

STORMWATER DETENTION ANALYSIS
BRANDYWINE ESTATES
BAX PROJECT NO. 85-2897A

PREPARED FOR:

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OCTOBER, 1988

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I. PURPOSE:

The purpose of this report is to estimate the attenuation characteristics of the detention facility that is proposed to be constructed as part of the subdivision improvements of the 16.63 Acre tract of land known as "Brandywine Estates" Subdivision.

II. SCOPE:

This report estimates the expected attenuation characteristics during the 25 year frequency storm of 20 minutes duration, utilizing the rational method of estimating storm runoff to the detention facility.

III. DETENTION CONCEPT:

The proposed site improvements include construction of a detention basin at a low point of the subdivision. The basin will be utilized as a stormwater detention facility, and outflow will discharge at the South property line of the tract. The storage volume and outflow rates have been analyzed to determine the peak rate of runoff leaving the detention facility, and hence the attenuation provided by the detention facility for the design storm.

IV. STORMWATER RUNOFF INFORMATION:

General Site Data and Runoff Calculations for the subdivision are shown on Exhibit 'A'.

Estimated Inflow Hydrograph calculations for the design storm are shown on Exhibit 'B'.

V. DETENTION BASIN CHARACTERISTICS:

The depth-storage characteristics of the proposed basin are shown on Exhibit 'C'.

The selection of an outflow pipe sizes and outflow pipe's performance calculations are shown on Exhibit 'D'.

VI. ROUTING PROCEDURE:

The modified Puls routing procedure was used to estimate the effects of storage volume on outflow rate.

Exhibit 'E' details the development of the routing curve and displays the routing calculations.

A storm of great intensity is checked for safe storage and overflow on Exhibit 'F'.

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Brandywine Estates
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VII. SUMMARY:

The proposed detention facility is allowed a maximum outflow of 358.79 c.f.s. In considering the design 25 year frequency storm of 20 minutes, the peak outflow is 355.0 c.f.s.

A summary is shown on Exhibit 'G'.

Graph's of the detention basin's inflow hydrograph, depth-storage characteristics, depth-outflow characteristics and the design routing curve are all shown on Exhibit 'H'.

EXHIBITS

GENERAL SITE DATA & RUNOFF CALCULATIONS

1. The tract of land to be developed into "BRANDYWINE ESTATES" Subdivision is situated in the Dardenne Creek watershed. Presently, the site is situated in a zone C, which is a zone of minimal flooding, on the Flood Insurance Rate Maps of St. Charles County, Missouri, No. 2903150250A.
2. For the purpose of estimating runoff, the following rational method factors will be used:

PRE-DEVELOPED CONDITIONS: 0-5% IMPERVIOUS

$$15 \text{ year Storm P.I.} = 1.87 \text{ c.f.s./A} \leq$$

$$25 \text{ year Storm P.I.} = 2.31 \text{ c.f.s./A} \leq$$

$$100 \text{ year Storm P.I.} = 2.95 \text{ c.f.s./A} \leq$$

POST-DEVELOPED CONDITIONS: 40% IMPERVIOUS

$$15 \text{ year Storm P.I.} = 2.64 \text{ c.f.s./A} \leq$$

$$25 \text{ year Storm P.I.} = 3.26 \text{ c.f.s./A} \leq$$

$$100 \text{ year Storm P.I.} = 4.17 \text{ c.f.s./A} \leq$$

3. Total Site Acreage = 16.629 Acres

EXHIBIT "A"

SHEET 1 OF 2

4. For the purpose of conforming to the City of O'Fallon's requirements, the 25 year P.I factors will be used. Differential rate of runoff = 0.95 cfs/Ac ($3.26 \text{ c.f.s./Ac} - 2.31 \text{ c.f.s./Ac}$).

5. The detention facility must "attenuate" flow so as not to increase the runoff for the total site. Considering the entire site area, 16.629 Acres, the "attenuation" of the basin equals 15.80 c.f.s. (site area * Differential rate of runoff; $16.629 \text{ Acres} * 0.95 \text{ c.f.s./Acres}$)

INFLOW HYDROGRAPH CALCULATIONS

1. of the flows that will inflow to the proposed detention basin, the most remote point of origination is the northeasternmost corner of Lot 9.

