

PROFESSIONAL
ENGINEERING
CONSULTANTS
PROFESSIONAL ASSOCIATION

DRAINAGE PLAN
AND
SUPPORTING CALCULATIONS

FOR
CORPORATE LAKES
AN ADDITION TO WICHITA, SEDGWICK COUNTY, KANSAS

PREPARED BY
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APRIL 21, 1989

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Project Corporate Lakes

Item Drainage Plan Introduction

INTRODUCTION

The proposed Corporate Lakes subdivision is a replat of Regency Pointe, which was recorded on April 3, 1989. A Drainage Plan was prepared by PEC, dated July, 1986, in conjunction with the platting of Regency Pointe. The 1986 plan proposed an RCB to carry runoff from Webb Road to the west line of the plat.

The proposed subdivision will provide open space for an open channel to carry the runoff from Webb Road westward. The calculations herein are based on various channel sections which may be utilized in the development of the site.



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Project Corporate Lakes

Item Drainage Plan Hydrology

HYDROLOGY

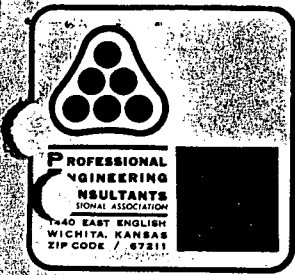
In the 1986 plan, M.W. Berry P.E. of PEC, computed the 100-year runoff entering the site at Webb Rd to be 559 cfs and the internal 100-year runoff to be 83 cfs. The combined flow was computed to be 600 cfs. These figures were based on the City of Wichita Drainage Manual procedures. Further estimations of the 100-year runoff were made using Rational Method, TR-11, FENL-H, SCS TR-55, and the Rossmiller Modified Rational Method. Estimates ranged from 596 cfs to 1,166 cfs. As a compromise, a value of 850 cfs was selected for the design Q_{100} for the project.

For the purposes of this study, the Q_{100} of 850 cfs will be used. Additional calculations for the 5-year storm will be based on 480 cfs.

$$\left(Q_5 = Q_{100} \times \frac{I_5}{I_{100}} = 850 \times \frac{1.83}{3.24} = 480 \right)$$

based on $t_c = 77 \text{ min.}$

Copies of Mr. Berry's calculations are included in this section for reference.



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Project NW Cor CENTRAL & WEBB
Item HYDROLOGY - OFF SITE

I. BASIN CHARACTERISTICS - OFF SITE

A check was made of the Drainage Map from plans for Sedgewick County Proj 833-P. It agrees with USGS Quad sheet & with the drainage map for KDOT Proj 874- (by P.E.C.) and will therefore be used.

The 26 acres lying south of Central and east of the main plant is 100% impervious.

The 24 acres lying south of Central and adjacent to Webb Rd which includes the main plant is 100%. An additional 21 acres abutting Webb Rd and south of the main plant is nearly 100% impervious. This combined area of 45 acres is conveyed thru a reinf. conc. box lying on the west side of Webb Road.

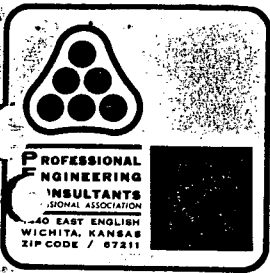
Of the 148 acres lying north of Central & east of Webb, the land use is mixed:

43 acres low density residential

37 acres impervious area (mainly parking lot)

68 acres agricultural

The majority of the agricultural land is under Beech ownership. It will be assumed that this land is developable to 45% imperviousness at some future date.



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Project NW COR CENTRAL WEBB

Item HYDROLOGY OFF SITE

II RUNOFF COMPUTATION

A. "C" FACTORS Reference, Attachment D, C.D.W. Drainage Manual

Basin NE

① 68 Acres, D Soil, Slope 1.5%

Assume 45% Imp $C_{100} = 0.68$ $C_5 = 0.54$

② 37 Acres, D Soil, 100% Imp., Slope 1.5% (Parking Lot)

$C_{100} = 0.89$ $C_5 = 0.87$

③ 43 Acres, D Soil, 1/2-Acres Res., Slope 1.0%

$C_{100} = 0.72$ $C_5 = 0.48$

Composite $C_{100} = (0.68)(68) + (0.89)(37) + (0.72)(43) / 148 = 0.74$

$C_5 = (0.54)(68) + (0.87)(37) + (0.48)(43) / 148 = 0.61$

Basin E

26 Acres, D Soil, Parking Areas

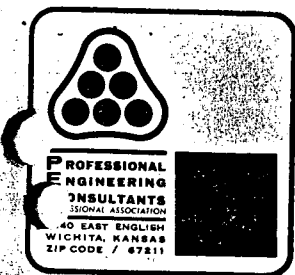
Slope 1.25%

$C_{100} = 0.89$ $C_5 = 0.87$

Basin C

24 Acres, D Soil Parking Area

$C_{100} = 0.89$ $C_5 = 0.87$



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Project NW COR CENTRAL & WEBB

Item HYDROLOGY OFF SITE

II. A. Cont.

BASIN S

21 Acres, D Soil, Parking Areas

Slope 1.25%

$C_{100} = 0.89$ $C_5 = 0.87$

III. TIMES OF CONCENTRATION

A. BASIN NE

2900' travel across pasture - no defined channel

slope = 1.5%

if $L > 500'$ use Kinematic Wave

$$t_c = 0.93 L^{0.6} n^{0.6} / i^{0.4} S^{0.3}$$

Trial	S	n	L	i	T_c
1	0.015	0.13	2900'	3.00	74
2	"	"	"	3.5	70
3	"	"	"	3.6	69
4	"	"	"	3.7	68
5	"	"	"	3.4	71

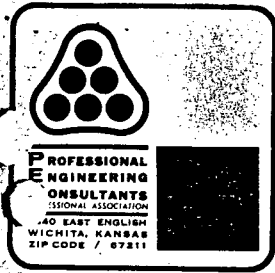
OK 100-Yr

$\therefore T_c = 71 \text{ min}, i_{100} = 3.4 \text{ in/hr}, \text{ Reach 1}$

5-Year:

1	0.015	0.13	2900	4.0	66
2	"	"	"	3.0	74
3	"	"	"	2.0	87
4	"	"	"	1.8	91
5	"	"	"	1.7	93
6	"	"	"	1.6	95

$T_c = 95 \text{ min}$ $i_5 = 1.6 \text{ in/hr}, \text{ Reach 1}$



III A Cont

Reach 2 500' Pav't Slope = 2%

Trial	S	n	L	t	T _c
1	0.02	0.015	500'	5.0	5.3
2	"	"	"	6.0	4.9
3	"	"	"	8.0	4.4
4	"	"	"	4.0	5.0
5	"	"	"	7.0	4.6
6	"	"	"	9.0	4.2
7	"	"	"	10.0	4.0
8	"	"	"	10.3	4.0
9	"	"	"	10.5	3.9

T₊ access^{let} < 4 minutes, and falls off IDF table. No 4 min

From velocity method V = 2 ft/sec

$$\frac{500 \text{ ft}}{2 \text{ ft/sec}} \times \frac{1 \text{ min}}{60 \text{ sec}} = 4.2 \text{ min} \text{ } \underline{\text{USE 4 min.}} \quad \Sigma = 75 \text{ min (100\%)} \\ \Sigma = 99 \text{ min (5Yr)}$$

Reach 3 - Storm sewer

2-54" RCP @ 1.5% 1500' long

Assume for 100 yr that they flow at capacity.

