



Stormwater Management Plan

Project:
**Hoover Park Section 8
Residential Development
City of Grove City, Ohio**

July 2003
Revised July 2004

**Hoover Park Section 8
City of Grove City, Ohio
Storm Water Detention Design
July 2004**

Purpose

To design an on-site storm water detention facility to be constructed in conjunction with the **Hoover Park Section 8** single-family residential development, using the storm water management policy for the City of Grove City. The enclosed study supports the design of the proposed storm water detention facilities.

Hydrologic Analyses

Hydrologic parameters such as Runoff Curve Number (RCN) and Times of Concentration (T_c) were determined using standard Natural Resources Conservation Service (NRCS) methodology. The 1-, 2-, 5-, 10-, 25-, 50-, and 100-year discharge amounts were calculated using the NRCS TR-20 method. This analyses reflects the NRCS Type II distribution, 24-hr storm duration. Rainfall depths were obtained from "Technical Paper No. 40, Rainfall Frequency Atlas of the United States". The detention basin was designed using Pond Pack 9.0 by Haestad Method's.

Existing Conditions

The development site is shown in an approximate manner on the storm sewer tributary area maps included in **Appendix C**. A preliminary design for the detention basin located in section 8 of the development can be found in the Hoover Park Section 2 stormwater detention analysis report, prepared in June of 1995, and a portion of this report has been included in **Appendix A**. The Section 2 report calculated the allowable release rate for the development, see **Table No. 1**. The Lower Basin constructed with Section 2 of the development was designed to meet this criteria provided that another detention basin, Upper Basin, was installed just east of State Route 62 with Section 8 of the development. This report analyzes the Upper Basin and its impact to the Lower Basin.

Table No. 1 – Allowable Release Rate – Hoover Park Section 2

Storm Event (year)	Peak Flow Rate (cfs)
1	33.3
2	38.2
5	58.0
10	81.5
25	99.6
50	122.3
100	136.4

Post Developed Conditions

The Section 2 detention report anticipated approximately 53.7 acres being tributary to the Upper Basin, see **Appendix D**. Upon a field investigation of the site along SR 62, it has been determined that 125.09 acres of offsite area west of SR 62 is tributary to a 18" culvert under SR 62 that will be intercepted and routed to the Upper Basin. State Route 62 creates a natural embankment for the 125.09 acres tributary to the 18" culvert. The invert of the 18" culvert is approximately 858.57 with the top of road being at an elevation of approximately 866.7. At an elevation of 867.0, there are approximately 11.9 acre-feet of

natural storage west of SR 62. During less frequent storms the runoff from the offsite area west of SR 62 overtops the road at an elevation of 866.7. A 50' wide weir at an elevation of 866.7 was added to the model to flood route the runoff to the Upper Basin. The calculated peak flow rates and water surface elevation from the natural storage area west of State Route 62 are shown on Table 2.

Table No. 2 – Peak Flow Rate through 18" Culvert @ State Route 62

Storm Event (year)	Peak Flow Rate (cfs)	Water Surface Elevation (ft)
1	12.11	865.25
2	12.35	865.58
5	12.77	866.25
10	12.83	866.62
25	23.65*	866.88
50	36.14*	867.01
100	44.90*	867.09

*SR 62 overtops

An additional 6.17 acres east of SR 62 also drains to the Upper Basin. Because the drainage areas changed, the original TR-20 model was converted to a Pond Pack model and modified to reflect these changes, see **Figure No. 1**. The Lower Basin and Upper Basin were reanalyzed to verify that the release rates from the development did not exceed the allowable from **Table No. 1**. From the original report, Subarea 004 was increased by 9.0 acres to reflect changes to the grading plan. The Upper Basin was then modified to reflect the proposed conditions as shown on the construction drawings. The proposed Upper Basin has less volume than originally calculated in the Section 2 report, which was another reason to analyze the impact to the Lower Basin. The outlet structure for the Upper Basin consists of a 24" culvert with an invert elevation of 858.00. The secondary outlet structure is a catch basin at an elevation of 862.25'. The tailwater condition for the outlet structures is a 24" storm sewer with a slope of 0.90%. The peak flow rate from the upper basin for a 100-year storm is 32.73 cfs.

Conclusion

Release rates from both the Upper and Lower basins are shown on **Table No. 3**. The resulting peak flow rates from the revised model meet the allowable discharge requirements as set forth in the Section 2 report. The revised peak flow rates and water surface elevations for each basin are shown on **Table No. 4**.

Table No. 3 – Summary of Stormwater Discharges – Hoover Park

Storm Event	Peak Discharge (cfs)		
	Upper Basin	Lower Basin	Allowable
1-yr	11.40	23.18	33.3
2-yr	11.90	32.18	38.2
5-yr	12.84	56.96	58.0
10-yr	13.37	77.48	81.5
25-yr	20.09	99.55	99.6
50-yr	25.04	118.14	122.3
100-yr	32.73	132.53	136.4

Table No. 4 – Detention Basin Characteristics

	Upper Basin	Lower Basin
Normal Pool Elevation	858.00 ft	833.00 ft
Emergency Overflow Elevation	863.00 ft	838.00 ft
Maximum Storage	4.793 ac-ft	17.787 ac-ft
100-Year Water Surface Elevation	862.73 ft	837.63 ft
100-Year Peak Flow Rate	32.73 cfs	132.53 cfs
100-Year Storage Used	4.482 ac-ft	16.273 ac-ft

APPENDIX A

Hoover Park Section 8

Hoover Park Section 2 – Detention Report

SUMMARY HYDROLOGIC AND HYDRAULIC REPORT
FOR
HOOVER PARK - SECTION 2
STORMWATER DETENTION ANALYSIS

February 1995
Revised June 1995

Prepared by:
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Introduction

The Hoover Park Section 2 project is located in the City of Grove City, just west of Haughn Road, and just south of Casa Boulevard. The project parcel is 119.5 acres in size; the proposed development consists of approximately 107 acres of single-family residential homes (including a 3.6-acre park), 9.5 acres of multi-family housing, and a commercial area of 3.0 acres. There are two significant drainage courses on the project parcel. The smaller of the two systems drains approximately 48 acres of the northern portion of the parcel. Its headwater area is within the parcel, and the drainage swale discharges towards the northeastern corner of the parcel. The main system drains approximately 59 acres of the southern and extreme western portions of the parcel, and also collects off-site area of approximately 89 acres from the west and south of the project.

For stormwater management purposes, two detention basins are proposed. One of the basins is located near the western end of the project parcel, and the other is located at the eastern (downstream) end of the parcel. The two basins have been placed in series; the outlet structure at the lower/eastern basin has been designed to meet critical storm requirements for the project. Since the lower/eastern basin also replaces an existing detention basin associated with Hoover Park Section 1, this new basin has been designed to also meet the critical storm requirements of Hoover Park Section 1. As shown in the Appendix of this report, the critical storm for the Hoover Park Section 2 project has been calculated to be a 25-year storm, and the critical storm for the Section 1 project has previously been calculated to be a 5-year storm.

Determination of Allowable Outflows

The detailed calculations for allowable outflows from the lower/eastern detention basin are presented in the Appendix to this report. As indicated above, the allowable outflows from the Hoover Park Section 2 project site have been calculated based on a 25-year critical storm. However, uncontrolled off-site areas are allowed to pass through the site undetained. Therefore, the total allowable outflows from the project are a combination of on-site and off-site allowable flows for the various frequency storm events. As shown in the Appendix, the allowable outflows have been calculated initially for the Hoover Park Section 2 and off-site areas, without considering Hoover Park Section 1 impact; then the allowable release rates have been modified to account for the 5-year critical storm release rates associated with Section 1.

During the course of this analysis, it was discovered that a storm sewer along Mayfair Courts North and South (from the Mayfair Park subdivision south of the project) is tributary to the project. This storm sewer collects drainage from approximately 9.2 acres within the Mayfair Park subdivision which lies outside of the natural drainage boundary of Section 2 and directs it to the channel along the southern edge of the project parcel. It appears that this sewer line will contribute flow to Section 2 up to the capacity of the pipe; once this capacity is exceeded, flows in excess of that capacity are assumed to be major-flood routed along the south side of Mayfair Drive, outside of the Section 2 drainage system. For purposes of this analysis, it is assumed that flow values from this system at or below the peak of the 5-year storm (16.03 cfs) will discharge to the Section 2 drainage system.

The following flows have been calculated to be the maximum allowable discharges from the Hoover Park Section 2 project:

1-year	=	33.3 cfs
2-year	=	38.2 cfs
5-year	=	58.0 cfs
10-year	=	81.5 cfs
25-year	=	99.6 cfs
50-year	=	122.3 cfs
100-year	=	136.4 cfs

Detention Basin Analysis

The Soil Conservation Service's TR-20 computer program has been utilized to compute runoff hydrographs and perform reservoir routings for the detention basin analysis. Hydrologic input parameters have been calculated for both pre-development and post-development conditions utilizing the SCS's TR-55 manual. Detailed calculations for these parameters can be found in the Appendix of this report. A schematic drawing showing the individual drainage areas associated with the pre-development and post-development conditions is also provided with the calculations.

To eliminate confusion in the TR-20 computer modelling, the area associated with Mayfair Park is always identified as Subarea 005 in the TR-20 models, and the area associated with Hoover Park Section 1 is always identified as Subarea 006. Separate computer models had to be established for the 10-, 25-, 50-, and 100-year storms because the flood-routed portion of the flood hydrographs had to be separated from the storm sewer flow portion in the Mayfair Park area, thus requiring the use of READHD cards to input the appropriate hydrograph for each storm.

Based on trial-and-error methodology, the outlet structures for the upper/western detention basin consist of a drop inlet and 15-inch RCP for the principal spillway, and a 20-ft. long, grassed, broad-crested weir as a secondary spillway. At the lower/eastern detention basin, the outlet consists of a concrete wall/weir, 4 feet long, with a breadth of 1 foot, which will discharge into the existing channel east of the basin. Calculations for the outlet ratings are presented in the Appendix of this report.

The results of the analyses are presented in the Table below, and in the Appendix. The results for the 1, 2, and 5-year storms can be found in the output summary for the computer input file HP2P123, while the 10, 25, 50, and 100-year storm results can be found in the output summaries for computer input files HP2P4, HP2P5, HP2P6, and HP2P7, respectively.

RESULTS OF TR-20 ANALYSES

Storm Recurrence Interval	Upper/West Basin		Lower/East Basin		
	Max. Outflow from Basin (cfs)	Max. Pool Elev. (Ft., MSL)	Max. Outflow from Basin (cfs)	Max. Pool Elev. (Ft., MSL)	Max. Allow. Outflow (cfs)
1-year	13.2	858.59	29.3	834.70	33.3
2-year	13.5	858.80	35.1	834.92	38.2
5-year	15.1	859.74	61.4	835.77	58.0
10-year	16.1	860.40	80.5	836.32	81.5
25-year	17.1	861.07	98.8	836.92	99.6
50-year	22.2	861.59	117.6	837.43	122.3
100-year	28.7	861.74	131.1	837.69	136.4

APPENDIX

CALCULATIONS OF HYDROLOGIC INPUT PARAMETERS



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VANS, MECHWART, HAMBLETON & TILTON, INC.

SUBJECT HOOVER PARK - Section 2

SUMMARY OF HYDROLOGY

DATE 2/15/95 Job File No. P-1742

COMPUTED BY RCD CHECKED BY _____

HYDROLOGY → PRE-DEVELOPMENT CONDITIONS

→ DRAINAGE AREA: Assume pre-development drainage area to be that area that drains to the outlet point of the parcel, namely, where the main tributary stream intersects with the eastern project property line

D.A. = 42.2 ac. off-site area upstream of Route 62

26.8 ac. off-site area below Rt. 62, not incl. Mayfair or Hoover Park Sect. 1

59.0 ac. on-site area naturally tributary to the outlet point

TOTAL = 128.0 ac. = 0.2000 sq. mi.