The travel time is:

- overland flow (TERRACE IN LOT 9)

$$\text{length} = 40'$$

$$\text{slope} = \frac{571 - 564}{40} = 17.5\%$$

$$\text{Velocity} \approx 6.4 \text{ ft/s}$$

$$\text{travel time} \approx 6.3 \text{ sec.}$$

- overland flow (LOT 9)

$$\text{length} = 105'$$

$$\text{slope} = \frac{564 - 563.6}{105} = 0.4\%$$

$$\text{velocity} \approx 1 \text{ ft/s}$$

$$\text{travel time} \approx 105 \text{ sec.}$$

- pavement flow (From Lot 9 to CI 120)

$$\text{length} = 185'$$

$$\text{slope} = \frac{563.6 - 559.9}{185} \approx 2\%$$

$$\text{velocity} = 2.75 \text{ ft/s}$$

$$\text{travel time} \approx 67.3 \text{ sec.}$$

- pipe flow (From CI 120 to EP 101)

$$\text{total length} = 939.2'$$

$$\text{Velocity (assumed)} = 7 \text{ ft/s}$$

$$\text{travel time} \approx 134.2 \text{ sec.}$$

The total travel time =

$$6.3 + 105 + 67.3 + 134.2 = 312.8 \text{ sec.}$$

$$t_c \approx 5.21 \text{ min.}$$

2. For Inflow Hydrograph use $t_c = 6 \text{ min.}$

3. A storm duration equal to the time of concentration of the site could be analyzed. Since the time of concentration of the site is so short, $t_c = 6 \text{ min.}$, it is the policy of this office that the shortest storm duration analyzed is a 20 minute storm. Therefore, a 20 min. duration storm was chosen.

4. From the Drainage Area Map, peak inflow to the basin for a 15 year 20 min. storm is as follows:

Pipe flow : 294.23 c.f.s.

Direct flow : 9.08 c.f.s.

TOTAL: 303.31 c.f.s.

5. Inflow Hydrograph for 15 year 20 min. storm, $t_c = 6$ min.

<u>TIME</u> (minutes)	<u>INFLOW</u> (c.f.s.)	<u>REMARKS</u>
0	0	DESIGN RAIN BEGINS
6	303.31	BEGIN PEAK INFLOW
12	303.31	
20	303.31	
26	0	INFLOW ENDS

6. To change Inflow from 15 year frequency storm to 25 year frequency storm, multiply inflow by:

$$\frac{\text{P.I. factor 25 yr.}}{\text{P.I. factor 15 yr.}} = \frac{3.26}{2.64} = 1.235$$

$$Q_{25} = 303.31 \text{ cfs} \times 1.235 = 374.59 \text{ c.f.s.}$$

EXHIBIT "B"

SHEET 3 OF 5

7. Inflow Hydrograph for 25 year 20 min. storm, $t_c = 6$ min.

<u>TIME</u> (minutes)	<u>INFLOW</u> (c.f.s.)	<u>REMARKS</u>
0	0	DESIGN RAIN BEGINS
6	374.59	BEGIN PEAK INFLOW
12	374.59	
20	374.59	
26	0	INFLOW ENDS

8. To change inflow from a 15 year frequency storm to a 100 year frequency storm, multiply inflow by:

$$\frac{\text{P.I. factor 100 yr.} = 4.17}{\text{P.I. factor 15 yr.} = 2.64} = 1.58$$

$$Q_{100} = 303.31 \text{ c.f.s.} \times 1.58 = 479.23 \text{ c.f.s.}$$

9. Inflow Hydrograph for 100 year, 20 min. storm, $t_d = 6$ min.

<u>TIME</u> (minutes)	<u>INFLOW</u> (c.f.s.)	<u>REMARKS</u>
0	0	DESIGN RAIN BEGINS
6	479.23	BEGIN PEAK INFLOW
12	479.23	↑ INFLOW RATE CONSTANT @ PEAK RATE ↓
⋮	⋮	
20	479.23	DESIGN RAIN ENDS
26	0	INFLOW ENDS

