Pebblebrook Hydraulic Study

Clark Dietz Project No. H21020

Hamilton County, Indiana

March 2005

Clark Dietz, Inc.
9000 Keystone Crossing, Suite 350
Indianapolis, Indiana 46240
317.582.0300

TABLE OF CONTENTS

EXECUTIVE SUMMARY 1

1.0 INTRODUCTION 3
1.1 Project Background and Purpose 3
1.2 Scope of Services 4

2.0 DATA COLLECTION/EVALUATION 5
2.1 Hamilton County GIS 5
2.2 Pebblebrook Place Survey 5
2.3 Pebblebrook Main Channel Field Reconnaissance 5
2.4 Public Input 9
2.5 Previous Reports/Studies 9
2.6 As-Built Plans 9
2.7 Hydrologic Data 9

3.0 PEBBLEBROOK MAIN CHANNEL WATERSHED 11
3.1 Shed Description 11
3.2 Hydrologic/Hydraulic Analysis 11
3.3 Future Development: Stormwater Detention Policy Evaluation 17
3.4 Andover Development Evaluation 18
3.5 Problem Identification 19
3.6 Solutions 21
3.6 Summary – Pebblebrook Main Channel 25

4.0 PEBBLEBROOK SOUTH WATERSHED 26
4.1 Watershed Description 26
4.2 Problem Identification 26
4.3 Hydrologic/Hydraulic Analysis 29
4.4 Alternatives Solutions 34
4.5 Summary – Pebblebrook South Watershed 42

List of Figures

Figure 1-1 USGS Map Showing Study Area 3
Figure 2-1 Photo Log from Pebblebrook Main Channel Walkthrough....6
Figure 2-2 Central Indiana Precipitation August 31 – September 2, 2003 10
Figure 3-1 Pebblebrook North Watershed Map 12
Figure 3-2 HEC-HMS Model Graphical Interface....13
Figure 3-3 Schematic of HEC-HMS model 16
Figure 3-4 Hydrologic Impact of Future Development 17
Figure 3-5 Pebblebrook Place Road Profile....20
Figure 3-6 HY8 Performance Curve of the Proposed 12 foot by 6 foot Culvert 22
Figure 3-7 Profile of the Pebblebrook Main Channel at Pebblebrook Place 23
Figure 3-8 HY8 Performance Curve of the Proposed 7 foot by 3 foot Culvert 24
Figure 4-1 Pebblebrook South Watershed Map 27
Figure 4-2 Pond 2 Flooding Area....28
Figure 4-3 XP-SWMM schematic from model....30
Figure 4-4 Existing Pond Parameters....31
Figure 4-5 Pond 2 Detention Elevations during a 100-year Storm Event....33
Figure 4-6 Pond 3 Detention Elevations during a 100-year Storm Event....34
Figure 4-7 New 21 inch Outfall Profile Route....36
Figure 4-8 Graph showing Pond 2 stages 37
Figure 4-9 Pond 3 Stage Analysis with Pond 2 Improvements....38
Figure 4-10 Alternative 2 Improvements 39
Figure 4-11 Pond 3 Stage Analysis – Alternative Comparison 40
Figure 4-12 Graph showing total flow discharged to Pebblebrook South Channel....41

List of Tables

Table 3-1 Design Rainfall Depths 14
Table 3-2 Design Rainfall Time Distributions 14
Table 3-3 Pebblebrook Main Channel - Subbasin Hydrologic Parameters 15
Table 3-4 Summary of Flows from HEC-HMS Model for Pebblebrook Main Channel....16
Table 3-5 Pre-developed Flow Calculations for Andover Basin 18
Table 3-6 Post-developed Flow Calculations for Andover Basin 18
Table 3-7 Proposed Culvert Overtopping Evaluation....22
Table 4-1 Existing Pond Parameters....30
Table 4-2 Pebblebrook South - Subbasin Hydrologic Parameters....31
Table 4-3 Existing Pond Parameters 32

List of Appendices

Appendix A Detailed Cost Opinions

EXECUTIVE SUMMARY

Hamilton County retained Clark Dietz to prepare a hydrologic and hydraulic analysis of the Pebblebrook Main Channel and South Pebblebrook Watershed located north of State Route 32 and west of Little Chicago Road. The study was undertaken to identify the cause of flooding problems in the watershed and determine appropriate solutions. The following sections present key findings and recommendations for the study areas.

Pebblebrook Main Channel Study Area

Key Findings

- Triple 30-inch culverts at Pebblebrook Place are significantly undersized (less than 2-year storm) and are prone to clogging with debris.

