EMERGENCY REPAIRS
GOLF COURSE ROAD
RIO RANCHO, NEW MEXICO

EMERGENCY DRAINAGE REPORT

SEPTEMBER 1988

PREPARED FOR
THE
CITY OF RIO RANCHO, NEW MEXICO

Portions Revised July 1991

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Gannett Fleming

EMERGENCY REPAIRS FOR GOLF COURSE ROAD

EMERGENCY DRAINAGE REPORT

I. **AUTHORITY:**

This emergency storm drainage report is a direct result of the City Counsel - City of Rio Rancho vote on September 14, 1988, to affect interim-emergency repairs on Golf Course Road between Southern Boulevard and the Sandoval/Bernalillo County Line necessary due to damage to the road section from storm runoff during the evening hours of September 13, 1988. Interim repairs are specified as follows:

1. Repair damaged and destroyed portions of the pavement sections.

2. Correct "stopping sight distance" problems present in the existing grade.

3. Overlay the existing pavement to facilitate a "one way crown" section sloping to the west.

4. Repair storm drainage "ditch" on the west side of Golf Course by installing a paved channel section and repairing culverts damaged by storm runoff (constraints - avoid existing water line at west right-of-way, remain within the existing 50-foot right-of-way).

II. **PURPOSE AND SCOPE:**

The purpose of this preliminary storm drainage report is to present an interim plan for drainage facilities to handle storm water from Golf Course Road and the immediate vicinity during a low intensity event. The area under consideration is bounded on the north by Southern Boulevard and on the south by the Sandoval/Bernalillo County Line.

III. PRESENT CONDITIONS:

West Side - The drainage basins west of Golf Course Road are mostly undeveloped with the south-west corner of Golf Course Road and Southern Boulevard and occasional development along the balance of Golf Course Road. Present drainage patterns are generally toward the east and south in an overland sheet flow. The roadway template intercepts this sheet flow and concentrates it in a shallow earthen swale section flowing generally to the south. Blocked or damaged culverts at 13th Avenue and 17th Avenue force flows collected from the north of each location to turn west and follow the unpaved streets (13th & 17th) until it is discharged into the east fork of Black's Arroyo. Flows collected south of 17th Avenue continue flowing to the south until it reaches adverse grade approximately at 21st Avenue where they flow across Golf Course Road to the east side where it continues to flow south along the pavement shoulder. Accumulated flow collected south of the crest (approximately 21st Avenue) continues to flow south along the west edge of pavement on Golf Course Road until it begins to spread to the south and west approximately 500 feet north of the County Line.

East Side - In contrast to the west side of Golf Course Road, the east side is mostly developed. Presently, the City of Rio Rancho project - Special Assessment District No. 3 is ongoing developing the infrastructure east of Golf Course Road along the entire length of this project. The storm drainage design for SAD No. 3, as developed by Wilson and Company, collects storm runoff and conveys it generally to the south and east until it is ultimately discharged (controlled discharge from a detention pond of 14 cfs) into 22nd Avenue where it is confined within the street section by curb and gutter and flows west to Golf Course Road. This runoff sheet flows across Golf Course Road to the west side. Additional flow is collected at Golf Course Road from 23rd Avenue which conveys storm runoff, confined by curb and gutter, from an area south of 22nd Avenue to the County Line (approximately 8 acres). The runoff flows south along the east edge of pavement until it reaches the county line where it turns to the southwest across Golf Course Road and begins to spread at about 45^{o} (southwest) to the centerline of Golf Course.

IV. FLOWS DURING STORM

On the afternoon of September 13, 1988, a rain storm occurred in the vicinity of Golf Course Road. The intensity was sufficient to cause ditch erosion and pavement undermining on both the east and west sides of the roadway section beginning approximately 400-feet north of 16th Avenue and ending at 23rd Avenue. The most extensive damage to the pavement section occurred between 22nd and 23rd Avenue where the west half of the southbound lane was undermined by depths of 2-1/2 to 6 feet. This section of pavement subsequently collapsed. The city closed Golf Course Road at that time and began filling and compacting voids on the morning of September 14, 1988.

A. **Improvements Completed Since 1988**

SAD 3 to the west has been completed such that the only contributions from east of Golf Course Road occur at 22nd Avenue and at 23rd Avenue. Also SAD 3 installed "water blocks" to eliminate runoff from the west entering the subdivisions east of Golf Course Road. The exception is 20th Avenue where runoff is allowed to surface flow from Golf Course Road east into 20th Avenue and allowed to flow through two 24-inch culverts under Golf Course and along the pavement shoulder.

When the pavement section washed out in 1988, a shallow "vee" channel was constructed along the west pavement shoulder of Golf Course Road from 17th Avenue to the County Line. This channel has limited capacity but is effective in helping to control erosion and protect the pavement section on Golf Course Road. No major vertical realignments were incorporated into the emergency roadway repair at that time.

Previously, runoff from the south side of Southern Boulevard between Golf Course Road and Nicklaus Drive was allowed to flow south, via 29th Street and 11th Avenue, into Golf Course. This runoff has been intercepted on Southern Boulevard, thereby eliminating its contribution to Golf Course Road at 11th Avenue.

V. PROPOSED STORM DRAINAGE IMPROVEMENTS (Updated June 1991)

A. **Proposed East Side Improvements**

The proposed typical section for Golf Course Road is a standard center crown with two 12-foot lanes, curb and gutter on the east and a 2-foot shoulder on the west.

Due to the crown, the east side curb and gutter, and the water blocks constructed on the intersecting roads by SAD 3, runoff collected in this side of Golf Course Road is confined there. To alleviate this channel effect, we propose to place longitudinal slot drains in the east gutter flowline at Ann Circle and at 23rd Avenue. The lateral pipe from the slot drains will be taken under Golf Course Road and discharged in the west roadside ditch. Runoff will also be turned into 20th Avenue where it will discharge into the existing detention pond east of Golf

Course Road between 20th and 21st Avenues. This additional flow into the pond is offset by the removal of the two existing 24-inch culverts under Golf Course Road. These two culverts currently convey flows form the west side of Golf Course Road to the pond.

The pond east of Golf Course Road (between 20th and 21st Avenues) discharges at a controlled rate of 14 cfs into 22nd Avenue where flows are conveyed west into Golf Course Road. We propose the installation of two "double-c" inlets in the returns on 22nd Avenue at Golf Course. A lateral will direct the flows under Golf Course Road into the west roadside channel.

Additional runoff is concentrated in 23rd Avenue from the watershed between 22nd and 23rd Avenues and east of Golf Course Road (Drainage Basin 7). This runoff flows west in 23rd Avenue to Golf Course Road. We propose to place two "double-c" inlets on 23rd Avenue east of the returns at Golf Course Road. A water block will also be installed between the inlets and Golf Course Road to contain the runoff before it crosses Golf Course Road on the surface. The lateral pipe from the "double-c" inlets will convey the intercepted flows under Golf Course Road into the new detention ponds west of Golf Course Road.

B. <u>Proposed West Side Improvements</u>

As discussed in the section above, the west side of the Golf Course Road typical section south of 11th Avenue consists of a 2-foot wide pave shoulder. In order to control the runoff, we have proposed a shallow paved "vee" channel along the roadside. The channel configuration is a 6:1 slope for 14 feet from the edge of shoulder to the flowline, then a 1:1 slope from the flowline up to existing grade. The channel will divert flow from up to a 10 year to the west at 13th and 17th Avenues to discharge into the graded street network. These two diversions are existing and Gannett Fleming West, Inc., was directed by the City Engineer to utilize and provide erosion control for them. The City Engineer informed Gannett Fleming West, Inc., that although the diverted flow may intrude into private property, the City intends to masterplan the area between Golf Course Road and the Black Arroyo and will provide easements or storm sewers to convey the 13th and 17th Avenues flow to the arroyo as part of the masterplan.

South of 17th Avenue the roadside channel will convey the flows south to discharge into the proposed new detention ponds at approximately 23rd Avenue. Throughout the length of the west roadside channel, two parallel 20-inch span by 28-inch rise culverts will be placed under roads and turnouts. The ends of the culverts will be treated with bar grates to keep a car's wheels from dropping into the culvert ends.

The west roadside channel will have the capacity to convey runoff from low intensity, frequent storms. Higher intensity storm runoff will expand beyond the channel confines and flow into the southbound lane of Golf Course Road. The encroachment into the roadway will be for a short duration and after the peak rainfall has subsided.

C. Proposed Golf Course Detention Pond

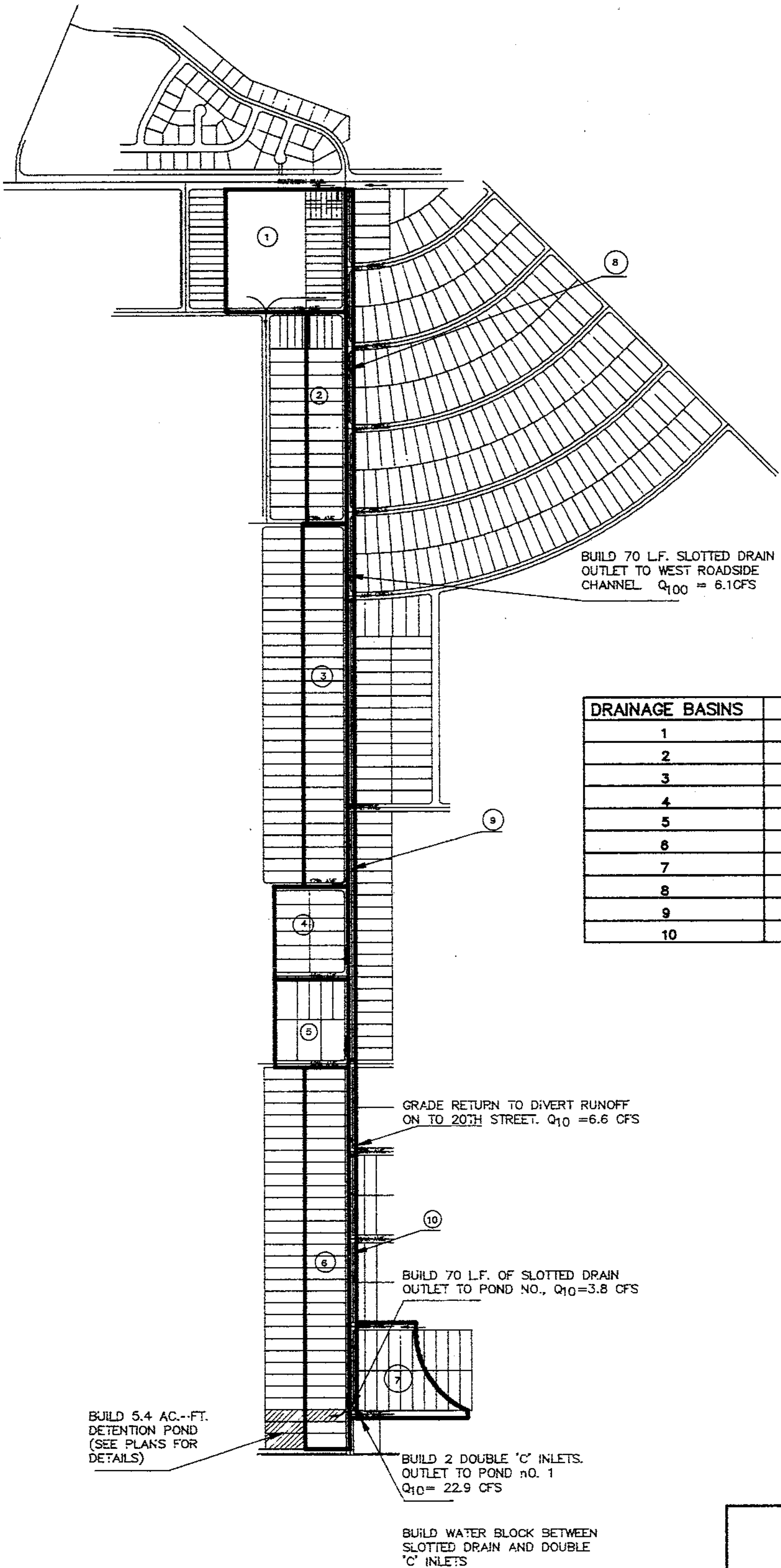
During development of Rehabilitation of Golf Course Road in 1988, several meetings were held with AMAFCA and City of Albuquerque personnel to discuss methods of conveying runoff to the Black Arroyo. The outcome from these meetings was generally that the City of Albuquerque and AMAFCA had no plans to improve Golf Course Road or the Black Arroyo for several years. Therefore, the City of Rio Rancho could not expect any assistance from AMAFCA or the City of Albuquerque. After reviewing several methods of controlling the runoff, the City of Rio Rancho directed Gannett Fleming West, Inc., to design a detention pond to reduce the peak discharge rates prior to discharging into the Black Arroyo at the Sandoval/Bernalillo County Line.

Due to the previously plated lot configuration and sloping terrain, we proposed serial pond or cascading pond concept. This method allows the ponds to "step" down the slope and allows the use of less expensive (off of Golf Course Road) lots.

The pond design provides 5.4 acre/feet of available storage. The calculated runoff totals 4.4 acre/feet. The additional 1 acre/foot of storage provides approximately 1 foot of freeboard above high water from a 100 year storm. The current pond design (July 1991) does not provide for additional runoff to be diverted into the facility. Therefore, all existing diversions upstream of the ponds must remain and be functional or the ponds will be overtopped. A routing has been included in Appendix D to show the effect of removing the diversions at 13th and 17th Avenues.

VI. METHODOLOGY

The drainage areas contributing runoff to Golf Course Road were delineated on an enlarged scale (1"=500') USGS topographic quad map and field checked. The drainage aerial map is included as Plate 1.



DRAINAGE BASINS	AREA (AC.)	Q_{100} (CFS)	Q_{10} (CFS)	Q_5 (CFS)
1	25.25	78.8	53.6	43.3
2	9.81	34.3	23.3	18.9
3	16.90	58.4	39.7	32.1
4	8.26	27.6	18.8	15.2
5	8.26	27.6	18.8	15.2
6	17.7	62.5	42.5	34.4
7	8.25	33.7	22.9	18.5
8	2.38	9.0	6.1	5.0
9	3.19	12.0	8.2	6.6
10	1.48	5.6	3.8	3.1

NO.	DESCRIPTION	DATE	BY

REVISIONS

CITY OF RIO RANCHO

GOLF COURSE ROAD IMPROVEMENTS

DRAINAGE BASINS

Gannett Fleming West, Inc.
ALBUQUERQUE, NEW MEXICO

ENGR'S FILE NO.	DRAWN: JLG & AFV	DATE: JULY, 1991
25596	CHECKED: WLB	SCALE: 1"=50'

7-23-91 GOLFDRN (1"=500')

SHEET ____ OF ____

The runoff volumes and associated hydrographs were generated utilizing the August 1988 draft of the revision of Section 22.2 of the City of Albuquerque Development Process Manual. Specifically, the Rational method as modified by Richard J. Heggan and presented in the August 1988 Draft Revision. A copy of this document is included in Appendix C. Hydrographs for drainage areas contributing runoff to the proposed detention ponds were routed using the lag time method to avoid the conservative approach of adding "peak-on-peak". However, no in route storage was considered due to the short distances between drainage areas and relatively small west roadside channel.

The percentage of land in each of the land treatments (see Appendix C, page 3) categories was based on visual observation in areas that were developed and on existing zoning in undeveloped areas.

The series detention pond were designed utilizing the Storage Indication of Modified Puls method as described in "Introduction to Hydrology" by Viessman, Knapp, Lewis, and Harbaugh, 1977.

The resulting calculations are included in the appendices of this report. The results of this study and design specifics are presented in the July 1991 pan set for the project.

APPENDIX A

HYDROLOGY & INLET SIZING

BY [initials] DATE 3/20/90 CHKD. BY [initials] DATE 3/20/90

SUBJECT GOLF COURSE ROAD 13 & 17 TURNOUT SUPER ELEV CALCS

JOB. NO. 25596

VOLUME OF RUNOFF @ 13th STREET TURNOUT

From Golf Course Intersection Drainage Report dtd March 1990

@ 11th Street Q_{100} = 28.9 cfs

From Golf Course "Emergency Repairs" Report dtd Sept 1988

Basin 2 (11th to 13th) Q_{100} = 34.3 cfs

TOTAL @ 13th Q_{100} = 63.2 cfs

VOLUME OF RUNOFF @ 17th STREET TURNOUT

FROM GOLF COURSE (SEPT 1988)

BASIN 3 Q_{100} = 55.8 cfs

TOTAL @ 17th Q_{100} = 58.4 cfs

13TH STREET

IN → Q_{100} = 63.2 cfs   V = 11.3 fps   d = 1.2 ft

(S = 0.0346 '/ft)

$$f_r = \frac{V}{\sqrt{gd}} = \frac{11.3}{\sqrt{32.2 \times 1.2}} = 1.8 > 1 \therefore \text{Super Critical}$$

OUT → $V = 9.3$ fps $d = 0.93$ ft ($S = 2.0\%$)

$f_r = \frac{9.3}{\sqrt{32.2 \times .93}} = 1.7 > 1$ Supercritical

IN → $S = 1.3 \times \frac{V^2 (b + 2zD)}{gr}$ (DPM 22.3 Pg 59)

$= 1.3 \times \frac{(11.3)^2 (0 + 2(3)(1.2))}{32.2 \times 50'}$

$S = 0.74$ ft

OUT $S = 1.3 \times \frac{(9.3)^2 (5 + 2(3)(.93))}{32.2 \times 50'}$

$S = 0.57'$

$D_{in} = 1.2' + 0.74' = 1.94$ ft
$D_{out} = 0.93' + 0.57' = 1.50$ ft
$< 2.33'$ OK

Min freeboard $= 2.33' - 1.94' = 0.39$ ft

17th STREET

IN → $Q_{100} = 58.4$ cfs $V = 8.0$ fps $d = 1.4$ ft ($S = 1.48\%$)

$f_r = \frac{8.0}{\sqrt{32.2 \times 1.4}} = 1.2 > 1$ ∴ Supercritical

Out → $Q_{100} = 58.4$ cfs $V = 5.6$ fps $d = 1.3$ ft ($S = 1.0\%$)

BY	DATE	SUBJECT	SHEET NO. 3 OF 5
CHKD. BY	DATE		JOB. NO.

$$f_r = \frac{5.6}{\sqrt{32.2 \times 1.3'}} = 0.87 < 1 \therefore \text{Subcritical}$$

$$S_{IN} = 1.3 \times \frac{V^2 (b + 2zD)}{gr}$$

$$= 1.3 \times \frac{(8.0^2)(0 + 2(6)(1.4))}{32.2 \times 50'}$$

$$= 0.87 \text{ ft}$$

$$S_{OUT} = 1.15 \frac{V^2 (b + 2zD)}{2gr}$$

$$= 1.15 \times \frac{(5.6)^2 (5 + 2(3)(1.3))}{2(32.2)(50)}$$

$$= 0.14 \text{ ft}$$

$$d_{in} = 1.4 \text{ ft} + 0.87 \text{ ft} = 2.27 \text{ ft}$$

$$d_{out} = 1.3 \text{ ft} + 0.14 \text{ ft} = 1.44 \text{ ft}$$

$$\text{min freeboard} = 2.33 \text{ ft} - 2.27 \text{ ft} = 0.06 \text{ ft}$$

17th @ Golf Course
4 of 5

PROJECT : GOLF COURSE 03-21-1990

slope 1 = 6 :1 slope 2 = 1 :1 bottom = 0
Manning coeff. = .017 Slope = .0148 ft/ft
Area = 6.950938 sq ft Perim = 10.56511 ft Hyd. Rad = .6579147
Depth= 1.409249 ft Velocity= 8.032937 fps Capacity= 55.83645 cf

PROJECT : GOLF COURSE 03-21-1990

slope 1 = 3 :1 slope 2 = 1 :1 bottom = 5
Manning coeff. = .025 Slope = .01 ft/ft
Area = 10.01254 sq ft Perim = 11.00876 ft Hyd. Rad = .909507
Depth= 1.312961 ft Velocity= 5.578005 fps Capacity= 55.84999 cf

PROJECT : GOLF COURSE 03-21-1990

slope 1 = 3 :1 slope 2 = 1 :1 bottom = 5
Manning coeff. = .025 Slope = .02 ft/ft
Area = 7.825941 sq ft Perim = 9.988249 ft Hyd. Rad = .7835148
Depth= 1.089972 ft Velocity= 7.138475 fps Capacity= 55.86528 cf

PROJECT : GOLF COURSE 03-21-1990

slope 1 = 3 :1 slope 2 = 1 :1 bottom = 5
Manning coeff. = .025 Slope = .03 ft/ft
Area = 6.854874 sq ft Perim = 9.502428 ft Hyd. Rad = .7213813
Depth= .9838167 ft Velocity= 8.271988 fps Capacity= 56.70344 cf

PROJECT : GOLF COURSE 03-21-1990

slope 1 = 3 :1 slope 2 = 1 :1 bottom = 5
Manning coeff. = .025 Slope = .04 ft/ft
Area = 6.134355 sq ft Perim = 9.126478 ft Hyd. Rad = .6721492
Depth= .9016686 ft Velocity= 9.109842 fps Capacity= 55.88301 cf