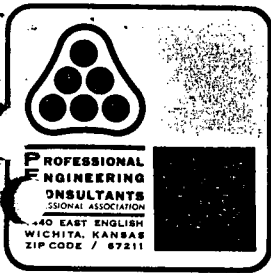
$$V = \frac{1.49}{n} R^{2/3} S^{1/2} = \frac{1.49}{0.013} (1.125)^{2/3} (0.015)^{1/2} = 15 \text{ ft/sec}$$

$$\therefore Q_{\text{max}} = (2)(15.9 \text{ ft}^2)(15 \text{ ft/sec}) = \underline{477 \text{ cfs}}$$

(If 48" ϕ , not 54" ϕ , V = 14 ft/s
 $\phi = 353$)

$$Q_{100}: T_{\text{Reach 3}} = \frac{1500}{15(60)} = 1.67 \text{ min!} \quad \Sigma T_{\text{emp}} = 77 \text{ min}$$

$$\text{For } Q_5, \text{ assume } 8 \text{ ft/s} \quad T_{\text{f}} = 3 \text{ min} \quad \Sigma T_{\text{c5}} = 102 \text{ min.}$$



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Project NW COR CENTRAL & WEBB

Item HYDROLOGY OFF SITE

1600 str/stm sewer

III
(B) BASIN E

Reach 1 1200' travel across pavement as sheet flow

1 slope = 1.25%

KINEMATIC WAVE - 100 yr

Travel	S	n	L	v	T _a
1	0.0125	0.015	1200'	5.0	10.3
2	"	"	"	4.0	11.3
3	"	"	"	3.5	11.9
4	"	"	"	3.0	12.6
5	"	"	"	2.0	14.9
6	"	"	"	6.0	9.6
7	"	"	"	7.0	9.0
8	"	"	"	8.0	8.5
9	"	"	"	9.0	8.1
10	"	"	"	10.0	7.8
11	"	"	"	11.0	7.5
12	"	"	"	9.1	8.1
13	"	"	"	9.2	8.1
14	0.0125	0.015	1200'	8.9	8.2

← USE 8 min
100 yr

FIVE YEAR

1	0.0125	0.015	1200'	5.0	10.3
2	"	"	"	5.2	10.1
3	"	"	"	5.4	10.0
4	"	"	"	5.6	9.8
5	"	"	"	5.8	9.7
6	"	"	"	6.0	9.6
				5.3	10.06

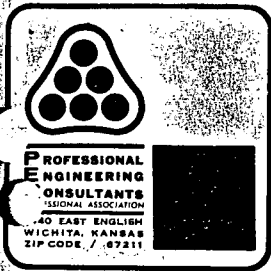
← USE 10 min
5-yr

Reach 2 1600' thru 2-54" storm sewer.

Use same velocities as before for travel time.

$$T_{100} = \frac{1600}{15.60} = 1.78 \text{ min}$$

$$T_5 = \frac{1600}{8.160} = 3.33 \text{ min}$$



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Project NW COR CENTRAL & WEBB

Item HYDROLOGY OFF SITE

III (C) BASIN C

$L=1750'$, $\gamma=1.25\%$, 100% Imp roofs & parking lots

KINEMATIC WAVE ANALYSIS - 100 YR

Trial	S	n	L	i	T _c
1	0.0125	0.015	1750	8.54	10.4
2	"	"	"	8.75	10.3
3	"	"	"	8.25	10.6
4	"	"	"	8.00	10.7
5	"	"	"	9.00	10.2
6	"	"	"	8.3	10.6
7	0.0125	0.015	1750	8.4	10.5

← USE 11 MIN

KINEMATIC WAVE ANALYSIS - 5 YR

1	0.0125	0.015	1750	5.0	12.9
2	"	"	"	4.75	13.2
3	"	"	"	4.5	13.5
4	"	"	"	4.6	13.4
5	"	"	"	4.7	13.2
6	"	"	"	4.8	13.1
7	"	"	"	4.9	13.0

← USE 13 MIN

III (D) BASIN S

$L=2200'$, $\gamma=1.25\%$, 100% Parking Lots & Roofs

KINEMATIC WAVE ANALYSIS - 100 YR

Trial	S	n	L	i	T _c
1	0.0125	0.015	2200	7.4	12.7
2	"	"	"	8.0	12.3
3	"	"	"	7.75	12.4
4	"	"	"	8.25	12.1
5	"	"	"	7.8	12.4
6	"	"	"	7.9	12.3
7	"	"	"	"	"

100 YR T_c
USE 12 MIN



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Project NW COR. CENTRAL & WEBB

Item HYDROLOGY OFF SITE

III (B) cont.

TRIAL	S	n	L	i	T _E
1	0.0125	0.015	2200	4.5	15.5
2	"	"	"	4.4	15.6
3	"	"	"	4.45	15.5

15 YR T_E
USE 15 min

IV PEAK RATE OF RUNOFF

(A) BASIN NE

① For entire basin DA = 148 Ac

$C_{100} = 0.74$ $T_{c100} = 77 \text{ min}$ $i_{100} = 3.24 \text{ in/hr}$ $Q_{100} = 355 \text{ cfs}$

$C_5 = 0.61$ $T_{c5} = 102 \text{ min}$ $i_5 = 1.51 \text{ in/hr}$ $Q_5 = 136 \text{ cfs}$

② Check imp. area only DA = 37 Ac

$C_{100} = 0.84$ $T_c = 15 \text{ min}$ $i_{100} = 7.37 \text{ in/hr}$ $Q_{100} = 242 \text{ cfs}$

$C_5 = 0.87$ $i_5 = 4.56 \text{ in/hr}$ $Q_5 = 147 \text{ cfs}$

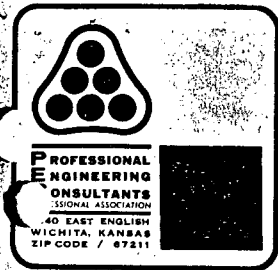
③ Add proportion of basin balance DA = 111 Ac

$C_{100} = 0.7$ $T_{c100} = 71 \text{ min}$ $i_{100} = 3.39 \text{ in/hr}$ $Q_{100} = 263 \text{ cfs}$

$C_5 = 0.51$ $T_{c5} = 95 \text{ min}$ $i_5 = 1.59 \text{ in/hr}$ $Q_5 = 90 \text{ cfs}$

$\Sigma Q_{100} = 242 + \frac{15}{71}(263) = 298 \text{ cfs}$ DOES NOT CONTROL, USE ① ABOVE

$\Sigma Q_5 = 147 + \frac{15}{95}(90) = 161 \text{ cfs}$



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Project NW COR CENTRAL & WEBB

Item HYDROLOGY OFF SITE

IV CONT

(B) BASIN E

26 Acres

DEFAULT
MINIMUM VALUE

$C_{100} = 0.89$ $T_{c100} = 15 \text{ min}$ $i_{100} = 7.37 \text{ in/hr}$ $Q_{100} = 170 \text{ cfs}$

$C_5 = 0.87$ $T_{c5} = 15 \text{ min}$ $i_5 = 4.56 \text{ in/hr}$ $Q_5 = 103 \text{ cfs}$

(C) BASIN C

24 Acres

$C_{100} = 0.89$ $T_c = 15 \text{ min}$ $i_{100} = 7.37 \text{ in/hr}$ $Q_{100} = 157 \text{ cfs}$

$C_5 = 0.87$ $T_c = 15 \text{ min}$ $i_5 = 4.56 \text{ in/hr}$ $Q_5 = 95 \text{ cfs}$

(D) BASIN S

21 Ac

{ same as above $T_c = 15 \text{ min}$ minimum } $Q_{100} = 138 \text{ cfs}$
 $Q_5 = 83 \text{ cfs}$

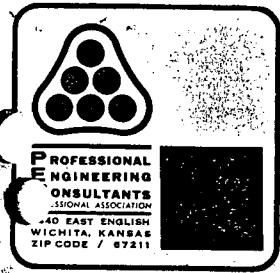
V COMBINATION OF FLOWS @ DESIGN POINT

The use of 15 minute time of concentration for the paved areas is somewhat arbitrary. The flows from these three basins will be added peak-on-peak at the design point. The attenuation effect of travel time will be ignored.

$\Sigma Q's \text{ E, C, + S}$

$Q_{100} = 465 \text{ cfs}$

$Q_5 = 281 \text{ cfs}$



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Project NW COR CENTRAL & WEBB

Item HYDROLOGY OFF SITE

CONT

Combine flows

$$100 \text{ yr: } 355 + \frac{3.24}{7.37} (465) = 559 \text{ cfs} \quad \leftarrow \text{CONTROLS}$$

$$\text{OR} \\ 465 + \frac{15}{77} (355) = 534 \text{ cfs}$$

$$5 \text{ yr: } 136 + \frac{1.51}{4.56} (281) = 230 \text{ cfs}$$

$$\text{OR} \\ 281 + \frac{15}{102} (136) = 301 \text{ cfs} \quad \leftarrow \text{CONTROLS}$$