- The existing 36-inch culvert at Moontown Road is undersized (less than 2-year storm). The downstream channel is also silted in and overgrown with vegetation in the channel bottom.

- The proposed Andover Development west of Moontown Road is providing release rates for their detention facilities that are more restrictive than the County's requirements (i.e. over-detaining). This will help alleviate downstream flooding, but will not preclude the need for improvements.

Recommendations

- Pebblebrook Place Improvements
  - ⇒ Replace the existing triple 30-inch culverts with a new 12-foot by 4-foot concrete box culvert
  - ⇒ Re-grade approximately 675 feet of channel in the vicinity of the culvert crossing
  - ⇒ The opinion of probable total project cost is $132,000.

- Moontown Road Improvements
  - ⇒ Replace the existing 36-inch culvert with a new 7-foot by 3-foot concrete box culvert
  - ⇒ Re-grade approximately 1,850 feet of channel downstream of the Moontown Road culvert
  - ⇒ The opinion of probable total project cost is $186,000

- Other Recommendations:
  - ⇒ Future watershed developments should have detention policy release rates at least as restrictive as those provided in the Andover Development (0.32 cfs/acre for a 100-year storm event, 0.19 cfs/acre for a 10-year storm event).

South Pebblebrook Study Area

Key Findings

- Two detention ponds in the Pebblebrook Subdivision have flooding problems:
  - ⇒ The pond at the north end of Pebblebrook Boulevard (near the entrance to Pebblebrook Golf Course) floods during events exceeding a 2-year storm. A total of 10 other ponds ultimately drain through this pond. The pond is significantly undersized to handle this

volume of stormwater runoff. The street along Pebblebrook Boulevard floods when the pond overtops; however, no homes are flooded.

⇒ The pond north of Ashbrook Drive, adjacent to the No. 2 tee of Pebblebrook Golf Course, floods during a 100-year event. During the Labor Day 2003 storm (greater than a 100-year storm), the pond overtopped, resulting in first floor flooding of three homes along Ashbrook Drive.

Recommendations

- Ashbrook Drive Pond Improvements
  - ⇒ Construct a new 21-inch outfall across golf course
  - ⇒ Construct approximately 300 feet of channel from the new outfall to the downstream receiving stream
  - ⇒ Place 5-inch restrictor plate on the existing 12-inch outfall
  - ⇒ The opinion of probable total project cost is $76,000

- Pebblebrook Boulevard Pond Improvements
  - ⇒ Construct the new 21-inch outfall from the Ashbrook Pond first. This improvement by itself will significantly improve flooding at the Pebblebrook Boulevard Pond (level of protection increases to nearly a 25-year storm event).
  - ⇒ To further reduce flooding (100-year protection), the outlet sewer from the pond should be upsized from a 15-inch to a 24-inch pipe.
  - ⇒ The opinion of probable total project cost is $13,000

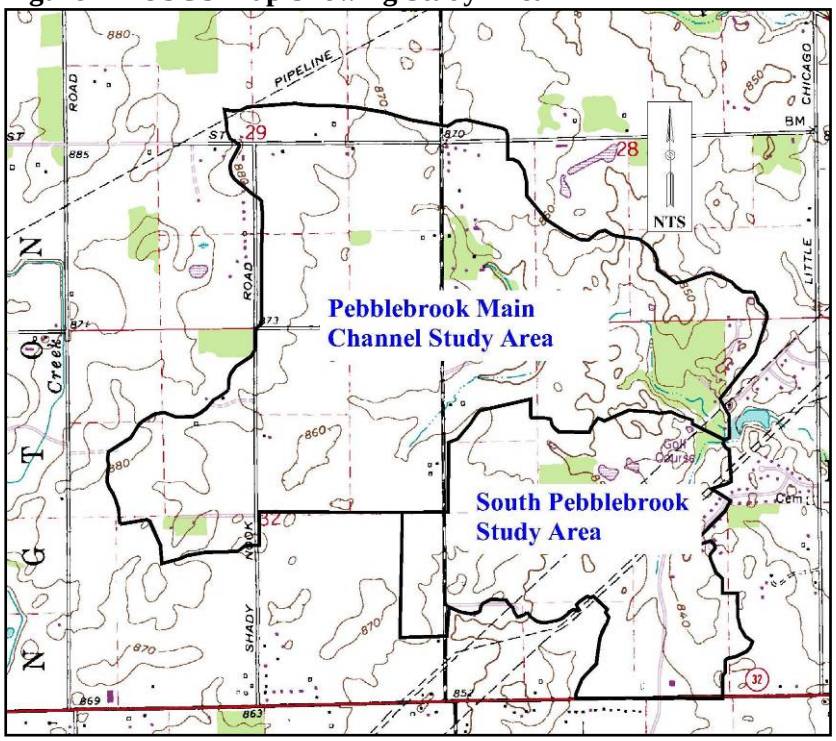
- Other Recommendations
  - ⇒ No new drainage areas should be added to the Pebblebrook South Watershed. Areas draining to the Fred Hines drain should continue to discharge as the do currently.
  - ⇒ Any new developed areas within the Pebblebrook South Watershed should be required to provide detention that restricts the 100-year release rate to 0.08 cfs/acre.