PROJECT : GOLF COURSE 03-21-1990

slope 1 = 3 :1 slope 2 = 1 :1 bottom = 5
Manning coeff. = .025 Slope = .05 ft/ft
Area = 5.675457 sq ft Perim = 8.879364 ft Hyd. Rad = .6391738
Depth= .8476721 ft Velocity= 9.847555 fps Capacity= 55.88937 cf

13th @ Golf Course

5 of 5

PROJECT : GOLF COURSE	03-21-1990	
slope 1 = 6 :1	slope 2 = 1 :1	bottom = 0
Manning coeff. = .017	Slope = .0346 ft/ft	
Area = 5.420396 sq ft	Perim = 9.329699 ft	Hyd. Rad = .580983
Depth= 1.244462 ft	Velocity= 11.30049 fps	Capacity= 61.25315 cfs
PROJECT : GOLF COURSE	03-21-1990	
slope 1 = 3 :1	slope 2 = 1 :1	bottom = 5
Manning coeff. = .017	Slope = .01 ft/ft	
Area = 8.126636 sq ft	Perim = 10.13428 ft	Hyd. Rad = .801896
Depth= 1.121881 ft	Velocity= 7.539265 fps	Capacity= 61.26887 cfs
PROJECT : GOLF COURSE	03-21-1990	
slope 1 = 3 :1	slope 2 = 1 :1	bottom = 5
Manning coeff. = .017	Slope = .02 ft/ft	
Area = 6.367152 sq ft	Perim = 9.249496 ft	Hyd. Rad = .6883782
Depth= .9285491 ft	Velocity= 9.625628 fps	Capacity= 61.28783 cfs
PROJECT : GOLF COURSE	03-21-1990	
slope 1 = 3 :1	slope 2 = 1 :1	bottom = 5
Manning coeff. = .017	Slope = .03 ft/ft	
Area = 5.54413 sq ft	Perim = 8.807466 ft	Hyd. Rad = .6294808
Depth= .8319618 ft	Velocity= 11.10321 fps	Capacity= 61.55767 cfs
PROJECT : GOLF COURSE	03-21-1990	
slope 1 = 3 :1	slope 2 = 1 :1	bottom = 5
Manning coeff. = .017	Slope = .04 ft/ft	
Area = 4.999381 sq ft	Perim = 8.503248 ft	Hyd. Rad = .5879378
Depth= .7654877 ft	Velocity= 12.24763 fps	Capacity= 61.23056 cfs
PROJECT : GOLF COURSE	03-21-1990	
slope 1 = 3 :1	slope 2 = 1 :1	bottom = 5
Manning coeff. = .017	Slope = .05 ft/ft	
Area = 4.629102 sq ft	Perim = 8.290602 ft	Hyd. Rad = .5583553
Depth= .7190229 ft	Velocity= 13.22772 fps	Capacity= 61.23247 cfs
PROJECT : GOLF COURSE	03-21-1990	
slope 1 = 6 :1	slope 2 = 1 :1	bottom = 0
Manning coeff. = .017	Slope = .0346 ft/ft	
Area = 5.420396 sq ft	Perim = 9.329699 ft	Hyd. Rad = .580983
Depth= 1.244462 ft	Velocity= 11.30049 fps	Capacity= 61.25315 cfs
PROJECT : GOLF COURSE	03-21-1990	
slope 1 = 3 :1	slope 2 = 1 :1	bottom = 5

BY ______ DATE ______
CHKD. BY JLG DATE 6/25/81

SUBJECT GOLF COURSE ROAD
STORM DRAINAGE - RUNOFF
VOLUMES - PRESENT CONDITIONS

SHEET NO. 1 OF 6
JOB. NO. GF/M 25596
SUB 05J

ASSUMPTIONS: 1. USE RATIONAL METHOD AS MODIFIED BY THE AUG 1988 DRAFT REVISION OF DPM SECTION 22.2 BY R.J. HEGGA

2. PER THE ABOVE REFERENCE - THIS PROJECT IS LOCATED IN:

"ZONE 1 - WEST OF THE RIO GRANDE"

AND CONSISTS OF LAND TREATMENTS:

"2. SOIL COMPACTED BY HUMAN ACTIVITY. MINIMAL VEGITATION. UNPAVED PARKING, ROADS, TRAILS. MOST VACANT LOTS. LAWNS, PARKS, AND GOLF COURSES."

AND

"3. IMPERVIOUS AREAS. PAVEMENT. ROOFS"

3. POND FACILITIES WILL BE DESIGNED TO HANDLE A 100 YR EVENT. CONDITIONS DURING A 5 YR EVENT WILL BE CHECKED.

100 YR EVENT

BASIN 1 $L = 1400'$ $H = 22'$ $S = 1.57\%$ $A = 25.25\ ac$

$T_p = 15 - 5P_3 = 15 - 5\left(\frac{0.30}{25.25}\right) = 14.9\ min$

$V =$

Treatment	3	7.58 ac ×	4.76 =	36.08
Treatment	2	7.58 ac ×	3.22 =	24.41
Treatment	1	10.10 ac ×	1.81	18.28
				78.77 cfs

78.77 cfs

BY ______ DATE ______ SUBJECT ______ SHEET NO. ~ OF 4
CHKD. BY JLG DATE 6/05/89 JOB. NO. 25596

BASIN 2 L = 1425', H = 10' S = 0.70% A = 9.81 ac

$Tc = 15 - 5\left(\frac{1.71}{9.81}\right) = 14.1$ MIN

V =

Treatment 3 1.71 ac × 4.76 = 8.17 cfs
Treatment 2 8.10 ac × 3.22 = 26.08 cfs
34.25 cfs

ACCUMULATED Q100 AT 13th AVENUE → 113.02 cfs

BASIN 3 L = 2450', H = 45', S = 1.84%, A = 16.9 ac

$Tc = 15 - 5\left(\frac{2.56}{16.9}\right) = 14.2$ min

V =

Treatment 3 2.56 ac × 4.76 = 12.19 cfs
Treatment 2 14.34 ac × 3.22 = 46.17 cfs
58.36 cfs

Q100 AT 17th AVENUE S → 58.36 cfs

BASIN 4 L = 600', H = 22', S = 3.67%, A = 8.26 ac

$Tc = 15 - 5\left(\frac{0.61}{8.26}\right) = 14.6$ MIN

V =

Treatment 3 0.61 ac × 4.76 = 2.92 cfs
Treatment 2 7.65 ac × 3.22 = 24.63 cfs
27.55 cfs

27.55 cfs

BY ______ DATE ______ SUBJECT ______ SHEET NO. ______ OF ______
CHKD. BY JLG DATE 6/06/89 JOB. NO. ______

BASIN 5 $L = 600'$, $H = 18'$, $S = 3.00\%$, $A = 8.26$ ac

$T_c = 15 - 5\left(\frac{0.61}{8.26}\right) = 14.6$ min ✓

$V =$

Treatment 3 0.61 ac × 4.76 = 2.92 cfs ✓
Treatment 2 7.65 ac × 3.22 = 24.63 cfs ✓
27.55 cfs ✓

55.10 cfs

BASIN 6 $L = 2575'$, $H = 90'$, $S = 3.50\%$ $A = 17.7$ ac

$T_c = 15 - 5\left(\frac{3.60}{17.70}\right) = 14.0$ min ✓

$V =$

Treatment 3 3.60 ac × 4.76 = 17.13 cfs ✓
Treatment 2 14.08 ac × 3.22 = 45.35 cfs ✓
62.48 cfs ✓

117.58 cfs

BASIN 7 – NEXT PAGE

BASIN 7

$1.52\ in^2 \times 500^2 - 380{,}000\ ft^2 = 8.72\ ac$ = D.AREA

$L = 1020\ ft \qquad W = 580\ ft$

LAND TREATMENT 2 - 60% = 5.23 ac

LAND TREATMENT 3 - 40% = 3.49 ac

PEAK RUNOFF - ZONE 1 Q_{100}

TREATMENT 2 = $5.23\ ac \times 3.29\ ^{cfs}/_{ac} = 17.2\ cfs$

TREATMENT 3 = $3.49\ ac \times 4.74\ ^{cfs}/_{ac} = 16.5\ cfs$

TOTAL = 33.7 cfs

$t_p = 15 - 5\,(A_3) = 15 - 5\,(0.40) = 13\ min$

$$P_e = \frac{(0.93)(5.23) + (1.98)(3.49)}{8.72} = 1.350\ in$$

$Q_p = (3.29\ ^{cfs}/_{ac})(.60) + (4.74\ ^{cfs}/_{ac})(.40) = 3.87\ ^{cfs}/_{ac}$

$t_b = 121\left(\frac{1.350}{3.87}\right) - 15(.40) = 36.2\ min$

Duration of Peak = $15(.4) = 6\ min$

$Q_{10} = 0.68 \times 33.7\ cfs = 22.9\ cfs$

$Q_5 = 0.55 \times 33.7\ cfs = 18.5\ cfs$

DISCHARGE PIPES NEEDED - 10 YR STORM

FROM 2 - DOUBLE 'C' 's @ 23rd

$Q_i = 22.2$ cfs or 11.1 cfs / inlet

$Q = C_1 S^{1/2}$ (Eq 2 Pg 7 Concrete Pipe Design Manual)

INVERT NORTH SIDE = 5200.91 - .83 - 1.5 - .33
= 5198.25

$S = 1.0\%$ $S^{1/2} = 0.10$

INVERT SOUTH SIDE ⇒ 0.01 × 31' = 0.31'

5198.25 - 0.31' = 5197.94

$Q = (0.10)(105) = 10.5$ cfs < 11.1 cfs

NEED SLOPE: $11.1 = (S^{1/2})(105)$

$S^{1/2} = 0.1057$

$S = 0.0112 \Rightarrow 1.12\%$

∴ SOUTH INVERT = 5198.25 - 31(.0112) = 5197.90

18" - Q_{FULL} = (.1057)(105) = 11.1 cfs OK

TRY 24" CROSSING GOLF COURSE

$S = 1.0\%$ $S^{1/2} = 0.10$

$Q = (0.10)(226) = 22.6$ cfs > $Q_{10} = 22.2$ cfs OK

24" INVERT @ SOUTH = 5195.00 + 75(0.01) = 5195.75

TRY 2- Double Grate Type 'C' Inlets (COA style)

$A = 6.33' \times 2.08' = 13.17 ft^2$

$P = 2(2.08) + 6.33 = 10.49 ft$ $\quad d = 6'' + 2''(\text{depress}) = 8''$

CHECK CAPACITY AS WEIR

$Q_i = C_w P d^{1.5}$ $\quad C_w = 3.0$ (HEC-12 Pg 69 Eq 17)

$Q_i = (3.0)(10.49)(.67)^{1.5} = 17.3 cfs$

CHECK CAPACITY AS ORIFICE

$Q_i = C_o A (2gd)^{0.5}$ $\quad C_o = 0.67$ (HEC-12 Pg 69 Eq 18)

$Q_i = (0.67)(13.17)(2 * 32.2 * 0.67)^{0.5} = 58 cfs$

USE INTERCEPTION BY WEIR FLOW

$Q_{10} = \frac{1}{2}(22.2 cfs) = 11.5 cfs < Q_i = 17.3 cfs$

ALLOWABLE CLOGGING $= \frac{11.5}{17.3} = 0.66$ or 34 % OK

USE 2 DOUBLE GRATE TYPE 'C' INLETS
@ 23rd EAST OF GOLF COURSE.

NOTE: DEPTH OF PONDING BASE OF A 6" HIGH WATER BLOCK BETWEEN THE INLETS AND GOLF COURSE ROAD.

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SUBJECT GOLF COURSE | SHEET NO. | OF
JOB NO.
BY [illegible] DATE 5/91 CHKD. BY DATE

CHECK VERTICAL DEPTH

$$V_1 = C.F. + 0.5 + 1.2\frac{V_1^2}{2g} + \frac{d}{\cos S}$$

$$= 0.67 + 0.5 + 1.2\left(\frac{6.3^2}{64.4}\right) + 1.5$$

$$= 3.4\ ft$$

$$V_1 = \frac{11.1\ cfs}{1.77\ ft^2} = 6.3\ fps$$

ACTUAL = 3.32 ft ⟶ (NORTH BASIN)

$$V_2 = C.F._1 + 0.5 + H_1 + 1.2\frac{V_2^2}{2g} + \frac{d_2}{\cos S_2} - G$$

$$= 0.67 + 0.5 + 4.2 + 1.2\left(\frac{7.1^2}{64.4}\right) + 2 - 0$$

$$= 8.3\ ft$$

$$V_2 = \frac{22.2\ cfs}{3.14\ ft^2} = 7.1\ fps$$

ACTUAL = 0.67 + (5200.90 - 5195.75)

= 5.82 ft

NOTE: ALTHOUGH A "BACKWATER CONDITION" WILL EXIST FOR A SHORT PERIOD OF TIME NEAR THE PEAK RUNOFF PERIOD IT WILL LAST ONLY A SHORT PERIOD OF TIME.

THE TWO OPTIONS TO AVOID THE BACKWATER AT THIS INSTALLATION ARE:

- LARGER OUTLET PIPE
- DEPER INVERTS IN THE INLETS.

HOWEVER, THE PHYSICAL RESTRAINTS OF THE PRRJECT WILL NOT ALLOW THE USE OF EITHER OPTION.

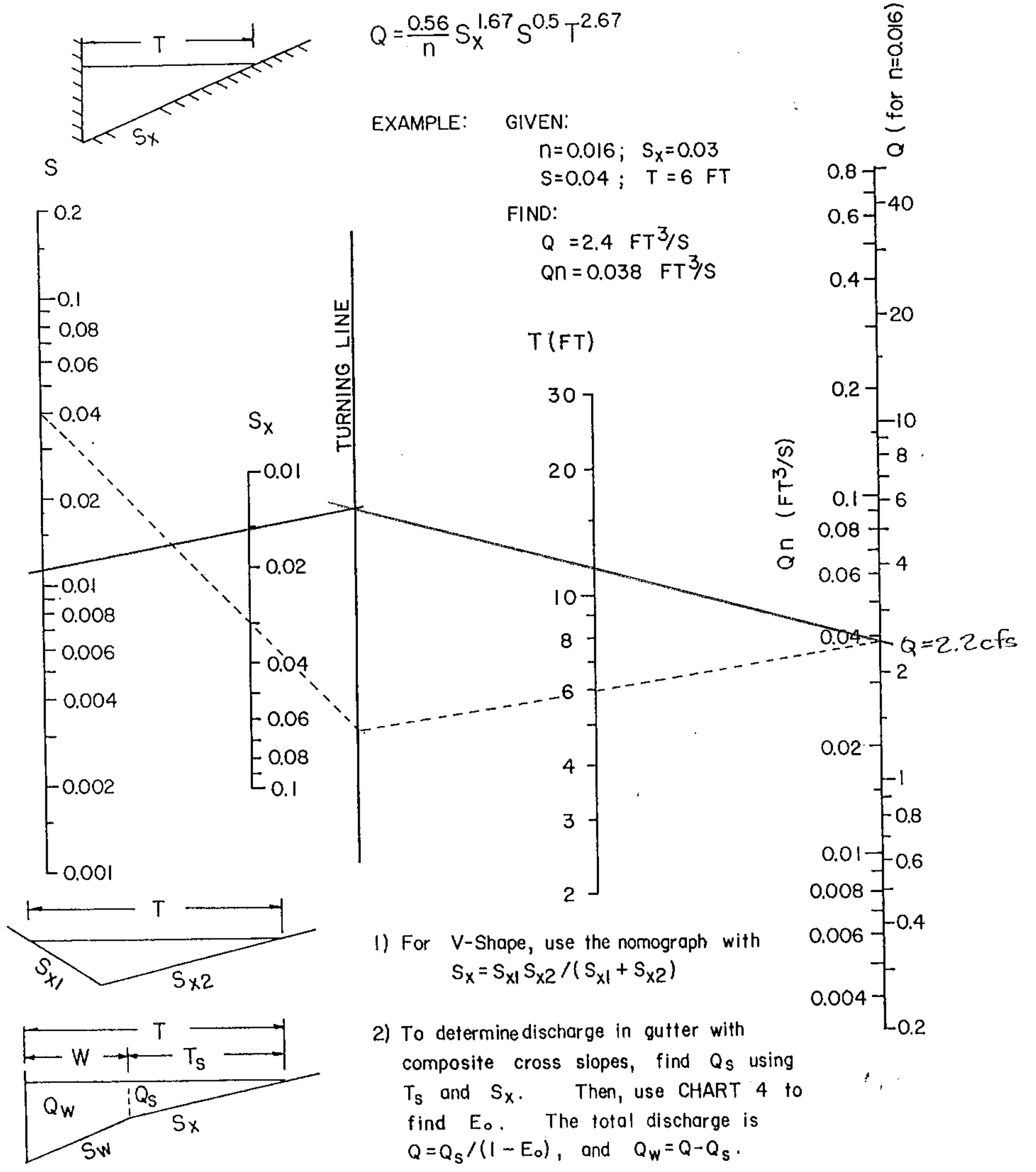


CHART 3. Flow in triangular gutter sections.

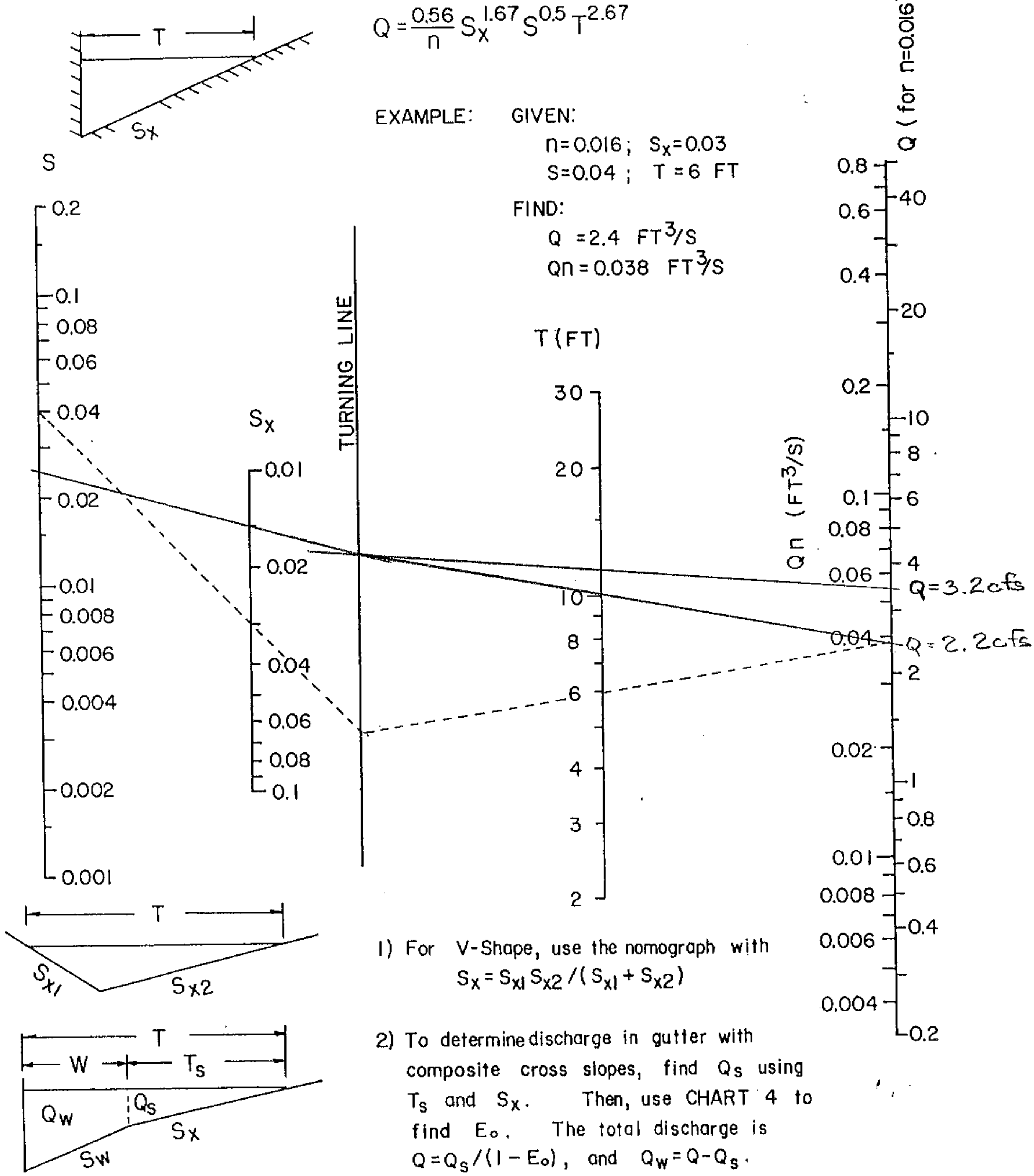
CAPACITY OF SECTION (HALF STREET)
SOUTHERN TO ANN CIRCLE

$Q_{MAX} = 2.2$ cfs

$$Q = \frac{0.56}{n} S_x^{1.67} S^{0.5} T^{2.67}$$

EXAMPLE: GIVEN:
$n=0.016$; $S_x=0.03$
$S=0.04$; $T=6$ FT

FIND:
$Q = 2.4\ FT^3/S$
$Qn = 0.038\ FT^3/S$



1) For V-Shape, use the nomograph with $S_x = S_{x1} S_{x2} / (S_{x1} + S_{x2})$

2) To determine discharge in gutter with composite cross slopes, find Q_s using T_s and S_x. Then, use CHART 4 to find E_o. The total discharge is $Q = Q_s/(1-E_o)$, and $Q_w = Q - Q_s$.