EXHIBIT "B"

SHEET 5 OF 5

DEPTH-STORAGE VOLUME CALCULATIONS

<u>ELEV.</u>	<u>AREA</u>	<u>AVE. AREA</u>	<u>INCR. OF DEPTH</u>	<u>INCR OF VOLUME</u>	<u>TOTAL VOLUME</u>
FLOWLINE 539.6	0				0
		0.007	0.4	0.003	
540.0	0.013				0.003
		0.065	2	0.130	
542.0	0.117				0.133
		0.196	2	0.392	
544.0	0.274				0.525
		0.325	2	0.650	
546.0	0.375				1.175
		0.427	2	0.854	
TOP BERM 548.0	0.479				2.029

DEPTH - OUTFLOW CALCULATIONS

1. Maximum Permissible Outflow equals
Peak Inflow - Attenuation:

$$374.59 \text{ c.f.s.} - 15.80 \text{ c.f.s.} = 358.79 \text{ c.f.s.}$$

Outflow pipes: 1-48" RCP & 2-42" RCP

2. Performance Calculations:

Orifice Equation $\rightarrow Q = C a \sqrt{2gh}$

$$a = 1 \times 12.566 + 2 \times 9.621 = 31.81 \text{ ft}^2$$

$$C = 0.6$$

$$g = 32.2$$

h = height from Springline of pipes;
Springline of all pipes = 541.6

<u>ELEV.</u>	<u>h</u>	<u>Q_{out}</u>
539.6	0	0
542.0	0.4	96.87
543.0	1.4	181.23
544.0	2.4	237.28
545.0	3.4	282.42
546.0	4.4	321.28
547.0	5.4	355.92
548.0	6.4	387.48

EXHIBIT "D"

SHEET 1 OF 1

ROUTING CURVE CALCULATIONS

Let $\Delta t = 1 \text{ minute} = 0.0166 \text{ hrs}$

Then $\frac{25}{\Delta t} + \text{Outflow} = \frac{2.5}{0.0166} \times \frac{24 \text{ (hrs/day)}}{1.98 \text{ (Ac-FT)}} + 0$
(cfs-day)

$$\frac{25}{\Delta t} + 0 = 1454.5 * S + 0 \quad (\text{c.f.s.})$$

<u>ELEV.</u>	<u>S</u> (AC-FT)	<u>0</u> (c.f.s.)	<u>$\frac{25}{\Delta t} + 0$</u> (c.f.s.)
539.6	0	0	0
540.0	0.003	4.0 **	8.36
541.0	0.038 *	29.0 **	84.27
542.0	0.133	196.87	290.32
543.0	0.294 *	181.23	608.85
544.0	0.525	237.28	1,000.89
545.0	0.820 *	282.42	1,475.11
546.0	1.175	321.28	2,030.32
547.0	1.590 *	355.92	2,668.58
548.0	2.029	387.48	3,338.66

** FROM DEPTH OUTFLOW CURVE

* FROM DEPTH STORAGE CURVE

EXHIBIT "E"

SHEET 1 OF 3

25 Year, 20 minute Storm

Design Pond Routing

FORM 10Z	0	1	2	3	4	5	6	7
Line	Time	I_1	$I_1 + I_2$	$\frac{2S_1}{\Delta t} - O_1$	$\frac{2S_2}{t} + O_2$	Elev	Outflow O_2	Storage S_2
1	0	0		0			0	
2	1	62.43	62.43	0	62.43		212	
3	2	124.86	187.30	20.43	207.13		692	
4	3	187.30	312.16	69.73	381.89		1272	
5	4	249.73	437.03	126.89	563.92		1722	
6	5	312.16	561.89	219.92	781.81		2072	
7	6	374.59	686.75	366.81	1053.56		2442	
8	7	374.59	749.18	565.56	1319.74		2692	
9	8	374.59	749.18	775.74	1524.92		2872	
10	9	374.59	749.18	950.92	1700.10		2992	
11	10	374.59	749.18	1101.10	1850.28		3092	
12	11	374.59	749.18	1231.28	1980.46		3162	
13	12	374.59	749.18	1348.46	2097.64		3242	
14	13	374.59	749.18	1448.64	2197.82		3282	
15	14	374.59	749.18	1540.82	2290.00		3352	