1.0 INTRODUCTION

1.1 Project Background and Purpose

Hamilton County retained Clark Dietz to prepare a hydrologic and hydraulic analysis of the Pebblebrook Main Channel and South Pebblebrook Watershed. The following figure shows the project study areas.

Figure 1-1 USGS Map Showing Study Area



This project was separated into two primary study areas:

Pebblebrook Main Channel

Analysis of main channel condition and potential impacts of future development, culvert hydraulic and capacity analyses, stream stability and sedimentation consideration. This analysis included channel reaches within the regulated drain and upstream of the regulated drain.

South Pebblebrook Watershed

Identification and analysis of drainage problems associated with the existing detention pond east of Village Way, the existing detention pond west of Tee #2 on the Pebblebrook Golf Course, and the backyard drainage problems reported at the residences along the north side of Ashbrook Drive between Bentgrass Drive and Crown Point Court.

1.2 Scope of Services

Pebblebrook Main Study Area – Pebblebrook Main Channel

1. Develop a HEC-HMS stormwater runoff model to simulate existing runoff, flooding potential, and estimate possible increases that may be caused by future development.
2. Identify existing and potential future flooding problems in the existing downstream and proposed upstream subdivision.
3. Evaluate the effectiveness of existing on-site stormwater detention requirements on controlling future flood peaks for future development.
4. Evaluate the triple 30 inch culverts at Pebblebrook Place.
5. Develop a set of solutions for existing and potential future flooding problems under existing and future development conditions.
6. Review operation and maintenance practices and needs.
7. Obtain public and developer input for this problem area.
8. Prepare a draft and final report.

South Pebblebrook Watershed Neighborhood Drainage Problems

1. Develop a XP-SWMM stormwater model of existing ponds to determine the effectiveness, identify problems, and propose solutions to problems reported in this area.
2. Acquire and review GIS mapping, drainage system maps and records, subdivision plats/plans, past hydraulic studies, complaint records, and other available existing information in the drainage problem areas.
3. Assist the County Surveyor in conducting a neighborhood meeting to obtain input from the residents on their drainage concerns.
4. Locate and characterize existing drainage problems in the study area.
5. Evaluate the effectiveness of existing on-site stormwater detention.
6. Develop a set of solutions to existing drainage problems.
7. Prepare a draft and final report.

2.0 DATA COLLECTION AND EVALUATION

2.1 Hamilton County GIS

Geographic Information System (GIS) mapping and aerial photography of the study area were downloaded from the Hamilton County website. These data were used to generate working maps to illustrate existing buildings, streams, roadways, stormwater infrastructure, 2-foot contour elevations, and other items useful in determining existing conditions of the watershed. Based on contours from this mapping the watershed boundary was delineated and then subdivided into smaller subbasins within the two main watersheds.

2.2 Pebblebrook Place Survey (Pebblebrook Main Channel)

Surveying of the Pebblebrook Place culvert crossing was completed to more accurately determine site geometry, culvert elevations, overtopping elevations, and the extent of flooding. Elevations were tied into the GIS spot elevations downloaded from the Hamilton County website. Photographs of the culvert crossing are shown below.

