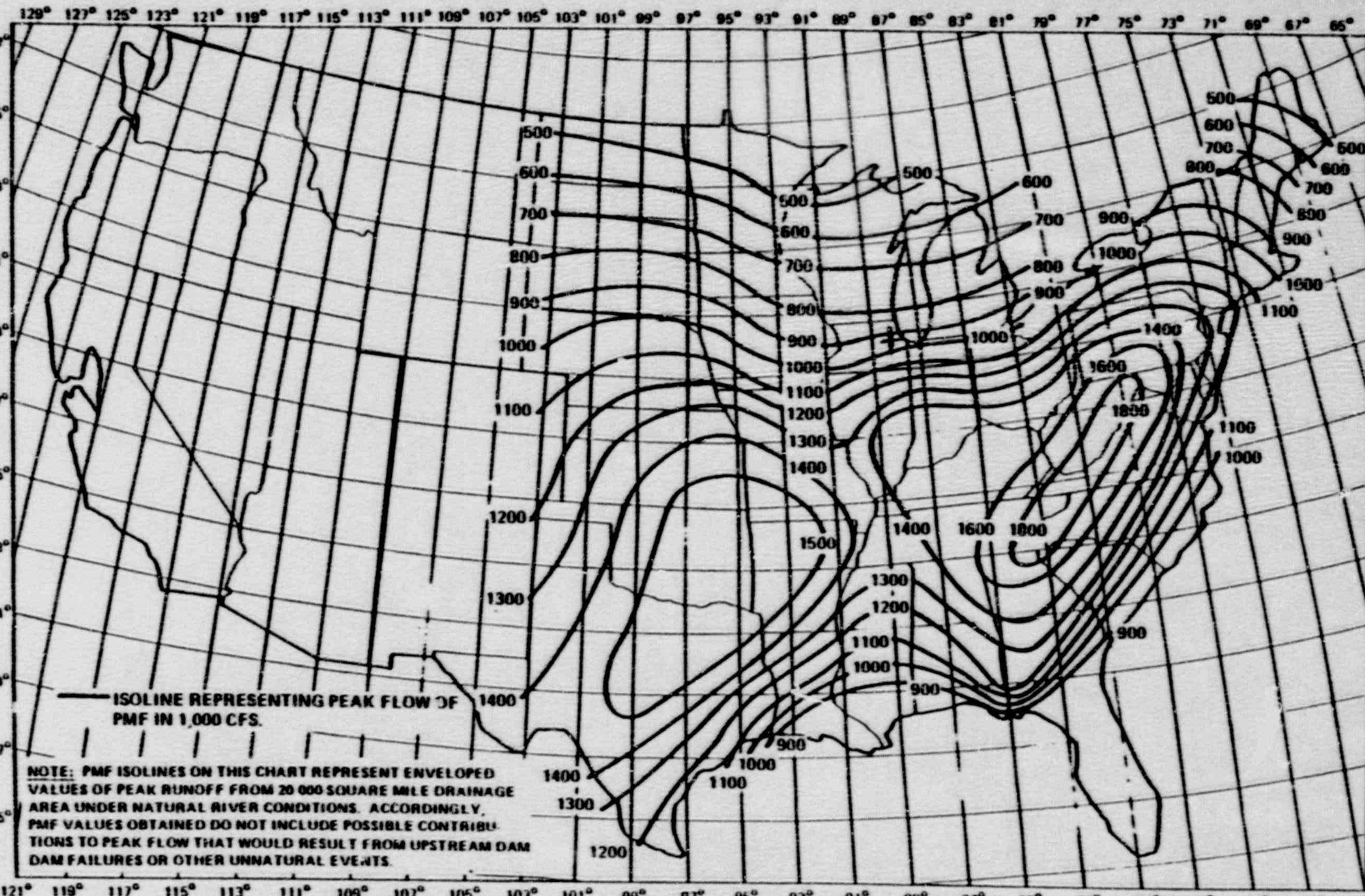

FIGURE B.7 PROBABLE MAXIMUM FLOOD (ENVELOPING PMF ISOLINES) FOR 20,000 SQUARE MILES

Maximum Floodflows in the Conterminous United States

By J. R. CRIPPEN and CONRAD D. BUE

GEOLOGICAL SURVEY WATER-SUPPLY PAPER 1887

Attachment II

UNITED STATES GOVERNMENT PRINTING OFFICE, WASHINGTON : 1977

FIGURE 1.—Map of the conterminous United States showing flood-region boundaries.