Design Pond Routing

25 Year; 20 minute storm



FORM 10Z	0	1	2	3	4	5	6	7
Line	Time	I_1	$I_1 + I_2$	$\frac{25I_1}{\Delta t} - O_1$	$\frac{25I_2}{t} + O_2$	Elev	Outflow O_2	Storage S_2
1	14	374.59	749.18	1,540.82	2,290.00		3350	
2	15	374.59	749.18	1,620.00	2,369.18		3390	
3	16	374.59	749.18	1,691.18	2,440.36		3430	
4	17	374.59	749.18	1,754.36	2,503.54		3460	
5	18	374.59	749.18	1,811.54	2,560.72		3495	
6	19	374.59	749.18	1,861.72	2,610.90		3525	
7	20	374.59	749.18	1,905.90	2,655.08	547.0	3550	69260 CU. FT.
8	21	312.16	686.75	1,945.08	2,631.83		3530	1.59 ASFT
9	22	249.73	561.89	1,925.83	2,487.72		3455	
10	23	187.30	437.03	1,796.72	2,233.75		3315	
11	24	124.86	312.16	1,570.75	1,882.91		3110	
12	25	62.43	187.30	1,260.91	1,448.21		2815	
13	26	0	62.43	885.21	947.64		2300	
14								
15								

STORM OF GREAT INTENSITY CHECK /
OUTFALL STRUCTURE DESIGN

100 year Storm $Q = 479.23$ c.f.s.

Since the outflow pipes are 42" (2) and 48" (1) R.C. pipes, it is safe to assume that these pipes will not become clogged. The outfall structure will be sized by a trial and error process using the weir equation and the orifice equation.

Try an outfall structure of a Open Top Box with Weir Length of $(6.5)4 = 26$ FT.
Set Sill at 547.0

Assume water Elev. 548.1

height above springline of pipes = h

$$h = 548.10 - 541.60 = 6.5$$

$$Q_{\text{orifice}} = C_d \sqrt{2gh}$$

$$a = 31.81 \text{ ft}^2$$

$$Q = 0.6(31.81) \sqrt{64.4(6.5)} = 390.49 \text{ c.f.s.}$$

$$Q_{\text{weir}} = Q_{100} - Q_{\text{orifice}} = 479.23 - 390.49$$

$$Q_w = 88.74 \text{ c.f.s.}$$

$$Q = CLH^{3/2}$$

$$H = \left(\frac{Q}{CL}\right)^{0.67} = \left(\frac{88.74}{3(26.9)}\right)^{0.67} = 1.1'$$

$$548.1 - 547.0 = 1.1' = 1.1' \text{ assumed } \checkmark \checkmark \quad (\text{OK})$$

Therefore,

- Use an Open Top Box (with Re-Bar Grate as Lid) for Outfall Structure
- Set the sill at ELEV. 547.00.

Check capacity of 48" R.C.P. with extra 88.74 c.f.s. from 100 year storm:

$$Q_{2-42"} = C a \sqrt{2gh} \quad a=19.24 ; h=6.5 ; c=0.6$$

$$Q_{2-42"} = 236.19 \text{ c.f.s.}$$

$$Q_{48"} = 384.44 \text{ c.f.s.} - 236.19 \text{ c.f.s.} = 148.25 \text{ c.f.s.}$$

plus additional flow of 88.74 c.f.s.

$$\text{new } Q_{48"} = 236.99 \text{ c.f.s.}$$

Change 48" RCP to 60" R.C.P. FROM
OUTFALL STRUCTURE TO END OF PIPE AT
CREEK. KEEP 48" RCP IN BASIN.