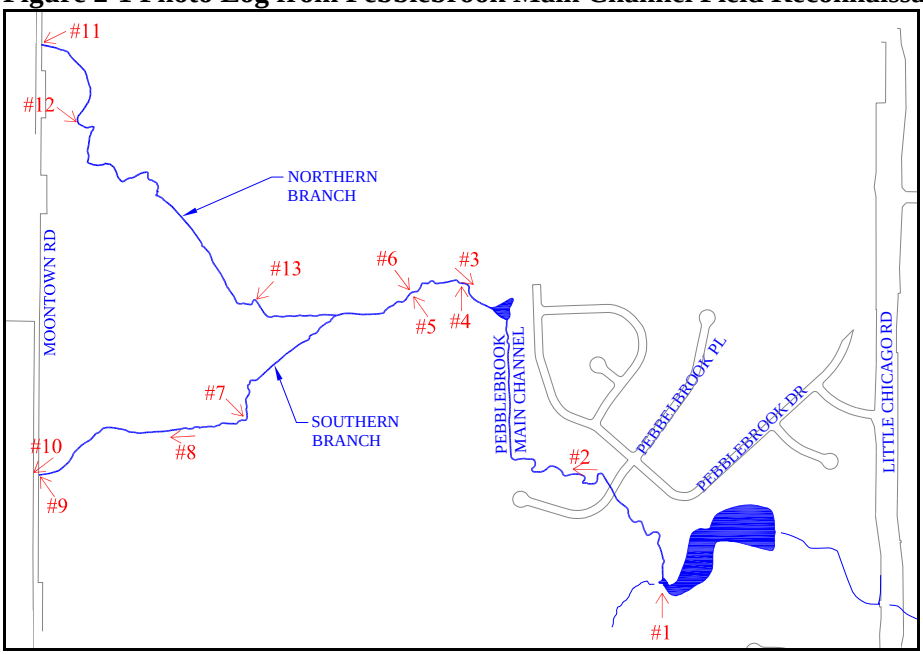
Existing triple 30-inch culverts under Pebblebrook Place

View of Pebblebrook Place above existing triple 30-inch culverts

2.3 Pebblebrook Main Channel Field Reconnaissance

A field reconnaissance of Pebblebrook Main Channel was completed to evaluate and current conditions. Cross sectional measurements of the channel and photographs of representative reaches were obtained. Figure 2-1 shows the location where selected photographs were taken during the field reconnaissance.

Figure 2-1 Photo Log from Pebblebrook Main Channel Field Reconnaissance



The following photographs and captions summarize the channel field reconnaissance.

Photograph #1: *Southernmost detention pond at Pebblebrook Main Channel*

Photograph #2: *Typical stream section between Pebblebrook Place and Pebblebrook Golf Course*

Photograph #1 is a view of the detention pond to which the Pebblebrook Main Channel and the Pebblebrook South Watershed discharge. Shown in the photograph is a dry-weather discharge into the detention pond. This is presumably from a golf course irrigation system but should be further investigated to verify that it is not an illicit discharge.

Photograph #3: *Typical stream section at the Pebblebrook Golf Course*

Photograph #4: *Log Jam in Channel*

Photograph #5: *15-inch outfall from Pebblebrook Golf Course*

Photograph #6: *Minor bank erosion of channel*

Photograph #7: *Minor bank erosion of channel*

Photograph #8: *Dense vegetation in channel*

Photograph #8 shows heavy vegetation growth and silt in the south branch of the Pebblebrook Main Channel, downstream of Moontown Road.

Photograph #9: *Downstream view of 36-inch Culvert at Moontown Road at the southern branch of Pebblebrook Main Channel*

Photograph #10: *Upstream view of 36-inch Culvert at Moontown Road at the southern branch of Pebblebrook Main Channel*

Photograph #9 shows the downstream end of the 36-inch culvert that crosses under Moontown Road. The culvert was found to be half full of sediment at the downstream end.

Minutes from an August 23rd, 2004 Hamilton County Drainage Board meeting indicated that the Moontown Road culvert has had flooding problem for years. While the restriction at the downstream side of the culvert contributes to the road overtopping, it is not main cause of the overtopping. The culvert is also undersized. Existing conditions and solutions will be discussed further in Section 3.3.

Photograph #11 *View of the downstream end of the 32"x48" culvert under Moontown Road at the northern branch of Pebblebrook Main Channel*

Photograph #12 *Typical channel view of the northern branch of Pebblebrook Main Channel.*

Photograph #13: *View of the northern branch of Pebblebrook Main Channel as it transitions from a wooded area into the golf course.*

2.4 Public Input

On October 12th, 2004, a presentation to the Pebblebrook Homeowners Association was made. The purpose and progress of the project were reviewed. Input from residents concerning flooding problems was noted and considered during the evaluation.

Minutes from Hamilton County Drainage Board meetings were reviewed and problems concerning the Pebblebrook area were noted. Complaints of frequent overtopping of the roadway at Pebblebrook Place and Moontown road were noted. In addition, complaints of detention pond flooding problems in the Pebblebrook neighborhood were reviewed in preparation for analysis of the system.