In addition to these areas, Mayfair Park and Hoover Park Sect. 1 also contribute drainage to the project area. Mayfair Park contains a storm sewer which discharges into the site area. The drainage area associated with the storm sewer is 9.2 acres; however, the discharge is limited to what the sewer can carry, up to a 5-year peak flow from the 9.2 acres. When flows exceed the 5-yr. peak, the remaining flow is major flood routed to the east along Mayfair Drive. Hoover Park Sect. 1 also contributes flow; a temporary detention basin is located where the proposed larger lower pond for Section 2 is to be located. Use the pre-development drainage area of 10.4 acres to assist in calculating allowable discharges for Section 2 (Section 1 contributes to the total allowable discharge from Section 2 lower pond).

→ TIME OF CONCENTRATION Main stream

Sheet flow: $l = 130' \pm$ to beginning of roadside ditch along Rensch Rd.

$S = (885.9 - 882) / 130 = 0.03$; $n = 0.15$ (short grass assumed).

$$T_t = \frac{0.007 (nL)^{0.8}}{(P_2)^{0.5} (S)^{0.4}} = 0.19 \text{ hr.}$$

Shallow conc. flow: $l = 6,390 \text{ ft.}$; $S = (882 - 851.5) / 6390 = .0048$

$$\bar{V} = 16.1345 (S)^{0.5} = 1.11 \text{ ft/sec.}$$

$$T_t = 6390 / 1.11 = 5757 \text{ sec.} = 1.60 \text{ hrs.}$$

CONT'D on NEXT PAGE



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SUMMARY OF HYDROLOGY

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→ TIME OF CONCENTRATION (Cont'd)

PRE-DEVELOPMENT CONDITIONS

Main channel flow: $L = 2930 \text{ ft}$; $S = (851.5 - 832.0) / 2930 = .0067$

$\bar{V} = 2.1 S^{0.5}$, where "S" is in %

$\bar{V} = 1.72 \text{ ft/sec}$

$T_t = 2930 / 1.72 = 1703 \text{ sec} = 0.47 \text{ hr}$

TOTAL $T_c = 0.19 + 1.60 + 0.47 = 2.26 \text{ hrs}$

As indicated in discussion in "drainage areas", Mayfair Park and Hoover Park Section 1 also contribute to pre-development flows.

→ Mayfair Park: Flow affecting project site comes from storm sewer extending along Mayfair Courts North & South. Assume T_c is based on 10 minutes travel time to first inlet plus travel time in pipe at say 3 ft/sec. Assumed length of storm sewer $\approx 1100 \text{ ft}$.
Travel time in pipe $= 1100 / 3 = 367 \text{ sec} = 0.10 \text{ hr}$

Total $T_c = 0.17 + 0.10 = 0.27 \text{ hr}$

→ Hoover Park Section 1: From previous calculations associated with Hoover Park Sect. 1, pre-development $T_c = 0.53 \text{ hr}$

→ CURVE NUMBERS: Land use is mainly agricultural with scattered areas of residential development, mainly with large lot sizes. For main area of 128 acres, use an assumed $CN = 80$, as was used in previous analysis for Hoover Crossing Section 4 (downstream of this area, but containing this area in its watershed)

→ Mayfair Park area: Residential area with large lots & assume $CN = 81$

→ Hoover Park Section 1: From previous calculations, $CN = 79$



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SUMMARY OF HYDROLOGY

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①

HYDROLOGY

POST-DEVELOPMENT CONDITIONS

→ DRAINAGE AREA: For post-development condition, assume analysis will be split into 2 pieces drainage area directed and routed through a proposed upper pond near the west end of the parcel, and drainage area routed through a proposed pond near the eastern end of the parcel. (It is assumed that a temporary pond constructed for Hoover Park Section 1 detention will be removed, and become a part of the new proposed pond for Section 2).

- For the upper pond, assume that drainage area is split into 2 subareas; Area 001 for the off-site drainage area, and Area 002 for the on-site drainage area.

- Area 001 (off-site area) = 42.2 acres upstream of Rt 62 and 2.6 acres downstream of Rt 62. Total D.A. = $42.2 + 2.6 = 44.8$ acres = 0.0700 sq. mi. (see p. (5) for adjustment)

- Area 002 (on-site area) = 12.5 ac. = 0.0195 sq. mi. (see p. (5) for adjustment)

- For the lower pond, outflow from the upper pond will be combined with drainage area being directed to the lower pond. Again, assume that drainage area will be split into two subareas; Area 003 for the off-site area, and Area 004 for the on-site area. In addition, we need to consider the drainage area from Hoover Park Section 1, which will now be directed into this pond. Use subarea 006 for Hoover Park Section 1 drainage.

- Area 003 (off-site area, not including Mayfair Park or Hoover Park Sect. 1) = 24.2 ac. = 0.0378 sq. mi. (see p. (5) for adjustment)

- Area 004 (on-site area, assuming all area within property boundary drains to lower pond) = 107.0 ac. = 0.1672 sq. mi. (see p. (5) for adjustment)

- Area 006 (Hoover Park Sect. 1 drainage area for proposed conditions) = 17.3 ac. = 0.0271 sq. mi. (see p. (5) for adjustment)

(2)

HYDROLOGY - POST-DEVELOPMENT CONDITIONS (cont'd)

→ TIME OF CONCENTRATION:

- Upper pond: off-site area (001)

Assume hydraulic length to run from upper watershed boundary to western edge of project parcel.

Sheet flow: $l = 130' \pm$, $s = 0.03$, $n = 0.15$ as in pre-developed conditions; use $T_L = 0.19$ from previous calculations.

Shallow conc. flow: $l = 4210$ ft., measured from beginning of roadside ditch along Ransch Rd. to western property line of project parcel.

$$s = (882 - 862.5) / 4210 = 0.0046$$

$$\bar{V} = 16.1345 (s)^{0.5} = 1.10 \text{ ft/sec.}$$

$$T_L = 4210 / 1.1 = 3827 \text{ sec.} = 1.06 \text{ hrs.}$$

$$\text{Total } T_c = 0.19 + 1.06 = \underline{1.25 \text{ hrs.}}$$

- Upper pond: On-site area (002)

Assume travel time of 10 minutes to first storm sewer inlet plus travel time in pipe, at say 3 ft/sec. Assumed length of sewer line ≈ 800 ft. Travel time in pipe = $800 / 3 = 267 \text{ sec.} = 0.07 \text{ hr.}$

$$\text{Total } T_c = 0.17 + 0.07 = \underline{0.24 \text{ hr.}}$$

- Lower pond: Off-site area (003) - not including Mayfair Park or Hoover Sect

Assume hydraulic length on south side of project, from edge of watershed to Mayfair Drive to NE corner of Lot 5 at end of Mayfair Court.

Sheet flow: $l = 90' \pm$, $s = (862.8 - 862) / 90 = 0.0089$, $n \approx 0.17$

$$T_L = \frac{0.007 (nL)^{0.6}}{(P_2)^{0.5} (s)^{0.4}} = \frac{0.007 (0.17 \times 90)^{0.6}}{(2.55)^{0.5} (0.0089)^{0.4}} = 0.26 \text{ hr.}$$

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HYDROLOGY - POST-DEVELOPMENT CONDITIONS (cont'd)

- Lower pond: Off-site area (003) (cont'd)

Shallow conc. flow: $L \approx 1840$ ft.

$$S = (862 - 848) / 1840 = .0076$$

$$\bar{V} = 1.4 \text{ ft/sec.}$$

$$T_t = 1840 / 1.4 = 1314 \text{ sec.} = 0.37 \text{ hr.}$$

Main channel flow: $L \approx 1030$ ft.

$$S = (848 - 842) / 1030 = .0058$$

$$\bar{V} = 1.5 \text{ ft/sec.}$$

$$T_t = 1030 / 1.5 = 687 \text{ sec.} = 0.19 \text{ hr.}$$

$$\text{Total } T_c = 0.26 + 0.37 + 0.19 = 0.82 \text{ hr.}$$

- Lower pond: On-site area (004)

Assume 10 minutes travel time to first inlet, plus travel time in sewer, at say 3 ft/sec. Assumed length of storm sewer = 4000 ft.
Travel time in sewer = $4000 / 3 = 1333 \text{ sec.} = 0.37 \text{ hr.}$

$$\text{Total } T_c = 0.17 + 0.37 = 0.54 \text{ hr.}$$

- Hoover Park Section 1 area (006)

From previous calculations, $T_c = 0.24 \text{ hr.}$

4

HYDROLOGY - POST-DEVELOPMENT CONDITIONS (cont'd)

→ CURVE NUMBERS

- Upper pond: Off-site area (001)

Area is mainly agricultural, with some residential areas of larger lot sizes. Assume CN = 80, as used in Hoover Crossing Section 4 study downstream.

- Upper pond: On-site area (002)

According to plans, approximately 9.5 acres are proposed to be multi-family housing, and 3.0 acres are proposed to be commercial (including right-of-way). The weighted curve number is calculated as follows:

$$\begin{aligned} 9.5 / 12.5 \times \text{CN } 90 &= 68.4 \text{ (multi-family residential)} \\ 3.0 / 12.5 \times \text{CN } 94 &= 22.6 \text{ (commercial)} \\ \text{Total} &= 91.0 \rightarrow \text{USE } \text{CN } 91. \end{aligned}$$

- Lower pond: Off-site area (003)

Area is about 75% agricultural, with about 25% residential area of about 1/2 acre lot size. Use average CN = 80, as was used for upper pond off-site areas.

- Lower pond: On-site area (004)

On-site area is mainly single-family residential area with a proposed park of 3.6 acres. Weighted CN is calculated as follows:

$$\begin{aligned} 103.4 / 107 \times \text{CN } 82 &= 79.2 \text{ (residential)} \\ 3.6 / 107 \times \text{CN } 74 &= 2.5 \text{ (park)} \\ \text{Total} &= 81.7 \rightarrow \text{USE } \text{CN } 82. \end{aligned}$$

- Hoover Park Sect. 1 area (006): From previous calculations, use CN = 83



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SUBJECT HOOVER PARK - Sect. 2

SUMMARY OF HYDROLOGY

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(5)

HYDROLOGY - POST-DEVELOPMENT CONDITIONS (cont'd)

As a result of finalization of storm sewer layout and design, and also with the availability of more detailed topo information, the final drainage areas tributary to each pond is slightly different from those calculated in the initial analysis/calculations of 2/14/95. The comparison of total drainage areas to each pond are given below:

Upper pond: Initial calculation \rightarrow D.A. = 57.3 ac.

Final calculation \rightarrow D.A. = 53.7 ac.

Lower pond: Initial calculation \rightarrow D.A. = 131.2 ac.

Final calculation \rightarrow D.A. = 136.8 ac.