CHART 3. Flow in triangular gutter sections.

Q_{ACTUAL} = (.95)(

@ ANN CIRCLE Q = 2.2 cfs
(ie MAX CAPACITY FROM GOLF COURSE NORTH)
T = 10' d_{fl} = 0.15'
A = 0.75 sf V = 2.9 fps

SOUTHERN TO ANN CIRCLE – SLOTTED DRAIN

DRAINAGE AREA NO. 8

STREET – 12' × 2800' = 33,600 sf = 0.77 ac

YARDS – 25' × 2800' = 70,000 sf = 1.61 ac

Q_{10} (T-3) = (0.68)(4.74)(0.77) = 2.5 cfs

Q_{10} (T-2) = (0.68)(3.29)(1.61) = 3.6 cfs

TOTAL Q_{10} = 6.1 cfs

SINCE Q_{10} = 6.1 cfs > $Q_{CAP\ STREET}$ = 2.2 cfs

ONLY 2.2 cfs CONTINUES ON EAST SIDE

THE BALANCE (6.1 - 2.2 =) 3.9 cfs CROSSES

THE CROWN TO THE WEST DITCH.

L_T = 45 FT (LENGTH FOR TOTAL INTERCEPTION)

USE 70 FT ALLOWS 36% CLOGGING

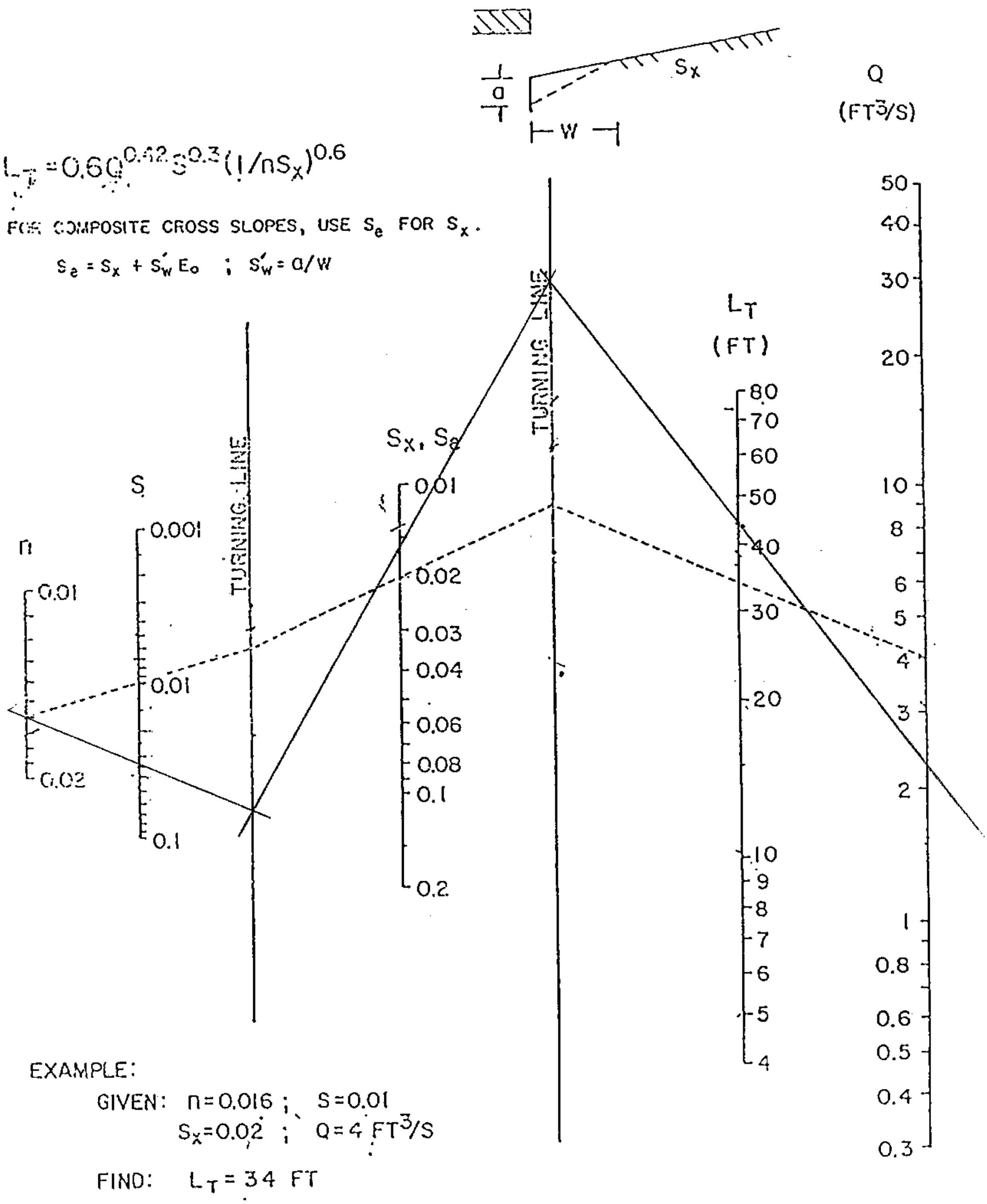


CHART 9. Curb-opening and slotted drain inlet length for total interception.

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SUBJECT Golf Course | SHEET NO. 7 OF 10
JOB NO.
BY [signature] DATE 5/91 CHKD. BY DATE

20th to 23rd Street

Drainage Area No 10

Street = 12' × 1750' = 21,000 sf = 0.48 ac

Yards = 25' × 1750' = 43,750 sf = 1.00 ac

Q_{10} (T-3) = (.68)(4.74)(0.48) = 1.6 cfs

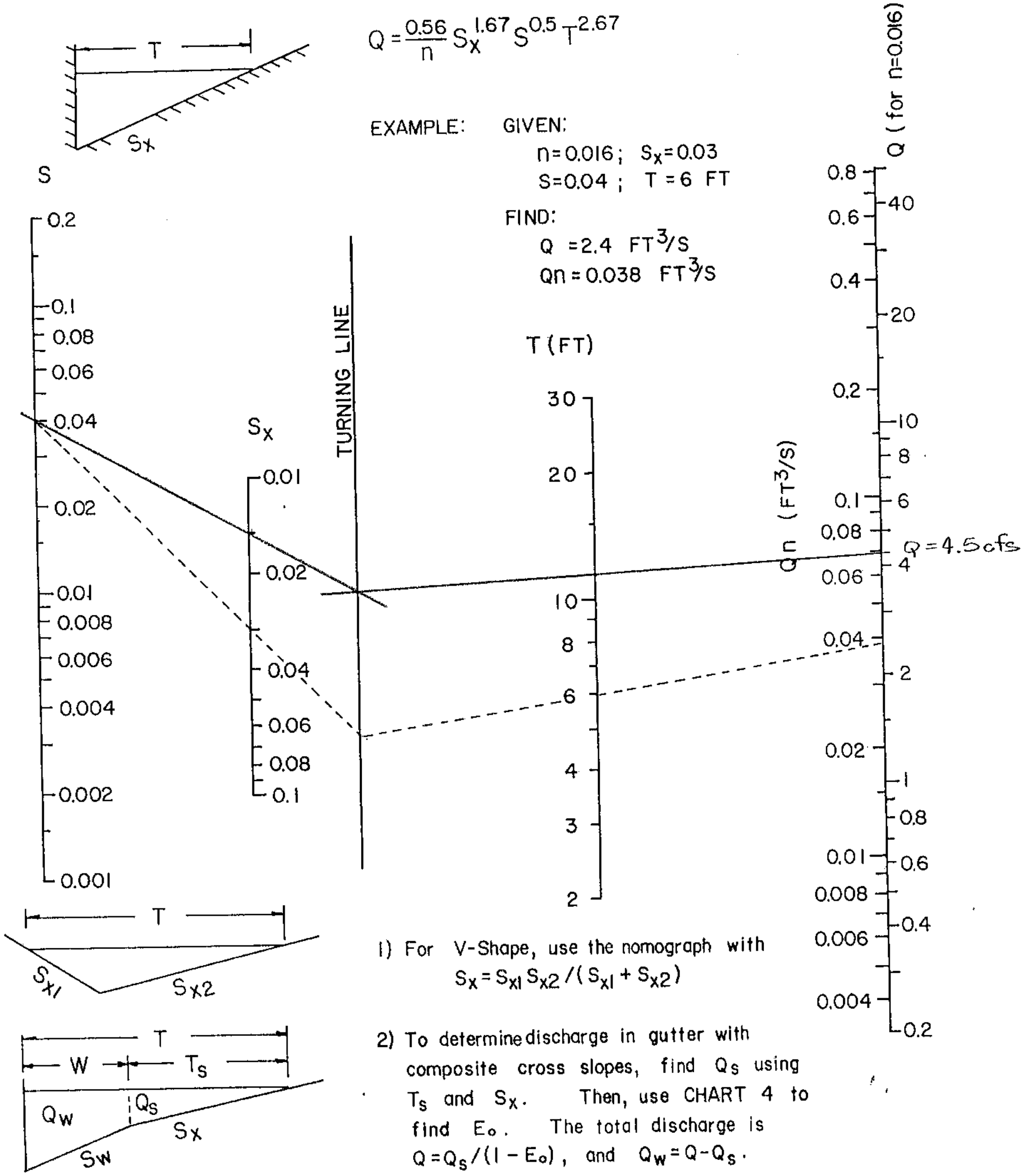
Q_{10} (T-2) = (.68)(3.29)(1.00) = 2.2 cfs

Total Q_{10} = 3.8 cfs

Since Q_{10} = 3.8 cfs < Q_{CAP} = 4.5 cfs (of street)
Flow remains on east side of roadway

L_T = 65 ft (Length for total interception)

Use 70 ft Allows 8% clogging



$$Q = \frac{0.56}{n} S_x^{1.67} S^{0.5} T^{2.67}$$

EXAMPLE: GIVEN:
$n = 0.016$; $S_x = 0.03$
$S = 0.04$; $T = 6$ FT

FIND:
$Q = 2.4$ FT³/S
$Qn = 0.038$ FT³/S

1) For V-Shape, use the nomograph with $S_x = S_{x1} S_{x2} / (S_{x1} + S_{x2})$

2) To determine discharge in gutter with composite cross slopes, find Q_s using T_s and S_x. Then, use CHART 4 to find E_o. The total discharge is $Q = Q_s / (1 - E_o)$, and $Q_w = Q - Q_s$.

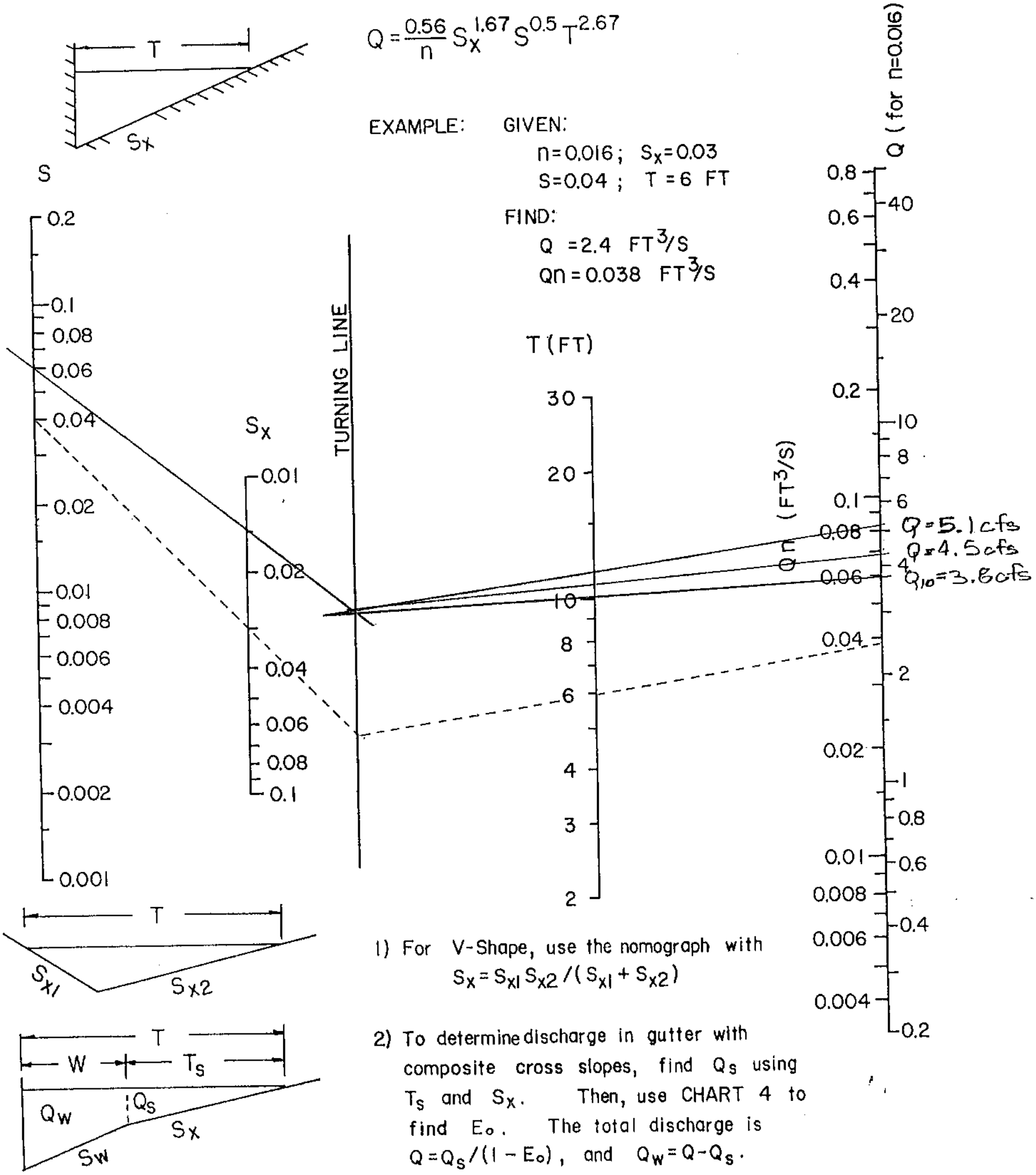
CHART 3. Flow in triangular gutter sections.

CAPACITY ON GOLF COURSE
from 20th to 23rd
$Q_{max} = 4.5$ cfs

$$Q = \frac{0.56}{n} S_x^{1.67} S^{0.5} T^{2.67}$$

EXAMPLE: GIVEN:
$n = 0.016$; $S_x = 0.03$
$S = 0.04$; $T = 6$ FT

FIND:
$Q = 2.4$ FT^3/S
$Qn = 0.038$ FT^3/S



1) For V-Shape, use the nomograph with $S_x = S_{x1} S_{x2} / (S_{x1} + S_{x2})$

2) To determine discharge in gutter with composite cross slopes, find Q_s using T_s and S_x. Then, use CHART 4 to find E_o. The total discharge is $Q = Q_s/(1 - E_o)$, and $Q_w = Q - Q_s$.

CHART 3. Flow in triangular gutter sections.

@ 23rd
$Q_{10} = 3.8$ cfs $T = 10'$ $d = 0.15'$
$A = 0.75$ sF $V = 5.1$ fps

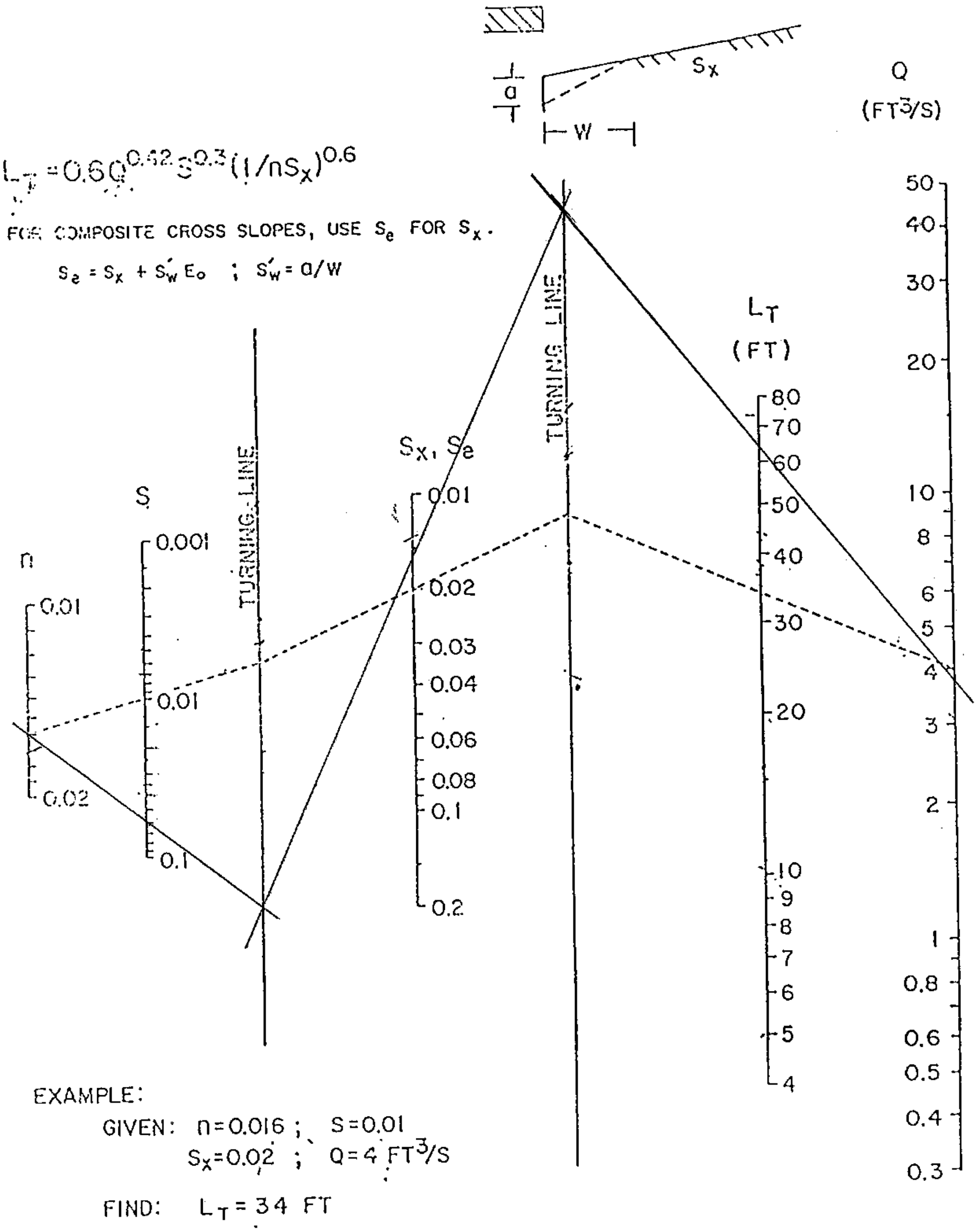


CHART 9. Curb-opening and slotted drain inlet length for total interception.

Q_i @ 23rd Q_{max} = 4.5cfs Q_i =

APPENDIX B

HYDROLOGIC RESERVOIR ROUTING

Hydrologic Reservoir Routing

Use Modified Puls Method (Storage Indication)

Flow over emergency spillway

$$O = CYH^{x}$$

where: O = outflow rate (cfs)
Y = length of spillway crest
H = reservoir depth above spillway crest (ft)
C = discharge coefficient for the weir, theoretically 3.0
x = exponent (3/2)

Flow through free outlet discharge pipe is described by:

Inlet Control Nomograph by Bureau of Public Roads (Jan 1963)

Assumptions:
1. Rainfall has previously saturated ponds and filled depression storage areas. (i.e. Inflow Quantity = Outflow Quantity)
2. No evaporation is accounted for.