2.5 Previous Reports/Studies

Beazer Homes is in the process of developing a residential subdivision in Hamilton County, hereafter referred to as Andover. The site is located west of Moontown Road and south of 186th Street. The *Andover Overall Drainage Report*, prepared by Stoeppelwerth & Associates, Inc., was submitted to Hamilton County (May 18th, 2004) for approval. This report was reviewed to determine the hydrologic and hydraulic impacts of the development on the watershed and to evaluate the effectiveness of the proposed on-site stormwater detention. After review of the report, Clark Dietz met with Stoeppelwerth & Associates to clarify remaining questions and obtain understanding of the drainage features of Andover (see Section 3.5).

2.6 As-Built Plans

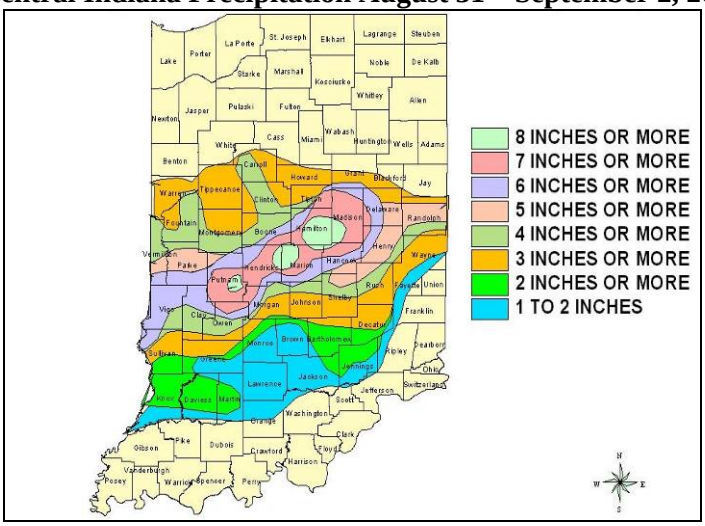
As-built data provided by Hamiliton County were reviewed and utilized in the modeling of the South Pebblebrook watershed. The data were used to develop a hydraulic model of the storm sewer system and the interconnected detention ponds within the area.

2.7 Hydrologic Data

Illinois State Water Survey Bulletin 71 rainfall depths and distributions were used to compute stormwater runoff from the 2-, 10-, 25-, 50-, and 100-year recurrence interval events. *Hydrologic Data* is explained in more detail in the Hydrologic Analysis section of this report.

During the 2003 Labor Day weekend, Hamilton County experienced a storm event exceeding the 100-yr magnitude. More than eight inches of rain fell in southeastern Hamilton County during a 2-3 day period, with most of the rain occurring on Labor Day. Figure 2-2 shows total precipitation zones during the Labor Day 2003 storm.