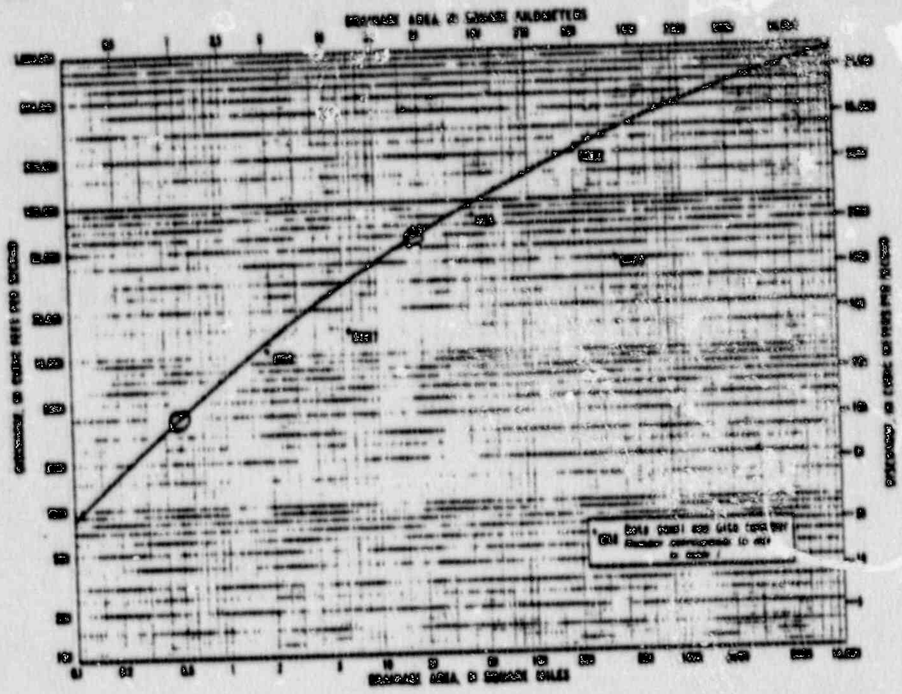

FIGURE 8.—Peak discharge versus drainage area, and envelope curve for region 6.