Hoover Park Sect 1: Initial calculation \rightarrow D.A. = 17.3 ac. = .0271 mi²

Final calculation \rightarrow D.A. = 12.0 ac. = .0188 mi²

In order to adjust TR-20 model for the new D.A., an assumption must be made relative to how to split the change between on-site and off-site areas. For purposes of this analysis, assume both on-site and off-site areas are adjusted by the same % increase or decrease needed. Therefore, adjust drainage areas as follows:

For upper pond: Reduce on-site and off-site DA's by ratio $\frac{53.7}{57.3}$

New on-site area = .937 x .0195 = 0.0183 sq. mi. (.002)

New off-site area = .937 x .0700 = 0.0656 sq. mi. (.001)

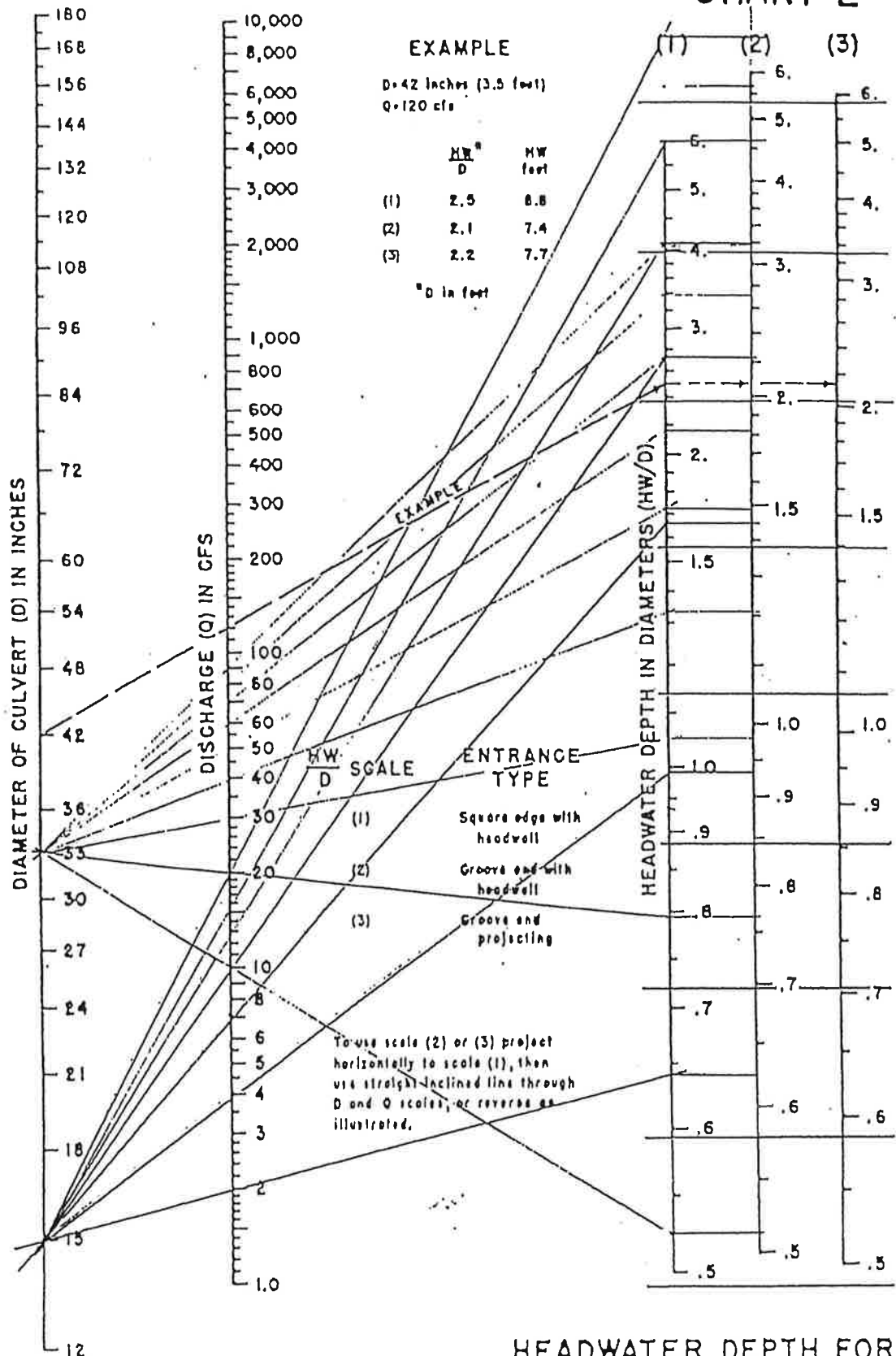
For lower pond: Increase on-site and off-site DA's by ratio $\frac{136.8}{131.2}$

New on-site area = 1.043 x .1672 = 0.1743 sq. mi.

New off-site area = 1.043 x .0378 = 0.0394 sq. mi.

OUTLET RATINGS AND STORAGE CURVES FOR UPPER/WESTERN POND

CHART 2



HEADWATER DEPTH FOR CONCRETE PIPE CULVERTS WITH INLET CONTROL

HEADWATER SCALES 2B3
REVISED MAY 1964

BUREAU OF PUBLIC WORKS JUL 1963



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SUBJECT HOOVER PARK - Sect. 2

OUTLET RATING - UPPER POND

DATE 2/16/95 Job File No. P-1742

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OUTLET RATING - UPPER POND

Assume a standard 2-4 catch basin to serve as a drop inlet principal spillway. The catch basin will have windows on all 4 sides, each 3.5 ft. long; and have an assumed breadth of 8". Crest of windows at normal pool elev. 858.0 ft. Assume window height = 6"; after a depth of 6", assume windows operate as a box culvert with inlet control. Flow will discharge into a 15" RCP with invert elevation at 854.5 ft. Assume this pipe will flow with inlet control. The pipe will begin to control flow from the drop inlet when the pool rises to near elevation 858.5 ft. (see plot of outlet rating). Therefore, box culvert rating and grate ratings can be ignored. Simply use rating for window operating as a weir until the culvert begins to control flow.

WINDOW OPERATING AS A WEIR

H	HW Elev	C	$Q = CLH^{3/2}$ (1 window)	Q for 4 windows
0	858.0	0	0	0
0.2	858.2	2.75	0.86	3.4 cfs
0.4	858.4	2.80	2.48	9.9 cfs
0.5	858.5	2.84	3.51	14.1 cfs

SECONDARY SPILLWAY RATING: Assume a grassed spillway with a length of 20 feet, a breadth of 10 feet, and a crest at elevation 861.4 feet. Water will discharge into street to east (major flood routing path).

ELEV	H	$H^{3/2}$	C	L	$Q = CLH^{3/2}$	Q_{pipe}	Q_{TOTAL}
861.4	0	0	-	20	0	17.7 cfs	17.7 cfs
861.6	0.2	0.089	2.49	↓	4.4 cfs	18.0	22.4
861.8	0.4	0.253	2.56		13.0	18.3	31.3
862.0	0.6	0.465	2.70		25.1	18.6	43.7

46 1320

855 IN X 10 TO 12 INCHES
RATED FOR 12 INCHES

863

862

861

860

859

858

857

856

855

854

ELEV.,
FT.

WINDOW RATING

RATING
FOR
15"
RCP

HOOVER PARK - SECTION 2

OUTLET RATING FOR
UPPER POND

RCP 2/14/95

REV. 6/8/95

Q IN CFS

0 2 4 6 8 10 12 14 16 18 20



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SUBJECT HOOVER PARK - Sect. 2

DATE 6/5/95 Job File No. _____

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STORAGE CURVE - REVISED - upper pond

ELEV.	POOL AREA (in ²)	POOL AREA (ac)	AVG. POOL AREA (ac)	Δ DEPTH (ft)	Δ VOLUME (ac-ft)	TOTAL VOLUME (ac-ft)
858	4.83	1.11				0
			1.18	1	1.18	
859	5.46	1.25				1.18
			1.35	1	1.35	
860	6.29	1.44				2.53
			1.52	1	1.52	
861	6.96	1.60				4.05

46 1320

10 X 10 TO 1. INCH - 5 IN. SECTION
MURKIN & DISORDER, AND MOUNTAIN

861

860

859

ELEV., FT., MSL

3.580

STORAGE IN AC-FT.

HOOVER PARK - SECT. 2

STORAGE CURVE @
UPPER POND

REV. RCD 6/6/95

OUTLET RATINGS AND STORAGE CURVES FOR LOWER/EASTERN POND



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SUBJECT HOOVER PARK - Section 2

OUTLET RATING - LOWER POND

DATE 2/13/95 Job File No. _____

COMPUTED BY RCD CHECKED BY _____

OUTLET RATING - LOWER POND

Assume principal spillway is a concrete weir with length of 4 feet,
a breadth of 1 foot, and a crest at elevation 833.0 ft.

ELEV.	H	$H^{3/2}$	C	L	$Q = CLH^{3/2}$
833.0	0	0	0	4	0
.2	0.2	0.089	2.69		1.0
.4	0.4	0.253	2.72		2.8
.6	0.6	0.465	2.75		5.1
.8	0.8	0.716	2.85		8.2
834.0	1.0	1.000	2.98		11.9
.2	1.2	1.315	3.08		16.2
.4	1.4	1.657	3.20		21.2
.6	1.6	2.024	3.28		26.6
.8	1.8	2.415	3.31		32.0
835.0	2.0	2.828	3.30		37.3
.5	2.5	3.953	3.31		52.3
836.0	3.0	5.196	3.32		69.0
.5	3.5	6.548	3.32		87.0
837.0	4.0	8.000	3.32		106.2
.5	4.5	9.546	3.32		126.8
838.0	5.0	11.180	3.32		148.5



CONSULTING ENGINEERS & SURVEYORS

EVANS, MECHWART, HAMBLETON & TILTON, INC.

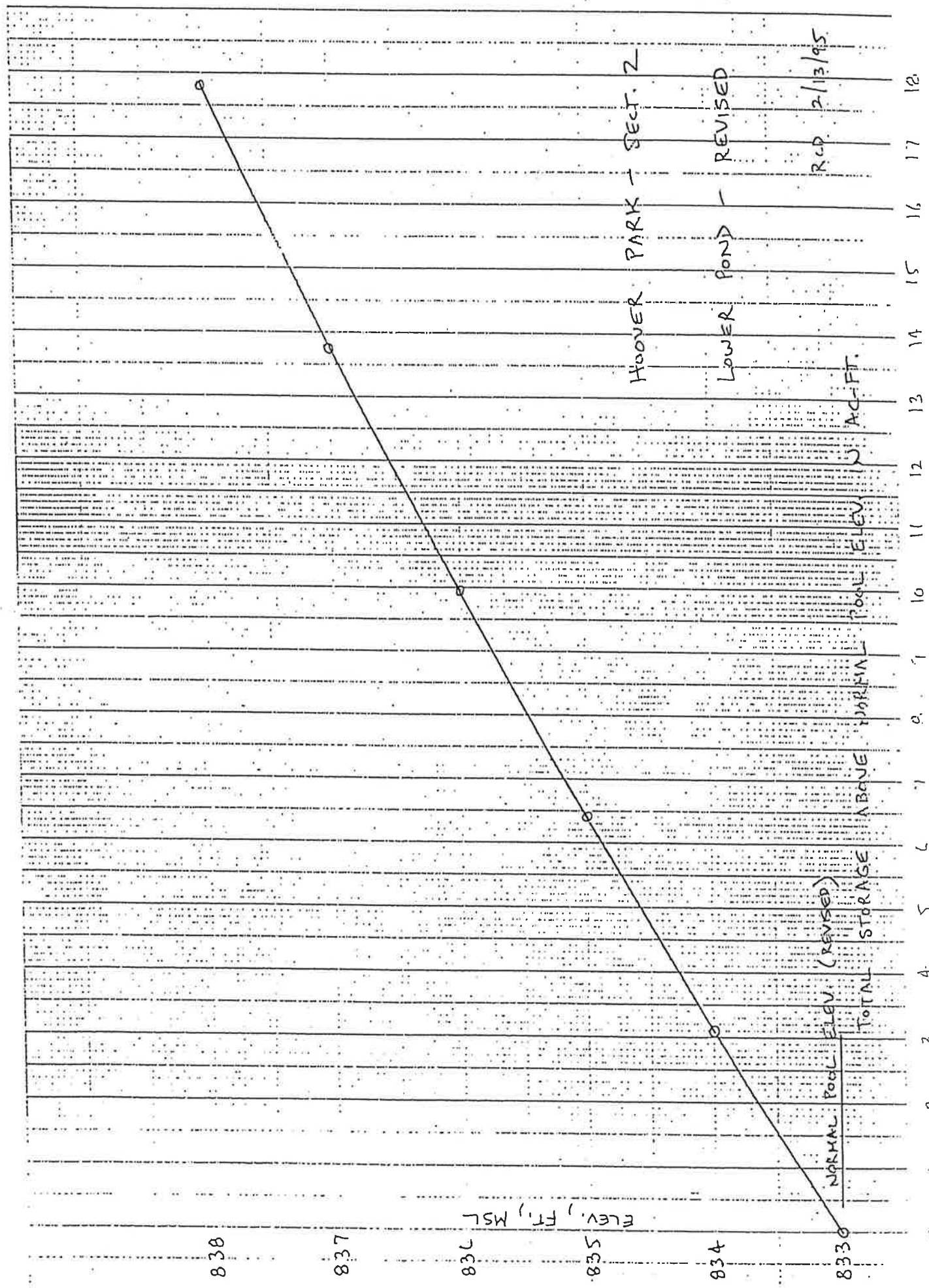
SUBJECT HOOVER PARK - Sect. 2

REVISED AREA-CAPACITY DATA - lower portion

DATE 2/13/95 Job File No. _____

COMPUTED BY RCD CHECKED BY _____

ELEV.	POOL AREA (in ²)	POOL AREA (ac)	AVG, POOL AREA (ac)	Δ DEPTH (ft)	Δ VOLUME (ac-ft)	TOTAL VOLUME (ac-ft)
833	12.98	2.98				0
			3.09	1	3.09	
834	13.92	3.20				3.09
			3.32	1	3.32	
835	14.96	3.43				6.41
			3.55	1	3.55	
836	15.97	3.67				9.96
			3.80	1	3.80	
837	17.06	3.92				13.76
			4.04	1	4.04	
838	18.14	4.16				17.80



CRITICAL STORM AND ALLOWABLE DISCHARGE CALCULATIONS



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SUBJECT HOOVER PARK - Sect. 2

DATE 2/16/95 Job File No. P-1742

COMPUTED BY RCP CHECKED BY _____

CRITICAL STORM CALCULATIONS

Critical storm is based on the percentage increase in runoff volume for the 1-year storm. Since all of the northern area of the parcel is proposed to be drained to the southern drainage system, across the natural drainage divide for the southern system, the drainage areas for pre-development and post-development conditions will be significantly different. Based on calculation of hydrologic parameters, the increase in runoff volume is calculated as follows:

PRE-DEVELOPMENT CONDITIONS:

59 ac. @ CN 80 \Rightarrow runoff for 2.35" precip. = 0.79 in.