Figure 2-2 Central Indiana Precipitation August 31 – September 2, 2003



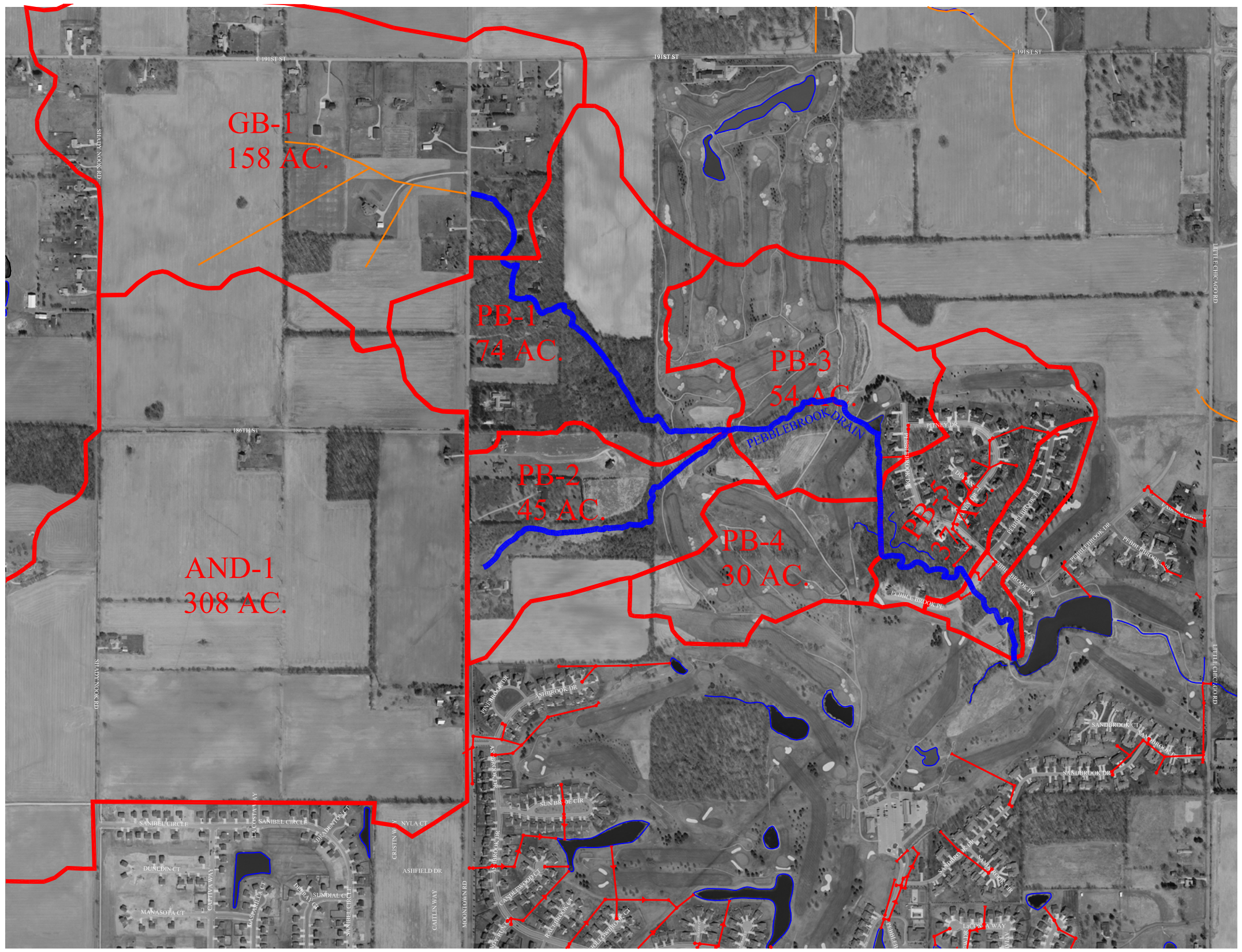
3.0 PEBBLEBROOK MAIN CHANNEL WATERSHED

3.1 Watershed Description

The Pebblebrook Main Channel Watershed is approximately 1.13 square miles. Land use includes agricultural, golf course, residential, and wooded areas. About half of the watershed is agricultural but portions are being developed. Most of the agricultural areas are in the upstream portion of the watershed. The Pebblebrook Golf Course and the Pebblebrook neighborhood represent the downstream portion of the watershed. Figure 3-1 shows the watershed in more detail.

3.2 Hydrologic/Hydraulic Analysis

Hydrologic analysis of the Pebblebrook Main Channel Watershed was performed to assist in problem identification and develop solutions and recommendations. The hydrologic computer model HEC-HMS (U. S. Army Corps of Engineers, Hydrologic Engineering Center – Hydrologic Modeling System, Version 2.2.1) was used to perform the analysis. HEC-HMS is a computer model capable of simulating runoff from various land uses and soil types, combining subbasin hydrographs, and routing flow through storage and conveyance facilities. Flows from the HEC-HMS model were used as inputs to the hydraulic analyses of the problem areas. The following sections describe the model development, evaluation results, and conclusions.



