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

PREPARED FOR:

TRAVIS SHANE CORPORATION
3910 OLD HIGHWAY 94 SOUTH, SUITE 114
ST. CHARLES MO 63303

PREPARED BY:

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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 outlfow 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'.

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 year Storm P.I. = 1.87 c.f.s./Ac

25 year Storm P.I = 2.31 c.f.s./Ac

100 year Storm P.I. = 2.95 c.f.s./Ac

POST-DEVELOPED CONDITIONS: 40% IMPERVIOUS

15 year Storm P.I = 2.64 c.f.s./Ac

25 year Storm P.I. = 3.26 c.f.s./Ac

100 year Storm P.I. = 4.17 c.f.s./Ac

3. Total Site Acreage = 16.629 Acres

EXHIBIT "A"

4. For the purpose of conforming to the City of O'fallon's requirements, the 25 year. P.I factors will be used. Differentia rate of runoff = 0.95 cfs/A± (3.26 c.f.s./A± - 2.31 c.f.s./A±).

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 * Differentia rate of runoff; 16.629 Acres * 0.95 c.f.s./Acres.

EXHIBIT "A"

SHEET 2 OF 2

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)
  - length = 40'
  - slope = $\frac{571-564}{40} = 17.5\%$
  - Velocity ≈ 6.4 ft/s
  - travel time ≈ 6.3 sec.
- overland flow (LOT 9)
  - length = 105'
  - slope = $\frac{564-563.6}{105} = 0.4\%$
  - velocity ≈ 1 ft/s
  - travel time ≈ 105 sec.
- pavement flow (From Lot 9 TO CI 120)
  - length = 185'
  - slope = $\frac{563.6-559.9}{185} \approx 2\%$
  - Velocity = 2.75 ft/s
  - travel time ≈ 67.3 sec.
- pipe flow (From CI 120 TO EP 101)
  - total length = 939.2'
  - Velocity (assumed) = 7 ft/s
  - travel time ≈ 134.2 sec.

EXHIBIT "B

The total travel time =

$6.3 + 105 + 67.3 + 134.2 = 312.8$ sec.

$t_c \approx 5.21$ min.

2. For Inflow Hydrograph use $t_c = 6$ 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, $t_c = 6$ 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.

TIME (minutes)	INFLOW (c.f.s.)	REMARKS
0	0	DESIGN RAIN BEGINS
6	303.31	BEGIN PEAK INFLOW
12	303.31	INFLOW RATE CONSTANT @ PEAK RATE
20	303.31	DESIGN RAIN ENDS
26	0	INFLOW ENDS

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

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

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

EXHIBIT "B"

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

TIME (minutes)	INFLOW (c.f.s.)	REMARKS
0	0	DESIGN RAIN BEGINS
6	374.59	BEGIN PEAK INFLOW
12	374.59	INFLOW RATE CONSTANT @ PEAK RATE
20	374.59	DESIGN RAIN ENDS
26	0	INFLOW ENDS

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

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

Q_{100} = 303.31 c.f.s. * 1.58 = 479.23 c.f.s.

EXHIBIT "B"

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

TIME (minutes)	INFLOW (c.f.s.)	REMARKS
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"

DEPTH-STORAGE VOLUME CALCULATIONS

	ELEV.	AREA	AVE. AREA	INCR. OF DEPTH	INCR OF VOLUME	TOTAL VOLUME
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

EXHIBIT "C"

SHEET 1 OF 1

DEPTH-OUTFLOW CALCULATIONS

1. Maximum Permissable Outflow equals Peak Inflow - Attenuation:

   374.59 c.f.s. − 15.80 c.f.s. = 358.79 c.f.s.