OFF-SITE HYDROLOGY SUMMARY

$$Q_{100} = 559 \text{ cfs @ } T_c = 77 \text{ min}$$

$$Q_5 = 301 \text{ cfs @ } T_c = 15 \text{ min}$$

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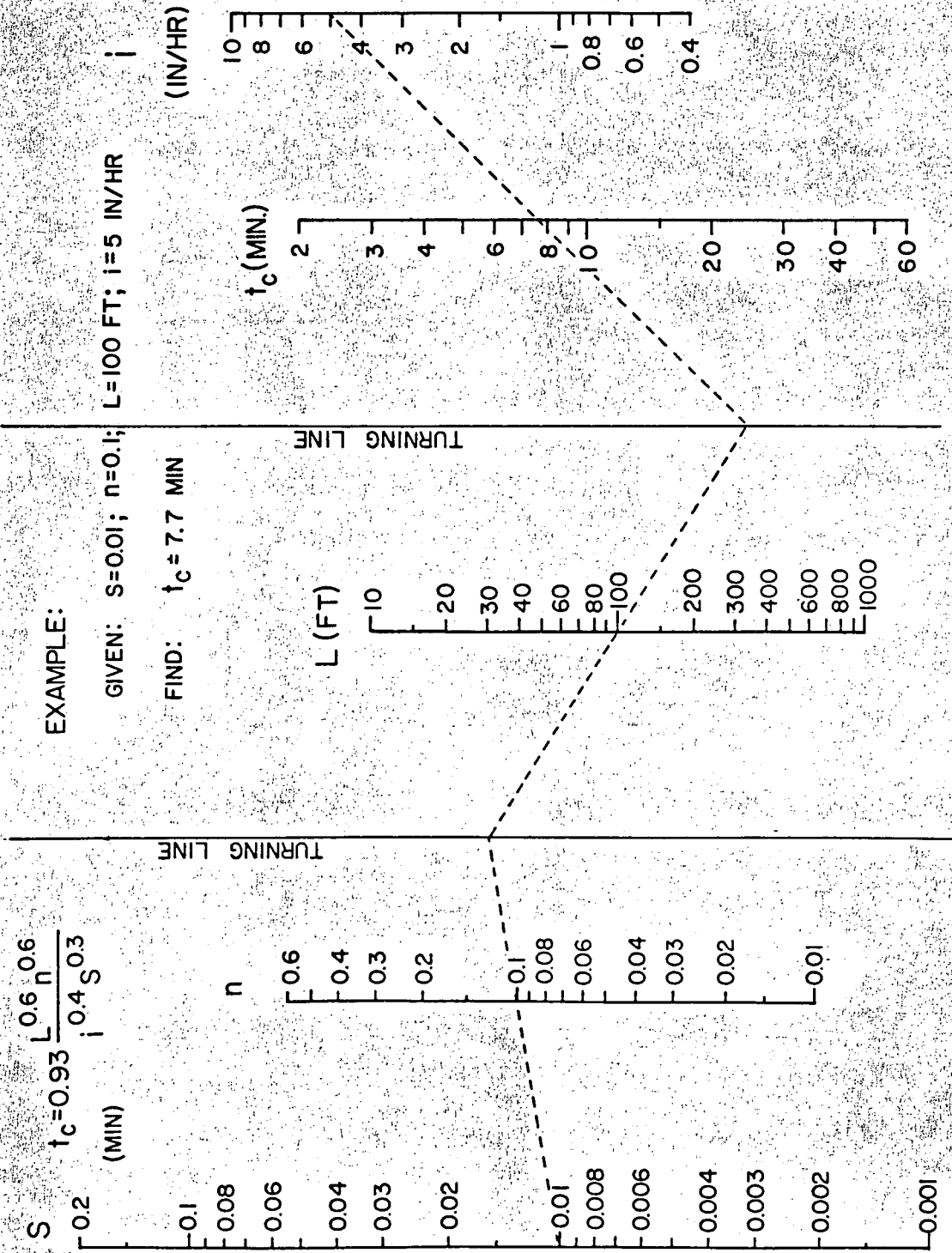
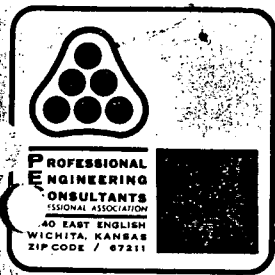


CHART 1. Kinematic wave formulation for determining time of concentration.



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Project NW COR CENTRAL & WEBB

Item HYDROLOGY - ON SITE

I. Analyze gross area for box culvert sizing. More detailed analysis will be done later for inlet pipe sizing.

DA = 14Ac
Slope = 1% in final configuration

L = 1600' sheet flow, mostly paved.

For simplicity, assume schoolyard is at same density as light commercial parcel

From Attachment D, p.1, COW Drainage Manual,

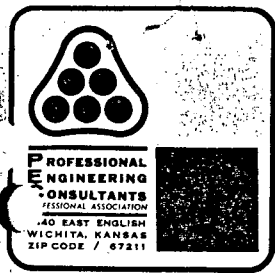
C_s = 0.69 C₁₀₀ = 0.80

KINEMATIC WAVE ANALYSIS - 100 Yr

Trial	S	n	L	i	T _c
1	0.01	0.015	600	7	6.4
2	"	"	"	8	6.0
3				9	5.7
4				10	5.5
5				11	5.3
6				9.2	5.7
				9.4	5.6
				9.6	5.6
				9.8	5.6

} USE 5min. DEFAULT TO 15 MIN.

By inspection T_c = 15 min default for 5-year Design



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Project NW COR CENTRAL & WEBB

Item HYDROLOGY - ON SITE

$$TQ_{100} = 0.80 \times 7.37 \times 14 = 83 \text{ cfs}$$

$$Q_5 = 0.67 \times 4.56 \times 14 = 44 \text{ cfs}$$

Combine this flow with off-site flows

$$Q_{100} = 355 + \frac{3.24}{7.37} (465 + 83) = 596 \text{ cfs} \quad \text{SAY } 600 \text{ cfs}$$

$$Q_5 = 281 + 44 + \frac{15}{102} (136) = 345 \text{ cfs} \quad \text{SAY } 350 \text{ cfs}$$

FIRST CUT RCB SIZE
USE 8 ft/sec

$$\frac{600 \text{ cfs}}{8 \text{ ft/s}} = 75 \text{ ft}^2$$

^{would} It appear that existing

3-10'x5' RCB is oversized.



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Project NW COR CENTRAL & WEBB

Item ROUGH HYDROLOGY CHECK

I. Q_{100} computed by MMB per COW drainage manual procedure.
= 600 cfs

II Q_{100} from Proj File 32-82172-1382, Wichita Art Assoc bridge,
= 800 cfs (Reportedly from C.D.W.)

III Rough Rational Method Check:

$$DA = 148 + 24 + 21 + 26 + 14 = 233 \text{ Ac}$$

$$C = [148 \times 0.74 + (24 + 21 + 26)(0.89) + 14 \times 0.80] / 233 = 0.79$$

$$i_{100} = 3.24 \text{ in/hr} \quad T_c = 77 \text{ min}$$

$$Q_{100} = 0.79 \times 3.24 \times 233 = 596 \text{ cfs}$$

IV TR-11 by Ks Div of H₂O Res. (See Sh Nos 3-4)

$$Q_{100} = 83.8 (A_c)^{0.524} (P_2)^{2.529}$$

$$A_c = \frac{233}{640} = 0.36 \text{ mi}^2 \quad P_{2,24} = 3.5 \text{ in}$$

$$Q_{100} = 83.8 (0.36)^{0.524} (3.5)^{2.529} = 1166 \text{ cfs} \quad (+74, -42 \text{ std error})$$

(TR-11 range $0.41 \text{ mi}^2 < DA < 19,260 \text{ mi}^2$)

V FENL-H (See Sh Nos 5-10)

$$I_{mp} = (43 \times 0.25) + (68)(0.45) + (37 + 24 + 2 \times 26)(0.96) + (14)(0.70) / 233 = 0.66$$

MAR = 5 in $X_R = 4.8$

$$k = 0.93$$

$$X_u = X_R (1 - I_{mp} + I_{mp}/k) = X_R (1.05) = 5.03$$

$$T_H = T_R = 1.7 \text{ hr}$$

$$T_H X_u = 8.6$$

$$Z = 0.98$$

$$Q_p = 640 (0.36) (5.03) (0.98) = 1136 \text{ cfs} \quad (\text{NOTE: THIS BASIN IS PROBABLY TOO SMALL FOR THIS METHOD})$$



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Project NW COR CENTRAL & WEBB

Item ROUGH HYDROLOGY CHECK

VI SCS Method TR-55

$Q_{100} = 1139$ cfs

VII Rossmiller Rational

$Q_{100} = 845$ cfs

} See Sh 11-13

HYDROLOGICAL ESTIMATES RANGE FROM 596 cfs TO 1166 cfs.