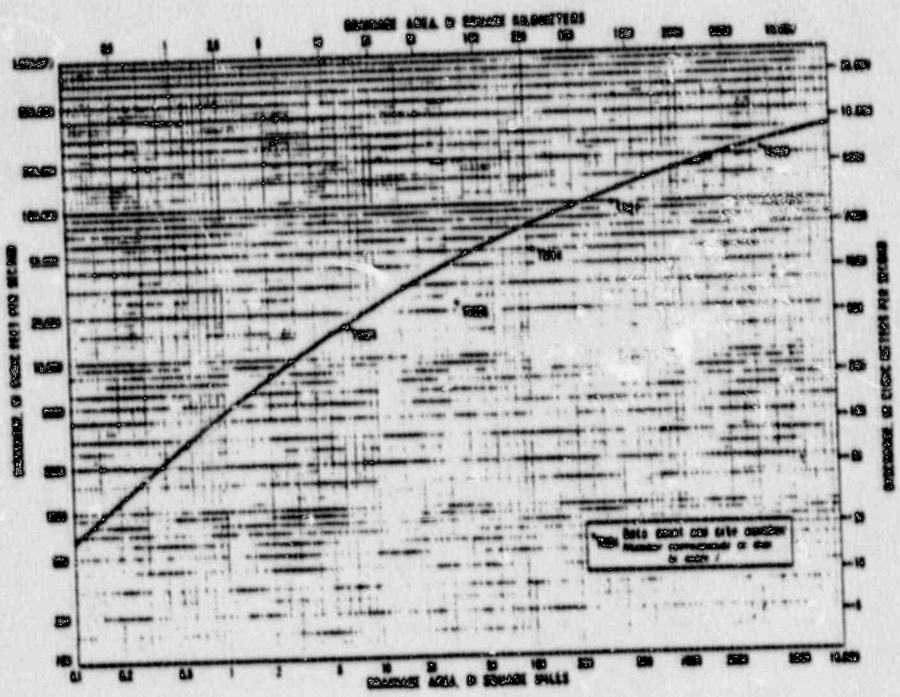

FIGURE 9.—Peak discharge versus drainage area, and envelope curve for region 7.

JAMES L. GRANT AND ASSOCIATES, INC.

CALCULATION SUMMARY SHEET

PROJECT NO: 809307

CALCULATION NUMBER: ______

DATE: 7-17-90

DESCRIPTION OF CALCULATION:

Calculation of flow velocity adjacent to the disposal cell during large (PMF) flow in Kress Creek

REFERENCES USED:

- Method - Normal depth in Kress Creek will give conservative velocities (suggested by NRC staff)

NUREG 1.59 for PMF Flow

Chow (Open Channel Hydraulics) for n value

USGS Quad sheet for channel slope & X-sections

CALCULATION MADE BY: N. [illegible] DATE: 7/17/90

CALCULATION CHECKED BY: [signature] DATE: 7/17-90

CALCULATION APPROVED BY: [signature] DATE: 7/17/90

FIGURE 8

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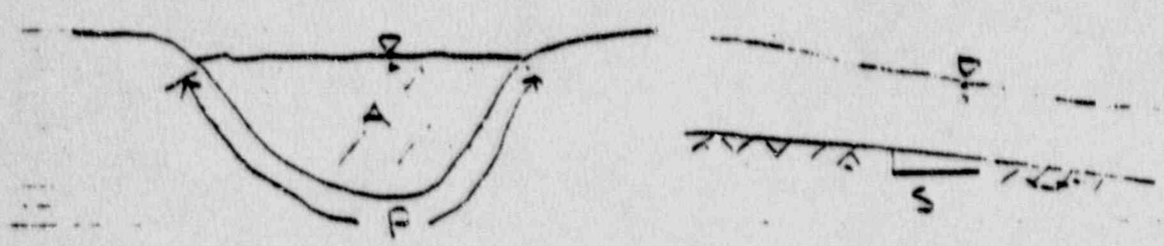
MANNING'S EQN

$$Q = \frac{1.49}{n} A R^{2/3} S^{1/2} \qquad V = \frac{Q}{A} = \frac{FT}{SEC} \text{ UNITS}$$

ELEVATION (ELEVATION)	$AR^{2/3}$	MANNINGS	Q = .067 $AR^{2/3}$ (CFS)	V (FPS)		
725	1292	.067	2164	3.0		
730	15,598	.067	26,127	4.7		
735	63,442	.067	106,265	7.0		
740	125,513	.067	210,234	9.7		

FLOW RATE IN CHANNEL

VELOCITIES OF CHANNEL FLOW WHEN WATER LEVEL AT ELEVATIONS NOTED IN COLUMN 1



REFER TO CROSS-SECTIONAL DRAWING OF CHANNEL

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JOB NAME [illegible]
BY [illegible] DATE [illegible]
CHECKED BY [illegible] DATE 7/1/90

ANALYSIS OF [illegible] CREEK FLOW RATE

USE MANNING'S EQUATION

$Q = \frac{1.49}{n} A R^{2/3} S^{1/2}$ WHERE n = MANNING'S COEFFICIENT
S = CHANNEL SLOPE (ft/ft)
A = CHANNEL AREA (ft²)
R = HYDRAULIC RADIUS (ft)

n = 0.04 FOR THESE CHANNEL CONDITIONS (ESTIMATED)

$S = \frac{5 \text{ ft}}{2400 \text{ ft}} = 0.002$ FT/FT (see attached profile)