3. Precipitation directly on the pond is based on the average surface area of the pond.

4. Precipitation based on 100-year, 6-hour storm = 2.20 inches cumulative (August 1988 Draft - Revision of DPM Section 22.2 DPM - Zone 1)

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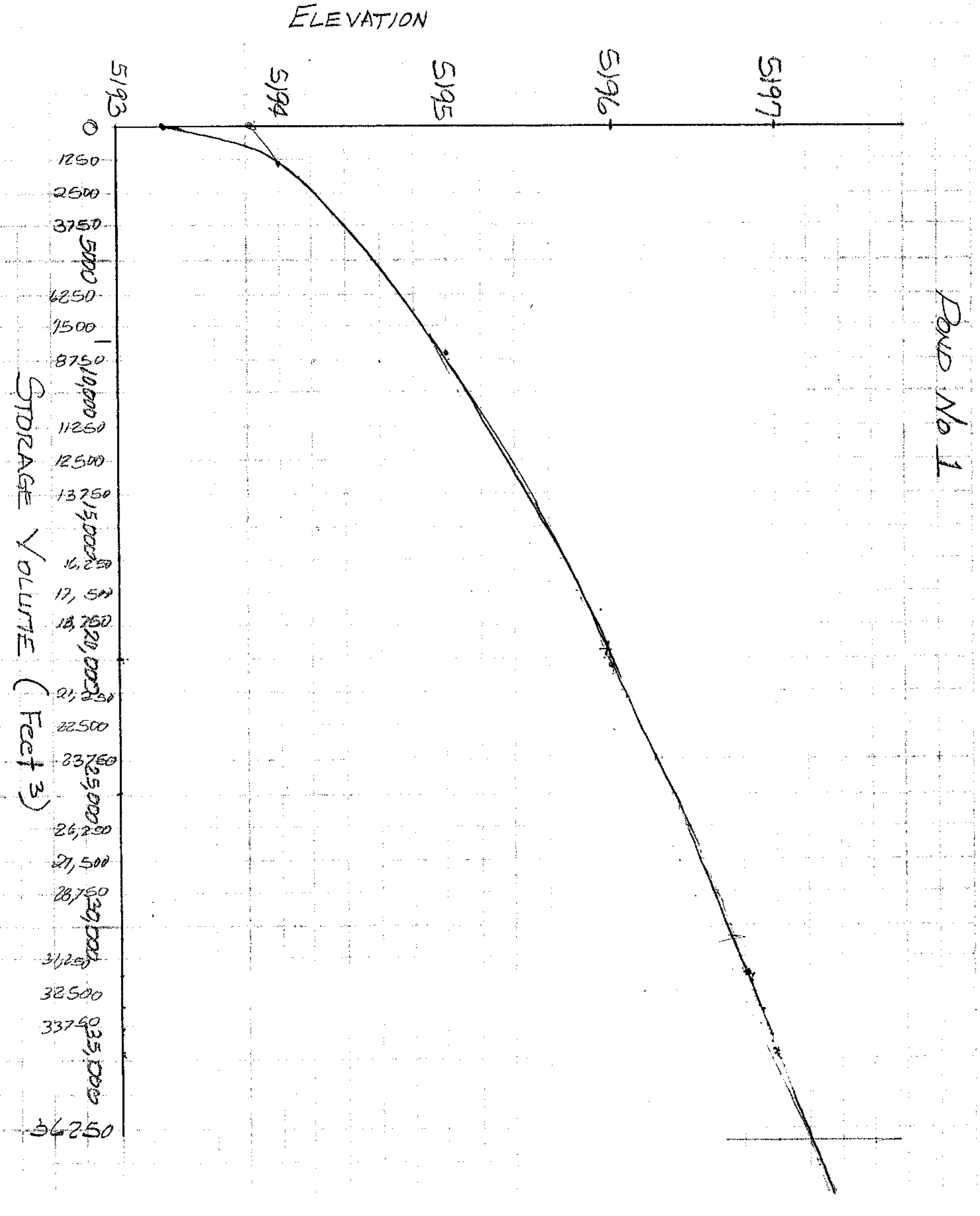
SUBJECT ______________________ SHEET NO. ______ OF ______

JOB NO. ______

BY ______ DATE ______ CHKD. BY ______ DATE ______



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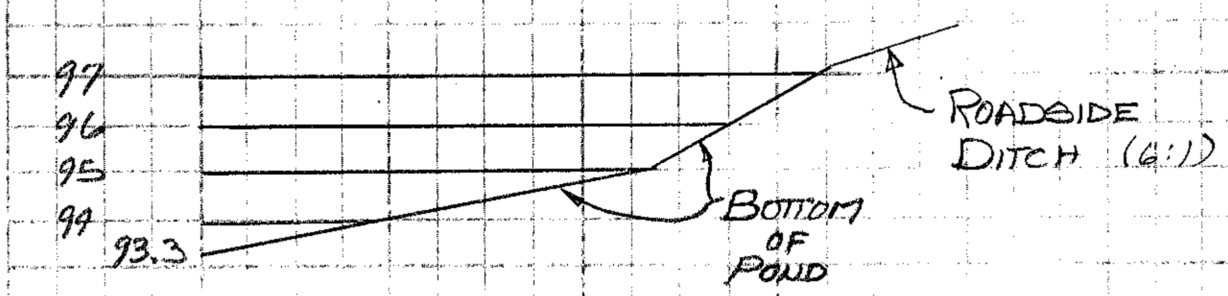
SUBJECT | SHEET NO. | OF
JOB NO.
BY | DATE | CHKD. BY | DATE

STORAGE VOLUME

POND No 1



ELEV	ΔELEV	SURFACE AREA (FT^2)	AVERAGE SURFACE AREA (FT^2)		Σ STORAGE VOLUME (FT^3)
93.3		0		0	0
	0.7'		1835		
94		3670		1285	1285
	1.0'		7184		
95		10698		7184	8469
	1.0'		11,797		
96		12895		11,797	20,266
	1.0'		13,796		
97		14697		14697	34,963

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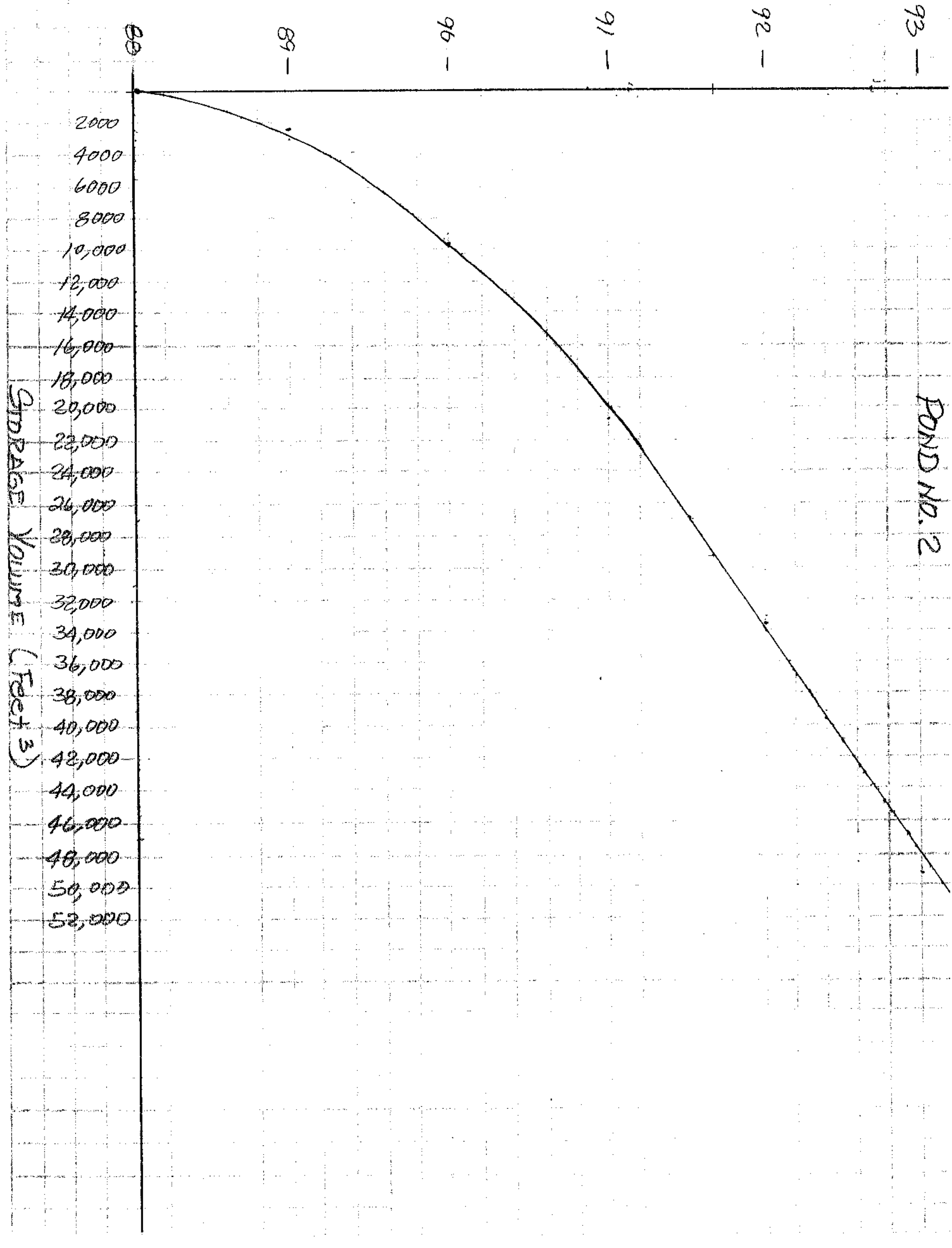
SUBJECT
SHEET NO. OF
JOB NO.
BY DATE CHKD. BY DATE



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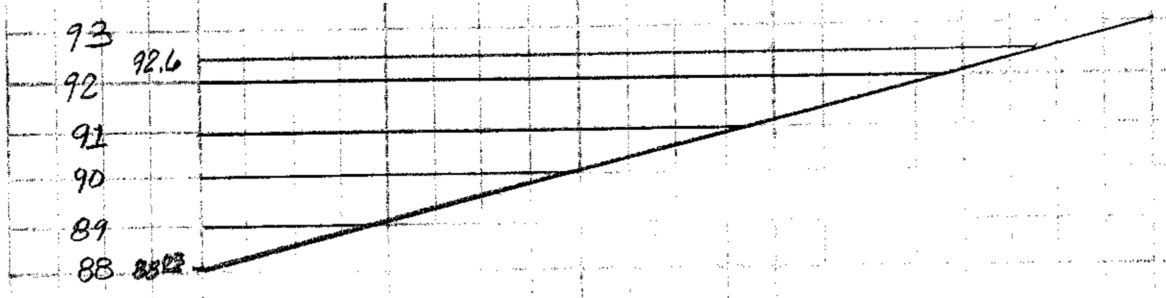
SUBJECT
BY DATE CHKD. BY DATE
SHEET NO. OF
JOB NO.

STORAGE VOLUME

POND NO 2



ELEV	Δ ELEV	SURFACE AREA (FT^2)	AVERAGE SURFACE AREA (FT^2)		Σ STORAGE VOLUME (FT^3)
88.05		0			
	0.95'		2538	2411.10	
89		5076			2411.10
	1.0'		7414.5	7414.5	
90		9753			9825.60
	1.0'		10,921.5	10,921.5	
91		12090			20,747.10
	1.0'		12,955.5	12955.5	
92		13821			33,702.60
	0.60'		14,839.8	14,839.8	
92.60					48,542.4
	0.40'				
93		15519	15519		49,221.60

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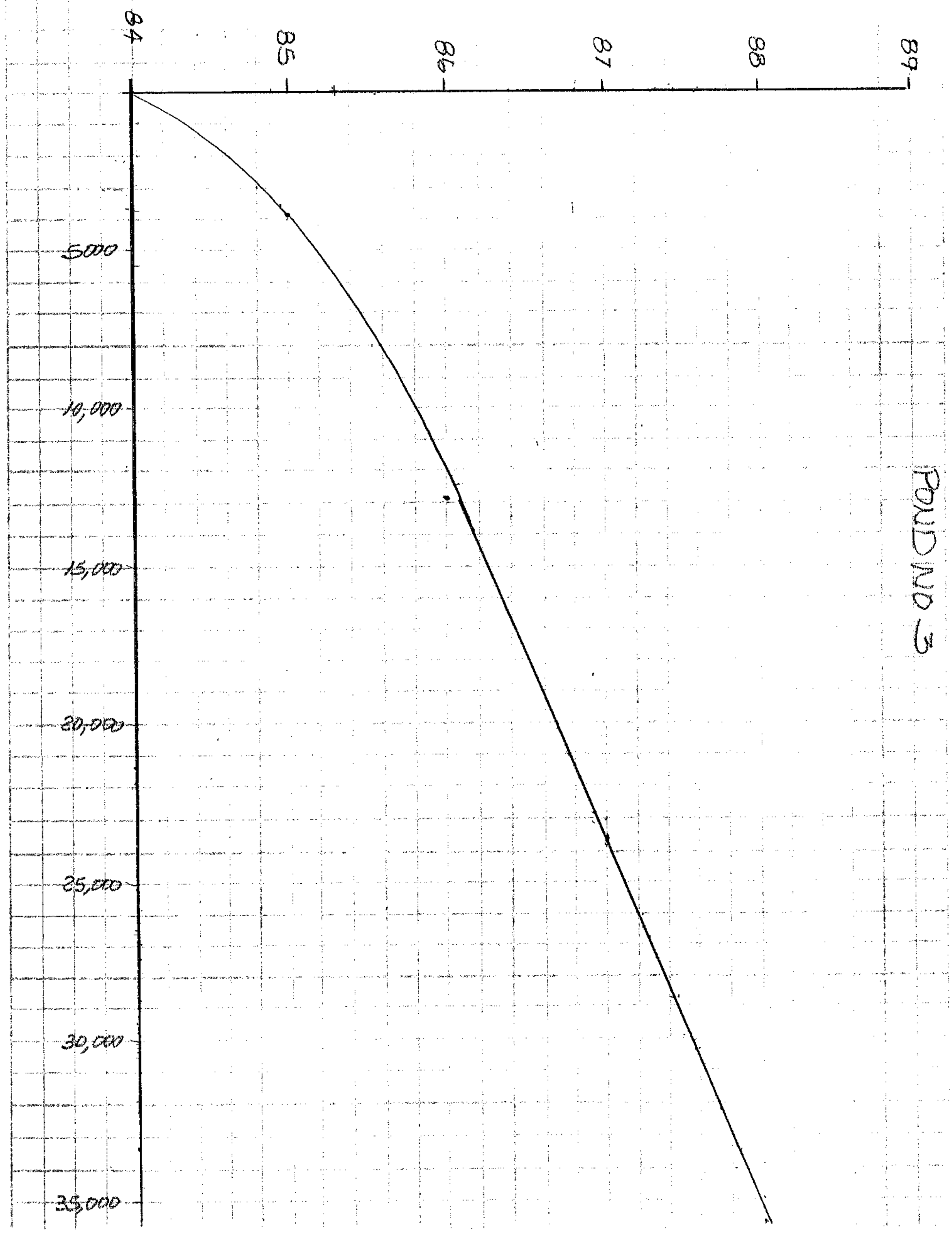
SUBJECT

BY DATE CHKD. BY DATE

SHEET NO. OF

JOB NO.



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SUBJECT
BY DATE CHKD. BY DATE
SHEET NO. OF
JOB NO.

STORAGE VOLUME

POND No. 3

ELEV	Δ ELEV	SURFACE AREA (FT^2)	AVG SURFACE AREA (FT^2)		Σ STORAGE VOLUME (FT^3)
84		0		0	
	1.0'		3985		
85		7969		3985	3985
	1.0'		8911		
86		9853		8911	12,896
	1.0'		10669		
87		11,486		10669	23,565
	1.0'		12353		
88		13,220		12353	35,918

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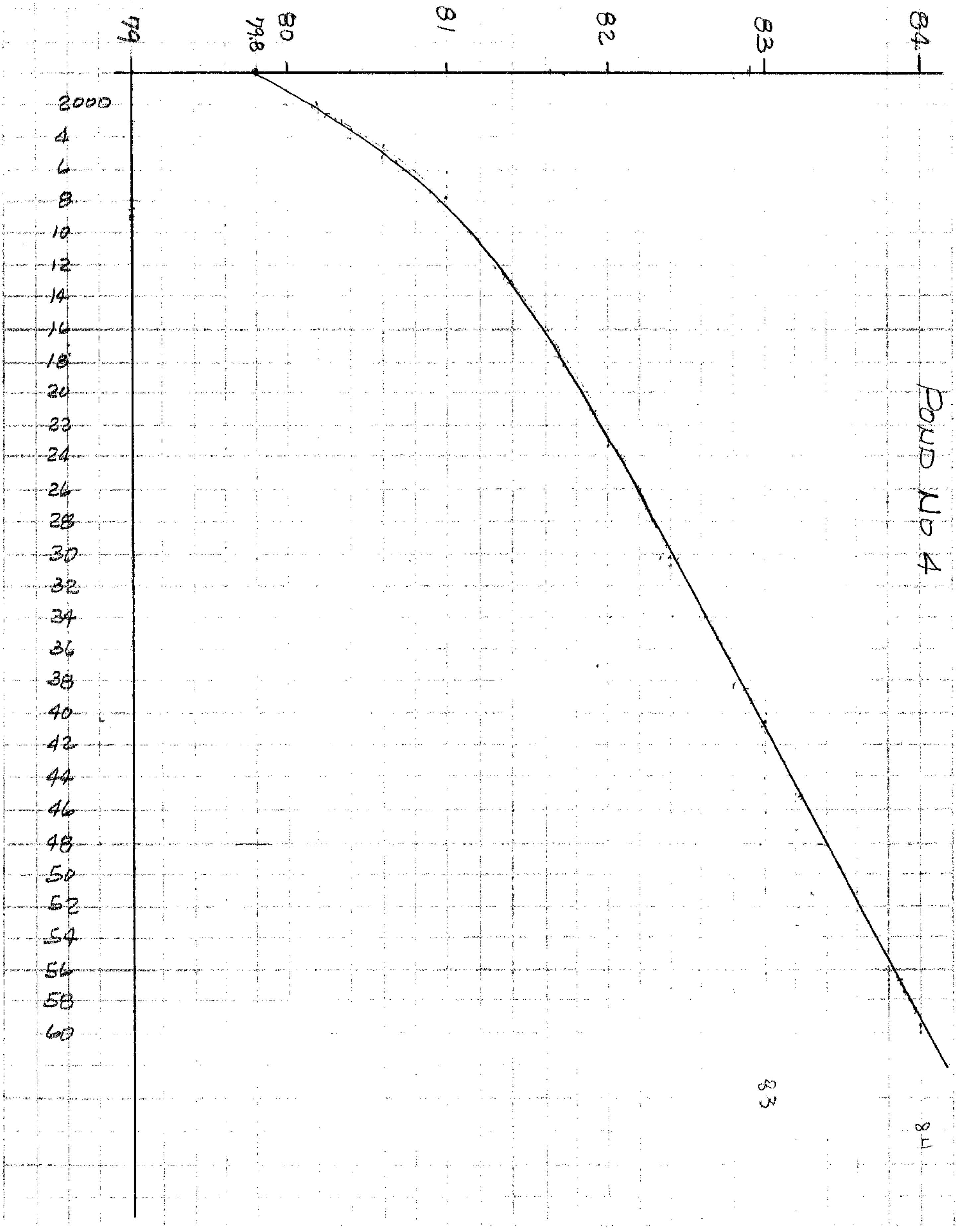
SUBJECT

SHEET NO. OF

JOB NO.

BY DATE CHKD. BY DATE



STORAGE VOLUME

POND NO 4

ELEV	ΔELEV	SURFACE AREA (FT^2)	AVG SURFACE AREA (FT^2)		Σ STORAGE VOLUME (FT^3)
79.80		0			
	0.20		472.5	94	
80		945			94
	1.0		7717.5	7717	
81		14,490			7811
	1.0		15,432	15432	
82		16,374			23,243
	1.0		17,286	17,286	
83		18,198			40,529
	1.0		19,127	19127	
84		20,056			59,656

POND ROUTING
HYDROGRAPH DIMENSIONS

DRAINAGE AREA	T_p (MIN)	P_e (IN)	Q_p (cfs/ac)	T_b (MIN)	DUR PEAK (MIN)	Q_5 (cfs)	Q_{100} (cfs)
4	14.6	1.01	3.40	34.7	1	15.2	27.55
5	14.6	1.01	3.40	34.7	1	15.2	27.55
6	14.0	1.14	3.58	34.7	4	34.4	62.48
7 *	13.0	1.35	3.87	36.2	6	18.5	33.7 **

DA 4 & 5

$$P_e = \frac{7.65(.93) + 0.61(1.98)}{8.26} = 1.01 \text{ IN}$$

$$Q_p = 3.29(7.65) + 4.74(0.61) = 28.1 \text{ cfs} / 8.26\text{ac} = 3.40 \text{ cfs/ac}$$

$$t_b = 121\left(\frac{1.01}{3.40}\right) - 15(.08) = 34.7 \text{ min}$$

$$\text{Dur Peak} = 15 * \left(\frac{.61}{8.26}\right) = 1.1 \text{ min}$$

* SEE PAGE _ of _ FOR CALCS

DA 6

$$P_e = \frac{14.08(.93) + 3.60(1.98)}{17.7} = 1.14 \text{ in}$$

$$Q_p = (3.29(14.08) + 4.74(3.60))/17.7 = 3.58 \text{ cfs/ac}$$

$$t_b = 121\left(\frac{1.14}{3.58}\right) - 15\left(3.60/14.08\right) = 34.7 \text{ min}$$

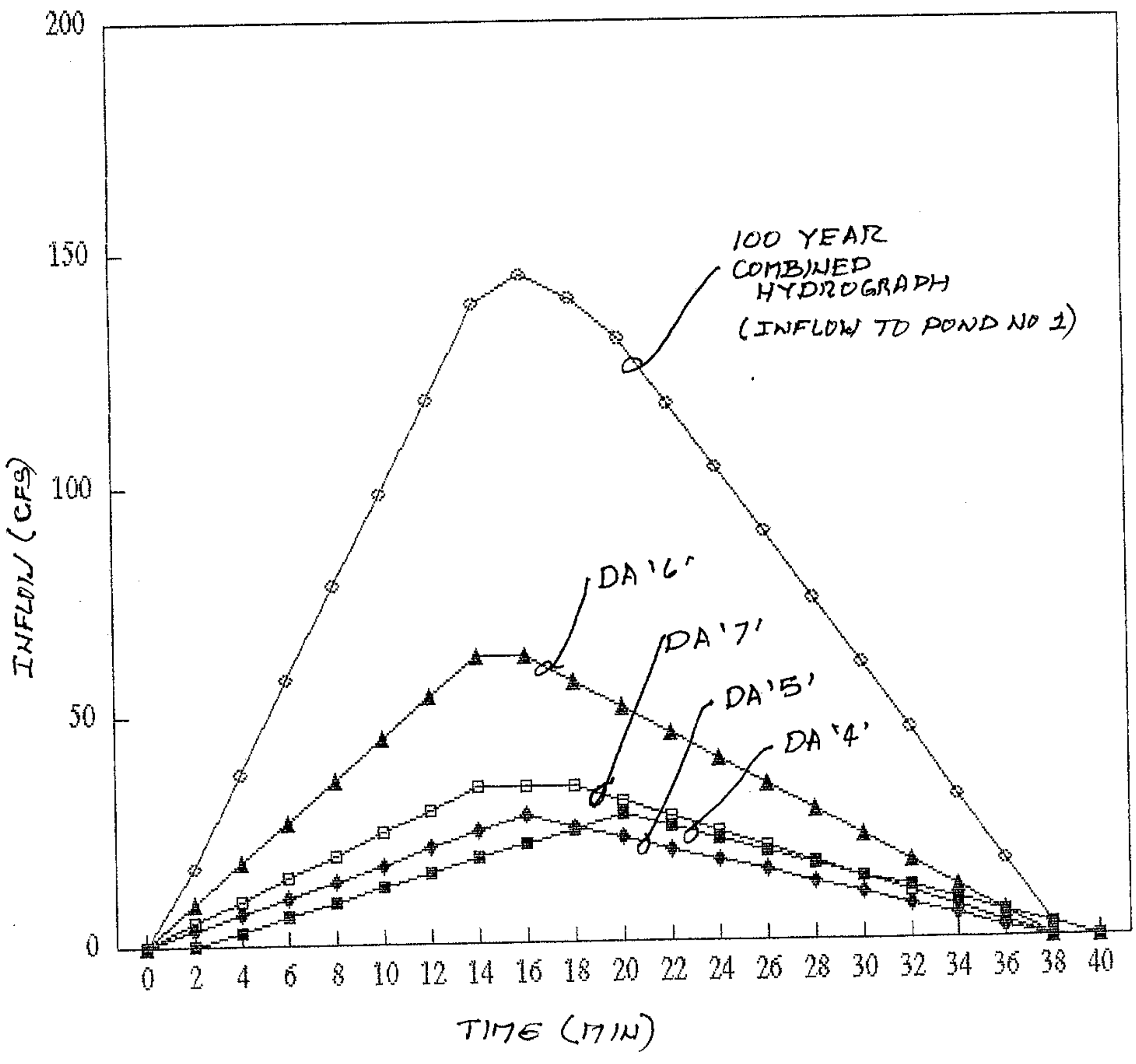
$$\text{Dur Peak} = 15 * \left(3.60/14.08\right) = 3.8 \text{ MIN}$$

** CAPACITY OF INLETS = 2 · DOUBLE 'C' @ 24 $Q_i = 2 * 17.3 = 34.6$ cfs > $Q_{100} = 33.7$ ASSUME FULL Q_{100} INTO PONDS.