TWO OF THREE ESTIMATES ON THE HIGH SIDE ARE SUSPECT DUE TO THE SMALL DRAINAGE AREA. THE THIRD (SCS) IS NOTORIOUS FOR OVERESTIMATING.

RECOMMEND BARREL DESIGN FOR $Q_{100} = 600$ cfs W/
PROVISION OF ONE (1) FOOT FREEBOARD.

The final regression computations show that equations using only contributing drainage area and 2-year 24-hour rainfall give results that are nearly as accurate as equations using all statistically significant variables. These equations for Q_N in cfs and their standard errors of estimate are as follows:

	<u>Regression equation</u>	<u>Standard error of estimate</u>
Q_2	$= 0.707 A_c^{0.548} P_2^{4.752}$	+51,-34 percent
Q_5	$= 3.98 A_c^{0.530} P_2^{4.021}$	+50,-33
Q_{10}	$= 9.92 A_c^{0.525} P_2^{3.591}$	+52,-34
Q_{25}	$= 25.6 A_c^{0.524} P_2^{3.127}$	+59,-37
Q_{50}	$= 47.6 A_c^{0.523} P_2^{2.821}$	+66,-40
Q_{100}	$= 83.8 A_c^{0.524} P_2^{2.529}$	+74,-42

The regression equations are for use throughout Kansas, within the limitations described in the next section of this report. Nomographs for these equations are given in figures 2, 3, and 4.

A more complete tabulation of the results of regression computations is given in the appendix.

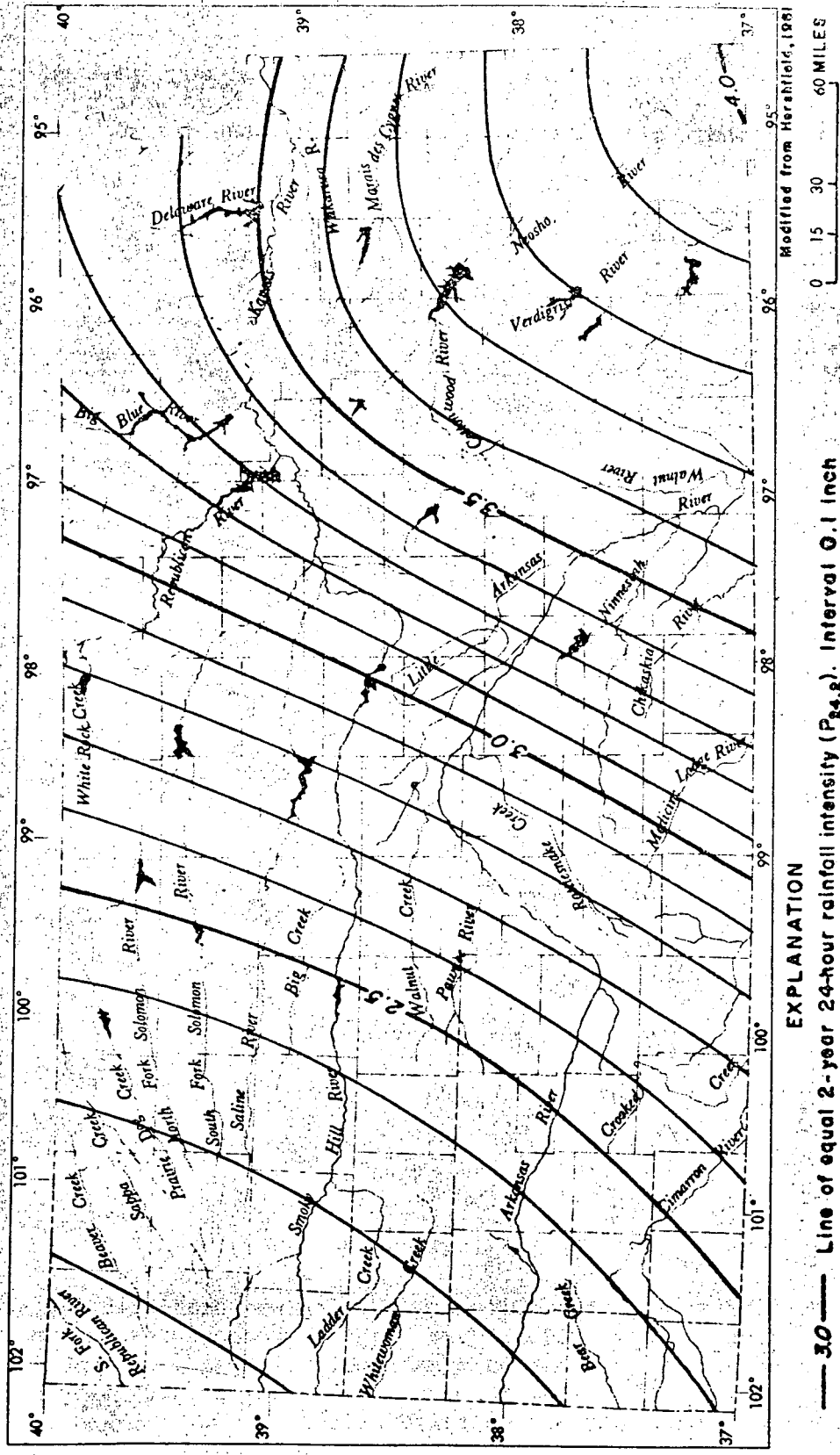
The standard error of estimate shown with each equation indicates the approximate accuracy. The accuracy of 100-year floods calculated from the regression equation is equivalent to the accuracy that would be obtained from about 12 years of record of peak flows at the site.

NOTE: A_c in square miles; P_2 in inches (2-year, 24-hour storm);
 Q in cfs

Limits of Definition

Because the equations were developed from data for streams in Kansas and a very few streams in nearby parts of bordering states, it is important that the equations not be used for streams outside Kansas. In many parts of other states, P_2 is outside the range for which the equations are defined, and hydrologic factors other than P_2 and A_c have effects that are too large to ignore. The range of contributing drainage areas used in the regression calculations was 0.41 to 19,260 sq mi; the scale for A_c in sq mi on figures 2, 3, and 4 extends from 0.4, the lower limit of applicability, to 10,000, which is larger than the contributing-drainage area of any currently unregulated stream in Kansas.

From: "Kansas Stream Flow Characteristics, Magnitude and Frequency of Floods in Kansas, Unregulated Streams" Technical Report No. 11, Kansas Water Resources Board, Feb. 1975. Written by P.R. Jordon & T.J. Izra.



EXPLANATION
 — 3.0 — Line of equal 2-year 24-hour rainfall intensity ($P_{24,2}$). Interval 0.1 inch

Figure 1. Distribution of 24-hour rainfall for 2-year recurrence interval

4/13

1A

FENL-H 5/13

HYDROLOGIC DESIGN UTILIZING FREQUENCY-EQUIVALENT HYDROGRAPHS

by
ROBERT L. SMITH
Deane Ackers Professor
Department of Civil Engineering

The University of Kansas
Lawrence, Kansas 66045
June, 1982

Final Report for Period October 16, 1980 to June 15, 1982
Prepared for
Kansas Department of Transportation

VOLUME I



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Engineering Dept.
City of Hutchinson

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decreases much more rapidly with area in western Kansas than in eastern Kansas [6].

The frequency-equivalent storm reflecting rural conditions as outlined above is presumed to also produce frequency-equivalent hydrographs for the urbanized area. However, the rate of rainfall excess, X , and the hydrograph response time need to be adjusted to account for the impact of adding impervious surface to the watershed area. The general concepts on which these adjustments are made are as follows.

$$X_U = X_R(1 - IMP) + i_R(IMP) \quad (7)$$

where

X_U = rate of rainfall excess in the urban environment,

X_R = rate of rainfall excess in the rural environment,

IMP = the decimal fraction of the area covered by impervious surface, and

i_R = rate of rainfall corresponding to the duration T for the specified i - T frequency relationship.

If the ratio of X_R/i_R is replaced by a coefficient k then equation (7) may be rewritten as

$$X_U = X_R[1 - IMP + IMP/k] \quad (8)$$

The increase in X due to the addition of impervious surface can be expected to shorten hydrograph time response. Normal treatment of this issue modeled as flow on a plane surface, and assuming kinematic wave applicable, suggests

$$T_e \approx X^q \quad (9)$$

with a value of a negative 0.4 or negative 0.5 usually assigned to q . Jundi [7] showed that the power on X in equation (9) would be highly variable for alluvial channels ranging from plus 0.1 to minus 0.8. The results encountered in this study confirm that q is negative, but, on average, its absolute value increases as size of area increases. The apparent reasons for this are discussed in Section III.