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

2. Performance Calculations:

   Orifice Equation → $Q = Ca\sqrt{2gh}$

   $a = 1 * 12.566 + 2 * 9.621 = 31.81\ ft^2$

   $C = 0.6$

   $g = 32.2$

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

ELEV.	h	Qout
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 Δt = 1 minute = 0.0166 hrs

Then $\frac{2S}{\Delta t}$ + Outflow $= \frac{2S}{0.0166} \times \frac{24 \left(\text{hrs/day}\right)}{1.98 \left(\frac{\text{Ac-FT}}{\text{cfs-day}}\right)} + O$

$$\frac{2S}{\Delta t} + O = 1454.5 * S + O \qquad (\text{c.f.s.})$$

ELEV.	S (AC-FT)	O (c.f.s.)	$\frac{2S}{\Delta t}+O$ (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	96.87	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"

Design Pond Routing 25 Year; 20 minute Storm

FORM 102	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		$21^{\underline{0}}$	
3	2	124.86	187.30	20.43	207.73		$69^{\underline{0}}$	
4	3	187.30	312.16	69.73	381.89		$127^{\underline{5}}$	
5	4	249.73	437.03	126.89	563.92		$172^{\underline{0}}$	
6	5	312.16	561.89	219.92	781.81		207.5	
7	6	374.59	686.75	366.81	1,053.56		$244^{\underline{0}}$	
8	7	374.59	749.18	565.56	1,314.74		$269^{\underline{5}}$	
9	8	374.59	749.18	775.74	1,524.92		$287^{\underline{0}}$	
10	9	374.59	749.18	950.92	1,700.10		$299^{\underline{5}}$	
11	10	374.59	749.18	1,101.10	1,850.28		$309^{\underline{5}}$	
12	11	374.59	749.18	1,231.28	1,980.46		$316^{\underline{0}}$	
13	12	374.59	749.18	1,348.46	2,097.64		$324^{\underline{5}}$	
14	13	374.59	749.18	1,448.64	2,197.82		$328^{\underline{5}}$	
15	14	374.59	749.18	1,540.82	2290.00		$335^{\underline{0}}$	

EXHIBIT "E"
SHEET 2 OF 3

Design Pond Routing 25 Year; 20 minute storm

FORM 102	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	14	374.59	749.18	1,540.82	2290.00		335.0	
2	15	374.59	749.18	1,620.00	2369.18		339.0	
3	16	374.59	749.18	1,691.18	2,440.36		343.0	
4	17	374.59	749.18	1,754.36	2,503.54		346.0	
5	18	374.59	749.18	1,811.54	2,560.72		349.5	
6	19	374.59	749.18	1,861.72	2,610.90		352.5	
7	20	374.59	749.18	1,905.90	2,655.08	547.0	355.0	69,260 CU. FT. 1.59 AC FT
8	21	312.16	686.75	1,945.08	2,631.83		353.0	
9	22	249.73	561.89	1,925.83	2,487.72		345.5	
10	23	187.30	437.03	1,796.72	2,233.75		331.5	
11	24	124.86	312.16	1,570.75	1,882.91		311.0	
12	25	62.43	187.30	1,260.91	1,448.21		281.5	
13	26	0	62.43	885.21	947.64		230.0	
14								
15								

PEAK INFLOW

EXHIBIT "E"
SHEET 3 OF 3

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

$$h = 548.10 - 541.60 = 6.5$$

$$Q_{orifice} = Ca\sqrt{2gh}$$

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

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

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

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

$$Q = CLH^{3/2} \qquad H = \left(\frac{Q}{CL}\right)^{0.67} = \left(\frac{88.74}{3(26.0)}\right)^{0.67} \simeq 1.1'$$

$$548.1 - 547.0 = 1.1' = 1.1' \text{ assumed } \checkmark\checkmark$$ OK.