Runoff volume = 59 ac. \times 0.79 in. = 46.61 ac-in.

POST-DEVELOPMENT CONDITIONS:

12.5 ac. @ CN 91 \Rightarrow runoff for 2.35" precip. = 1.48 in.

107.0 ac. @ CN 82 \Rightarrow runoff for 2.35" precip. = 0.89 in.

Runoff volume = 12.5 ac. \times 1.48 in. = 18.50 ac-in.

107.0 ac. \times 0.89 in. = 95.23 ac-in.

TOTAL = 113.73 ac-in.

INCREASE IN RUNOFF VOLUME = $\frac{113.73 - 46.61}{46.61} = \frac{67.12}{46.61} = 144\%$

\therefore Critical storm is a 25-year storm.



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EVANS, MECHWART, HAMBLETON & TILTON, INC.

SUBJECT HOOVER PARK - Sect. 2

ALLOWABLE OUTFLOWS

DATE 2/16/95 Job File No. P-1742

COMPUTED BY RCD CHECKED BY _____

DETERMINATION OF ALLOWABLE OUTFLOWS:

	Q_p 1 major subarea plus Mayfair Park storm sewer to E. property line *	Computer file pertinent to results	Q_p Combined effect of off-site areas plus Mayfair Park to ponds	Computer file pertinent to results
1-yr.	27.62 cfs	HP2E125	25.15 cfs	OFFSITE
2-yr.	32.94 cfs	HP2E125	30.05 cfs	OFFSITE
5-yr.	54.58 cfs	HP2E125	49.83 cfs	OFFSITE
10-yr.	69.48 cfs	HP2E10	64.31 cfs	OFFS10
25-yr.	86.13 cfs	HP2E25	78.98 cfs	OFFS25
50-yr.	100.92 cfs	HP2E50	100.92 cfs	HP2E50
100-yr.	109.25 cfs	HP2E100	109.25 cfs	HP2E100

* Not including Hoover Park Sect 1 impact; total runoff hydrographs for Mayfair Park storm sewer are found associated with computer file HP2E.MF.

Based on the figures in the above table, the peak flow (Q_p) associated with the on-site area for a 1-year storm $\approx 27.62 - 25.15$ cfs, or 2.47 cfs. Use 2.47 cfs as the pre-development runoff to be added to the off site runoffs for all storms up through the 25-year critical storm.

Ignoring the impact of Hoover Park Section 1 for now (which has a different critical storm), the allowable outflows for the Section 2 project without Hoover Park Sect. 1 impact would be as follows:

$$\begin{aligned} 1\text{-year} &\rightarrow 25.15 + 2.47 = 27.62 \text{ cfs} \\ 2\text{-year} &\rightarrow 30.05 + 2.47 = 32.52 \text{ cfs} \\ 5\text{-year} &\rightarrow 49.83 + 2.47 = 52.30 \text{ cfs} \\ 10\text{-year} &\rightarrow 64.31 + 2.47 = 66.78 \text{ cfs} \\ 25\text{-year} &\rightarrow 78.98 + 2.47 = 81.45 \text{ cfs} \\ 50\text{-year} &\rightarrow \quad \quad \quad = 100.92 \text{ cfs} \\ 100\text{-year} &\rightarrow \quad \quad \quad = 109.25 \text{ cfs} \end{aligned}$$



CONSULTING ENGINEERS & SURVEYORS

EVANS, MECHWART, HAMBLETON & TILTON, INC.

SUBJECT HOOVER PARK - Sect. 2

ALLOWABLE OUTFLOWS

DATE 2/16/95 Job File No. 9-1742

COMPUTED BY RCD CHECKED BY _____

Now add the allowable outflows associated with the Hoover Park Section 1 analysis. Hoover Park Section 1 has a 5-year critical storm. The pre-development peak flows from that analysis are as follows: (See output for computer file HOPK.E)

1-yr.	→	5.70 cfs
2-yr.	→	6.84 cfs
5-yr.	→	11.46 cfs
10-yr.	→	14.76 cfs
25-yr.	→	18.18 cfs
50-yr.	→	21.34 cfs
100-yr.	→	23.11 cfs

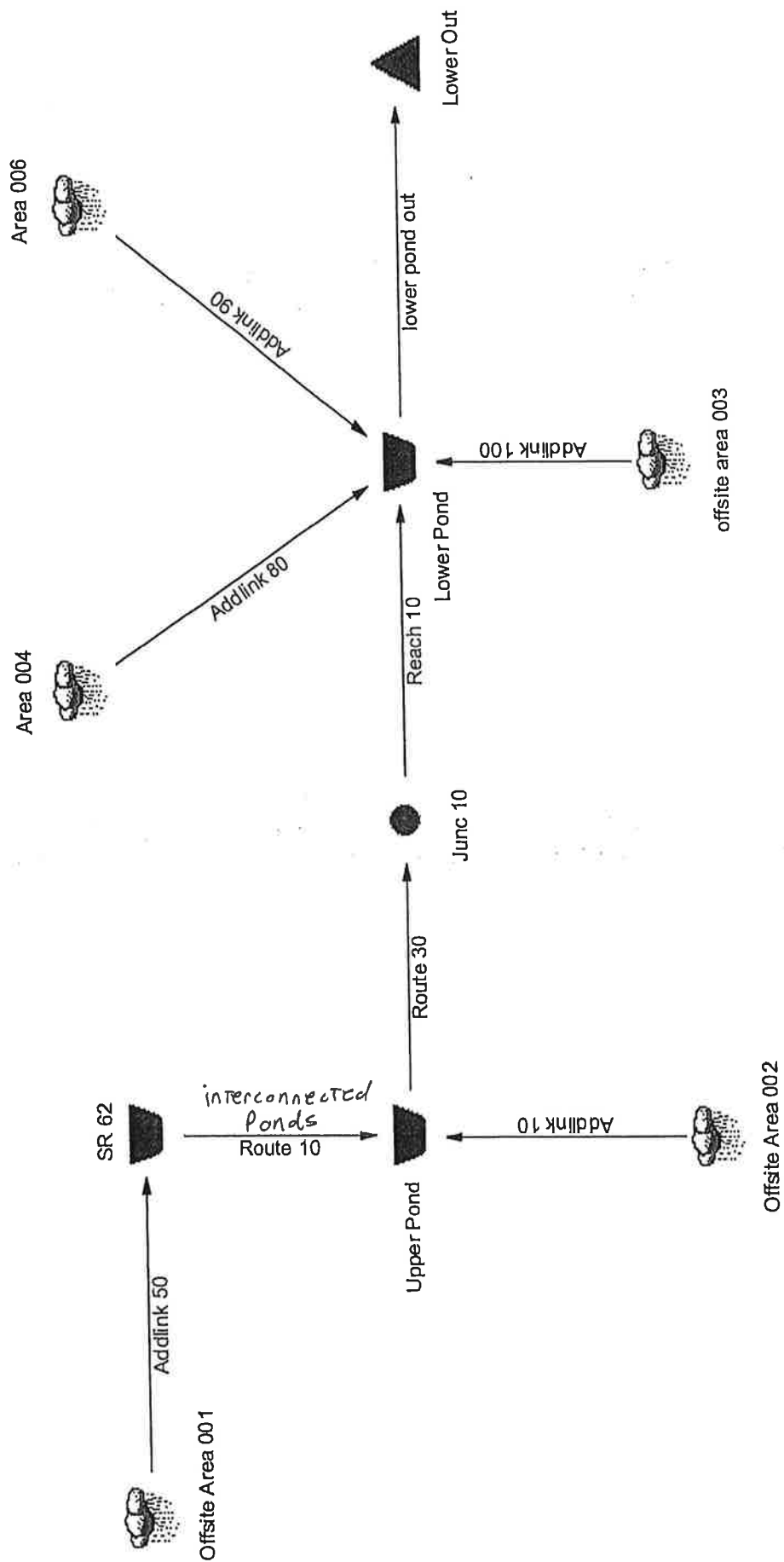
∴ The total allowable outflows from lower pond detention structure should be as follows:

1-year	→	27.62 + 5.7	=	33.32 cfs	,	say	33.3 cfs
2-year	→	32.52 + 5.7	=	38.22 cfs	,	say	38.2 cfs
5-year	→	52.30 + 5.7	=	58.00 cfs	,	say	58.0 cfs
10-year	→	66.78 + 14.76	=	81.54 cfs	,	say	81.5 cfs
25-year	→	81.45 + 18.18	=	99.63 cfs	,	say	99.6 cfs
50-year	→	100.92 + 21.34	=	122.26 cfs	,	say	122.3 cfs
100-year	→	109.25 + 23.11	=	136.36 cfs	,	say	136.4 cfs

APPENDIX B

Hoover Park Section 8

Pond Pack Output



Job File: Q:\PROJECT\stormwater\stormwater\20011352\pondpack\20011352.PPW
Rain Dir: Q:\PROJECT\stormwater\stormwater\20011352\pondpack\

=====

JOB TITLE

=====

Project Date: 7/21/2003
Project Engineer: EMHT INC
Project Title: Hoover Park Section 7
Project Comments:

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MASTER DESIGN STORM SUMMARY

Network Storm Collection: Franklin County

Return Event	Total Depth in	Rainfall Type	RNF ID	
1 yr	2.3000	Synthetic Curve	TypeII	24hr
2 yr	2.6000	Synthetic Curve	TypeII	24hr
5 yr	3.3000	Synthetic Curve	TypeII	24hr
10 yr	3.8000	Synthetic Curve	TypeII	24hr
25 yr	4.3000	Synthetic Curve	TypeII	24hr
50 yr	4.7000	Synthetic Curve	TypeII	24hr
100 yr	5.0000	Synthetic Curve	TypeII	24hr

ICPM CALCULATION TOLERANCES

Target Convergence= .000 cfs +/-
Max. Iterations = 35 loops
ICPM Time Step = .0500 hrs
Output Time Step = .0500 hrs
ICPM Ending Time = 35.0000 hrs

MASTER NETWORK SUMMARY
SCS Unit Hydrograph Method

(*Node=Outfall; +Node=Diversion;)
(Trun= HYG Truncation: Blank=None; L=Left; R=Rt; LR=Left&Rt)

Node ID	Return Type Event	HYG Vol ac-ft	Trun	Qpeak hrs	Qpeak cfs	Max WSEL ft	Max Pond Storage ac-ft
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Name.... Watershed