SUBJECT:
BY
DATE
CHKD. BY
DATE
JOB NO.

COMBINED HYDROGRAPH
100-YEAR STORM

TIME (MIN)	DRAINAGE AREA 4	DRAINAGE AREA 5	DRAINAGE AREA 6	DRAINAGE AREA 7	COMBINED HYD. @ POND NO. 1	
	Time Lag = 1 min	Time Lag = 4 min	Time Lag = 1 min	Time Lag = 1 min		
0	0	0	0	0	0.0	0
2	0.0	3.5	8.9	4.8	17.2	1032
4	3.1	6.9	17.9	9.6	37.5	4310
6	6.1	10.4	26.8	14.4	57.7	10020
8	9.2	13.8	35.7	19.3	78.0	18161
10	12.3	17.3	44.6	24.1	98.2	28733
12	15.3	20.7	53.6	28.9	118.5	41737
14	18.4	24.2	62.5	33.7	138.8	57171
16	21.5	27.6	62.5	33.7	145.3	74212
18	24.5	25.1	56.8	33.7	140.1	91337
20	27.6	22.6	51.1	30.3	131.6	107644
22	24.8	20.1	45.5	27.0	117.3	122583
24	22.1	17.6	39.8	23.6	103.0	135803
26	19.3	15.1	34.1	20.2	88.7	147304
28	16.6	12.5	28.4	16.8	74.4	157087
30	13.8	10.0	22.7	13.5	60.0	165152
32	11.0	7.5	17.0	10.1	45.7	171498
34	8.3	5.0	11.4	6.7	31.4	176125
36	5.5	2.5	5.7	3.4	17.1	179034
38	2.8	0.0	0.0	0.0	2.8	180224
40	0.0	0.0	0.0	0.0	0.0	180390



100 YEAR COMBINED HYDROGRAPH (INFLOW TO POND NO 1)
DA '6'
DA '7'
DA '5'
DA '4'
INFLOW (CFS)
TIME (MIN)
0
50
100
150
200
0 2 4 6 8 10 12 14 16 18 20 22 24 26 28 30 32 34 36 38 40

HYDROLOGIC RESERVOIR ROUTING
GOLF COURSE DETENTION PONDS
100-YEAR STORM
W/O 13th & 17th Avenues Diversion

JUNE 30, 1991

TIME PERIOD	INFLOW 100-YR STORM	FLOW OUT THRU 12 12" PIPES NO.1 SPILL	DELTA STORAGE IN POND NO. 1	CUM STORAGE IN POND NO 1	CUM STORAGE ABOVE 19500	DEPTH OF FLOW OVER SPILLWAY NO. 1	FLOW OVER SPILLWAY NO. 1 2.5
		0	0	0	0	0.000	0
0-2 MIN	1	1	0	0	0	0.000	0
2-4 MIN	2063	2063	0	0	0	0.000	0
4-6 MIN	4494	4494	0	0	0	0.000	0
6-8 MIN	6925	6480	445	445	0	0.000	0
8-10 MIN	10358	6480	3878	4323	0	0.000	0
10-12 MIN	13790	6480	7310	11633	0	0.000	0
12-14 MIN	17810	6480	11330	22963	3463	0.299	1615
14-16 MIN	21831	6480	13736	36699	17199	1.483	17873
16-18 MIN	24202	6480	-151	36548	17048	1.470	17638
18-20 MIN	25176	6480	1057	37605	18105	1.561	19304
20-22 MIN	25746	6480	-37	37568	18068	1.558	19245
22-24 MIN	23978	6480	-1746	35822	16922	1.407	16524
24-26 MIN	22211	6480	-792	35030	15530	1.339	15336
26-28 MIN	19481	6480	-2334	32696	13196	1.138	12012
28-30 MIN	16751	6480	-1740	30956	11456	0.988	9716
30-32 MIN	14021	6480	-2175	28781	9281	0.800	7085
32-34 MIN	11291	6480	-2274	26507	7007	0.604	4648
34-36 MIN	8562	6480	-2565	23942	4442	0.383	2346
36-38 MIN	5832	6480	-2993	20949	1449	0.125	437
38-40 MIN	3102	6480	-3815	17184	0	0.000	0
40-42 MIN	1760	6480	-4720	12414	0	0.000	0
42-44 MIN	748	6480	-5732	6682	0	0.000	0
44-46 MIN	374	6480	-6106	576	0	0.000	0
46-48 MIN	0	576	-576	0	0	0.000	0
48-50 MIN	0				0	0.000	0
50-52 MIN	0						

HWL
EL 5197.5

TOP OF BERM EL 5198.1
FREEBOARD 0.6'

EL. 5197.5

TIME PERIOD	NET FLOW INTO POND NO. 2	FLOW OUT THRU 12 12" PIPES	DELTA STORAGE IN POND NO 2	CUM STORAGE IN POND NO 2	CUM STORAGE ABOVE 36000	DEPTH OF FLOW OVER SPILLWAY NO. 2	FLOW OVER SPILLWAY NO. 2 25
	0	0	0	0	0	0.000	0
0-2 MIN	1	1	0	0	0	0.000	0
2-4 MIN	2063	2063	0	0	0	0.000	0
4-6 MIN	4494	4494	0	0	0	0.000	0
6-8 MIN	6480	6480	0	0	0	0.000	0
8-10 MIN	6480	6480	0	0	0	0.000	0
10-12 MIN	6480	6480	0	0	0	0.000	0
12-14 MIN	8095	6480	1614	1614	0	0.000	0
14-16 MIN	24353	6480	17873	19487	0	0.000	0
16-18 MIN	24118	6480	17638	37125	1125	0.080	226
18-20 MIN	25784	6480	19078	56203	20203	1.443	17162
20-22 MIN	25725	6480	2082	58285	22285	1.592	19882
22-24 MIN	23004	6480	-3358	54927	18927	1.352	15562
24-26 MIN	21816	6480	-226	54701	18701	1.336	15284
26-28 MIN	18492	6480	-3272	51429	15429	1.102	11454
28-30 MIN	16196	6480	-1737	49692	13692	0.978	9575
30-32 MIN	13565	6480	-2490	47202	11202	0.800	7086
32-34 MIN	11128	6480	-2437	44765	8765	0.626	4904
34-36 MIN	8826	6480	-2558	42207	6207	0.443	2923
36-38 MIN	6917	6480	-2485	39722	3722	0.266	1357
38-40 MIN	6480	6480	-1357	38365	2365	0.169	687
40-42 MIN	6480	6480	-687	37678	1678	0.120	411
42-44 MIN	6480	6480	-410	37268	1268	0.091	270
44-46 MIN	6480	6480	-269	36999	999	0.071	189
46-48 MIN	0	6480	-6668	30331	0	0.000	0
48-50 MIN	0	6480	-6480	23851	0	0.000	0
50-52 MIN		6480	-6480	17371			
		6480	-6480	10891			
		6480	-6480	4411			
		4411	-4411	0			
		0	0	0			

HWL
EL 5193.8

TOP OF BERM EL 5194.3
FREEBOARD 0.5'

EL 5193.7

TIME PERIOD	NET FLOW INTO POND NO. 3	FLOW OUT THRU 12 12" PIPES	DELTA STORAGE IN POND NO 3	CUM STORAGE IN POND NO 3	CUM STORAGE ABOVE 20000	DEPTH OF FLOW OVER SPILLWAY NO. 3	FLOW OVER SPILLWAY NO. 3 18
	0	0	0	0	0	0.000	0
0-2 MIN	1	1	0	0	0	.000	0
2-4 MIN	2063	2063	0	0	0	.000	0
4-6 MIN	4494	4494	0	0	0	.000	0
6-8 MIN	6480	6480	0	0	0	.000	0
8-10 MIN	6480	6480	0	0	0	.000	0
10-12 MIN	6480	6480	0	0	0	.000	0
12-14 MIN	6480	6480	0	0	0	.000	0
14-16 MIN	6480	6480	0	0	0	.000	0
16-18 MIN	6706	6480	225	225	0	.000	0
18-20 MIN	23642	6480	17161	17386	0	.000	0
20-22 MIN	26362	6480	19882	37268	17268	1.470	12699
22-24 MIN	22042	6480	2862	40130	20130	1.713	15984
24-26 MIN	21764	6480	-699	39431	19431	1.654	15158
26-28 MIN	17934	6480	-3704	35727	15727	1.338	11038
28-30 MIN	16055	6480	-1462	34265	14265	1.214	9535
30-32 MIN	13566	6480	-2449	31816	11816	1.006	7188
32-34 MIN	11384	6480	-2283	29533	9533	0.811	5209
34-36 MIN	9403	6480	-2286	27247	7247	0.617	3453
36-38 MIN	7837	6480	-2095	25152	5152	0.438	2070
38-40 MIN	7167	6480	-1382	23770	3770	0.321	1295
40-42 MIN	6891	6480	-884	22886	2886	0.246	868
42-44 MIN	6750	6480	-597	22289	2289	0.195	613
44-46 MIN	6669	6480	-424	21865	1865	0.159	451
46-48 MIN	6480	6480	-450	21415	1415	0.120	298
48-50 MIN	6480	6480	-297	21118	1118	0.095	209
50-52 MIN	6480	6480	-209	20909	909	0.077	153
	6480	6480	-153	20756	756	0.064	116
	6480	6480	-116	20640	640	0.054	91
	4411	6480	-2159	18481	0	.000	0
	0	6480	-6480	12001	0	.000	0
		6480	-6480	5521	0	.000	0
		5521	-5521	0			
		0	0	0			
		0	0	0			

HWL
EL 5189

TOP OF BERM EL 5189
FREEBOARD Ø

EL 5188.4

TIME PERIOD	NET FLOW INTO POND NO. 4	FLOW OUT THRU 9 12" PIPES	DELTA STORAGE IN POND NO 4	CUM STORAGE IN POND NO 4	CUM STORAGE ABOVE 49000	DEPTH OF FLOW OVER SPILLWAY NO. 4	FLOW OVER SPILLWAY NO. 4 15	DISCHARGE TO BLACK ARROYO CF/2 MIN
	0	0	0	0	0	0.000	0	
0-2 MIN	1	1	0	0	0	0.000	0	
2-4 MIN	2063	2063	0	0	0	0.000	0	20
4-6 MIN	4494	4494	0	0	0	0.000	0	44
6-8 MIN	6480	4860	1620	1620	0	0.000	0	48
8-10 MIN	6480	4860	1620	3240	0	0.000	0	48
10-12 MIN	6480	4860	1620	4860	0	0.000	0	48
12-14 MIN	6480	4860	1620	6480	0	0.000	0	48
14-16 MIN	6480	4860	1620	8100	0	0.000	0	48
16-18 MIN	6480	4860	1620	9720	0	0.000	0	48
18-20 MIN	6480	4860	1620	11340	0	0.000	0	48
20-22 MIN	19179	4860	14319	25659	0	0.000	0	48
22-24 MIN	22464	4860	17603	43262	0	0.000	0	48
24-26 MIN	21638	4860	16778	60040	11040	0.649	3109	79
26-28 MIN	17518	4860	9549	69589	20589	1.211	7917	127
28-30 MIN	16015	4860	3237	72826	23826	1.402	9856	147
30-32 MIN	13668	4860	-1047	71779	22779	1.340	9219	140
32-34 MIN	11689	4860	-2384	69395	20395	1.200	7805	126
34-36 MIN	9933	4860	-2732	66663	17663	1.039	6291	111
36-38 MIN	8550	4860	-2601	64062	15062	0.886	4954	98
38-40 MIN	7775	4860	-2038	62024	13024	0.766	3983	88
40-42 MIN	7348	4860	-1495	60529	11529	0.678	3317	81
42-44 MIN	7093	4860	-1084	59445	10445	0.614	2861	77
44-46 MIN	6931	4860	-789	58656	9656	0.568	2543	74
46-48 MIN	6778	4860	-624	58032	9032	0.531	2300	71
48-50 MIN	6689	4860	-471	57561	8561	0.504	2123	69
50-52 MIN	6633	4860	-349	57212				48
	6596	4860	1736	58948				48
	6571	4860	1710	60658				48
	6480	4860	1620	62278				48
	6480	4860	1620	63898				48
	6480	4860	1620	65518				48
		4860	-4860	60658				48
		4860	-4860	55798				48
		4860	-4860	50938				48
		4860	-4860	46078				48
		4860	-4860	41218				48
		4860	-4860	36358				48
		4860	-4860	31498				48
		4860	-4860	26638				48
		4860	-4860	21778				48
		4860	-4860	16918				48
		4860	-4860	12058				48
		4860	-4860	7198				48
		4860	-4860	2338				48
		2338	-2338	0				23
		0	0	0				

EL 5184.9

HWL
EL 5185

TOP OF BERM 5185.5
FREEBOARD 0.5'

CAPACITY OF
OUTFALL
CHANNEL
52 cfs

HYDROLOGIC RESERVOIR ROUTING
GOLF COURSE DETENTION PONDS
100-YEAR STORM

TIME PERIOD	INFLOW 100-YR STORM	FLOW OUT THRU 12 12" PIPES NO.1 SPILL	DELTA STORAGE IN POND NO. 1	CUM STORAGE IN POND NO 1	CUM STORAGE ABOVE 19500	DEPTH OF FLOW OVER SPILLWAY NO. 1	FLOW OVER SPILLWAY NO. 1 25
		0	0	0	0	0.000	0
0-2 MIN	1	1	0	0	0	0.000	0
2-4 MIN	2063	2063	0	0	0	0.000	0
4-6 MIN	4494	4494	0	0	0	0.000	0
6-8 MIN	6925	6480	445	445	0	0.000	0
8-10 MIN	9357	6480	2877	3322	0	0.000	0
10-12 MIN	11788	6480	5308	8630	0	0.000	0
12-14 MIN	14219	6480	7739	16369	0	0.000	0
14-16 MIN	16650	6480	10170	26539	7039	0.607	4680
16-18 MIN	17432	6480	6272	32811	13311	1.148	12169
18-20 MIN	16817	6480	-1832	30979	11479	0.990	9746
20-22 MIN	15798	6480	-427	30552	11052	0.953	9207
22-24 MIN	14079	6480	-1607	28945	9445	0.814	7274
24-26 MIN	12361	6480	-1392	27553	8053	0.694	5726
26-28 MIN	10642	6480	-1564	25989	6489	0.559	4142
28-30 MIN	8924	6480	-1698	24291	4791	0.413	2628
30-32 MIN	7205	6480	-1902	22389	2889	0.249	1230
32-34 MIN	5487	6480	-2223	20166	666	0.057	136
34-36 MIN	3768	6480	-2848	17818	0	0.000	0
36-38 MIN	2050	6480	-4430	12888	0	0.000	0
38-40 MIN	331	6480	-6149	6739	0	0.000	0
40-42 MIN	0	6480	-6480	259	0	0.000	0
42-44 MIN	0	259	-259	0	0	0.000	0
44-46 MIN	0	0			0	0.000	0
46-48 MIN	0	0			0	0.000	0
48-50 MIN	0				0	0.000	0
50-52 MIN	0						

TIME PERIOD	NET FLOW INTO POND NO. 2	FLOW OUT THRU 12 12" PIPES	DELTA STORAGE IN POND NO 2	CUM STORAGE IN POND NO 2	CUM STORAGE ABOVE 36000	DEPTH OF FLOW OVER SPILLWAY NO. 2	FLOW OVER SPILLWAY NO. 2 25
	0	0	0	0	0	0.000	0
0-2 MIN	1	1	0	0	0	0.000	0
2-4 MIN	2063	2063	0	0	0	0.000	0
4-6 MIN	4494	4494	0	0	0	0.000	0
6-8 MIN	6480	6480	0	0	0	0.000	0
8-10 MIN	6480	6480	0	0	0	0.000	0
10-12 MIN	6480	6480	0	0	0	0.000	0
12-14 MIN	6480	6480	0	0	0	0.000	0
14-16 MIN	11160	6480	4679	4679	0	0.000	0
16-18 MIN	18649	6480	12169	16848	0	0.000	0
18-20 MIN	16226	6480	9745	26593	0	0.000	0
20-22 MIN	15687	6480	9206	35799	0	0.000	0
22-24 MIN	13754	6480	7273	43072	7072	0.505	3554
24-26 MIN	12206	6480	2172	45244	9244	0.660	5312
26-28 MIN	10622	6480	-1169	44075	8075	0.577	4337
28-30 MIN	9108	6480	-1708	42367	6367	0.455	3036
30-32 MIN	7710	6480	-1805	40562	4562	0.326	1842
32-34 MIN	6616	6480	-1705	38857	2857	0.204	913
34-36 MIN	6480	6480	-912	37945	1945	0.139	518
36-38 MIN	6480	6480	-512	37433	1433	0.102	324
38-40 MIN	6480	6480	-324	37109	1109	0.079	221
40-42 MIN	6480	6480	-220	36889	889	0.064	158
42-44 MIN	259	6480	-6379	30510	0	0.000	0
44-46 MIN	0	6480	-6480	24030	0	0.000	0
46-48 MIN	0	6480	-6480	17550	0	0.000	0
48-50 MIN	0	6480	-6480	11070	0	0.000	0
50-52 MIN		6480	-6480	4590			
		4590	-4590	0			
		0	0	0			
		0	0	0			
		0	0	0			

TIME PERIOD	NET FLOW INTO POND NO. 3	FLOW OUT THRU 12 12" PIPES	DELTA STORAGE IN POND NO 3	CUM STORAGE IN POND NO 3	CUM STORAGE ABOVE 20000	DEPTH OF FLOW OVER SPILLWAY NO. 3	FLOW OVER SPILLWAY NO. 3 18
	0	0	0	0	0	0.000	0
0-2 MIN	1	1	0	0	0	.000	0
2-4 MIN	2063	2063	0	0	0	.000	0
4-6 MIN	4494	4494	0	0	0	.000	0
6-8 MIN	6480	6480	0	0	0	.000	0
8-10 MIN	6480	6480	0	0	0	.000	0
10-12 MIN	6480	6480	0	0	0	.000	0
12-14 MIN	6480	6480	0	0	0	.000	0
14-16 MIN	6480	6480	0	0	0	.000	0
16-18 MIN	6480	6480	0	0	0	.000	0
18-20 MIN	6480	6480	0	0	0	.000	0
20-22 MIN	6480	6480	0	0	0	.000	0
22-24 MIN	10034	6480	3554	3554	0	.000	0
24-26 MIN	11792	6480	5311	8865	0	.000	0
26-28 MIN	10817	6480	4336	13201	0	.000	0
28-30 MIN	9516	6480	3036	16237	0	.000	0
30-32 MIN	8322	6480	1841	18078	0	.000	0
32-34 MIN	7393	6480	912	18990	0	.000	0
34-36 MIN	6993	6480	512	19502	0	.000	0
36-38 MIN	6804	6480	324	19826	0	.000	0
38-40 MIN	6701	6480	220	20046	46	0.004	2
40-42 MIN	6638	6480	156	20202	202	0.017	16
42-44 MIN	6480	6480	-16	20186	186	0.016	14
44-46 MIN	6480	6480	-14	20172	172	0.015	13
46-48 MIN	6480	6480	-12	20160	160	0.014	11
48-50 MIN	6480	6480	-11	20149	149	0.013	10
50-52 MIN	6480	6480	-10	20139	139	0.012	9
	4590	6480	-1899	18240	0	.000	0
	0	6480	-6480	11760	0	.000	0
	0	6480	-6480	5280	0	.000	0
	0	5280	-5280	0	0	.000	0
		0	0	0	0	.000	0
		0	0	0			
		0	0	0			
		0	0	0			