P = WETTED PERIMETER = A/R

TO DETERMINE R, MEASURE A AND P FROM CHANNEL CROSS SECTIONS

LOCATION (ELEV.)	AREA (SQ. FT.)	P (FT).	R (FT.)	$R^{2/3}$	ΣA (SQ. FT.)	$AR^{2/3}$
720 - 725	717.85	300	2.4	1.8	718	1292
725 - 730	5549.25	1600	3.9	2.5	6267	15,599
730 - 735	8904.7	1900	7.98	4	15,171	60,442
735 - 740	10488.1	2400	10.7	4.9	25,659	125,817
740 - 744	12297.2	3900	— ([illegible])	—	37,956	—

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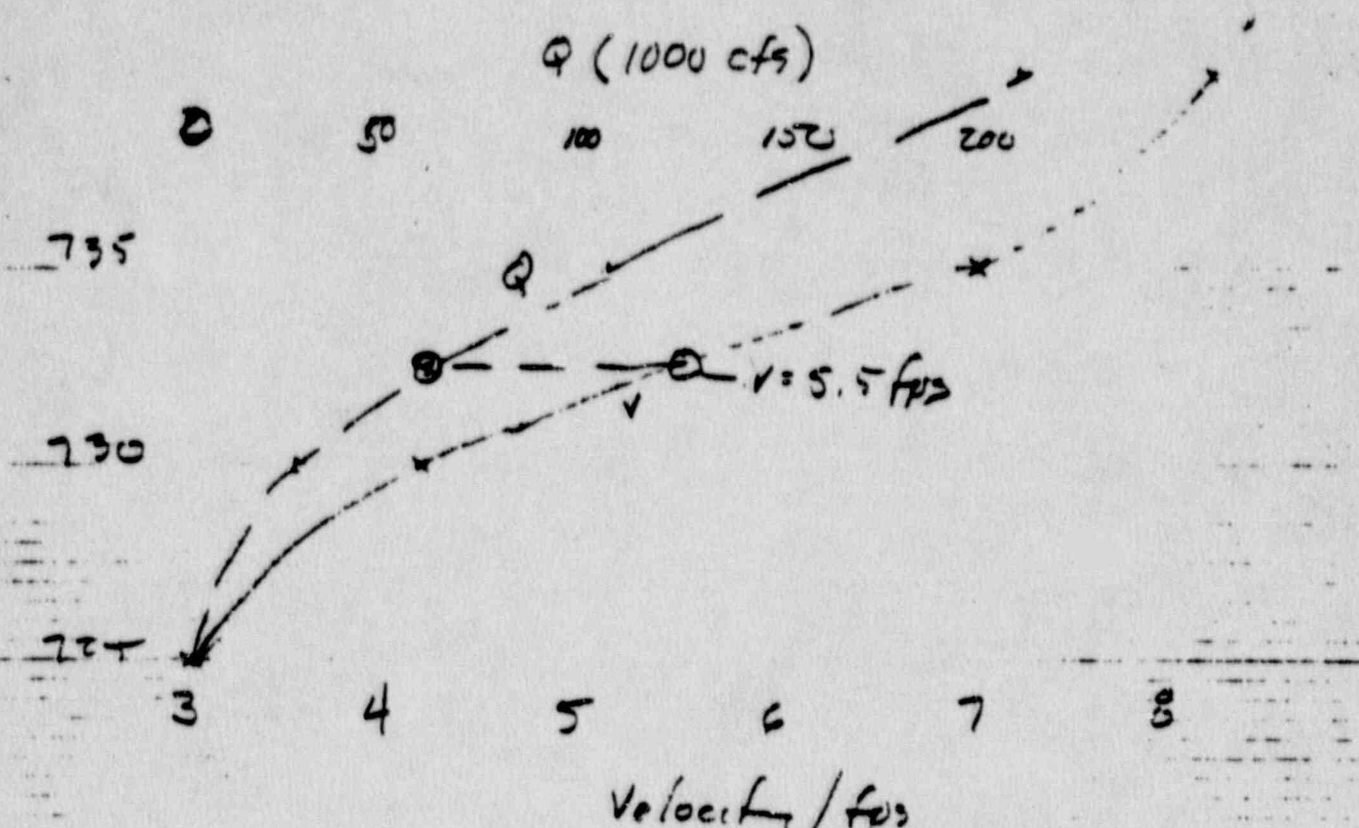