File.... Q:\PROJECT\stormwater\stormwater\20011352\pondpack\20011352.PPW

ICPM CALCULATION TOLERANCES

Target Convergence= .000 cfs +/-
 Max. Iterations = 35 loops
 ICPM Time Step = .0500 hrs
 Output Time Step = .0500 hrs
 ICPM Ending Time = 35.0000 hrs

MASTER NETWORK SUMMARY
 SCS Unit Hydrograph Method

(*Node=Outfall; +Node=Diversion;)
 (Trun= HYG Truncation: Blank=None; L=Left; R=Rt; LR=Left&Rt)

Node ID	Type	Return Event	HYG Vol ac-ft	Trun	Qpeak hrs	Qpeak cfs	Max WSEL ft	Max Pond Storage ac-ft
AREA 004	AREA	1	8.580		12.2500	79.10		
AREA 004	AREA	2	10.773		12.2500	100.73		
AREA 004	AREA	5	16.269		12.2000	154.95		
AREA 004	AREA	10	20.432		12.2000	195.85		
AREA 004	AREA	25	24.737		12.2000	237.78		
AREA 004	AREA	50	28.261		12.2000	271.84		
AREA 004	AREA	100	30.943		12.2000	297.60		
AREA 006	AREA	1	.907		12.0500	13.22		
AREA 006	AREA	2	1.132		12.0500	16.60		
AREA 006	AREA	5	1.692		12.0500	24.89		
AREA 006	AREA	10	2.114		12.0500	31.03		
AREA 006	AREA	25	2.549		12.0500	37.27		
AREA 006	AREA	50	2.905		12.0500	42.31		
AREA 006	AREA	100	3.175		12.0500	46.12		
JUNC 10	JCT	1	7.267		16.3000	11.40		
JUNC 10	JCT	2	9.387		16.6500	11.90		
JUNC 10	JCT	5	14.786		16.5500	12.84		
JUNC 10	JCT	10	18.878		15.9000	13.37		
JUNC 10	JCT	25	23.141		15.3000	20.09		
JUNC 10	JCT	50	26.695		15.3000	25.04		
JUNC 10	JCT	100	29.416		15.0500	32.73		

Name.... Watershed

File.... Q:\PROJECT\stormwater\stormwater\20011352\pondpack\20011352.PPW

ICPM CALCULATION TOLERANCES

Target Convergence= .000 cfs +/-
 Max. Iterations = 35 loops
 ICPM Time Step = .0500 hrs
 Output Time Step = .0500 hrs
 ICPM Ending Time = 35.0000 hrs

MASTER NETWORK SUMMARY
 SCS Unit Hydrograph Method

(*Node=Outfall; +Node=Diversion;)
 (Trun= HYG Truncation: Blank=None; L=Left; R=Rt; LR=Left&Rt)

Node ID	Type	Return Event	HYG Vol ac-ft	Trun	Qpeak hrs	Qpeak cfs	Max WSEL ft	Max Pond Storage ac-ft
*LOWER OUT	JCT	1	18.299		13.2000	23.18		
*LOWER OUT	JCT	2	23.256		13.0500	32.18		
*LOWER OUT	JCT	5	35.778		12.9000	56.96		
*LOWER OUT	JCT	10	45.271		12.8500	77.48		
*LOWER OUT	JCT	25	55.125		12.8000	99.55		
*LOWER OUT	JCT	50	63.257		12.8000	118.14		
*LOWER OUT	JCT	100	69.463		12.7500	132.53		
LOWER POND	IN POND	1	18.300		12.2000	93.80		
LOWER POND	IN POND	2	23.257		12.2000	120.23		
LOWER POND	IN POND	5	35.779		12.2000	186.41		
LOWER POND	IN POND	10	45.272		12.2000	236.74		
LOWER POND	IN POND	25	55.126		12.2000	288.93		
LOWER POND	IN POND	50	63.258		12.2000	331.50		
LOWER POND	IN POND	100	69.464		12.2000	363.71		
LOWER POND	OUT POND	1	18.299		13.2000	23.18	834.47	4.618
LOWER POND	OUT POND	2	23.256		13.0500	32.18	834.81	5.753
LOWER POND	OUT POND	5	35.778		12.9000	56.96	835.64	8.646
LOWER POND	OUT POND	10	45.271		12.8500	77.48	836.24	10.825
LOWER POND	OUT POND	25	55.125		12.8000	99.55	836.83	13.072
LOWER POND	OUT POND	50	63.257		12.8000	118.14	837.29	14.893
LOWER POND	OUT POND	100	69.463		12.7500	132.53	837.63	16.273

Name.... Watershed

File.... Q:\PROJECT\stormwater\stormwater\20011352\pondpack\20011352.PPW

ICPM CALCULATION TOLERANCES

Target Convergence= .000 cfs +/-
 Max. Iterations = 35 loops
 ICPM Time Step = .0500 hrs
 Output Time Step = .0500 hrs
 ICPM Ending Time = 35.0000 hrs

MASTER NETWORK SUMMARY
 SCS Unit Hydrograph Method

(*Node=Outfall; +Node=Diversion;)
 (Trun= HYG Truncation: Blank=None; L=Left; R=Rt; LR=Left&Rt)

Node ID	Type	Return Event	HYG Vol ac-ft	Trun	Qpeak hrs	Qpeak cfs	Max WSEL ft	Max Pond Storage ac-ft
OFFSITE AREA 001 AREA		1	6.894		12.5500	36.76		
OFFSITE AREA 001 AREA		2	8.897		12.5500	49.41		
OFFSITE AREA 001 AREA		5	14.043		12.5500	81.91		
OFFSITE AREA 001 AREA		10	18.024		12.5500	106.93		
OFFSITE AREA 001 AREA		25	22.192		12.5500	133.00		
OFFSITE AREA 001 AREA		50	25.634		12.5500	154.42		
OFFSITE AREA 001 AREA		100	28.269		12.5500	170.74		
OFFSITE AREA 002 AREA		1	.525		12.4500	3.56		
OFFSITE AREA 002 AREA		2	.647		12.4500	4.43		
OFFSITE AREA 002 AREA		5	.948		12.4500	6.56		
OFFSITE AREA 002 AREA		10	1.172		12.4500	8.12		
OFFSITE AREA 002 AREA		25	1.402		12.4500	9.71		
OFFSITE AREA 002 AREA		50	1.590		12.4500	10.99		
OFFSITE AREA 002 AREA		100	1.732		12.4500	11.96		
OFFSITE AREA 003 AREA		1	1.545		12.4500	10.40		
OFFSITE AREA 003 AREA		2	1.965		12.4000	13.56		
OFFSITE AREA 003 AREA		5	3.033		12.3500	21.66		
OFFSITE AREA 003 AREA		10	3.849		12.3500	27.86		
OFFSITE AREA 003 AREA		25	4.699		12.3500	34.28		
OFFSITE AREA 003 AREA		50	5.398		12.3500	39.53		
OFFSITE AREA 003 AREA		100	5.931		12.3500	43.51		

Name.... Watershed

File.... Q:\PROJECT\stormwater\stormwater\20011352\pondpack\20011352.PPW

ICPM CALCULATION TOLERANCES

Target Convergence= .000 cfs +/-
 Max. Iterations = 35 loops
 ICPM Time Step = .0500 hrs
 Output Time Step = .0500 hrs
 ICPM Ending Time = 35.0000 hrs

 MASTER NETWORK SUMMARY
 SCS Unit Hydrograph Method

(*Node=Outfall; +Node=Diversion;)
 (Trun= HYG Truncation: Blank=None; L=Left; R=Rt; LR=Left&Rt)

Node ID	Type	Return Event	HYG Vol ac-ft	Trun	Qpeak hrs	Qpeak cfs	Max WSEL ft	Max Pond Storage ac-ft
SR 62	POND	1	6.894		12.5500	36.76		
SR 62	POND	2	8.897		12.5500	49.41		
SR 62	POND	5	14.043		12.5500	81.91		
SR 62	POND	10	18.024		12.5500	106.93		
SR 62	POND	25	22.192		12.5500	133.00		
SR 62	POND	50	25.634		12.5500	154.42		
SR 62	POND	100	28.269		12.5500	170.74		
SR 62	OUT POND	1	6.893		13.1000	12.11	865.25	1.885
SR 62	OUT POND	2	8.896		13.2500	12.35	865.58	3.051
SR 62	OUT POND	5	14.042		13.0000	12.77	866.25	6.345
SR 62	OUT POND	10	18.022		14.2000	12.83	866.62	9.081
SR 62	OUT POND	25	22.180		14.2500	23.65	866.88	10.992
SR 62	OUT POND	50	25.618		13.8500	36.14	867.01	12.004
SR 62	OUT POND	100	28.253		13.7500	44.90	867.09	12.824
UPPER POND	POND	1	7.418		12.5500	14.94		
UPPER POND	POND	2	9.543		12.5500	16.19		
UPPER POND	POND	5	14.990		12.4500	18.70		
UPPER POND	POND	10	19.194		12.4500	20.48		
UPPER POND	POND	25	23.582		14.2000	24.85		
UPPER POND	POND	50	27.207		13.8000	37.95		
UPPER POND	POND	100	29.985		13.7000	47.08		

Name.... Watershed

File.... Q:\PROJECT\stormwater\stormwater\20011352\pondpack\20011352.PPW

ICPM CALCULATION TOLERANCES

```

-----
Target Convergence=      .000 cfs +/-
Max. Iterations   =       35 loops
ICPM Time Step    =       .0500 hrs
Output Time Step  =       .0500 hrs
ICPM Ending Time  =      35.0000 hrs
-----

```

MASTER NETWORK SUMMARY
SCS Unit Hydrograph Method

(*Node=Outfall; +Node=Diversion;)
(Trun= HYG Truncation: Blank=None; L=Left; R=Rt; LR=Left&Rt)

Node ID	Type	Return Event	HYG Vol ac-ft	Trun	Qpeak hrs	Qpeak cfs	Max WSEL ft	Max Pond Storage ac-ft
UPPER POND	OUT POND	1	7.267		16.3000	11.40	859.97	1.637
UPPER POND	OUT POND	2	9.387		16.6500	11.90	860.03	1.686
UPPER POND	OUT POND	5	14.786		16.5500	12.84	860.13	1.780
UPPER POND	OUT POND	10	18.878		15.9000	13.37	860.18	1.833
UPPER POND	OUT POND	25	23.141		15.3000	20.09	860.96	2.574
UPPER POND	OUT POND	50	26.695		15.3000	25.04	862.11	3.785
UPPER POND	OUT POND	100	29.416		15.0500	32.73	862.73	4.482

File.... Q:\PROJECT\stormwater\stormwater\20011352\pondpack\
Title... Project Date: 7/21/2003
Project Engineer: EMHT INC
Project Title: Hoover Park Section 7
Project Comments:

DESIGN STORMS SUMMARY

Design Storm File, ID = Franklin County

Storm Tag Name = 1 yr

Data Type, File, ID = Synthetic Storm TypeII 24hr
Storm Frequency = 1 yr
Total Rainfall Depth= 2.3000 in
Duration Multiplier = 1
Resulting Duration = 24.0000 hrs
Resulting Start Time= .0000 hrs Step= .1000 hrs End= 24.0000 hrs

Storm Tag Name = 2 yr

Data Type, File, ID = Synthetic Storm TypeII 24hr
Storm Frequency = 2 yr
Total Rainfall Depth= 2.6000 in
Duration Multiplier = 1
Resulting Duration = 24.0000 hrs
Resulting Start Time= .0000 hrs Step= .1000 hrs End= 24.0000 hrs

Storm Tag Name = 5 yr

Data Type, File, ID = Synthetic Storm TypeII 24hr
Storm Frequency = 5 yr
Total Rainfall Depth= 3.3000 in
Duration Multiplier = 1
Resulting Duration = 24.0000 hrs
Resulting Start Time= .0000 hrs Step= .1000 hrs End= 24.0000 hrs