From equation (5) the time to equilibrium before insertion of impervious surface is

$$T_{eR} = T_{BR} - T_R \quad (10)$$

Then from equation (9) the time to equilibrium after insertion of impervious surface is

$$T_{eU} = T_{eR} \left(\frac{X_U}{X_R} \right)^q \quad (11)$$

Since T_U is presumed to equal T_R , we may state on the basis of equations (2), (1) and (5) that

$$Z_U = \frac{2 T_R}{T_{eU} + T_R} ,$$

$$Q_U = X_U Z_U A \quad \text{and}$$

$$T_{BU} = T_U + T_{eU} .$$

8/13

SECTION IV - SAMPLE CALCULATIONS

This section presents sample calculations which serve to illustrate the application of the frequency-equivalent method using the design charts presented in the prior section.

Example #1 - Rural Frequency Curves

Assume there is need to determine the flood frequency curve for an eight square mile rural watershed located at the southwest corner of Sedgwick County, Kansas.

1 - Examination of Figure 4 indicates the watershed is located in the 4 inch MAR zone.

2 - Frequency-equivalent storm dimensions, X and T, can be obtained from charts 6 through 17. The values so determined are listed in columns 2 and 3 of the tabulation presented on the next page.

3 - Hydrograph volume expressed in inches of runoff is the product XT and appears in column 4.

4 - The peak reduction factor, Z, may be read from Figure 18 and is tabulated in column 5.

5 - The peak discharge at each return interval is obtained from equation (1) or $Q_p = 640 XZA = 5120 XZ$. Values are listed in column 6.

9/13

(1) Return Interval years	(2) X in/hr	(3) T hr	(4) Vol=XT inches	(5) Z	(6) Q _p cfs
2	0.30	2.30	.69	0.51	783
5	0.55	2.55	1.40	0.64	1802
10	0.72	2.85	2.05	0.73	2691
25	0.95	3.15	2.99	0.80	3891
50	1.16	3.45	4.00	0.85	5048
100	1.38	3.60	4.97	0.87	6147

Example #2 - Urban Frequency Curves

Re-calculate the foregoing frequency relationship assuming the watershed has been urbanized and that IMP, the ratio of impervious area to total area, equals 0.40. The required steps are as follows.

1 - Calculate X_U from equation (8) which states $X_U = X_R[1 - IMP + IMP/k]$ where the values of k may be read from Figure 22. Values of X_R , k and X_U are listed in columns 2, 3 and 4 of the tabulation on the next page.

2 - The underlying assumption that the critical urban storm is equivalent to the critical rural storm means $T_U = T_R$ and T_U is listed in column 5.

3 - The urban hydrograph volume is the product of $T_U X_U$ and appears in column 6.

4 - The urban Z is read from Figure 18 and is posted in column 7.

5 - The urban peak discharge is obtained from equation (1) and $Q_p = 640 A X_U Z_U$. Values appear in column 8.

10/13

(1) Return Interval years	(2) X_R in/hr	(3) k	(4) X_U	(5) T_U hrs	(6) $X_U T_U$	(7) Z	(8) Q cfs
2	0.30	.41	.47	2.30	1.08	.60	1444
5	0.55	.56	.72	2.55	1.84	.70	2580
10	0.72	.65	.88	2.85	2.51	.76	3424
25	0.95	.78	1.06	3.15	3.34	.81	4396
50	1.16	.86	1.24	3.45	4.28	.86	5460
100	1.38	.93	1.42	3.60	5.11	.87	6325

Comment

The discharge values as computed for the rural and urban conditions specified in Examples #1 and #2 have been plotted in Figure 23. In each instance a smooth curve has been drawn through the plotted points and this procedure is recommended. The slight scatter encountered is due to graphical inaccuracies in chart construction and/or human error in reading the charts.

The urban analysis as presented above accounts only for the impact of impervious additions. It corrects for the added volume and for the shortened time response associated with an increased rate of rainfall excess. It does not allow for any significant change in channel alignment or reduction in channel resistance characteristics often experienced in urban area. In instances where major channel improvements are identifiable an additional correction may be desired. This issue is illustrated in the next example problem.

1/13

Computation of Peak Discharge

Date: 06-27-1986

Title: nw cor central & webb-5 yr

Rainfall 4.5 inches
Recurrence Interval (yrs.) 5
Runoff Curve Number 95
Hyd. Len. 4900 feet
Slope 1.5 %
% of HLM 40 %
% Imp. 66 %
Area of basin 233 acres

Computed Data

Basic Lag Factor (hrs.) 0.52
Hydr. Length Adj. 0.87
Imp. Area Adj. 0.83
Runoff Volume (in.) 3.92
Computed Time of Conc. (hrs.) 0.64

Peak Discharge by Technical Release No. 55 (1975)

Peak Dis. (cfs) = 620.70
Csm/in. = 434.41
Tc (hrs.) = 0.64

Peak Discharge by Modified Rational Formula (Rossmiller)
Discharge for 5 year freq.

Time of Conc. (Tc) = 38.34 minutes
Intensity = ~~3.25~~ 2.64 inches/hr.
C factor = 0.63
Peak Dis. (cfs) = ~~473.19~~ 417

12/13

Computation of Peak Discharge
Date: 06-27-1986

Title: na cor central & webb--10 yr

Rainfall 5.3 inches
Recurrence Interval (yrs.) 10
Runoff Curve Number 95
Hydr. Len. 4900 feet
Slope 1.5 %
% of ILM 40 %
% Imp. 66 %
Area of basin 233 acres

Computed Data

Basic Lag Factor (hrs.) 0.52
Hydr. Length Adj. 0.39
Imp. Area Adj. 0.83
Runoff Volume (in.) 4.72
Computed Time of Conc. (hrs.) 0.64

Peak Discharge by Technical Release No. 55 (1975)

Peak Dis. (cfs) = 745.98
Csm/in. = 434.41
Tc (hrs.) = 0.64

Peak Discharge by Modified Rational Formula (Rossmiller)
Discharge for 10 year freq.

Time of Conc. (Tc) = 33.34 minutes
Intensity = 3.31 ~~3.78~~ inches/hr.
C factor = 0.65
Peak Dis. (cfs) = 501 ~~575.51~~

13/13

Computation of Peak Discharge
Date: 06-25-1986

Title: nw cor central & webb

Rainfall 7.8 inches
Recurrence Interval (yrs.) 100
Runoff Curve Number 95
Hyd. Len. 4900 feet
Slope 1.5 %
% of HLM 40 %
% IMP. 66 %
Area of basin 233 acres

Computed Data

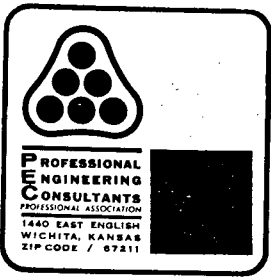
Basic Lag Factor (hrs.) 0.52
Hydr. Length Adj. 0.87
IMP. Area Adj. 0.83
Runoff Volume (in.) 7.20
Computed Time of Conc. (hrs.) 0.64

Peak Discharge by Technical Release No. 55 (1975)

Peak Dis. (cfs) = 1,137.03
Csm/in. = 434.41
Tc (hrs.) = 0.64

Peak Discharge by Modified Rational Formula (Rossmiller)
Discharge for 100 year freq.