TIME PERIOD	NET FLOW INTO POND NO. 4	FLOW OUT THRU 9 12" PIPES	DELTA STORAGE IN POND NO 4	CUM STORAGE IN POND NO 4	CUM STORAGE ABOVE 49000	DEPTH OF FLOW OVER SPILLWAY NO. 4	FLOW OVER SPILLWAY NO. 4 15	DISCHARGE TO BLACK ARROYO CF/2 MIN	DISCHARGE TO BLACK ARROYO CFS
	0	0	0	0	0	0.000	0	0	0
0-2 MIN	1	1	0	0	0	0.000	0	1	0
2-4 MIN	2063	2063	0	0	0	0.000	0	2063	17
4-6 MIN	4494	4494	0	0	0	0.000	0	4494	37
6-8 MIN	6480	4860	1620	1620	0	0.000	0	4860	41
8-10 MIN	6480	4860	1620	3240	0	0.000	0	4860	41
10-12 MIN	6480	4860	1620	4860	0	0.000	0	4860	41
12-14 MIN	6480	4860	1620	6480	0	0.000	0	4860	41
14-16 MIN	6480	4860	1620	8100	0	0.000	0	4860	41
16-18 MIN	6480	4860	1620	9720	0	0.000	0	4860	41
18-20 MIN	6480	4860	1620	11340	0	0.000	0	4860	41
20-22 MIN	6480	4860	1620	12960	0	0.000	0	4860	41
22-24 MIN	6480	4860	1620	14580	0	0.000	0	4860	41
24-26 MIN	6480	4860	1620	16200	0	0.000	0	4860	41
26-28 MIN	6480	4860	1620	17820	0	0.000	0	4860	41
28-30 MIN	6480	4860	1620	19440	0	0.000	0	4860	41
30-32 MIN	6480	4860	1620	21060	0	0.000	0	4860	41
32-34 MIN	6480	4860	1620	22680	0	0.000	0	4860	41
34-36 MIN	6480	4860	1620	24300	0	0.000	0	4860	41
36-38 MIN	6480	4860	1620	25920	0	0.000	0	4860	41
38-40 MIN	6482	4860	1621	27541	0	0.000	0	4860	41
40-42 MIN	6496	4860	1636	29177	0	0.000	0	4860	41
42-44 MIN	6494	4860	1634	30811	0	0.000	0	4860	41
44-46 MIN	6493	4860	1632	32443	0	0.000	0	4860	41
46-48 MIN	6491	4860	1631	34074	0	0.000	0	4860	41
48-50 MIN	6490	4860	1630	35704	0	0.000	0	4860	41
50-52 MIN	6489	4860	1629	37333				4860	41
	6480	4860	1620	38953				4860	41
	6480	4860	1620	40573				4860	41
	6480	4860	1620	42193				4860	41
	5280	4860	420	42613				4860	41
	0	4860	-4860	37753				4860	41
		4860	-4860	32893				4860	41
		4860	-4860	28033				4860	41
		4860	-4860	23173				4860	41
		4860	-4860	18313				4860	41
		4860	-4860	13453				4860	41
		4860	-4860	8593				4860	41
		4860	-4860	3733				4860	41
		3733	-3733	0				3733	31
		0	0	0				0	0

HWL
EL 5196.9

TOP OF BERM EL 5198.1
FREEBOARD 1.2'

HWL
EL 5192.8

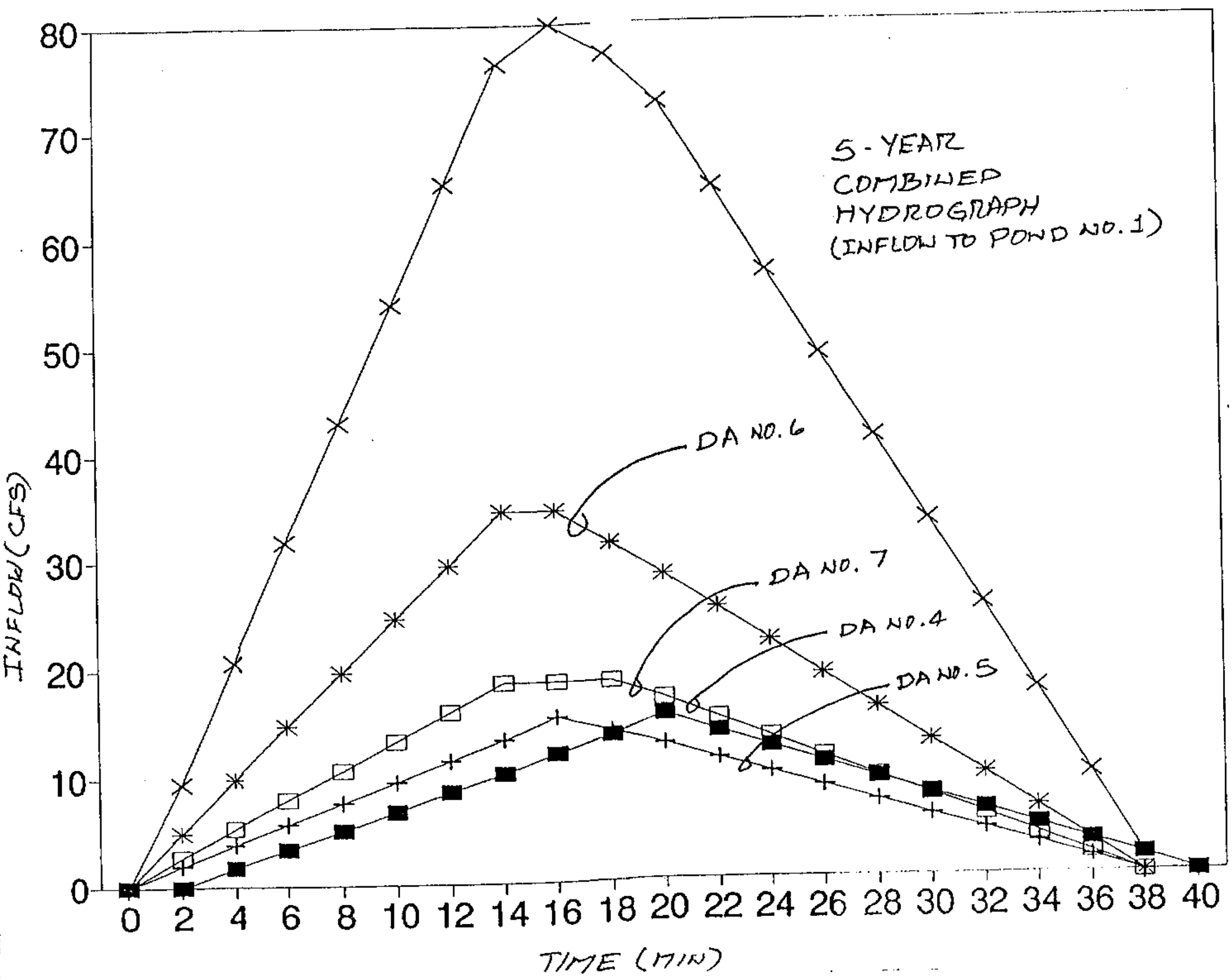
TOP OF BERM EL 5194.3
FREEBOARD 1.5'

HWL
EL 5186.9

TOP OF BERM EL 5189.0
FREEBOARD 2.1'

HWL
EL.5183.2

TOP OF BERM EL. 5185.5
FREEBOARD 2.3'



5-YEAR
COMBINED
HYDROGRAPH
(INFLOW TO POND NO. 1)
DA NO. 6
DA NO. 7
DA NO. 4
DA NO. 5
80
70
60
50
40
30
20
10
0
0 2 4 6 8 10 12 14 16 18 20 22 24 26 28 30 32 34 36 38 40
INFLOW (CFS)
TIME (MIN)

HYDROLOGIC RESERVOIR ROUTING
GOLF COURSE DETENTION PONDS
5-YEAR STORM

TIME PERIOD	INFLOW 5-YR STORM	FLOW OUT THRU 12 12" PIPES NO.1 SPILL	DELTA STORAGE IN POND NO. 1	CUM STORAGE IN POND NO 1	CUM STORAGE ABOVE 19500	DEPTH OF FLOW OVER SPILLWAY NO. 1	FLOW OVER SPILLWAY NO. 1 25
		0	0	0	0	0.000	0
0-2 MIN	1	1	0	0	0	0.000	0
2-4 MIN	1135	1135	0	0	0	0.000	0
4-6 MIN	2472	2472	0	0	0	0.000	0
6-8 MIN	3809	3809	0	0	0	0.000	0
8-10 MIN	5146	5146	0	0	0	0.000	0
10-12 MIN	6483	6480	3	3	0	0.000	0
12-14 MIN	7820	6480	1340	1343	0	0.000	0
14-16 MIN	9158	6480	2677	4020	0	0.000	0
16-18 MIN	9588	6480	3107	7127	0	0.000	0
18-20 MIN	9249	6480	2769	9896	0	0.000	0
20-22 MIN	8689	6480	2208	12104	0	0.000	0
22-24 MIN	7743	6480	1263	13367	0	0.000	0
24-26 MIN	6799	6480	318	13685	0	0.000	0
26-28 MIN	5853	6480	-626	13059	0	0.000	0
28-30 MIN	4908	6480	-1571	11488	0	0.000	0
30-32 MIN	3963	6480	-2517	8971	0	0.000	0
32-34 MIN	3018	6480	-3462	5509	0	0.000	0
34-36 MIN	2072	5509	-3436	2073	0	0.000	0
36-38 MIN	1128	2073	-945	1128	0	0.000	0
38-40 MIN	182	1128	-945	183	0	0.000	0
40-42 MIN	0	183	-183	0	0	0.000	0
42-44 MIN	0	0	0	0	0	0.000	0
44-46 MIN	0	0		0	0	0.000	0
46-48 MIN	0	0		0	0	0.000	0

TIME PERIOD	NET FLOW INTO POND NO. 2	FLOW OUT THRU 12 12" PIPES	DELTA STORAGE IN POND NO 2	CUM STORAGE IN POND NO 2	CUM STORAGE ABOVE 36000	DEPTH OF FLOW OVER SPILLWAY NO. 2	FLOW OVER SPILLWAY NO. 2 25
	0	0	0	0	0	0.000	0
0-2 MIN	0	0	0	0	0	0.000	0
2-4 MIN	1135	1135	0	0	0	0.000	0
4-6 MIN	2472	2472	0	0	0	0.000	0
6-8 MIN	3809	3809	0	0	0	0.000	0
8-10 MIN	5146	5146	0	0	0	0.000	0
10-12 MIN	6480	6480	0	0	0	0.000	0
12-14 MIN	6480	6480	0	0	0	0.000	0
14-16 MIN	6480	6480	0	0	0	0.000	0
16-18 MIN	6480	6480	0	0	0	0.000	0
18-20 MIN	6480	6480	0	0	0	0.000	0
20-22 MIN	6480	6480	0	0	0	0.000	0
22-24 MIN	6480	6480	0	0	0	0.000	0
24-26 MIN	6480	6480	0	0	0	0.000	0
26-28 MIN	6480	6480	0	0	0	0.000	0
28-30 MIN	6480	6480	0	0	0	0.000	0
30-32 MIN	6480	6480	0	0	0	0.000	0
32-34 MIN	6480	6480	0	0	0	0.000	0
34-36 MIN	5509	5509	0	0	0	0.000	0
36-38 MIN	2073	2073	0	0	0	0.000	0
38-40 MIN	1128	1128	0	0	0	0.000	0
40-42 MIN	183	183	0	0	0	0.000	0
42-44 MIN	0	0	0	0	0	0.000	0
44-46 MIN	0	0	0	0	0	0.000	0
46-48 MIN	0	0	0	0	0	0.000	0

TIME PERIOD	NET FLOW INTO POND NO. 3	FLOW OUT THRU 12 12" PIPES	DELTA STORAGE IN POND NO 3	CUM STORAGE IN POND NO 3	CUM STORAGE ABOVE 20000	DEPTH OF FLOW OVER SPILLWAY NO. 3	FLOW OVER SPILLWAY NO. 3 18
	0	0	0	0	0	0.000	0
0-2 MIN	0	0	0	0	0	.000	0
2-4 MIN	1134	1134	0	0	0	.000	0
4-6 MIN	2471	2471	0	0	0	.000	0
6-8 MIN	3808	3808	0	0	0	.000	0
8-10 MIN	5146	5146	0	0	0	.000	0
10-12 MIN	6480	6480	0	0	0	.000	0
12-14 MIN	6480	6480	0	0	0	.000	0
14-16 MIN	6480	6480	0	0	0	.000	0
16-18 MIN	6480	6480	0	0	0	.000	0
18-20 MIN	6480	6480	0	0	0	.000	0
20-22 MIN	6480	6480	0	0	0	.000	0
22-24 MIN	6480	6480	0	0	0	.000	0
24-26 MIN	6480	6480	0	0	0	.000	0
26-28 MIN	6480	6480	0	0	0	.000	0
28-30 MIN	6480	6480	0	0	0	.000	0
30-32 MIN	6480	6480	0	0	0	.000	0
32-34 MIN	6480	6480	0	0	0	.000	0
34-36 MIN	5509	5509	0	0	0	.000	0
36-38 MIN	2073	2073	0	0	0	.000	0
38-40 MIN	1128	1128	0	0	0	.000	0
40-42 MIN	183	183	0	0	0	.000	0
42-44 MIN	0	0	0	0	0	.000	0
44-46 MIN	0	0	0	0	0	.000	0
46-48 MIN	0	0	0	0	0	.000	0

TIME PERIOD	NET FLOW INTO POND NO. 4	FLOW OUT THRU 9 12" PIPES	DELTA STORAGE IN POND NO 4	CUM STORAGE IN POND NO 4	CUM STORAGE ABOVE 49000	DEPTH OF FLOW OVER SPILLWAY NO. 4	FLOW OVER SPILLWAY NO. 4 15	DISCHARGE TO BLACK ARROYO CF/2 MIN	DISCHARGE TO BLACK ARROYO CFS
	0	0	0	0	0	0.000	0	0	0
0-2 MIN	0	0	0	0	0	0.000	0	0	0
2-4 MIN	1134	1134	0	0	0	0.000	0	1134	9
4-6 MIN	2471	2471	0	0	0	0.000	0	2471	21
6-8 MIN	3808	3808	0	0	0	0.000	0	3808	32
8-10 MIN	5146	4860	286	286	0	0.000	0	4860	41
10-12 MIN	6480	4860	1620	1906	0	0.000	0	4860	41
12-14 MIN	6480	4860	1620	3526	0	0.000	0	4860	41
14-16 MIN	6480	4860	1620	5146	0	0.000	0	4860	41
16-18 MIN	6480	4860	1620	6766	0	0.000	0	4860	41
18-20 MIN	6480	4860	1620	8386	0	0.000	0	4860	41
20-22 MIN	6480	4860	1620	10006	0	0.000	0	4860	41
22-24 MIN	6480	4860	1620	11626	0	0.000	0	4860	41
24-26 MIN	6480	4860	1620	13246	0	0.000	0	4860	41
26-28 MIN	6480	4860	1620	14866	0	0.000	0	4860	41
28-30 MIN	6480	4860	1620	16486	0	0.000	0	4860	41
30-32 MIN	6480	4860	1620	18106	0	0.000	0	4860	41
32-34 MIN	6480	4860	1620	19726	0	0.000	0	4860	41
34-36 MIN	5509	4860	649	20375	0	0.000	0	4860	41
36-38 MIN	2073	4860	-2786	17589	0	0.000	0	4860	41
38-40 MIN	1128	4860	-3731	13858	0	0.000	0	4860	41
40-42 MIN	183	4860	-4676	9182	0	0.000	0	4860	41
42-44 MIN	0	4860	-4860	4322	0	0.000	0	4860	41
44-46 MIN	0	4922	-4922	0	0	0.000	0	4322	36
46-48 MIN	0	0	0	0	0	0.000	0	0	0

HWL
EL 5181.9

TOP OF BERM EL 5185.5
FREEBOARD 3.6'

Gannett Fleming
ENGINEERS AND PLANNERS

SUBJECT
SHEET NO. OF
JOB NO.
BY DATE CHKD. BY DATE

SPILLWAY PIPE SIZE

PER DETENTION POND OP STUDY

- NEED 12 - 12" PIPES IN SPILLWAYS 1, 2, & 3

FLOW = 6480 CF/2 MIN OR 54 CFS (4.5 cfs/ea)

HW/D = 2   HW = 24"

TRY 15"   HW/D = 24/15 = 1.6   Q = 7.4 cfs/ea

OR 54 cfs/7.4 cfs/ea = 7.29 $Q_{15/7}$ = 7.4 × 7 = 52 cfs

USE 7-15" CMP's EA IN SPILLWAYS 1, 2, & 3

- NEED 9-12" PIPES IN SPILLWAY NO. 4

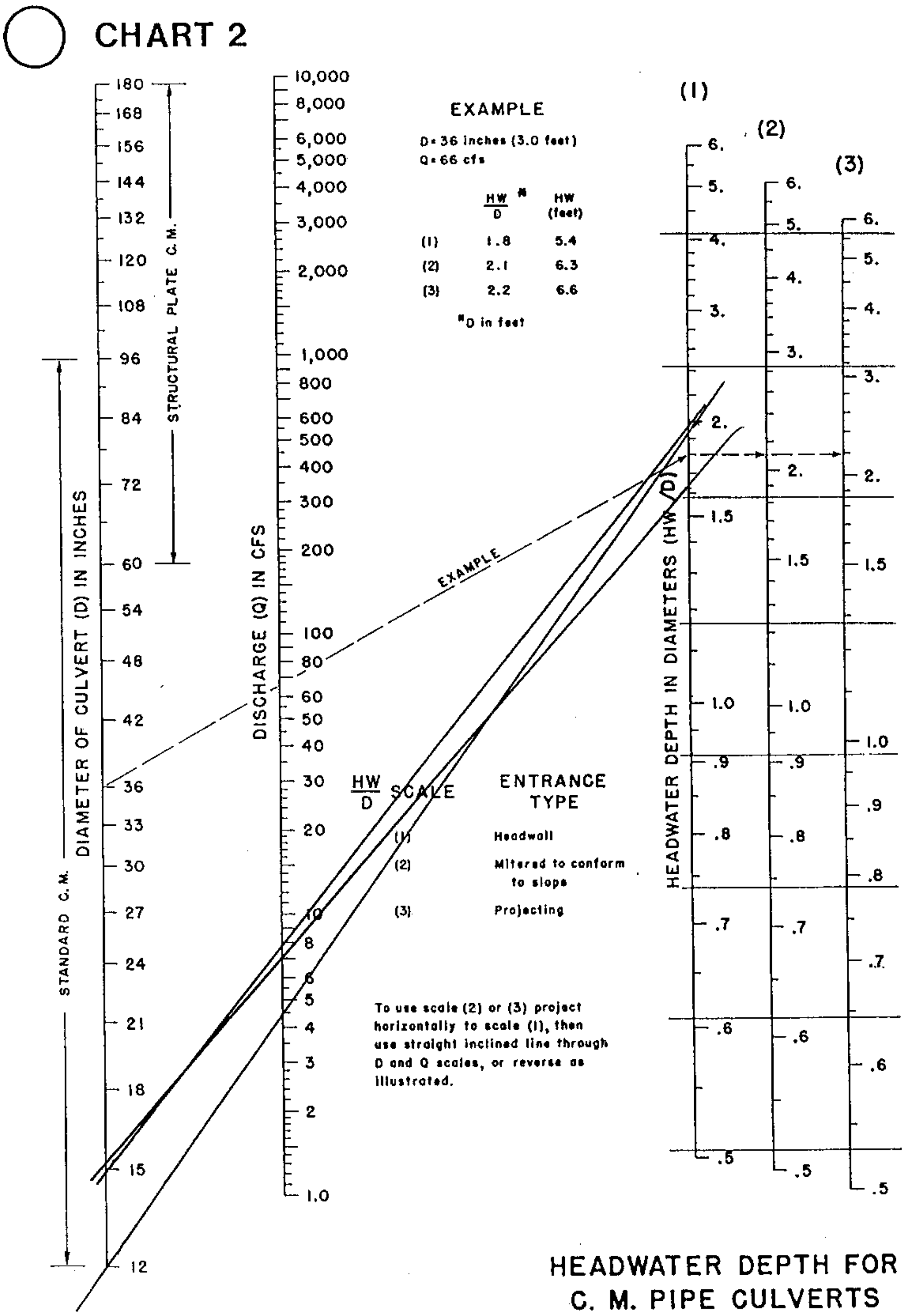
FLOW = 4860 cfs/2 min OR 40.5 cfs (4.5 cfs/ea)