Storm Tag Name = 10 yr

Data Type, File, ID = Synthetic Storm TypeII 24hr
Storm Frequency = 10 yr
Total Rainfall Depth= 3.8000 in
Duration Multiplier = 1
Resulting Duration = 24.0000 hrs
Resulting Start Time= .0000 hrs Step= .1000 hrs End= 24.0000 hrs

Storm Tag Name = 25 yr

Data Type, File, ID = Synthetic Storm TypeII 24hr
Storm Frequency = 25 yr
Total Rainfall Depth= 4.3000 in
Duration Multiplier = 1
Resulting Duration = 24.0000 hrs
Resulting Start Time= .0000 hrs Step= .1000 hrs End= 24.0000 hrs

Type.... Design Storms
Name.... Franklin County

Page 2.02

File.... Q:\PROJECT\stormwater\stormwater\20011352\pondpack\
Title... Project Date: 7/21/2003
Project Engineer: EMHT INC
Project Title: Hoover Park Section 7
Project Comments:

DESIGN STORMS SUMMARY

Design Storm File, ID = Franklin County

Storm Tag Name = 50 yr

Data Type, File, ID = Synthetic Storm TypeII 24hr
Storm Frequency = 50 yr
Total Rainfall Depth= 4.7000 in
Duration Multiplier = 1
Resulting Duration = 24.0000 hrs
Resulting Start Time= .0000 hrs Step= .1000 hrs End= 24.0000 hrs

Storm Tag Name = 100 yr

Data Type, File, ID = Synthetic Storm TypeII 24hr
Storm Frequency = 100 yr
Total Rainfall Depth= 5.0000 in
Duration Multiplier = 1
Resulting Duration = 24.0000 hrs
Resulting Start Time= .0000 hrs Step= .1000 hrs End= 24.0000 hrs

Type.... Tc Calcs
Name.... AREA 004

Page 3.01

File.... Q:\PROJECT\stormwater\stormwater\20011352\pondpack\20011352.PPW

.....
TIME OF CONCENTRATION CALCULATOR
.....

Segment #1: Tc: User Defined

Segment #1 Time: .5400 hrs

=====
Total Tc: .5400 hrs
=====

Type.... Tc Calcs
Name.... AREA 004

Page 3.02

File.... Q:\PROJECT\stormwater\stormwater\20011352\pondpack\20011352.PPW

Tc Equations used...

==== User Defined =====

Tc = Value entered by user

Where: Tc = Time of concentration

Type.... Tc Calcs
Name.... AREA 006

Page 3.03

File.... Q:\PROJECT\stormwater\stormwater\20011352\pondpack\20011352.PPW

.....
TIME OF CONCENTRATION CALCULATOR
.....

Segment #1: Tc: User Defined

Segment #1 Time: .2400 hrs

=====
Total Tc: .2400 hrs
=====

Type.... Tc Calcs
Name.... AREA 006

Page 3.04

File.... Q:\PROJECT\stormwater\stormwater\20011352\pondpack\20011352.PPW

Tc Equations used...

==== User Defined =====

Tc = Value entered by user

Where: Tc = Time of concentration

Type.... Tc Calcs
Name.... OFFSITE AREA 001

Page 3.05

File.... Q:\PROJECT\stormwater\stormwater\20011352\pondpack\20011352.PPW

.....
TIME OF CONCENTRATION CALCULATOR
.....

Segment #1: Tc: User Defined

Segment #1 Time: 1.0700 hrs

=====
Total Tc: 1.0700 hrs
=====

Type.... Tc Calcs
Name.... OFFSITE AREA 001

Page 3.06

File.... Q:\PROJECT\stormwater\stormwater\20011352\pondpack\20011352.PPW

Tc Equations used...

==== User Defined =====

Tc = Value entered by user

Where: Tc = Time of concentration

Type.... Tc Calcs
Name.... OFFSITE AREA 002

Page 3.07

File.... Q:\PROJECT\stormwater\stormwater\20011352\pondpack\20011352.PPW

.....
TIME OF CONCENTRATION CALCULATOR
.....

Segment #1: Tc: User Defined

Segment #1 Time: .8800 hrs

=====
Total Tc: .8800 hrs
=====

Type.... Tc Calcs
Name.... OFFSITE AREA 002

Page 3.08

File.... Q:\PROJECT\stormwater\stormwater\20011352\pondpack\20011352.PPW

Tc Equations used...

==== User Defined =====

Tc = Value entered by user

Where: Tc = Time of concentration

Type.... Tc Calcs
Name.... OFFSITE AREA 003

Page 3.10

File.... Q:\PROJECT\stormwater\stormwater\20011352\pondpack\20011352.PPW

Tc Equations used...

==== User Defined =====

Tc = Value entered by user

Where: Tc = Time of concentration

Type.... Runoff CN-Area
Name.... AREA 004

Page 4.01

File.... Q:\PROJECT\stormwater\stormwater\20011352\pondpack\20011352.PPW

RUNOFF CURVE NUMBER DATA

.....

Soil/Surface Description	CN	Area acres	Impervious Adjustment		Adjusted CN
			%C	%UC	
onsite area	82	120.600			82.00

COMPOSITE AREA & WEIGHTED CN ---> 120.600 82.00 (82)

.....

Type.... Runoff CN-Area
Name.... AREA 006

Page 4.02

File.... Q:\PROJECT\stormwater\stormwater\20011352\pondpack\20011352.PPW

RUNOFF CURVE NUMBER DATA

.....

Soil/Surface Description CN Area Impervious Adjusted
 acres Adjustment CN
 %C %UC -----
Hoover Park Section 1 83 12.000 83.00

COMPOSITE AREA & WEIGHTED CN ---> 12.000 83.00 (83)
.....

Type.... Runoff CN-Area
Name.... OFFSITE AREA 001

Page 4.03

File.... Q:\PROJECT\stormwater\stormwater\20011352\pondpack\20011352.PPW

RUNOFF CURVE NUMBER DATA

.....

Soil/Surface Description	CN	Area acres	Impervious Adjustment %C %UC	Adjusted CN
agricultural	78	125.090		78.00

COMPOSITE AREA & WEIGHTED CN ---> 125.090 78.00 (78)
.....

Type.... Runoff CN-Area
Name.... OFFSITE AREA 002

Page 4.04

File.... Q:\PROJECT\stormwater\stormwater\20011352\pondpack\20011352.PPW

RUNOFF CURVE NUMBER DATA

.....

Soil/Surface Description	CN	Area acres	Impervious Adjustment %C %UC	Adjusted CN
right of way	85	1.370		85.00
onsite area (minus pond)	82	4.050		82.00
pond	98	.750		98.00

COMPOSITE AREA & WEIGHTED CN ---> 6.170 84.61 (85)
.....

Type.... Runoff CN-Area
Name.... OFFSITE AREA 003

Page 4.05

File.... Q:\PROJECT\stormwater\stormwater\20011352\pondpack\20011352.PPW

RUNOFF CURVE NUMBER DATA

.....

Soil/Surface Description	CN	Area acres	Impervious Adjustment		Adjusted CN
			%C	%UC	
-----	-----	-----	-----	-----	-----
offsite area	80	24.600			80.00

COMPOSITE AREA & WEIGHTED CN ---> 24.600 80.00 (80)
.....

File.... Q:\PROJECT\stormwater\stormwater\20011352\pondpack\20011352.PPW

Elevation (ft)	Planimeter (sq.in)	Area (acres)	A1+A2+sq ^r (A1*A2) (acres)	Volume (ac-ft)	Volume Sum (ac-ft)
833.00	-----	2.9800	.0000	.000	.000
834.00	-----	3.2000	9.2680	3.089	3.089
835.00	-----	3.4300	9.9430	3.314	6.404
836.00	-----	3.6700	10.6480	3.549	9.953
837.00	-----	3.9200	11.3829	3.794	13.747
838.00	-----	4.1600	12.1182	4.039	17.787

POND VOLUME EQUATIONS

* Incremental volume computed by the Conic Method for Reservoir Volumes.

$$\text{Volume} = (1/3) * (\text{EL2}-\text{EL1}) * (\text{Areal} + \text{Area2} + \text{sq.rt.}(\text{Areal}*\text{Area2}))$$

where: EL1, EL2 = Lower and upper elevations of the increment
 Areal,Area2 = Areas computed for EL1, EL2, respectively
 Volume = Incremental volume between EL1 and EL2

File.... Q:\PROJECT\stormwater\stormwater\20011352\pondpack\20011352.PPW

Elevation (ft)	Planimeter (sq.in)	Area (acres)	A1+A2+sq ^{rt} (A1*A2) (acres)	Volume (ac-ft)	Volume Sum (ac-ft)
860.00	-----	.0000	.0000	.000	.000
864.00	-----	.1000	.1000	.133	.133
865.00	-----	1.9500	2.4916	.831	.964
866.00	-----	5.3500	10.5299	3.510	4.474
867.00	-----	9.6000	22.1166	7.372	11.846
868.00	-----	10.0000	29.3980	9.799	21.645

POND VOLUME EQUATIONS

* Incremental volume computed by the Conic Method for Reservoir Volumes.

$$\text{Volume} = (1/3) * (\text{EL2}-\text{EL1}) * (\text{Areal} + \text{Area2} + \text{sq.rt.}(\text{Areal}*\text{Area2}))$$

where: EL1, EL2 = Lower and upper elevations of the increment
 Areal,Area2 = Areas computed for EL1, EL2, respectively
 Volume = Incremental volume between EL1 and EL2

File.... Q:\PROJECT\stormwater\stormwater\20011352\pondpack\20011352.PPW

Elevation (ft)	Planimeter (sq.in)	Area (acres)	A1+A2+sqr(A1*A2) (acres)	Volume (ac-ft)	Volume Sum (ac-ft)
858.00	-----	.7500	.0000	.000	.000
859.00	-----	.8300	2.3690	.790	.790
860.00	-----	.9100	2.6091	.870	1.659
861.00	-----	1.0000	2.8639	.955	2.614
862.00	-----	1.0900	3.1340	1.045	3.659
863.00	-----	1.1800	3.4041	1.135	4.793

POND VOLUME EQUATIONS

* Incremental volume computed by the Conic Method for Reservoir Volumes.

$$\text{Volume} = (1/3) * (\text{EL2}-\text{EL1}) * (\text{Areal} + \text{Area2} + \text{sq.rt.}(\text{Areal}*\text{Area2}))$$

where: EL1, EL2 = Lower and upper elevations of the increment
 Areal,Area2 = Areas computed for EL1, EL2, respectively
 Volume = Incremental volume between EL1 and EL2

Type.... Outlet Input Data
Name.... lower pond out

Page 6.01

File.... Q:\PROJECT\stormwater\stormwater\20011352\pondpack\20011352.PPW

REQUESTED POND WS ELEVATIONS:

Min. Elev.= 833.00 ft
Increment = .50 ft
Max. Elev.= 838.00 ft

OUTLET CONNECTIVITY

---> Forward Flow Only (UpStream to DnStream)
<--- Reverse Flow Only (DnStream to UpStream)
<---> Forward and Reverse Both Allowed

Structure	No.		Outfall	E1, ft	E2, ft
User Defined Table	1	---	TW	.000	838.000
TW SETUP, DS Channel					

Type.... Outlet Input Data
Name.... lower pond out

Page 6.02

File.... Q:\PROJECT\stormwater\stormwater\20011352\pondpack\20011352.PPW

OUTLET STRUCTURE INPUT DATA

Structure ID = 1
Structure Type = User Defined Table

ELEV-FLOW RATING TABLE

Elev, ft	Flow, cfs
833.00	.00
833.20	1.00
833.40	2.80
833.60	5.10
833.80	8.20
834.00	11.90
834.20	16.20
834.40	21.20
834.60	26.60
834.80	32.00
835.00	37.30
835.50	52.30
836.00	69.00
836.50	87.00
837.00	106.20
837.50	126.80
838.00	148.50