SCALE: NTS

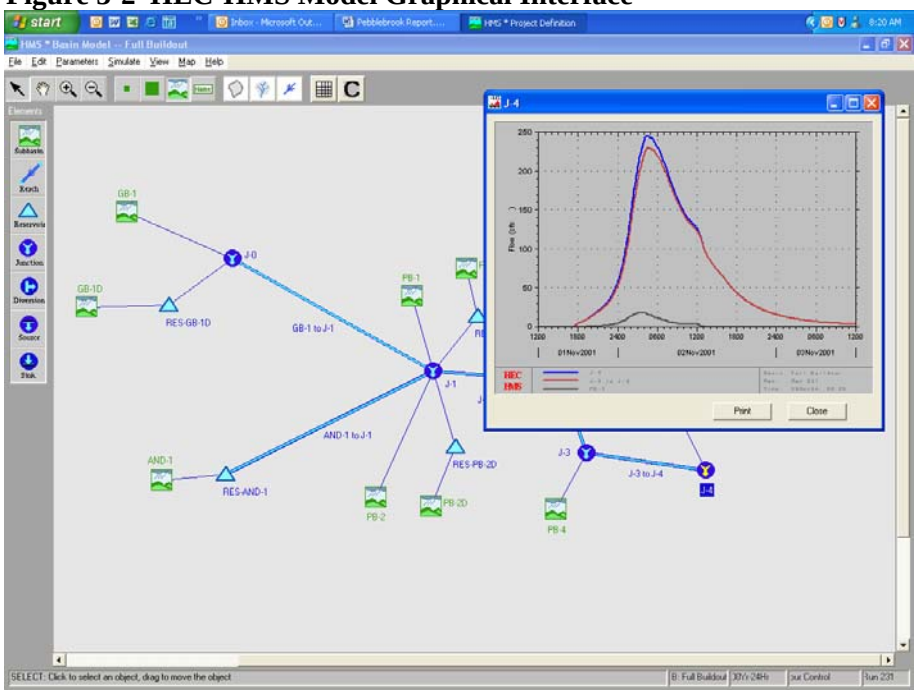
Figure 3-1
Pebblebrook Main Channel Watershed

Clark Dietz

HEC-HMS Model Development

HEC-HMS model development requires delineation of subbasins within the watershed, determining land use and runoff characteristics, and determining how subbasins are combined and routed downstream. The remainder of the HEC-HMS model input is divided into a series of operations. Each operation computes land surface runoff from a subbasin, combines two or more hydrographs, or performs flood routing through a channel reach or reservoir. Each operation produces a flow hydrograph as its output. Hydrographs can be added together (combined) to represent the confluence of two streams. The model graphical user interface and example of results are shown in Figure 3-2.

Figure 3-2 HEC-HMS Model Graphical Interface



The following sections describe the design rainfall data, subbasin parameters, routing of subbasin flows, and model verification for the watershed.

Design Rainfall

The watershed analyses focused on system performance for synthetic (predetermined) rainfall events. A design storm event is defined by precipitation depth, duration, and time distribution. Precipitation depths for various storm durations were obtained from "Bulletin 71 - Rainfall Frequency Atlas of the Midwest" (Midwestern Climate Center and Illinois State Water Survey, 1992). Time distributions (called Huff curves) were used as published in the above referenced Bulletin 71. These "Huff curves" distribute rainfall over the duration of the storm. Different curves (referred to as quartiles) are used for different duration storms. Storms less than 6 hours in duration use the first quartile distribution. Storms with durations of 6 to 12 hours use the second quartile distribution. Storms with durations greater than 12 hours but less than or equal to 24 hours use the third quartile distribution. A fourth quartile distribution is also available for storm

durations greater than 24 hours; however, storms longer than 24 hours are not typically used in urban stormwater management analyses. Tables 3-1 and 3-2 list design rainfall depths and distributions.

Table 3-1 Design Rainfall Depths

Storm Duration (hours)	Rainfall Depth by Recurrence Interval (inches)					
	2-yr	5-yr	10-yr	25-yr	50-yr	100-yr
1	1.37	1.71	2.00	2.43	2.80	3.21
3	1.87	2.33	2.72	3.30	3.81	4.28
6	2.19	2.73	3.19	3.87	4.46	5.13
12	2.54	3.17	3.70	4.49	5.18	5.95
24	2.92	3.64	4.25	5.16	5.95	6.84

Table 3-2 Design Rainfall Time Distributions

Cumulative Storm Time (%)	Cumulative Storm Rain (%)			
	First Quartile*	Second Quartile	Third Quartile	Fourth Quartile
5	12	3	2	2
10	25	6	5	4
15	38	10	8	7
20	51	14	12	9
25	62	21	14	11
30	69	30	17	13
35	74	40	20	15
40	78	52	23	18
45	81	63	27	21
50	84	72	33	24
55	86	78	42	27
60	88	93	55	30
65	90	87	69	34
70	92	90	79	40
75	94	92	86	47
80	95	94	91	57
85	96	96	94	74
90	97	97	96	88
95	98	98	98	95
100	100	100	100	100

Subbasin Parameters

The Pebblebrook Main Channel Watershed was subdivided into seven subbasins using critical analysis points as subbasin break points. Subbasin delineation was performed using the 2-foot contours in the Hamilton County GIS.

Stormwater runoff from each subbasin was computed using the Natural Resource Conservation Service (NRCS) curve number method available in HEC-HMS. Required parameters include subbasin area, curve number, and basin lag time. The time of concentration for each subbasin was estimated using the NRCS TR-55 method. Calculations were based on distance, surface characteristics, slope, and velocity of flow from the most remote point in the subbasin to the subbasin outlet. The time of concentration, measured in hours, was converted to the subbasin lag time using the HEC-HMS recommended factor of 0.6.