Time of Conc. (Tc) = 38.34 minutes
Intensity = 4.79 ~~5.61~~ inches/hr.
C factor = 0.76
Peak Dis. (cfs) = 848 ~~987.73~~

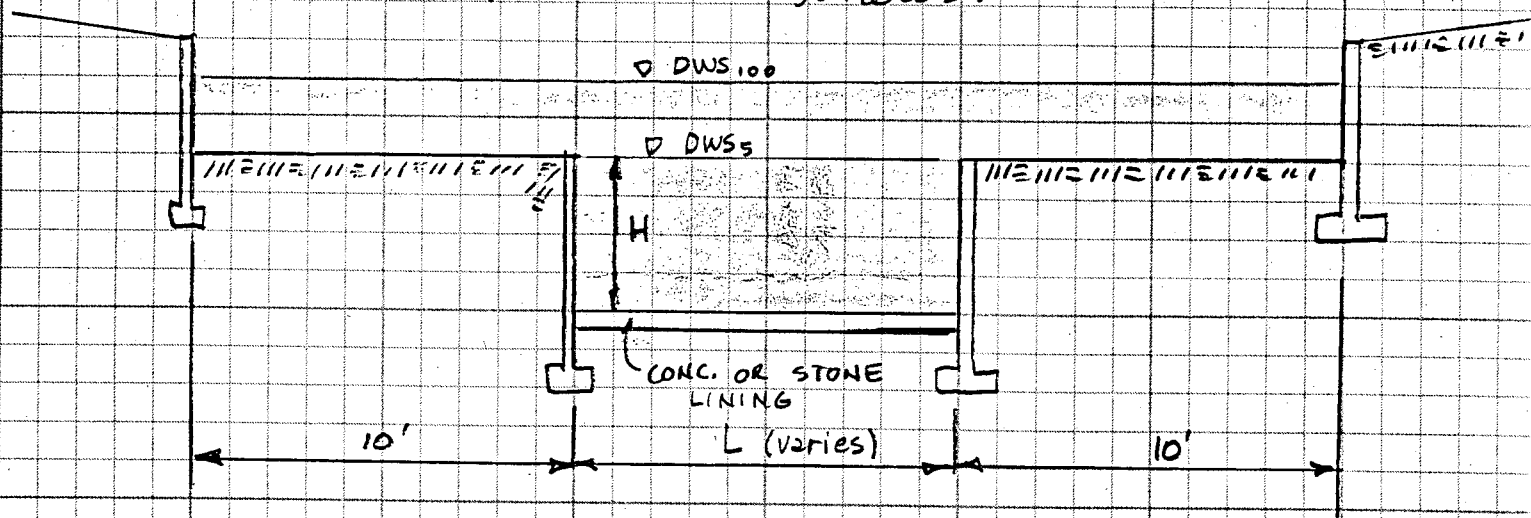


Date 4-20-89 Page 1 of 2

Project Corporate Lakes

Item Drainage Plan 5-Year Analysis

The development plan proposes 3 lakes within the project. The static pools of these 3 ponds will be controlled by the use of weirs. The west pond would be controlled via a weir on the upstream side of an RCB. An approximate cross-section is as follows:



CONFINE 5-YR FLOW (480 cfs to small channel)
 DETERMINE L by using weir equation $Q = CLH^{3/2}$
 $480 = 3.0 \times L \times H^{3/2}$
 $LH^{3/2} = 160$

w/:	L = 20'	H = 4.0'	
	L = 25'	H = 3.4'	← USE
	L = 30'	H = 3.1'	



Date 4.21.89 Page 1 of 6

Project Corporate Lakes

Item Drainage Plan 100-yr analysis

POND NO. 1 controlled by RCB @ W. E.

$$Q_{100} = 850 \text{ cfs.}$$

Assume Box $D = 6'$

Allowable $HW = 7'$

$$HW/D = 7/6 = 1.17$$

$$\therefore Q/B = 51 \text{ cfs}$$

$$W/Q = 850, B = 16.66$$

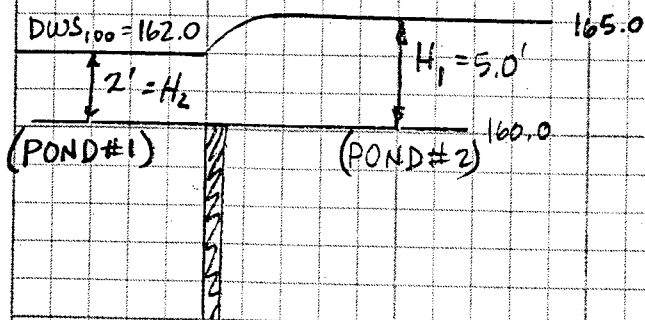
USE 2-9' x 6' RCB

$$DWS = E + 7' = 162.0$$

@ Pond #1

POND NO. 2 controlled by Weir.

During 100-yr storm, this weir is submerged by backwater from RCB (Pond No. 1)

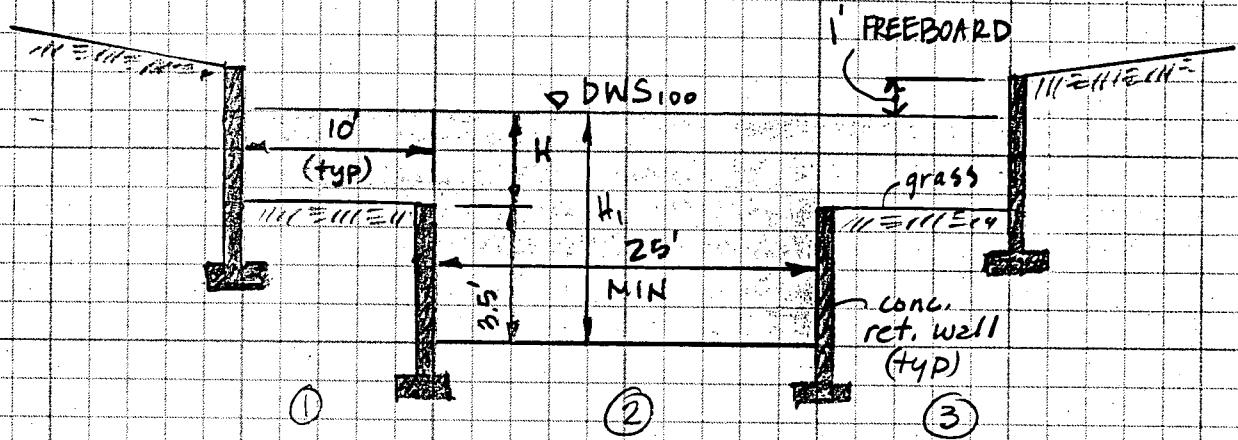




Date April 20, 1989 Page 2 of 6

Project Corporate Lakes

Item Drainage Plan 100-yr. analysis



SECTION @ WEIR

w/ $H_1 = 5$ $H = 1.5$

$$Q_{①} + Q_{③} = c L H^{3/2} = 3.0 \times 10 \times 1.5^{3/2} = 55 \times 2 = 110$$

$$Q_2: \frac{H_2}{H_1} = \frac{2'}{5} = 0.4 \quad \therefore \frac{Q}{Q_1} = 0.88$$

Normal Q in this section =

$$3 \times 25' \times 5^{3/2} = 838$$

$$\text{Submerged flow} = 0.88 \times 838 = 737$$

$$\begin{array}{r} \text{Total Flow} = 737 \\ + 110 \\ \hline 847 \end{array}$$

OK... (design Q = 850)

$$\begin{aligned} \text{DWS}_{100} \text{ pond 2} &= \text{Weir Elev.} + 5' = \\ &160 + 5 = 165.0 \end{aligned}$$

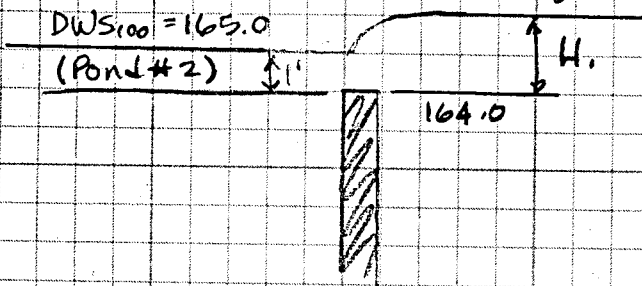


Date April 21, 1989 Page 3 of 6

Project Corporate Lakes

Item Drainage Plan 100-yr. analysis.

POND #3 - controlled by weir (see section on p.2)
 - during 100-year storm, center section of weir is submerged.