Structure ID = TW
Structure Type = TW SETUP, DS Channel

FREE OUTFALL CONDITIONS SPECIFIED

CONVERGENCE TOLERANCES...
Maximum Iterations= 30
Min. TW tolerance = .01 ft
Max. TW tolerance = .01 ft
Min. HW tolerance = .01 ft
Max. HW tolerance = .01 ft
Min. Q tolerance = .10 cfs
Max. Q tolerance = .10 cfs

Type.... Composite Rating Curve
Name.... lower pond out

Page 6.03

File.... Q:\PROJECT\stormwater\stormwater\20011352\pondpack\20011352.PPW

***** COMPOSITE OUTFLOW SUMMARY *****

WS Elev, Total Q		Converge		Notes
Elev. ft	Q cfs	TW Elev ft	Error +/-ft	Contributing Structures
833.00	.00	Free Outfall	1	
833.50	3.95	Free Outfall	1	
834.00	11.90	Free Outfall	1	
834.50	23.90	Free Outfall	1	
835.00	37.30	Free Outfall	1	
835.50	52.30	Free Outfall	1	
836.00	69.00	Free Outfall	1	
836.50	87.00	Free Outfall	1	
837.00	106.20	Free Outfall	1	
837.50	126.80	Free Outfall	1	
838.00	148.50	Free Outfall	1	

File.... Q:\PROJECT\stormwater\stormwater\20011352\pondpack\20011352.PPW

REQUESTED POND WS ELEVATIONS:

Min. Elev.= 858.00 ft
Increment = .25 ft
Max. Elev.= 863.00 ft

OUTLET CONNECTIVITY

----> Forward Flow Only (UpStream to DnStream)
<--- Reverse Flow Only (DnStream to UpStream)
<----> Forward and Reverse Both Allowed

Structure	No.		Outfall	E1, ft	E2, ft
Culvert-Circular	1	---->	TW	858.000	863.000
User Defined Table	2	---->	TW	.000	863.000
TW SETUP, DS Channel					

File.... Q:\PROJECT\stormwater\stormwater\20011352\pondpack\20011352.PPW

OUTLET STRUCTURE INPUT DATA

Structure ID = 1
Structure Type = Culvert-Circular

No. Barrels = 1
Barrel Diameter = 2.0000 ft
Upstream Invert = 858.00 ft
Dnstream Invert = 855.39 ft
Horiz. Length = 279.00 ft
Barrel Length = 279.01 ft
Barrel Slope = .00935 ft/ft

OUTLET CONTROL DATA...

Mannings n = .0130
Ke = .5000 (forward entrance loss)
Kb = .012411 (per ft of full flow)
Kr = .5000 (reverse entrance loss)
HW Convergence = .001 +/- ft

INLET CONTROL DATA...

Equation form = 1
Inlet Control K = .0098
Inlet Control M = 2.0000
Inlet Control c = .03980
Inlet Control Y = .6700
T1 ratio (HW/D) = 1.155
T2 ratio (HW/D) = 1.302
Slope Factor = -.500

Use unsubmerged inlet control Form 1 equ. below T1 elev.
Use submerged inlet control Form 1 equ. above T2 elev.

In transition zone between unsubmerged and submerged inlet control,
interpolate between flows at T1 & T2...

At T1 Elev = 860.31 ft ---> Flow = 15.55 cfs
At T2 Elev = 860.60 ft ---> Flow = 17.77 cfs

Type.... Outlet Input Data
Name.... PR 10

Page 6.06

File.... Q:\PROJECT\stormwater\stormwater\20011352\pondpack\20011352.PPW

OUTLET STRUCTURE INPUT DATA

Structure ID = 2
Structure Type = User Defined Table

ELEV-FLOW RATING TABLE

Elev, ft	Flow, cfs
858.00	.00
862.25	.01
862.50	4.60
862.75	6.50
863.00	7.90

Structure ID = TW
Structure Type = TW SETUP, DS Channel

FREE OUTFALL CONDITIONS SPECIFIED

CONVERGENCE TOLERANCES...
Maximum Iterations= 30
Min. TW tolerance = .01 ft
Max. TW tolerance = .01 ft
Min. HW tolerance = .01 ft
Max. HW tolerance = .01 ft
Min. Q tolerance = .10 cfs
Max. Q tolerance = .10 cfs

File.... Q:\PROJECT\stormwater\stormwater\20011352\pondpack\20011352.PPW

***** COMPOSITE OUTFLOW SUMMARY *****

WS Elev, Total Q		Converge		Notes
Elev. ft	Q cfs	TW Elev ft	Error +/-ft	Contributing Structures
858.00	.00	Free Outfall	2	
858.25	.24	Free Outfall	1 +2	
858.50	.91	Free Outfall	1 +2	
858.75	2.00	Free Outfall	1 +2	
859.00	3.43	Free Outfall	1 +2	
859.25	5.17	Free Outfall	1 +2	
859.50	7.15	Free Outfall	1 +2	
859.75	9.33	Free Outfall	1 +2	
860.00	11.63	Free Outfall	1 +2	
860.25	14.03	Free Outfall	1 +2	
860.50	16.42	Free Outfall	1 +2	
860.75	18.77	Free Outfall	1 +2	
861.00	20.36	Free Outfall	1 +2	
861.25	21.82	Free Outfall	1 +2	
861.50	23.21	Free Outfall	1 +2	
861.75	24.32	Free Outfall	1 +2	
862.00	24.80	Free Outfall	1 +2	
862.25	25.34	Free Outfall	1 +2	
862.50	30.50	Free Outfall	1 +2	
862.75	32.96	Free Outfall	1 +2	
863.00	34.93	Free Outfall	1 +2	

File.... Q:\PROJECT\stormwater\stormwater\20011352\pondpack\20011352.PPW

REQUESTED POND WS ELEVATIONS:

Min. Elev.= 858.57 ft
Increment = .25 ft
Max. Elev.= 868.00 ft

OUTLET CONNECTIVITY

---> Forward Flow Only (UpStream to DnStream)
<--- Reverse Flow Only (DnStream to UpStream)
<---> Forward and Reverse Both Allowed

Structure	No.		Outfall	E1, ft	E2, ft
Weir-Rectangular	2	--->	TW	866.700	868.000
Culvert-Circular	1	<---	TW	858.570	868.000

TW SETUP, DS Channel

Type.... Outlet Input Data
Name.... SR 62 outlet

Page 1.02

File.... Q:\PROJECT\stormwater\stormwater\20011352\pondpack\20011352.PPW

OUTLET STRUCTURE INPUT DATA

Structure ID	=	2
Structure Type	=	Weir-Rectangular

# of Openings	=	1
Crest Elev.	=	866.70 ft
Weir Length	=	50.00 ft
Weir Coeff.	=	2.630000
Weir TW effects	(Use adjustment equation)	

File.... Q:\PROJECT\stormwater\stormwater\20011352\pondpack\20011352.PPW

OUTLET STRUCTURE INPUT DATA

Structure ID = 1
Structure Type = Culvert-Circular

No. Barrels = 1
Barrel Diameter = 1.5000 ft
Upstream Invert = 858.57 ft
Dnstream Invert = 858.00 ft
Horiz. Length = 350.00 ft
Barrel Length = 350.00 ft
Barrel Slope = .00163 ft/ft

OUTLET CONTROL DATA...

Mannings n = .0130
Ke = .5000 (forward entrance loss)
Kb = .018213 (per ft of full flow)
Kr = .5000 (reverse entrance loss)
HW Convergence = .001 +/- ft

INLET CONTROL DATA...

Equation form = 1
Inlet Control K = .0098
Inlet Control M = 2.0000
Inlet Control c = .03980
Inlet Control Y = .6700
T1 ratio (HW/D) = 1.159
T2 ratio (HW/D) = 1.306
Slope Factor = -.500

Use unsubmerged inlet control Form 1 equ. below T1 elev.
Use submerged inlet control Form 1 equ. above T2 elev.

In transition zone between unsubmerged and submerged inlet control,
interpolate between flows at T1 & T2...

At T1 Elev = 860.31 ft ---> Flow = 7.58 cfs
At T2 Elev = 860.53 ft ---> Flow = 8.66 cfs

Type.... Pond Routing Summary Page 7.01
Name.... LOWER POND OUT Tag: 100 yr Event: 100 yr
File.... Q:\PROJECT\stormwater\stormwater\20011352\pondpack\20011352.PPW
Storm... TypeII 24hr Tag: 100 yr

LEVEL POOL ROUTING SUMMARY

HYG Dir = Q:\PROJECT\stormwater\stormwater\20011352\pondpack\
Inflow HYG file = NONE STORED - LOWER POND IN 100 yr
Outflow HYG file = NONE STORED - LOWER POND OUT 100 yr

Pond Node Data = LOWER POND
Pond Volume Data = LOWER POND
Pond Outlet Data = lower pond out

No Infiltration

INITIAL CONDITIONS

Starting WS Elev = 833.00 ft
Starting Volume = .000 ac-ft
Starting Outflow = .00 cfs
Starting Infiltr. = .00 cfs
Starting Total Qout= .00 cfs
Time Increment = .0500 hrs

INFLOW/OUTFLOW HYDROGRAPH SUMMARY

=====

Peak Inflow	=	362.46 cfs	at	12.2000 hrs
Peak Outflow	=	131.56 cfs	at	12.7500 hrs

Peak Elevation	=	837.61 ft
Peak Storage	=	16.181 ac-ft

=====

MASS BALANCE (ac-ft)

+ Initial Vol	=	.000
+ HYG Vol IN	=	66.248
- Infiltration	=	.000
- HYG Vol OUT	=	66.247
- Retained Vol	=	.001

Unrouted Vol = -.000 ac-ft (.000% of Inflow Volume)

Type.... ICPM Node Routing Summary Page 7.02
 Name.... SR 62 Tag: 100 yr Event: 100 yr
 File.... Q:\PROJECT\stormwater\stormwater\20011352\pondpack\20011352.PPW
 Storm... TypeII 24hr Tag: 100 yr

ICPM POND ROUTING SUMMARY

HYG Dir = Q:\PROJECT\stormwater\stormwater\20011352\pondpack\
 Inflow HYG file = SR 62 IN 100 yr
 Outflow HYG file = SR 62 OUT 100 yr

Pond Node Data = SR 62
 Pond Volume Data = SR 62
 Pond Outlet Data = SR 62 outlet

No Infiltration

INITIAL CONDITIONS

Starting WS Elev = 860.00 ft
 Starting Volume = .000 ac-ft
 Starting Outflow = .00 cfs

CALCULATION TOLERANCES

Target Convergence= .000 cfs +/-
 Max. Iterations = 35 loops
 ICPM Time Step = .0500 hrs
 Output Time Step = .0500 hrs
 ICPM Ending Time = 35.0000 hrs

MAXIMUM STORAGE

Tp, hrs	Elev, ft	Vol, ac-ft
13.7500	867.14	13.183

FORWARD FLOW PEAKS

Tp, hrs	Qp, cfs
---------	---------

REVERSE FLOW PEAKS

Tp, hrs	Qp, cfs
---------	---------

Pond Inflow.....	12.5500	170.74	.0000	.00
Pond Outflow....	13.7000	46.67	.0000	.00

TOTAL VOLUME IN

Vol, ac-ft	Direction
------------	-----------

TOTAL VOLUME OUT

Vol, ac-ft	Direction
------------	-----------

Pond Inflow.....	28.269	Forward	.000	Reverse
Pond Outflow....	.000	Reverse	25.790	Forward

MASS BALANCE (ac-ft)