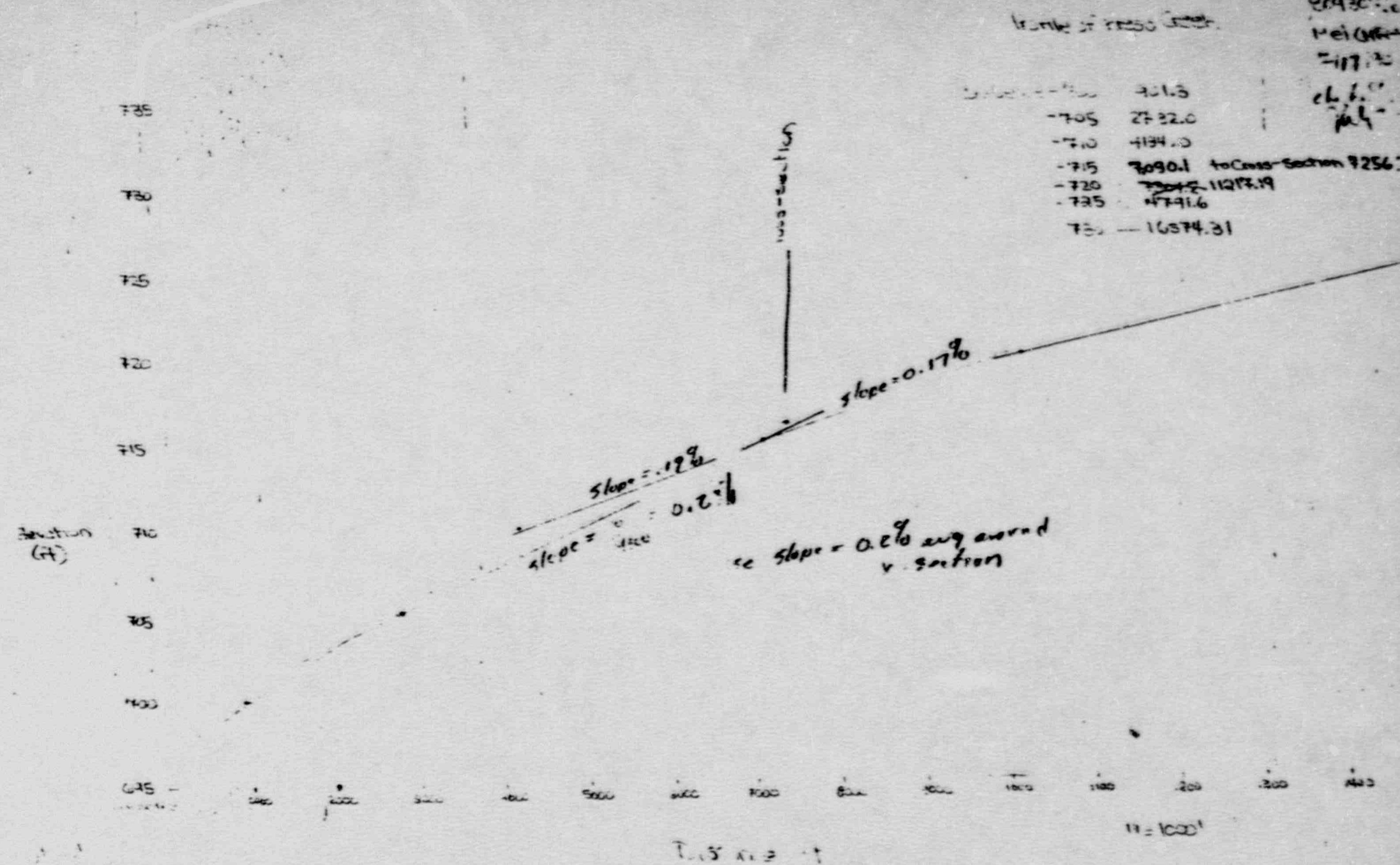
JOB NO. 809307 SHEET 1 OF 1
JOB NAME West Chicago
BY JLG DATE 7-17-90
CHECKED BY ______ DATE 7/17/90



used n=0.04

Kress Creek Normal Depth/Velocity Calculations

Figure 9



Elevation (ft)
735
730
725
720
715
710
705
700
695
Distance (ft)
1" = 1000'
slope = 0.17%
slope = .12%
slope = 0.2%
slope = 0.2% avg around
-705 2732.0
-710 4134.0
-715 7090.1 to Cross-Section 7256
-720 11297.19
-725 14791.6
735 — 16374.31