NO. 2400-20 DIETZEN BRASS PIPES
20 X 33 PER INCH

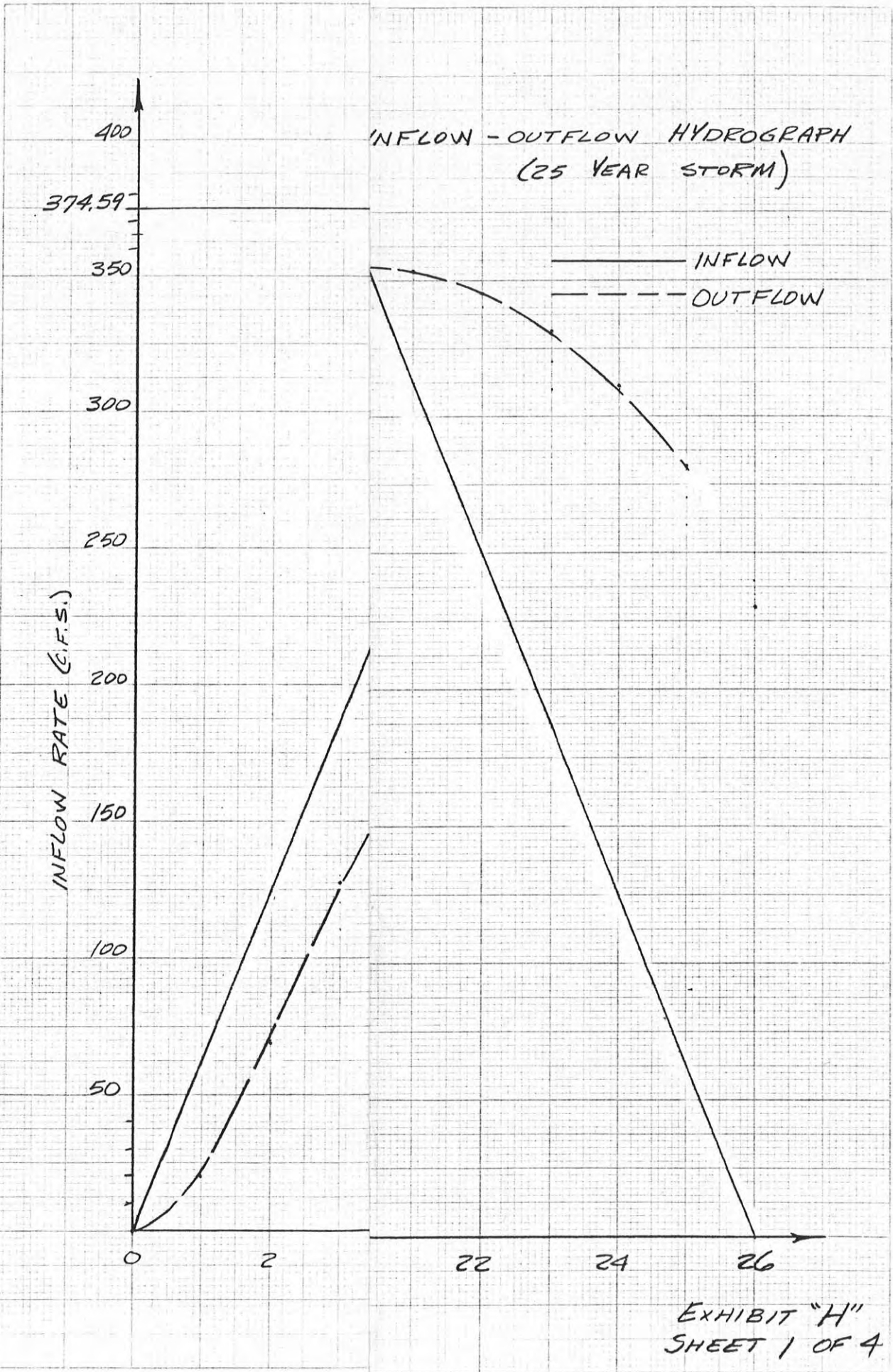