HW/D = 2   HW = 24"

TRY 15"   HW/D = 24/15 = 1.6   Q = 7.4 cfs/ea

OR 40.5 cfs/7.4 cfse = 5.4 $Q_{15/5}$ = 7.4 × 5 = 37 cfs

USE 5-15" CMPS IN SPILLWAY NO. 4



HEADWATER DEPTH FOR
C. M. PIPE CULVERTS
WITH INLET CONTROL

BUREAU OF PUBLIC ROADS JAN. 1963

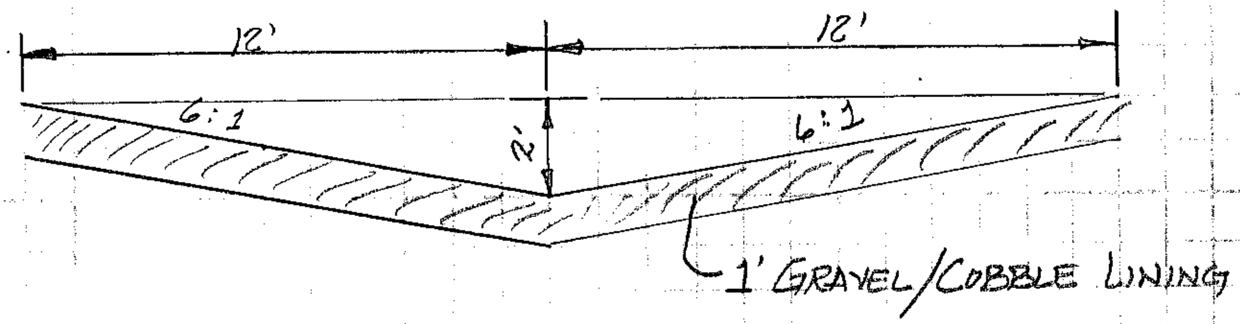
Gannett Fleming
ENGINEERS AND PLANNERS

SUBJECT | SHEET NO. OF | JOB NO. | BY | DATE | CHKD. BY | DATE

OUTLET CHANNEL

POND No 4 SPILLWAY to BLACK ARROYO

-MAIN CHANNEL



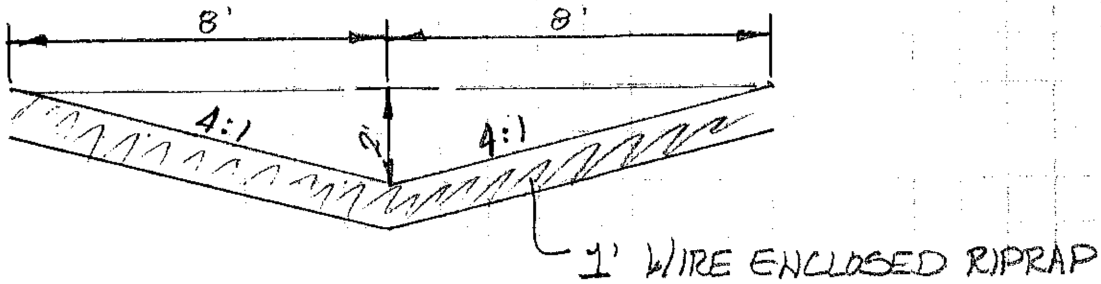
USE MANNING 'n' = 0.022

SLOPE = .0015'/ft $\qquad S^{1/2} = .0387 \quad R = {}^{24}/_{24.4} = 0.984$

$$Q = \frac{1.486}{.022}(.0387)(0.984)^{2/3}(24) = 62.1\text{ cfs} \qquad V = 2.6\text{ fps}$$

DISCHARGE FROM PONDS = 41 cfs OK

-TAIL CHANNEL (DISCHARGE AT BLACK ARROYO



USE MANNING 'n' = 0.04

SLOPE = 0.09'/ft $\quad S^{1/2} = 0.30$

$R = A/P = {}^{16}/_{16.5} = 0.970 \qquad R^{2/3} = 0.9798$

$$Q = \frac{1.486}{0.04}(.30)(.9798)(16) = 175 \text{ cfs} < 41 \text{ cfs DISCHARGE}$$

OK

$$V = 175/16 = 10.9 \text{ fps}$$

APPENDIX C

AUGUST 1988 DRAFT
REVISION OF SECTION 22.2
CITY OF ALBUQUERQUE
DEVELOPMENT PROCESS MANUAL

DRAFT DRAFT DRAFT

REVISION OF SECTION 22.2, DPM

Richard J. Heggen
Civil Engineering
UNM

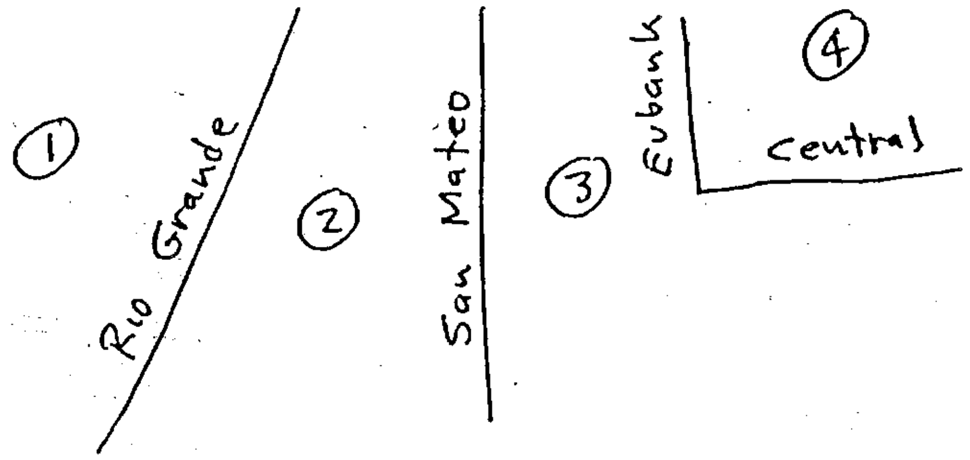
August, 1988

{Note: This document is prepared for discussion purposes only. A basis for possible revision to the City of Albuquerque's Development Process Maunal (DPM), Section 22.2, Hydrology is described. Should the DPM be revised, the Hydrology Section of Public Works Planning should maintain this document as a resource for future review and development.}

ZONES

Albuquerque's four precipitation zones are:

Zone	Location
1	West of the Rio Grande River
2	Beween the Rio Grande River and San Mateo
3	Between San Mateo and Eubank, North of Central and East of San Mateo, South of Central
4	East of Eubank, North of Central



Where a watershed extends across a zone boundry, use the zone which contains the largest portion of the watershed.

DESIGN STORM

The design storm is the 100-year 6-hour event defined by the NOAA Atlas 2, Precipitation-Frequency Atlas of the Western United States, Vol. IV - New Mexico. Assume an AMC II, a normally dry watershed. Cumulative depths for 100-year point precipitation for each zone are listed below. Assume linear variation in depths between adjacent time increments.

Cumulative Depth (inches)

Time (min)	Zone 1	Zone 2	Zone 3	Zone 4
0	0	0	0	0
5	0.06	0.06	0.07	0.07
10	0.14	0.15	0.16	0.16
15	0.25	0.27	0.29	0.30
20	0.41	0.44	0.47	0.49
25	0.70	0.77	0.82	0.85
30	1.24	1.35	1.44	1.50
35	1.46	1.59	1.69	1.77
40	1.60	1.74	1.85	1.93
45	1.68	1.83	1.95	2.03
50	1.75	1.90	2.03	2.11
55	1.80	1.96	2.09	2.18
60	1.85	2.01	2.14	2.23
65	1.87	2.04	2.17	2.27
70	1.89	2.06	2.20	2.30
75	1.91	2.08	2.22	2.33
80	1.93	2.09	2.24	2.35
85	1.94	2.10	2.25	2.37
90	1.95	2.11	2.26	2.39
95	1.95	2.12	2.27	2.41
100	1.96	2.12	2.28	2.42
105	1.96	2.13	2.29	2.43
110	1.97	2.13	2.29	2.44
115	1.97	2.13	2.29	2.45
120	1.97	2.13	2.30	2.46
180	2.05	2.20	2.40	2.61
360	2.20	2.35	2.60	2.90
1440	2.70	2.75	3.10	3.65

To estimate rainfall at return periods other than 100 years, multiply the above depths by the following factors:

Return Period (years)	Factor
50	0.93
25	0.79
10	0.68
5	0.55
2	0.48

(Note: The NOAA atlas underestimates depths for short intensity, long return period storms. The tabulation is low for the 5-30 minute depths.}

Example 1 Find the 10-year, 4-hour storm depth for zone 2.

$$P\ 100\text{-year } 4\text{-hour} = 2.40 + (2.60-2.40) * \frac{240-180}{360-180} = 2.47 \text{ in.}$$

$$P\ 10\text{-year } 2\text{-hour} = 0.68 * 2.47 = 1.68 \text{ in.}$$

TREATMENTS

Land treatments are:

Treatment	Land Condition
1	Soil uncompacted by human activity. Native grasses, weeds and shrubs in typical densities with minimal disturbance to grading, groundcover and infiltration capacity. Croplands. Landscaped areas. Unlined arroyos.
2	Soil compacted by human activity. Minimal vegetation. Unpaved parking, roads, trails. Most vacant lots. Lawns, parks and golfcourses.
3	Impervious areas. Pavement. Roofs.

ABSTRACTIONS

Initial abstraction is the rainfall depth which must be exceeded before direct runoff begins. Initial abstraction may be intercepted by vegetation, retained in surface depressions, or adsorbed on the watershed surface. Initial abstraction are:

Treatment	Initial abstraction (inches)
1	0.40
2	0.25
3	0.10

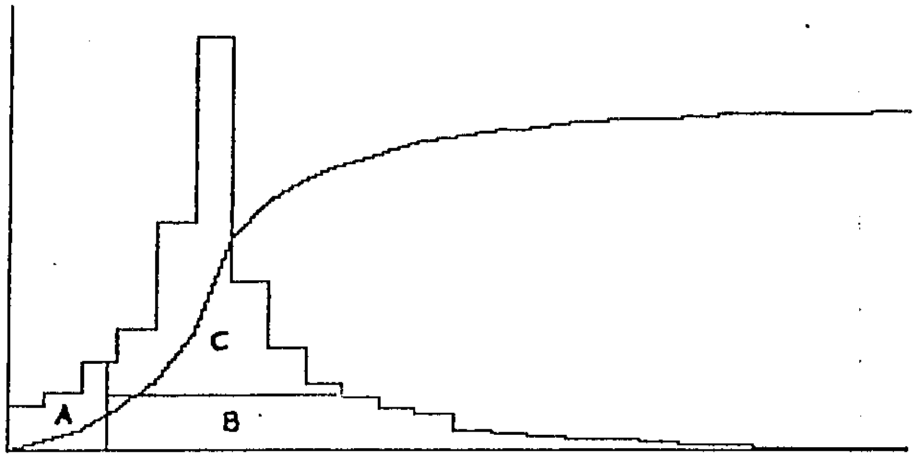
Infiltration is the only significant abstraction after the initial abstraction. After initial abstraction is satisfied, treat infiltration as a constant loss rate,

Treatment	Infiltration (inches/hour)
1	2.0
2	1.0
3	0.02

Runoff from a previous event can saturate a channel bed, rendering it minimally pervious for several days. Do not anticipate additional bed losses for design purposes.

VOLUMETRIC RUNOFF

Excess precipitation P_e is the depth of rainfall remaining after abstractions are removed. Excess precipitation does not depend on watershed area. Determine excess precipitation by subtracting the initial abstraction and infiltration from the design storm hyetograph. The following figure illustrates the development of the excess precipitation for zone 3, treatment 2. The curved line plots cumulative precipitation for zone 3. Rainfall intensities (in/hr) in 5-minute time increments are shown as a histogram. Initial abstraction for treatment 2 is area A. The horizontal line is at a height corresponding to the infiltration rate for treatment 2. Infiltration loss is area B. The remaining histogram area C is excess precipitation.



Excess precipitation by zone and treatment is summarized below.

Excess Precipitation (inches)

	Treatment		
Zone	1	2	3
1	0.55	0.93	1.98
2	0.65	1.06	2.14
3	0.73	1.17	2.38
4	0.79	1.25	2.68

To determine the volume of runoff,

1) Determine the area in each treatment, A_1, A_2 and A_3.

2) Compute the weighted P_e,

Runoff from a previous event can saturate a channel bed, rendering it minimally pervious for several days. Do not anticipate additional bed losses for design purposes.

VOLUMETRIC RUNOFF

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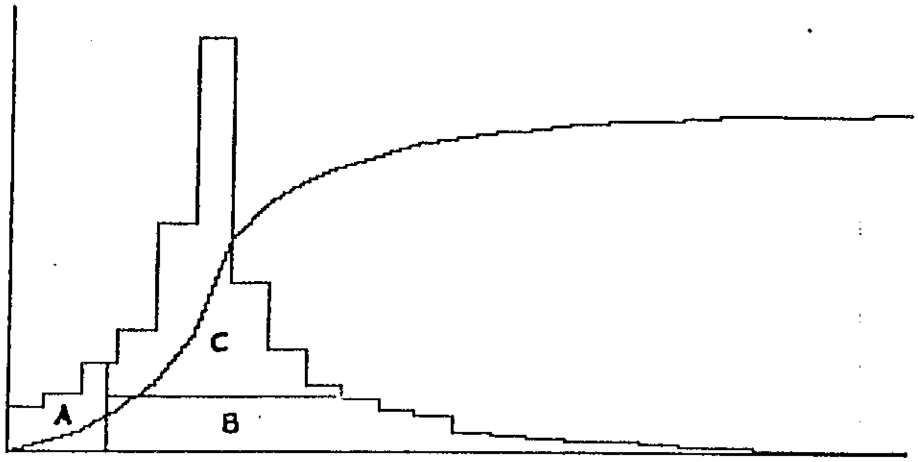


Excess precipitation by zone and treatment is summarized below.

Excess Precipitation (inches)

Zone	Treatment 1	Treatment 2	Treatment 3
1	0.55	0.93	1.98
2	0.65	1.06	2.14
3	0.73	1.17	2.38
4	0.79	1.25	2.68

To determine the volume of runoff,

1) Determine the area in each treatment, A_1, A_2 and A_3.

2) Compute the weighted P_e,

$$\text{Weighted } P_e = \frac{P_{e1}A_1 + P_{e2}A_2 + P_{e3}A_3}{A_1 + A_2 + A_3}$$

3) Multiply the weighted P_e by the watershed area.

Example 2 Find P_e for 100 acres in zone 1. Thirty acres are treatment 1, 50 acres are treatment 2 and 20 acres are treatment 3.

$$\text{Weighted } P_e = \frac{30 * 0.55 + 50 * 0.93 + 20 * 1.98}{100} = 1.03 \text{ in.}$$

$$\text{Volume} = \frac{1.03}{12} * 100 = 8.55 \text{ acre-ft.}$$

DISCHARGE RATE FOR SMALL WATERSHEDS

Small watersheds are less than or equal to 30 acres. Determine peak discharge by the Rational method, as subsequently developed.

{Note: the 30 acre limit is lower than guidelines suggested in some literature, e.g. that one to five square miles can be handled by the method. The areal limit, however, is constrained by the assumption of uniform rainfall intensity over the time of concentration. Thirty acres roughly corresponq to a 10-minute time of concentration. In Albuquerque, 10 minutes is the shortest time increment in which average intensity provides a reasonable description of the heaviest portion of rainfall. Thus as an estimator of peak runoff, the Rational method begins to fail for areas greater than 30 acres.}

Peak Discharge

Use a 10 minute time of concentration. The 10 minute peak intensities are:

Zone	Intensity (in/hr)
1	4.99
2	5.43
3	5.78
4	6.03

The 10-minute duration allows computation of a Rational C consistant with abstraction rates. Assuming peak discharge is attenuated by a factor of 0.6 for treatment 1, 0.8 for treatment 2 and 0.95 for treatment 3, the Rational C's in the following tabulation are the ratios of attenuated excess precipitation to total precipitation in the most intense 10 minutes.

Treatment	Rational Coefficient
1	0.39
2	0.66
3	0.95

{Note the quote from Chow: textbook Rational C values "are applicable for storms of 5 to 10-year frequeuencies. Less frequent higher-intensity storms will require the use of higher coefficients because infiltration and other abstractions have a proportionally smaller effect on peak runoff." Thus higher C's realized under heavy rainfall might be expected. The Denver COG Drainage Criteria Manual quantifies the C shift.}

Combining the intensities and C's, peak flows Q_p are,

Peak Discharge (cfs/acre)

	Treatment		
Zone	1	2	3
1	~~1.81~~ 1.95	~~3.22~~ 3.29	~~4.76~~ 4.74
2	2.07	3.57	5.18
3	2.29	3.86	5.52
4	2.44	4.06	5.76

To determine the peak rate of discharge,

1) Determine the area in each treatment, A_1, A_2 and A_3.

2) Multiply the peak rate for each treatment by the respective areas and sum to compute the total Q_p.

$$\text{Total } Q_p = Q_{p1}A_1 + Q_{p2}A_2 + Q_{p3}A_3$$

Example 3 Find Q_p for 10 acres in zone 1. Three acres are treatment 1, 5 acres are treatment 2 and 2 acres are treatment 3.

$$\text{Total } Q_p = 3 * 1.81 + 5 * 3.22 + 2 * 4.76 = 31 \text{ cfs}$$

Hydrograph

Base time t_b for a small watershed hydrograph is,

$$t_b = 121\ P_e/Q_p - 15\ P_3$$

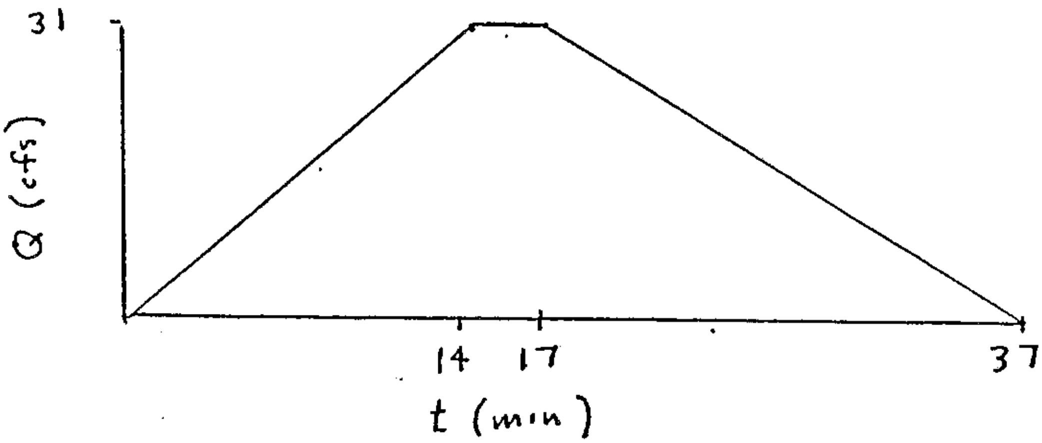
where t_b is in minutes, P_e is in inches, Q_p is in cfs/acre and P_3 is the proportion of the area in treatment 3. Time to peak t_p is $15 - 5\ P_3$ minutes. Continue the peak for 15 P_3 minutes. When P_3 is zero, the hydrograph will be triangular. When P_3 is not zero, the hydrograph will be trapaziodal, indicating a rapid achievement of peak runoff, a constant peak rate through the central portion of the storm, and a rapid diminishment.

Example 4 Determine the hydrograph for example 3. Because the treatments are proportional to those of example 2, P_e = 1.03 in., the same as that of example 2.

t_b = 121 * 1.03/3.1 - 15 * 0.2 = 37 min.

t_p = 15 - 5 * 0.2 = 14 min.

Duration of peak = 15 * 0.2 = 3 min.



DISCHARGE RATE FOR LARGE WATERSHEDS

Large watersheds exceed 30 acres. Determine peak discharge by the HYMO method (Problem-Oriented Computer Language for Hydrologic Modeling, by J.R. Williams and R.W. Hann, Jr., ARS-S-9, USDA, 1973) with the subsequent modifications.

HYMO was developed for watersheds exceeding 320 acres. The transition method discussed below provides a continuous Q_p from the upper boundary of small watersheds by adjusting the HYMO recession coefficient K as the area changes from 30 to 320 acres.

For watersheds containing treatment 3 (an impervious portion), the HYMO structure should be that of two superimposed watersheds, one pervious, the other impervious. The outflow hydrographs are computed independently and summed.

To compute the hydrograph,

Pervious Hydrograph

1) Determine the areas in treatments 1 and 2, A_1 and A_2.

2) HYMO uses curve numbers CN to compute excess precipitation. The following curve numbers yield the same excess precipitation as that

determined by the abstraction model developed earlier in this document.