41°52'30"
8°15'
12'30"
50'
WEST CHICAGO
Jr High Sch
Gary Sch
(WEST CHICAGO)
WEST
DU PAGE
Sewage Disposal
BURLINGTON
NORTHERN
Kregs
Creek
EASTERN
JOLIET
AND
ELGIN
BATAVIA
ROAD
BM 724
BRANCH
WEST
W I N F I E L D
NATIONAL
ACCELERATOR LABORATORY
Pit
MACK
18
16
19
20
21
22
28
29
30
730
740
750
760
735
738
744
739
725
713
705
715
710

Kerr-McGee West Chicago Facility

Evaluation of Potential Erosion

Influence of Retention Pond

Riprap for the western diversion ditch was sized based upon a critical channel section immediately upstream of the sedimentation basin. The same riprap will be continued around the side of the disposal cell that forms the northeastern side of the retention pond.

Most water enters the retention pond from the western diversion ditch. A lesser amount will flow into the pond from the southern diversion ditch (estimated to be about 400 cfs in section 5 of the report), and a small amount will flow directly from the surface of the disposal cell into the pond.

The retention pond discharges through a ten-inch discharge pipe. Flood flows discharge from the retention pond through an overflow spillway formed in the western berm of the pond. This spillway is approximately 75 feet wide, and the crest is one foot below the top of the berm. The spillway was sized to pass a reduced PMF without failure. Flows larger than the reduced PMF will overtop the berm forming the basin. The berm may erode if overtopped.