EXHIBIT "H"
SHEET 1 OF 4

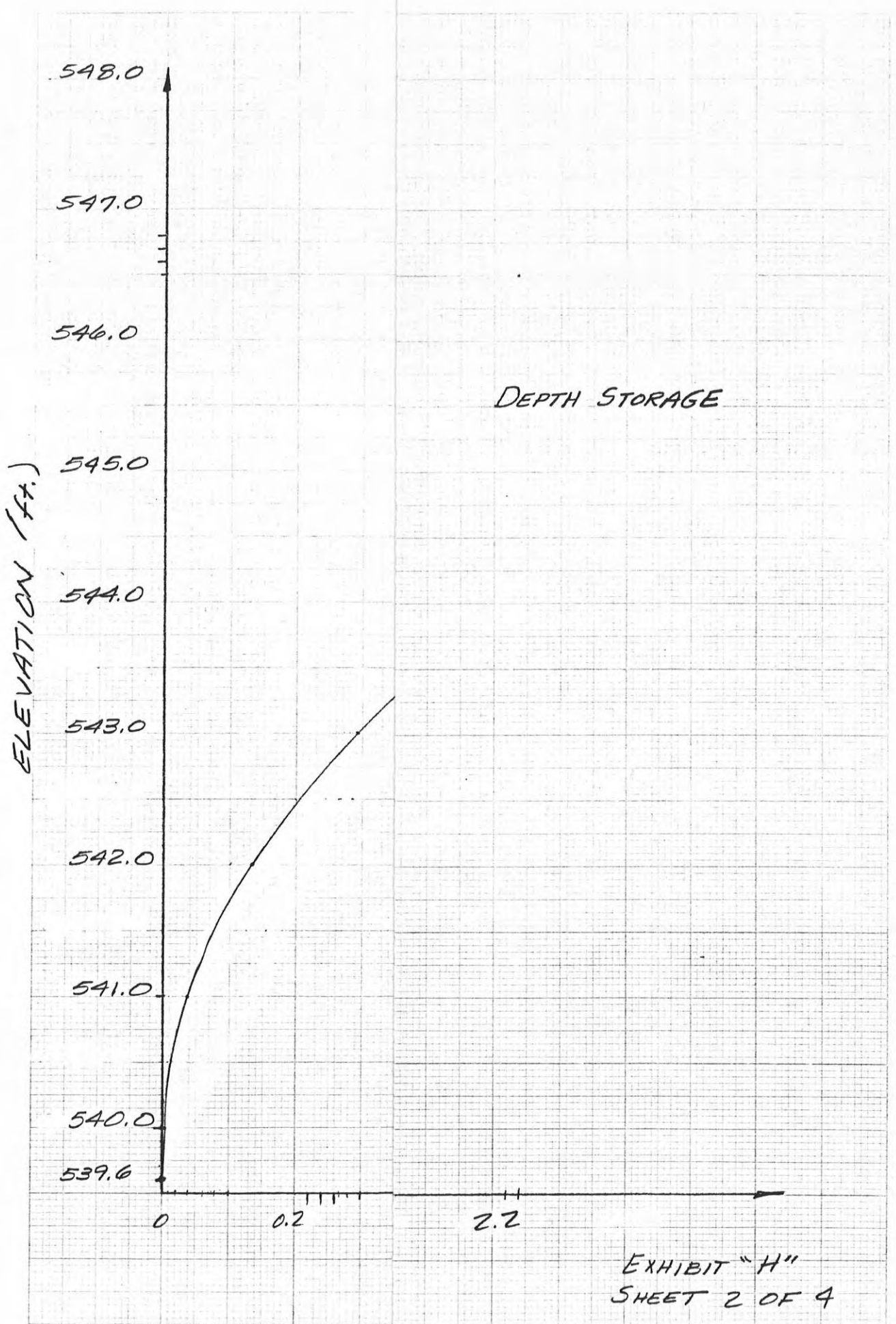