Assume $H_1 = 5'$ $H = 1.5'$

$$Q_1 \& Q_2 = CLH^{3/2} = 30 \times 10 \times 1.5^{3/2} = 55 \text{ cfs} \times 2 = 110 \text{ cfs}$$

$$Q_2 : \frac{H_2}{H_1} = \frac{1}{5} = 0.2 \quad \therefore Q/Q_1 = 0.96$$

$$\text{Normal } Q \text{ in center section} = CLH^{3/2} = 3 \times 25 \times 5^{3/2} = 838$$

$$\text{Submerged flow in center section} = 838 \times 0.96 = 804$$

$$\begin{array}{r} \text{Total flow} = 804 \\ + 110 \\ \hline 914 \end{array} \quad \text{OK} \quad (\text{design } Q = 850)$$

$$DWS_{100} \text{ Pond No. 3} = \text{Weir} + 5' = 164.0 + 5 = 169.0$$



Date 4-21-89 Page 4 of 6

Project Corporate Lakes

Item Drainage Plan 100-yr. analysis

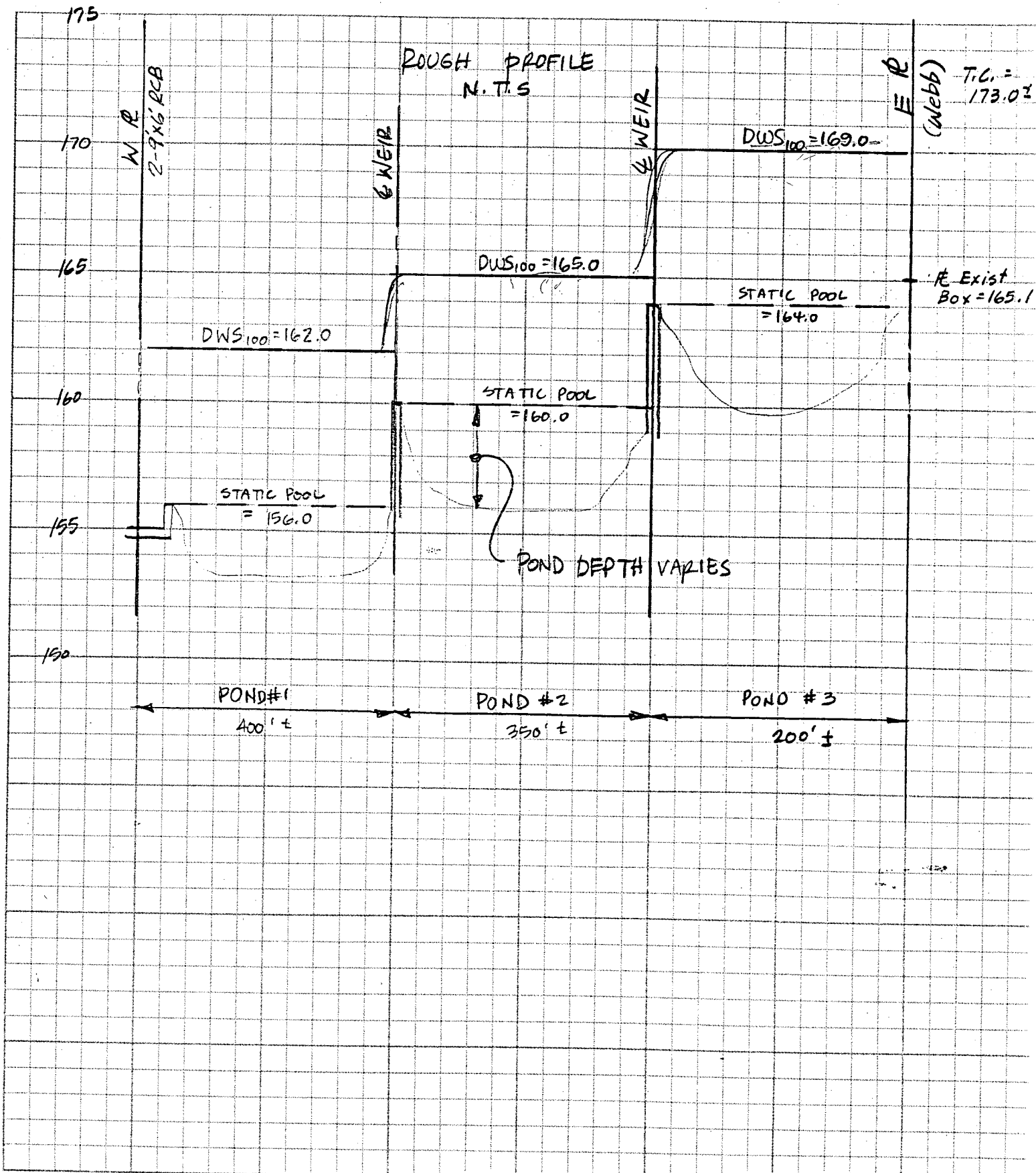
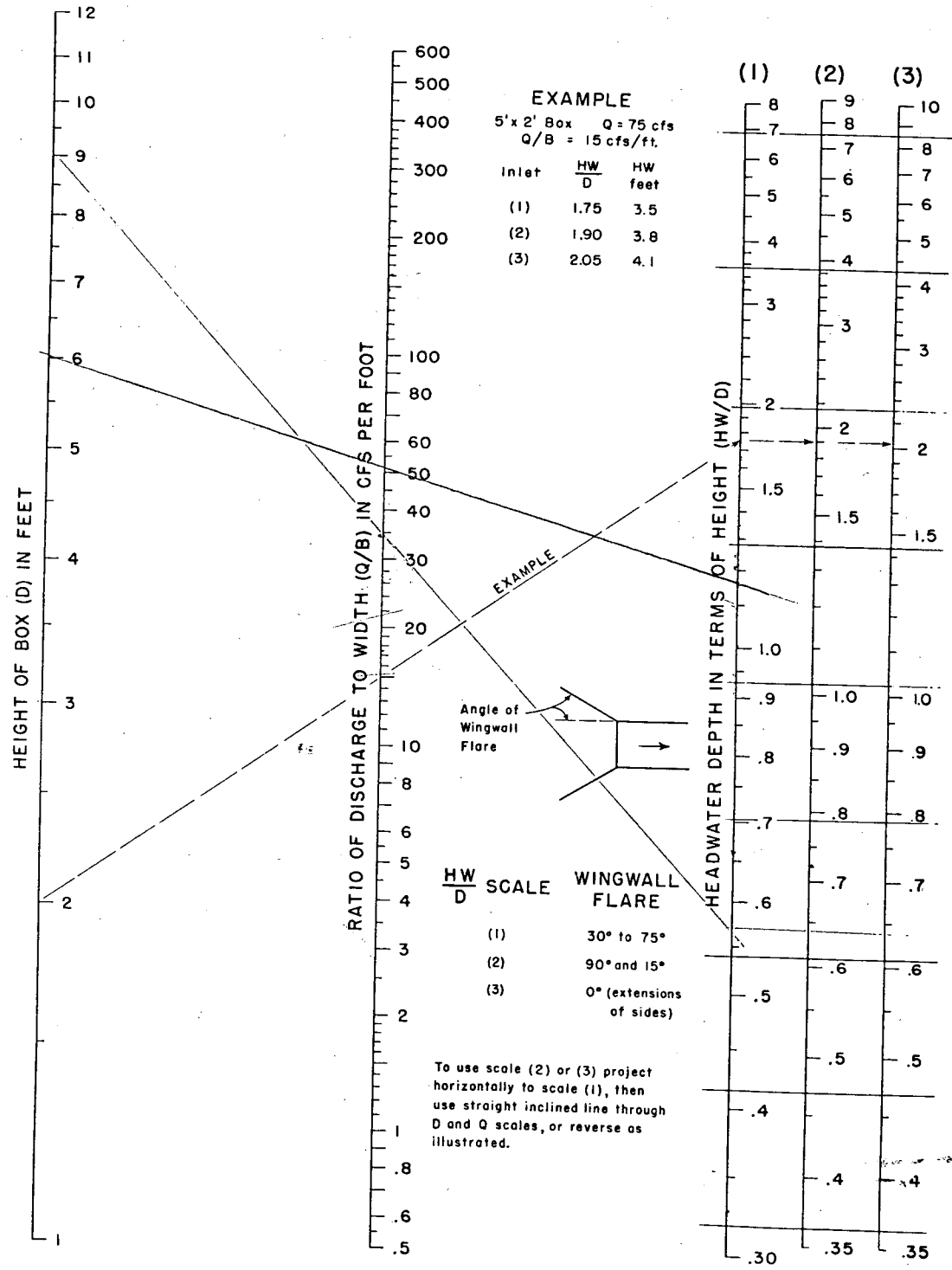


CHART I



HEADWATER DEPTH FOR BOX CULVERTS WITH INLET CONTROL

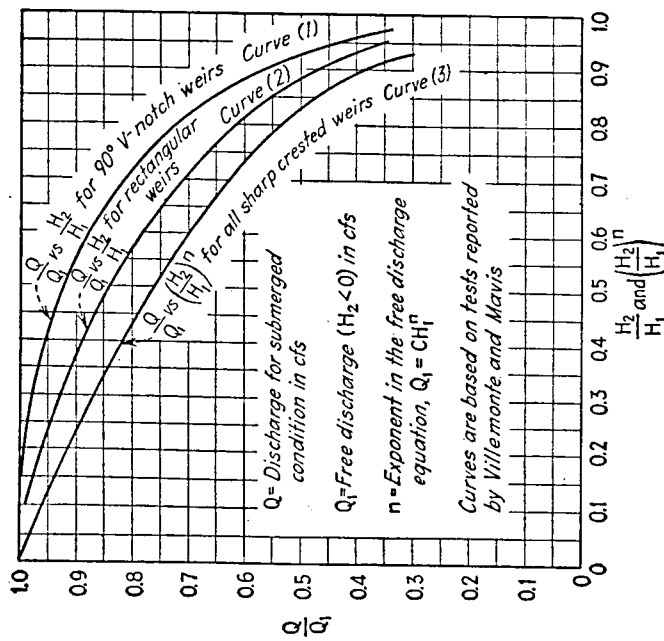
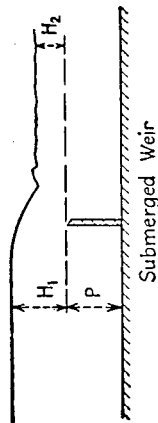