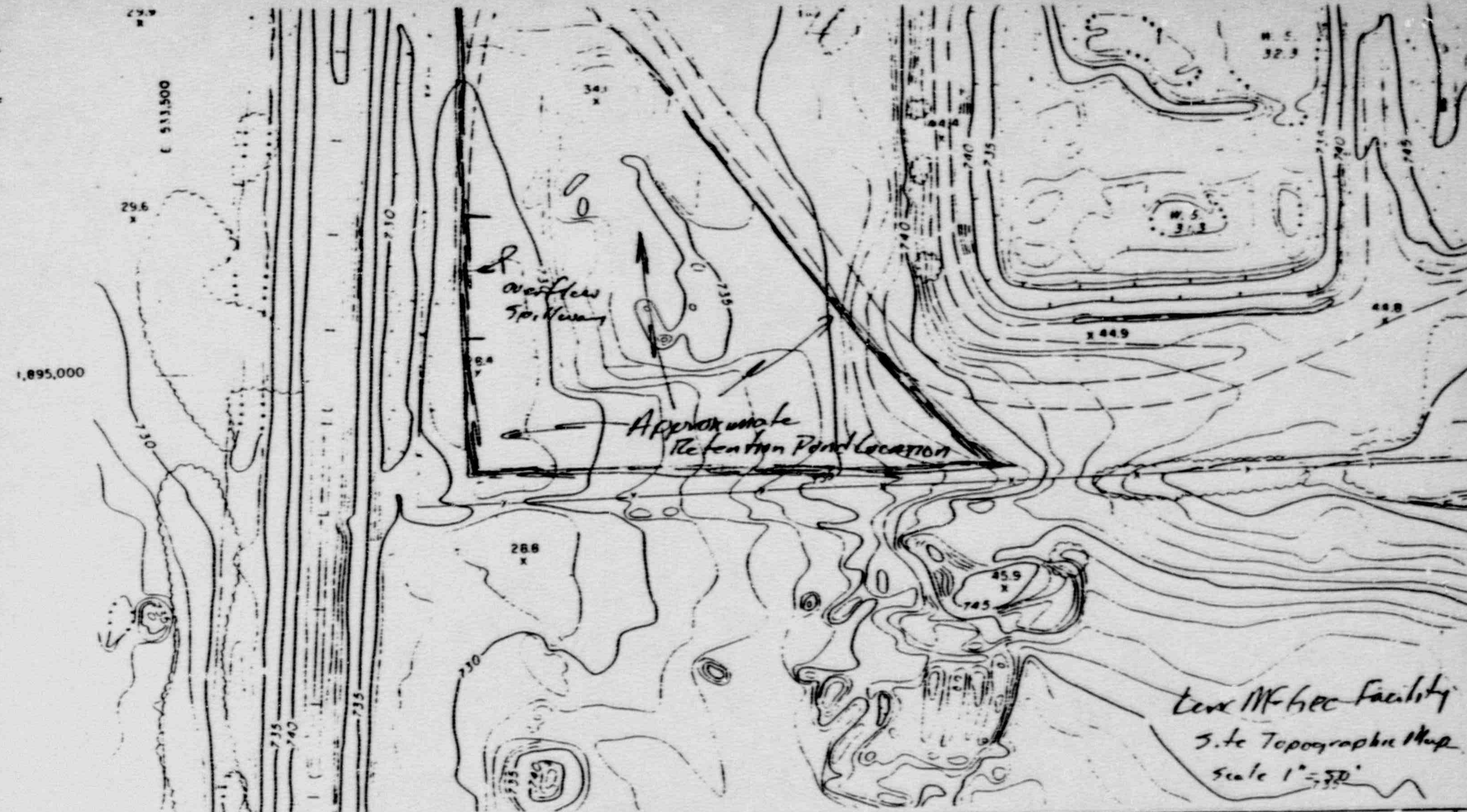
The position of the retention pond in relation to site topography and the disposal cell is shown in Figure 1 of the July 23 report. The pond is located at the southwest corner of the disposal cell, in a natural swale that extends to the south. The pond is located in a wide spot in the swale as is shown by the attached figure, which is a larger-scale drawing of the part of the site topographic map depicting existing ground surface around and within the area where the detention pond will be constructed. The approximate outline of the detention pond has been sketched to provide a reference on this figure. This sketch is only a rough outline of the proposed pond.

The swale widens at the retention pond because the disposal cell, which will form one side of the drainage channel, ends. Water flowing down the western side of the channel will spread upon entering the retention pond, and the velocity and depth of the water will decrease. Downstream of the proposed pond, the natural swale is constricted. This constriction will restrict flow, and will reduce velocities above the constriction.

The presence of the retention pond will have relatively little effect on flow conditions during large floods. The conservatively estimated depth of flow in the western diversion channel is ten feet. This depth is almost twice the depth of the retention pond, and represents a water surface that is about four feet above the berm forming the pond. The pond is constructed mostly above ground. The riprap in the diversion channel was sized based upon normal flow depths and velocities in the diversion ditch. Water entering the pond will spread into the wider channel that exists at the pond. The velocity will decrease. The riprap on the edge of the disposal cell will protect the cell from possible erosion by eddies in this flow.

Likewise, the failure of the retention pond berm will have little effect on the flow. The berm will be overtopped long before the peak flow (6500 cfs) used to size the riprap is reached. The flow still will spread and slow down as it enters the pond area because of the natural widening of the drainage swale and the fact that the disposal cell ends.

In summary, neither the presence of the retention pond nor its washing out will have much affect on flow conditions in the diversion ditches. The pond is located in a wide area of a natural drainage swale. The total depth of the pond is only slightly more than one-half of the depth of flow in the diversion ditch during the design flow, and the pond is constructed mostly above ground. The riprap was sized based upon normal flow depths and velocities in the diversion ditch. Because the swale downstream of the pond is wider than the design section of the diversion ditch, failure of the pond berm would merely remove a restriction in the swale, and would not lead to flow conditions more critical than those used to size the riprap for the channel.



REVISIONS	REFERENCE DRAWINGS
	Attachment to Retention Pond Discussion