EXHIBIT "H"
SHEET 2 OF 4

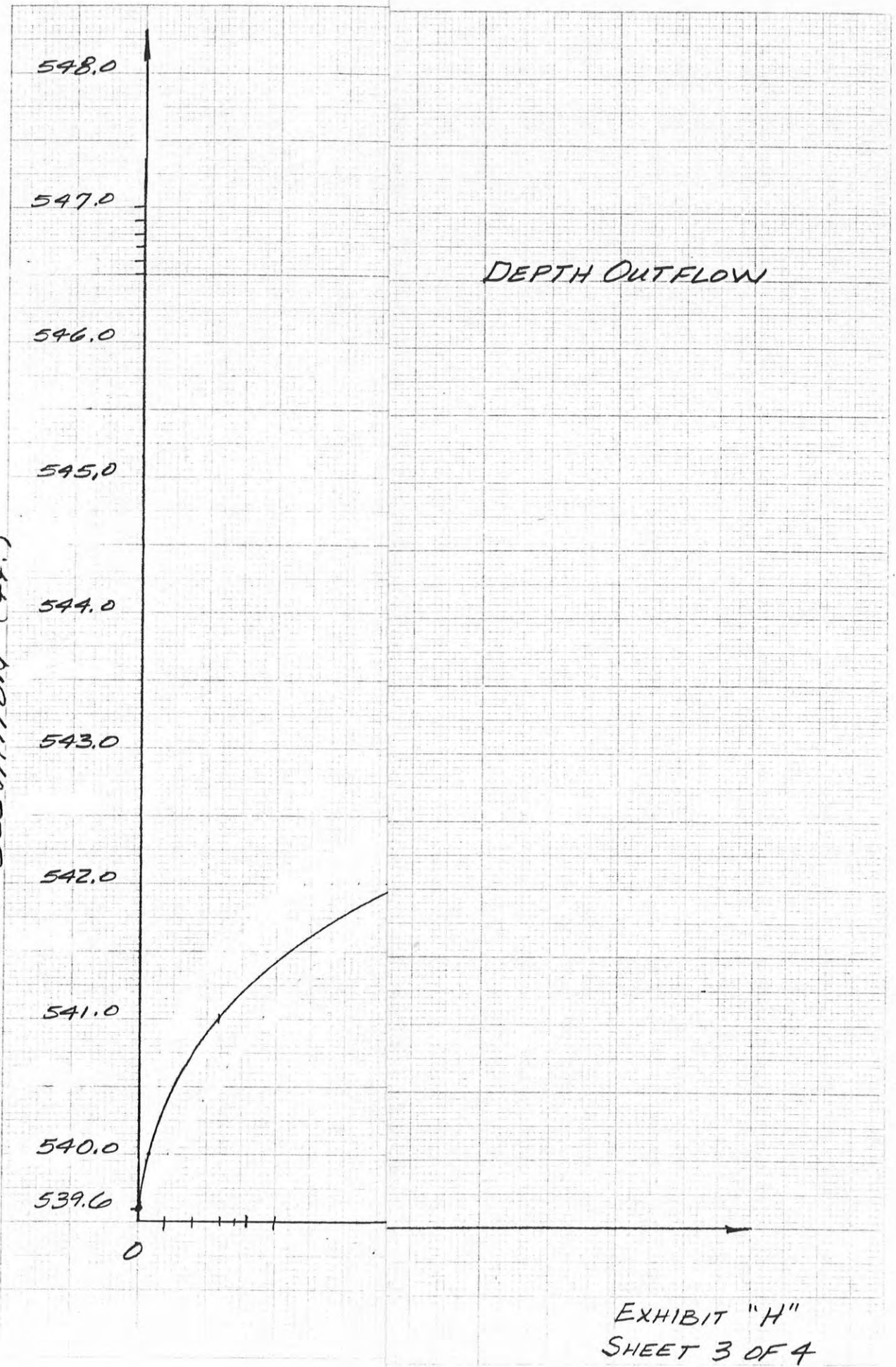
ELEVATION (ft.)