Fig. 5-5

the upstream side of the weir, H_1 , but also to the head on the downstream side, H_2 , and to a lesser extent, to the height of the weir crest above the floor of the channel, P . Early experiments by Francis (1848, 1883), Fteley and Stearns (1882), Bazin (1894), Cone (1916), and Cox (1928) have been summarized by Vennard and Weston.¹ They showed that the various test data could be presented in an orderly manner by selecting as variables Q/Q_1 and H_2/H_1 , where Q_1 is the discharge at the head H_1 , computed from the equation for free discharge (unsubmerged), which is expressed in general terms as follows:

$$Q_1 = CH_1^n \quad (5-49)$$

By plotting Q/Q_1 against H_2/H_1 , they found that the various data tended to fall on a single curve, except for small values of P/H_1 .

In 1947, Villemonte² presented the results of a series of tests on submerged (sharp-crested) weirs. He conducted tests on rectangular, triangular, parabolic, cusped, and proportional weirs. He showed that the results for all types could be represented by the single equation

$$\frac{Q}{Q_1} = \left[1 - \left(\frac{H_2}{H_1} \right)^n \right]^{0.385} \quad (5-50)$$

where n is the exponent in the free-discharge equation [Eq. (5-49)], and the other terms are as previously defined. This equation was found to satisfy all test results, with a maximum deviation of 5 per cent for some of the individual test results.

In 1949, Mavis³ presented results of tests on rectangular, triangular, parabolic, circular, sutor, and cusped weirs. He found that a single equation could be used to express the results for all the tests. His equation with subscripts changed to conform with usage in this book is

$$\frac{Q}{Q_1} = 1 - \left(0.45S + \frac{0.40}{2(10-10S)} \right) \quad (5-51)$$

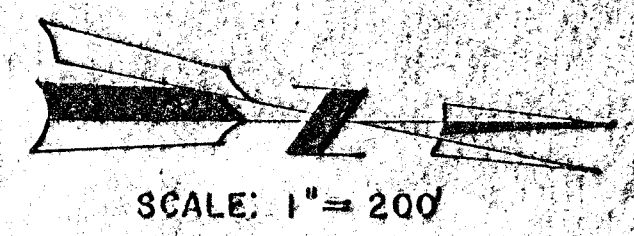
¹ John K. Vennard and Ray F. Weston, Submergence Effect on Sharp-crested Weirs, *Eng. News-Record*, June 3, 1943, p. 818.

² James R. Villemonte, Submerged-weir Discharge Studies, *Eng. News-Record*, Dec. 25, 1947, p. 866.

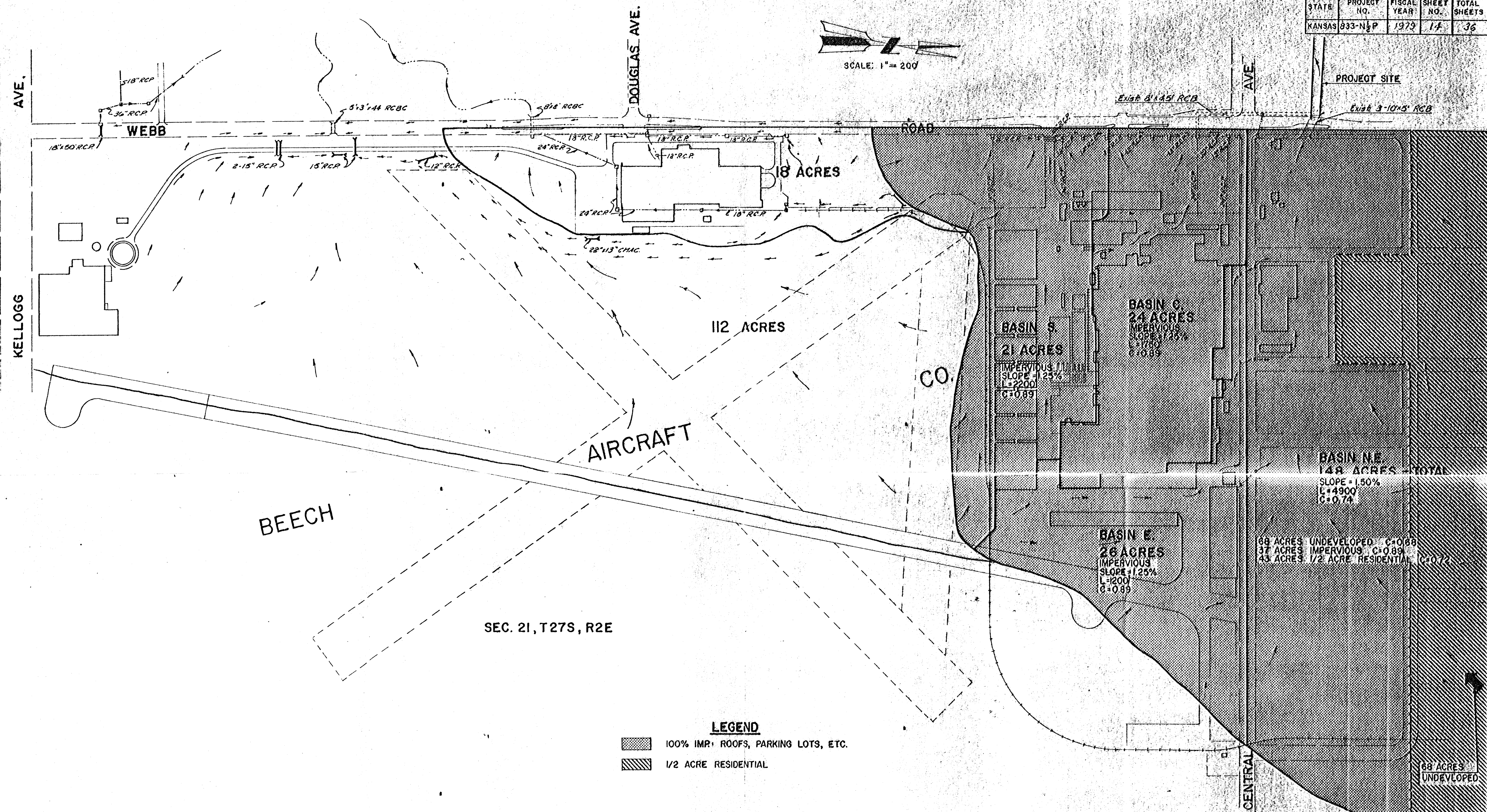
³ F. T. Mavis, How to Calculate Flow over Submerged Thin-plate Weirs, *Eng. News-Record*, July 7, 1949, p. 65.

6/6

STATE	PROJECT NO.	FISCAL YEAR	SHEET NO.	TOTAL SHEETS
KANSAS	833-N&P	1979	14	36



PROJECT SITE
 EXIST. 6" X 48" RCB
 EXIST. 3" X 10" RCB



LEGEND
 [Hatched Box] 100% IMP. ROOFS, PARKING LOTS, ETC.
 [Diagonal Lines Box] 1/2 ACRE RESIDENTIAL

NOTE: ENTIRE BASIN IS SCS HYDROLOGIC SOIL GROUP "D"

**DRAINAGE MAP
 RD 833-P**

PREPARED BY
 SEDGWICK COUNTY DEPARTMENT OF PUBLIC WORKS
 HIGHWAY DIVISION

6-27-86	1	PEC	Drainage Comp. - NW Cor. Webb & Central
DATE	REV	BY	FOR

REVISED	SCALE	DESIGNED	TRACED	CHECKED	SHEET NO.
	1" = 200'	JWR	JWR	PEB	14
	DATE:	7/78	9/78	3/79	
	PLANFILE	TOTAL SHEETS			