EXHIBIT "F"
SHEET 1 OF 3

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:

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

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

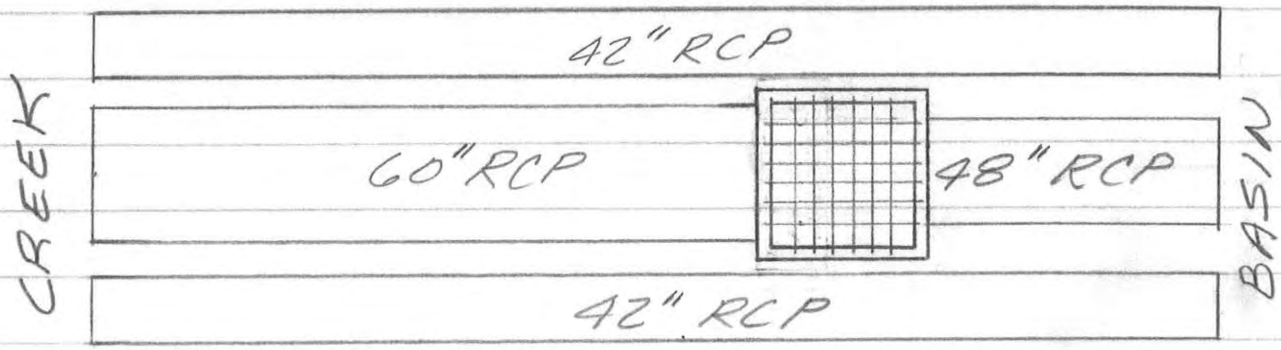
$Q_{48"} = 384.44$ c.f.s. $- 236.19$ c.f.s. $= 148.25$ c.f.s.

plus additional flow of 88.74 c.f.s.

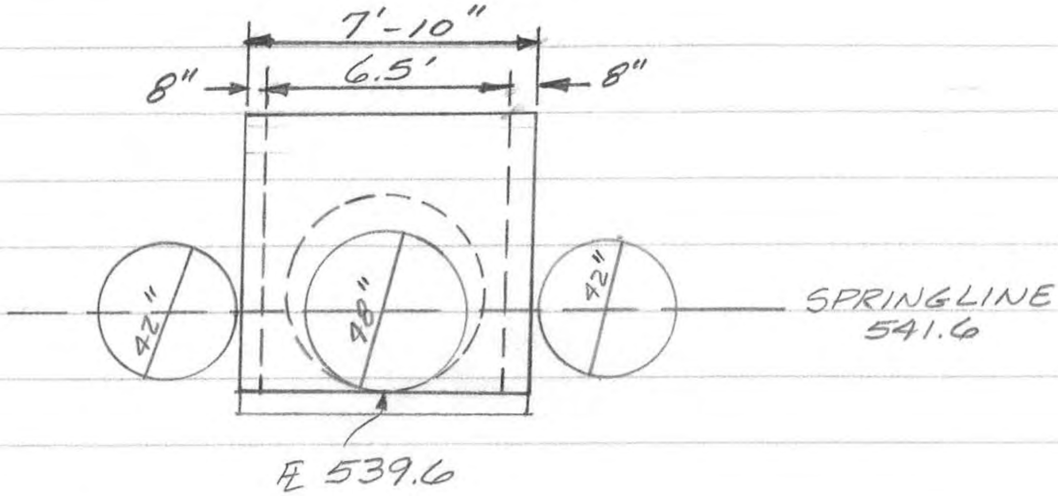
new $Q_{48"} = 236.99$ c.f.s.

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

PIPE CONFIGURATION



PLAN VIEW



7'-10" Square Box

EXHIBIT "F"

SHEET 3 OF 3

GENERAL SUMMARY

For 25 year frequency, 20 minute storm:

Peak Inflow Rate = 374.59 c.f.s.

Attenuation Required = 15.80 c.f.s.

Maximum Permissable Outflow = 358.79 c.f.s.

Peak Outflow = 355.0 c.f.s.

Attenuation Achieved = 19.59 c.f.s.

High Water Elevation = 547.00

Top of Berm = 548.50

Freeboard = 1.5'

EXHIBIT "G"