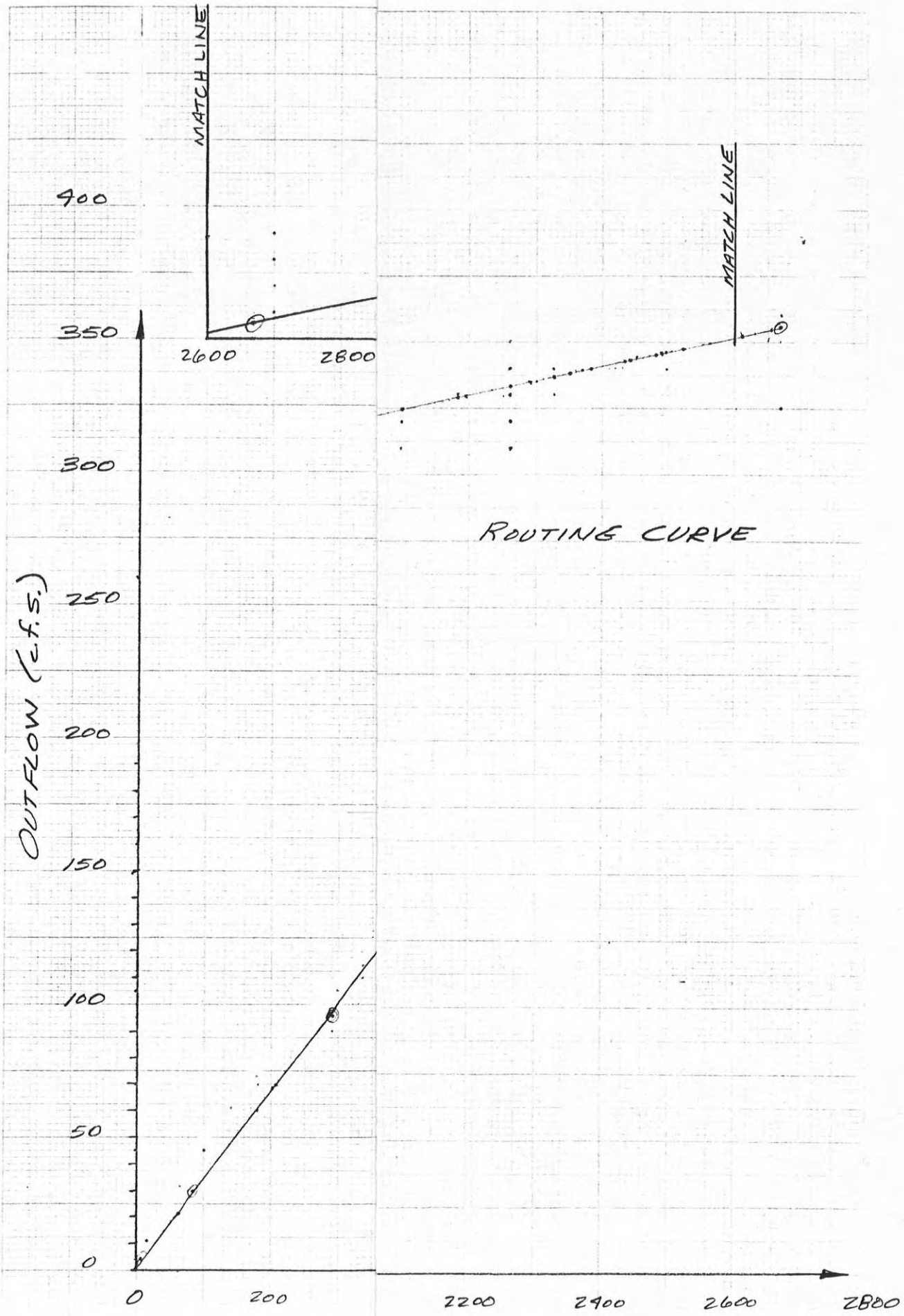
548.0
547.0
546.0
545.0
544.0
543.0
542.0
541.0
540.0
539.6

DEPTH OUTFLOW

0

EXHIBIT "H"
SHEET 3 OF 4





ROUTING CURVE