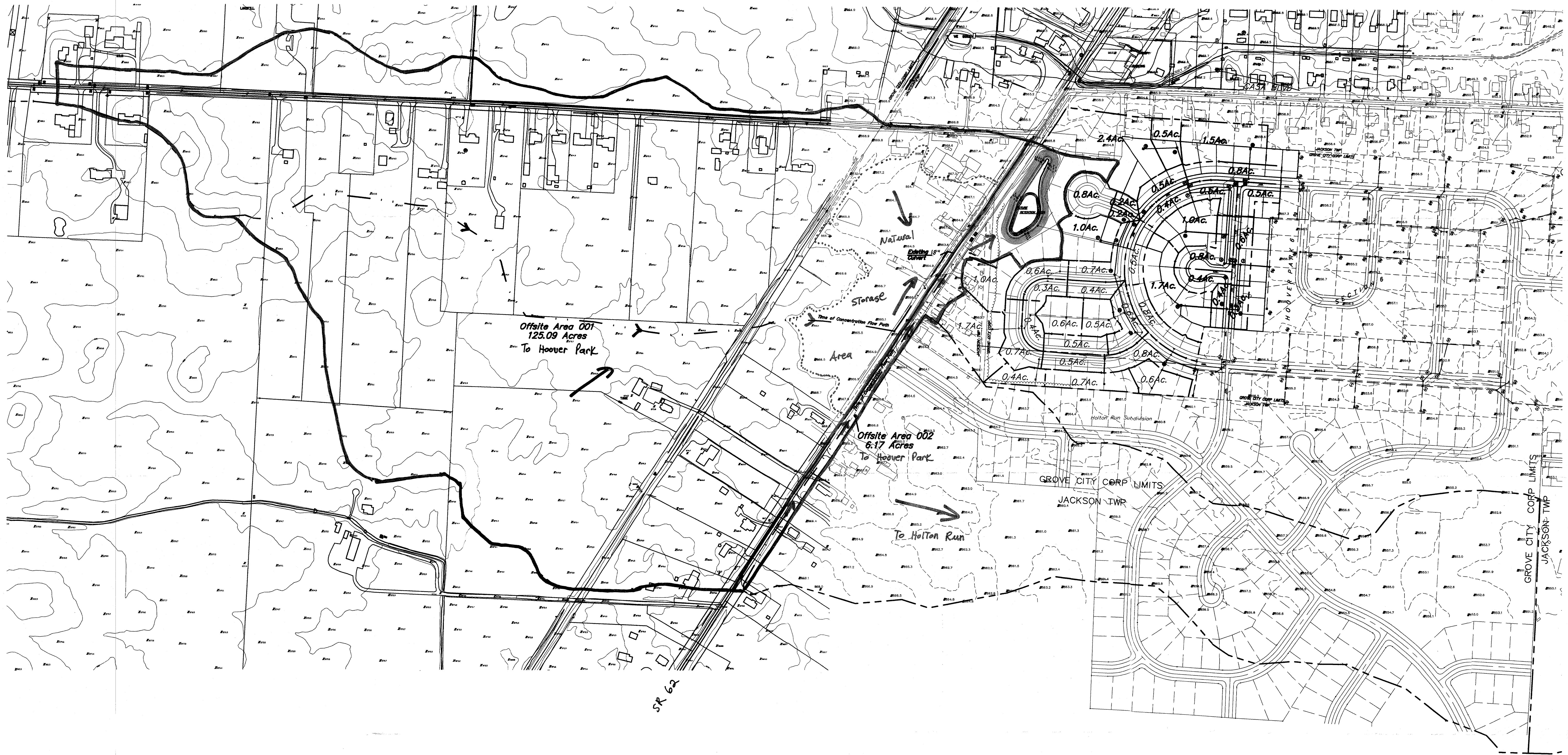
+ Initial Vol.....	.000
+ Total Vol IN....	28.269
- Total Vol OUT...	25.790
- Ending Pond Vol.	2.429 <-- (At 35.0000 hrs Elev.= 865.42 ft)
Difference.....	.050 ac-ft (.177% of Inflow Volume)

APPENDIX C

Hoover Park Section 8

Storm Sewer Tributary Area Maps

\\CHIDATA1\ADMIN\PROJECT\2001\352\DWG\STORMTRIB.DWG - 2 XREFS: 11352XBS 11352XST - PLOTTED BY DTHURNEY - June 07, 2004 - 9:33 AM



PREPARED BY
EVANS, MECHWART, HAMBLETON, & TILTON, INC.
CONSULTING ENGINEERS & SURVEYORS

STORM SEWER TRIBUTARY
AREA MAP
for
**HOOVER PARK
SECTION 7
PARTS 1 & 2**
City of Grove City, Ohio

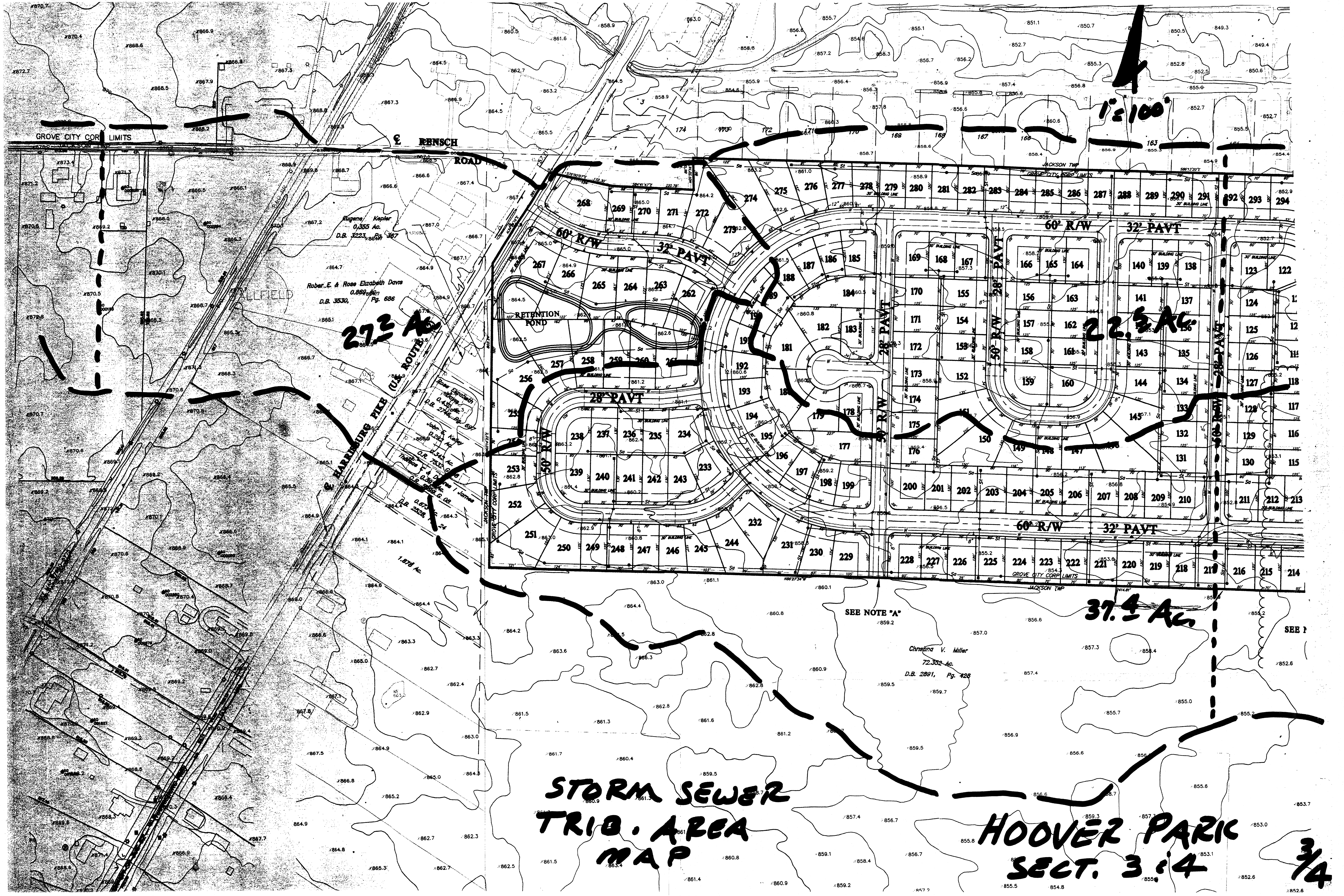
SCALE: 1" = 200'

June 2004

APPENDIX D

Hoover Park Section 8

Hoover Park Section 2 Exhibits



1"=100'

27.3 Ac

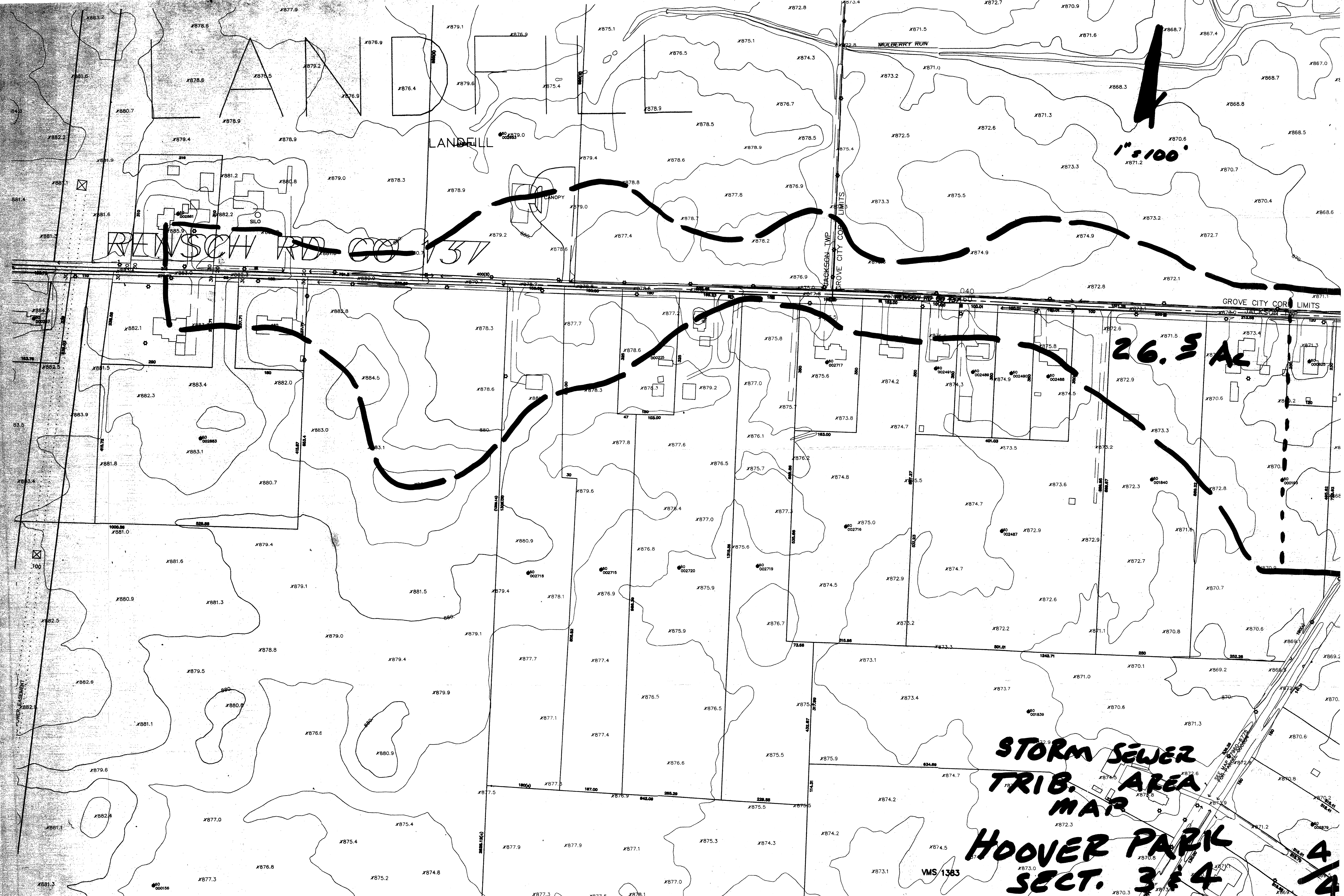
22.5 Ac

37.4 Ac

STORM SEWER
TRIB. AREA
MAP

HOOVER PARK
SECT. 3 of 4

3/4



1" = 100'

26.5 AC

STORM SEWER
TRIB. AREA
MAP

HOOPER PARK
SECT. 3 & 4

4/4