Curve Numbers

	Treatment	
Zone	1	2
1	76.8	84.8
2	77.0	85.1
3	75.4	83.7
4	72.8	81.2

Multiply the curve number for each treatment by the respective areal proportion and sum to compute the pervious composite curve number.

$$\text{Pervious composite CN} = \frac{CN_1A_1 + CN_2A_2}{A_1 + A_2}$$

3) Compute time of concentration t_c for the entire (pervious and impervious) watershed by the upland method, the sum of the travel times in the subreaches comprising the longest flow path to the watershed outlet.

$$t_c = L_1/V_1 + L_2/V_2 + \ldots$$

where L is the subreach length and V is the velocity in that subreach, as determined in the following equation,

$$V = k \sqrt{s}$$

where s is the slope in percent and k depends upon the conveyance,

k	Conveyance
0.7	Grass and landscaped areas
1	Bare ground
2	Paved areas (sheet flow) and small upland gullies
3	Street flow and channels

4) Set time to peak t_p equal to t_c, but never less than 10 minutes.

The upland method is not the default computation used in HYMO. The code uses an equation similar to the Kirpich equation, without considering channel type.

5a) For watersheds exceeding 320 acres, compute the constant K,

$$K = t_p/2$$

This computation reflects experience in Albuquerque. The default formula in HYMO is not appropriate. The fixed relation for K/t_p makes HYMO into a Snyder hydrograph where C_p is 0.88.

5b) For watersheds between 30 and 320 acres, compute K as follows.

a) Determine the areas in treatments 1 and 2, A_1 and A_2.

b) The following K's employed with a 10-minute t_c on a 30 acre watershed yield approximately the same Q_p as that obtained by the small watershed methodology.

	Treatment	
Zone	1	2
1	0.021	0.027
2	0.025	0.030
3	0.022	0.028
4	0.017	0.024

Multiply the K for each treatment by the respective areal proportion weighted by Rational C (0.39 for treatment 1, 0.66 for treatment 2) and sum to compute the composite K_c.

$$K_c = \frac{0.39\ K_1A_1 + 0.66\ K_2A_2}{0.39\ A_1 + 0.66A_2}$$

c) Compute K for a 320 acre watershed, K_{320}.

$$K_{320} = t_c \sqrt{80/A}$$

where A is the area in acres. If K_{320} is less than 5 minutes, set it equal to 5 minutes.

d) Compute the area-adjusted K by interpolation,

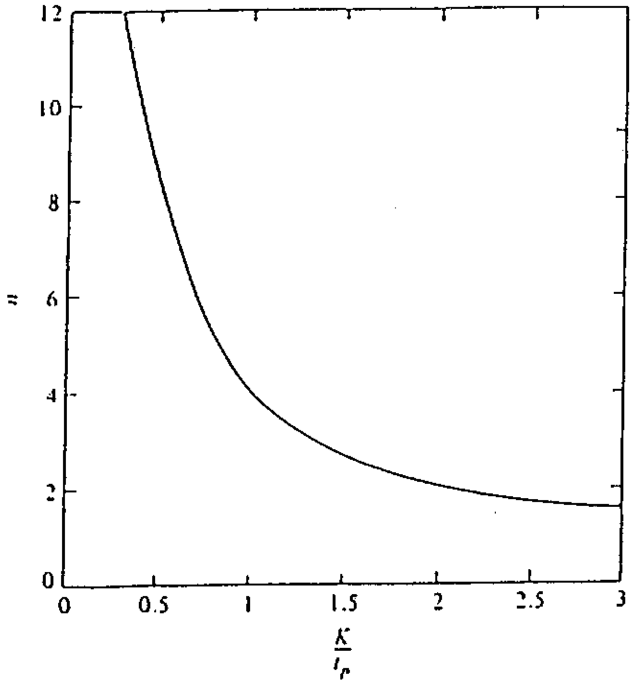
$$K = K_c + (A - 30)*(K_{320} - K_c)/290$$

5c) For watersheds less than 30 acres, where such watersheds are included in a large wtershed study, K equals K_c as determined above. (If a small watershed is being studied on its own, the small watershed methodology is appropriate.)

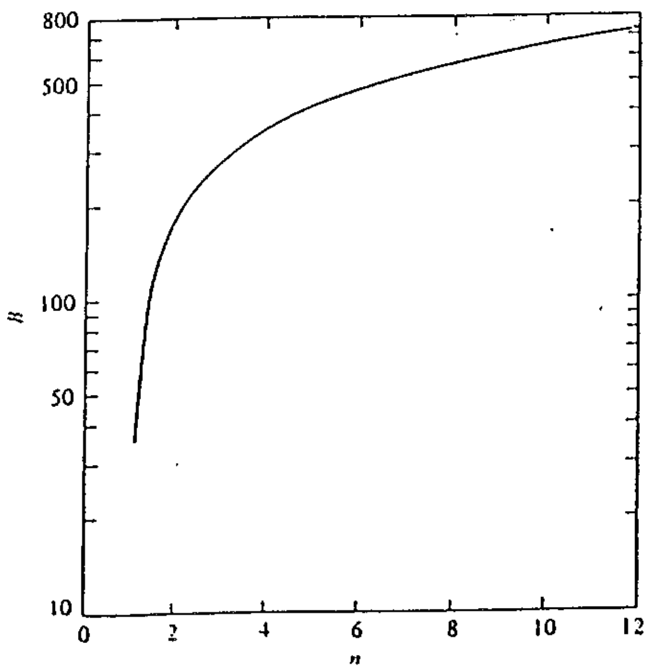
6) Determine the single time-step duration unit hydrograph from the HYMO procedure by inputting the pervious composite CN, t_p and K. Input only the pervious area. Input K and t_p as a negative values in hours in the COMPUTE HYD statement, as described in the HYMO manual.

The graphical solution that follows is carried out by the code.

a) Find the shape parameter n from the following figure. For watersheds over 320 acres, K/t_p is 0.5, so n will be 7.94



c) Find the watershed parameter B from the following figure. For watersheds over 320 acres, n is 7.94, so B will be 564.

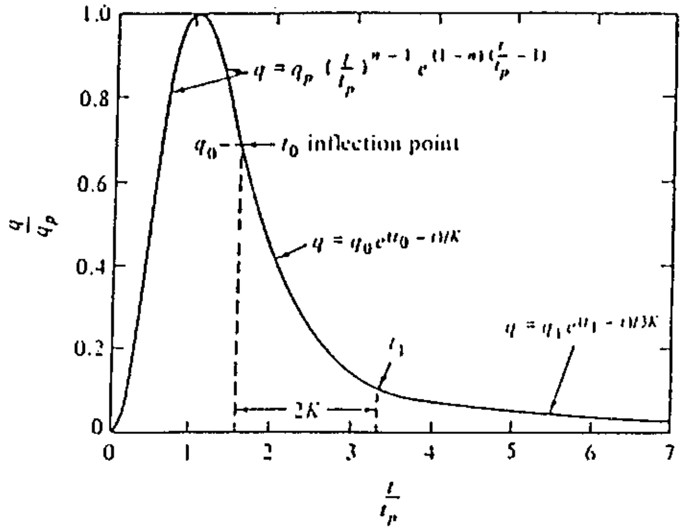


d) Compute peak unit hydrograph discharge,

$$U_p = BA/t_p$$

where U_p is in cfs, A is in mi^2, and t_p is in hours.

e) Compute a complete unit hydrograph using the following figure and equations. (The figure uses q_p in place of U_p.)



f) Multiply the vector of excess precipitation by the matrix of lagged unit hydrographs (the spreadsheet-type computation documented in hydrology texts) to determine the storm hydrograph. Use time increments of 5 minutes.

<u>Impervious Hydrograph</u>

The steps are the same as those for the pervious portion, with the following exceptions.

1) Consider only the impervious area A_3.

2) Use a curve number of 97.9 for all zones.

3) t_c is the same, that of the entire watershed.

4) t_p is the same, that of the entire watershed.

5a) K is the same, that of the entire watershed.

5b) K_{320} is the same. K_c is 0.56.

5c) K is 0.56.

6) The differences in the COMPUTE HYD statement are the ID, HYD NO, DA and CN. K will be different for basins less than 320 acres.

<u>Combined Hydrographs</u>

Use the ADD HYD command to combine the pervious and impervious hydrographs.

<u>Example 5</u> Find Q_p for 800 acres in zone 1. Treatments are 240 acres of treatment 1, 400 acres of treatment 2 and 160 acres of treatment 3, proportional to treatments in example 2. The flow path is: 600 ft. bare ground, s = 5 percent; 2000 ft. gullies, s = 3 percent ; and 6000 ft. lined

channel, s = 2 percent.

$$\text{Pervious curve number} = \frac{240 * 76.8 + 400 * 84.8}{640} = 81.8$$

$$t_p = \frac{600}{1 \sqrt{5}} + \frac{2000}{2 \sqrt{3}} + \frac{6000}{3 \sqrt{2}} = 2258 \text{ sec} = 0.627 \text{ hr}$$

$K = 0.627/2 = 0.313$ hr

$n = 7.94$, $B = 564$

$U_p = 564 * 1.00/0.627 = 901$ cfs

The full unit hydrograph specitication and the multiplication of the excess precipitation by the hydrograph matrix requires several pages of figures. Rather, use area = 1.00 mi^2, CN = 81.8, K = 0.313, t_p = 0.627 and the zone 1 hyetograph in the COMPUTE HYD statement in HYMO. Q_p for the pervious watershed is 398 cfs at 1.17 hrs.

For the impervious watershed, area = 0.25 mi^2 and CN = 97.9. HYMO yields a U_p of 225 cfs. Q_p of the impervious watershed is 293 cfs at 1.08 hrs.

Adding the two hydrographs, the combined Q_p = 679 cfs at 1.17 hrs.

<u>Example 6</u> Find Q_p for 120 acres in zone 1. Treatments are 36 acres of treatment 1, 60 acres of treatment 2 and 24 acres of treatment 3, proportional to the treatments of example 2. The flow path is: 232 ft. bare ground, s = 5 percent; 774 ft. gullies, s = 3 percent ; and 2322 ft. lined channel, s = 2 percent, proportional to the path in example 5.

Pervious curve number = 81.8 (example 5)

$$t_p = \frac{232}{1 \sqrt{5}} + \frac{774}{2 \sqrt{3}} + \frac{2322}{3 \sqrt{2}} = 874 \text{ sec} = 0.243 \text{ hr}$$

$$K_c = \frac{0.39 * 0.021 * 36 + 0.66 * 0.027 * 60}{0.39 * 36 + 0.66 * 60} = 0.025 \text{ hr}$$

$K_{320} = 0.243 \sqrt{80/120} = 0.198$ hr

$K = 0.025 + (120 - 30)*(0.198 - 0.025)/290 = 0.079$ hr

Plugging into HYMO, the pervious Q_p = 120 cfs

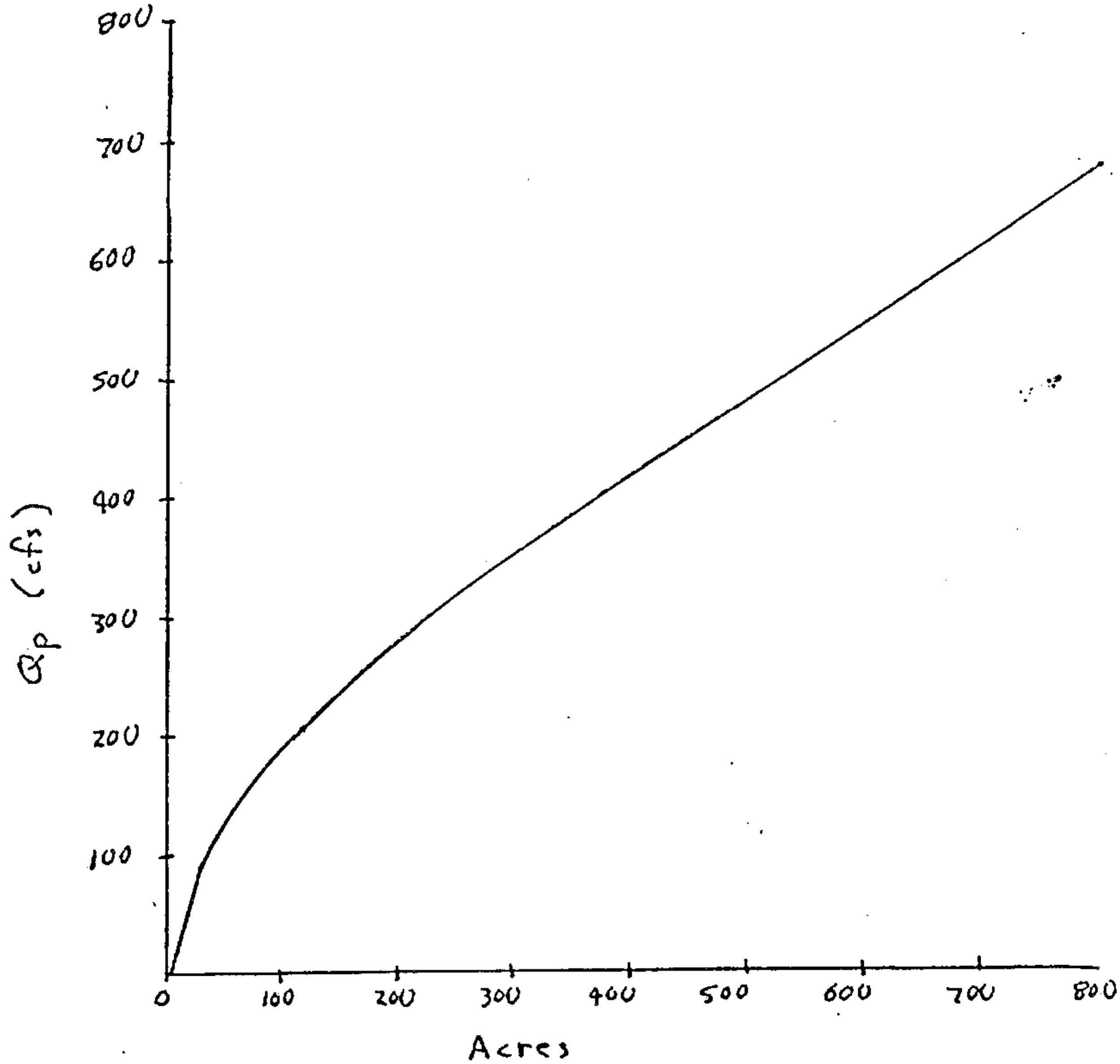
Impervious curve number = 97.9

$K = 0.056 + (120 - 30)*(0.198 - 0.056)/290 = 0.100$

Plugging into HYMO, the impervious Q_p = 86 cfs

Combined Q_p = 206 cfs

The figure below shows Q_p for the areas as calculated in examples 3, 5, and 6, all of which have the same proportional treatments.



APPENDIX D

HYDROLOGIC RESERVOIR ROUTING WITHOUT 13TH AND 17TH AVENUE DIVERSIONS

Gannett Fleming
ENGINEERS AND PLANNERS

BY LDS DATE July 9 CHKD. BY DATE

JOB NO.

13th and 17th Avenue

Diversions Removed

- Route thru Detention Ponds

Create Hydrographs

Basin #2 - Area = 9.81 acres

Land Usage

Treatment 2 - $8.10/9.81 = 82.6\% \times 3.29$ cfs/ac = 26.65 cfs

Treatment 3 - $1.71/9.81 = 17.4\% \times 4.74$ cfs/ac = 8.11 cfs

TOTALS 100.0% 34.76 cfs

$t_p = 15 - 5(.174) = 14.1$ min

$$P_e = \frac{(.93)(8.10) + (1.98)(1.71)}{9.81} = 1.113 \text{ in}$$

$Q_p = (3.29)(.826) + (4.74)(.174) = 3.542$ cfs/ac

$t_b = 121(1.113/3.54) - 15(.174) = 35.4$ min

Duration Peak = $15(.174) = 2.61$ min

Lag Time = 6050 ft / 9 ft/sec = 11.2 min

Basin #3 Area = 16.9 acres

Land Usage

Treatment #2 = 14.34/16.9 = 84.9% × 3.29 cfs/ac = 47.2 cfs

Treatment #3 = 2.56/16.9 = 15.1% × 4.74 cfs/ac = 12.1 cfs

Totals 100.0% 59.3 cfs

$t_p = 15 - 5(.15)) = 14.2$ min

$P_e = \frac{.93(14.34) + (1.98)(2.56)}{16.9} = 1.089$ in

$Q_p = (3.29)(.849) + (4.74)(.151) = 3.509$ cfs/ac

$t_b = 121\left(1.089/3.509\right) - 15(.151) = 35.3$ min

Duration of Peak = $15(.151) = 2.27$ min

Lag Time = 3570 ft / 9 ft/sec = 6.6 min

COMBINED HYDROGRAPH
5-YEAR STORM
W/ 13th and 17th DIVERSION

TIME (MIN)	DRAINAGE AREA 2	DRAINAGE AREA 3	DRAINAGE AREA 4	DRAINAGE AREA 5	DRAINAGE AREA 6	DRAINAGE AREA 7	COMBINED HYD. @ POND NO. 1		
	Time Lag = 11 min	Time Lag = 7 min	Time Lag = 1 min	Time Lag = 4 min	Time Lag = 1 min	Time Lag = 1 min			
0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0	0
2	0.0	0.0	0.0	1.9	4.9	2.6	9.5	567	1135
4	0.0	0.0	1.7	3.8	9.8	5.3	20.6	2371	2472
6	0.0	0.0	3.4	5.7	14.7	7.9	31.7	5511	3809
8	0.0	4.6	5.1	7.6	19.6	10.6	47.5	10264	5697
10	0.0	9.2	6.7	9.5	24.6	13.2	63.2	16905	7585
12	2.7	13.8	8.4	11.4	29.5	15.9	81.6	25595	9796
14	5.4	18.4	10.1	13.3	34.4	18.5	100.1	36496	12007
16	8.1	22.9	11.8	15.2	34.4	18.5	110.9	49155	13311
18	10.8	27.5	13.5	13.8	31.3	18.5	115.4	62734	13847
20	13.5	32.1	15.2	12.4	28.1	16.7	118.0	76737	14160
22	16.2	29.2	13.7	11.0	25.0	14.8	109.9	90411	13188
24	18.9	26.3	12.1	9.7	21.9	13.0	101.8	103113	12216
26	17.2	23.4	10.6	8.3	18.8	11.1	89.3	114578	10714
28	15.4	20.4	9.1	6.9	15.6	9.3	76.8	124542	9213
30	13.7	17.5	7.6	5.5	12.5	7.4	64.3	133004	7712
32	12.0	14.6	6.1	4.1	9.4	5.6	51.8	139965	6210
34	10.3	11.7	4.6	2.8	6.3	3.7	39.2	145425	4709
36	8.6	8.8	3.0	1.4	3.1	1.9	26.7	149383	3208
38	6.9	5.8	1.5	0.0	0.0	0.0	14.2	151840	1706
40	5.1	2.9	0.0	0.0	0.0	0.0	8.1	153177	968
42	3.4	0.0	0.0	0.0	0.0	0.0	3.4	153867	412
44	1.7	0.0	0.0	0.0	0.0	0.0	1.7	154175	206
46	0.0	0.0	0.0	0.0	0.0	0.0	0.0	154278	0

COMBINED HYDROGRAPH
100-YEAR STORM
W/O 13th and 17th DIVERSION

TIME (MIN)	DRAINAGE AREA 2	DRAINAGE AREA 3	DRAINAGE AREA 4	DRAINAGE AREA 5	DRAINAGE AREA 6	DRAINAGE AREA 7	COMBINED HYD. @ POND NO. 1		
	Time Lag = 11 min	Time Lag = 7 min	Time Lag = 1 min	Time Lag = 4 min	Time Lag = 1 min	Time Lag = 1 min			
0	0	0	0	0	0	0	0.0	0	0
2	0	0	0.0	3.5	8.9	4.8	17.2	1032	2063
4	0	0	3.1	6.9	17.9	9.6	37.5	4910	4494
6	0	0	6.1	10.4	26.8	14.4	57.7	10020	6925
8	0	8.3	9.2	13.8	35.7	19.3	86.3	18662	10358
10	0	16.7	12.3	17.9	44.6	24.1	114.9	30736	13790
12	4.9	25.0	15.3	20.7	53.6	28.9	148.4	46536	17810
14	9.8	33.4	18.4	24.2	62.5	33.7	181.9	66356	21831
16	14.7	41.7	21.5	27.6	62.5	33.7	201.7	89372	24202
18	19.6	50.1	24.5	25.1	56.8	33.7	209.8	114061	25176
20	24.5	58.4	27.6	22.6	51.1	30.3	214.5	139522	25746
22	29.4	53.1	24.8	20.1	45.5	27.0	199.8	164384	23978
24	34.3	47.8	22.1	17.6	39.8	23.6	185.1	187478	22211
26	31.2	42.5	19.3	15.1	34.1	20.2	162.3	208324	19481
28	28.1	37.2	16.6	12.5	28.4	16.9	139.6	226440	16751
30	24.9	31.9	13.8	10.0	22.7	13.5	116.8	241826	14021
32	21.8	26.5	11.0	7.5	17.0	10.1	94.1	254482	11291
34	18.7	21.2	8.3	5.0	11.4	6.7	71.3	264409	8562
36	15.6	15.9	5.5	2.5	5.7	3.4	48.6	271606	5832
38	12.5	10.6	2.8	0.0	0.0	0.0	25.9	276073	3102
40	9.4	5.3	0.0	0.0	0.0	0.0	14.7	278504	1760
42	6.2	0.0	0.0	0.0	0.0	0.0	6.2	279758	
44	3.1	0.0	0.0	0.0	0.0	0.0	3.1	280319	
46	0.0	0.0	0.0	0.0	0.0	0.0	0.0	280506	