Subbasin curve numbers were determined using a weighted average of curve numbers assigned to individual sub-areas of homogeneous land use and soil types. Existing conditions land use data was obtained from GIS maps and aerial photos. Measurements of channel widths taken during the field reconnaissance of the channel were input into the model along with slopes of the channel as estimated from the GIS mapping. The individual curve numbers for each land use and soil type were selected from tables in SCS Technical Release 55, Urban Hydrology for Small Watersheds, 1986. Subbasin locations were shown previously on in Figure 3-1. Subbasin parameters are summarized in Table 3-3.

Table 3-3 Existing Subbasin Hydrologic Parameters

Subbasin	Area (sq. mi.)	Time of Concentration (min)	Lag Time (min)	Curve Number
AND-1	0.48	85	51	81
GB-1	0.25	74	44	79
PB-1	0.12	84	50	62
PB-2	0.07	60	36	58
PB-3	0.08	62	37	64
PB-4	0.05	57	34	62
PB-5	0.06	40	24	66

Routings

A key feature of the HEC-HMS model is its capability to route stormwater runoff hydrographs through various drainage system components such as detention basins, culverts, and channel reaches. Appropriate flow routings enhance the accuracy of the representation of the watershed response to storm events by incorporating the attenuation of peak flows and time delay of hydrographs which occur as a flood wave travels through the watershed. Both detention pond storage and channel routings were utilized in the Pebblebrook Main Channel watershed HEC-HMS model.

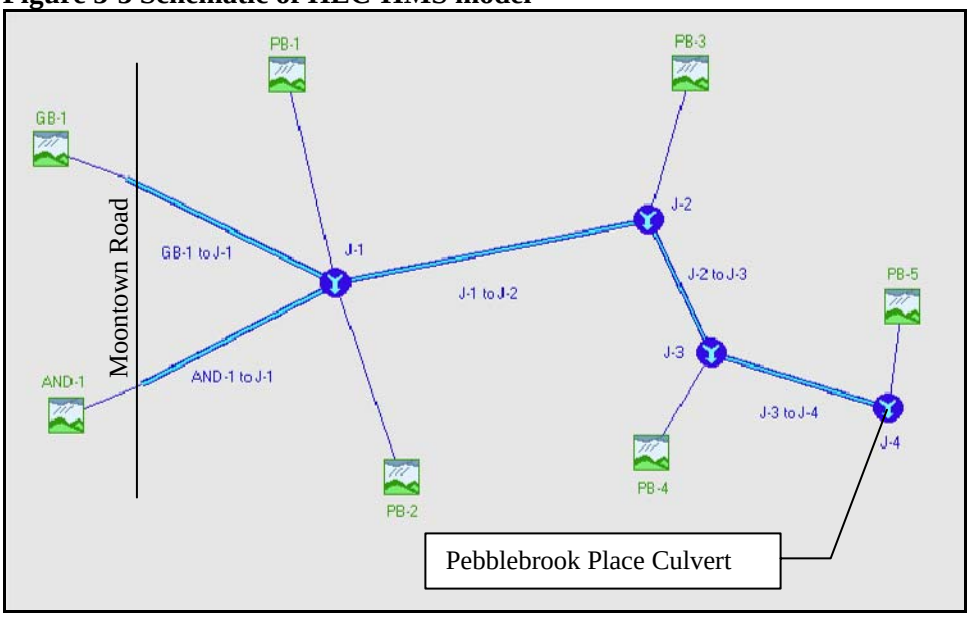
Modeled Scenarios

A HEC-HMS model was developed for the following scenarios:

- Existing
- Existing with Andover development
- Full buildout of Pebblebrook Main Channel Watershed

The full buildout scenario considers the watershed when all buildable areas have been developed (per current zoning). Each scenario was executed in HEC-HMS and peak flows throughout the watershed were determined. The following figure is a HEC-HMS schematic of the watershed.

Figure 3-3 Schematic of HEC-HMS model



Junction J-4, shown in Figure 3-4, is located at the triple 30-inch culverts at Pebblebrook Place. Flows at this junction pass through the culverts or over the roadway at Pebblebrook Place. Table 3-4 shows a summary of results for each modeled scenario at different locations in the watershed.

Table 3-4 Summary of Flows from HEC-HMS Model

Location	Existing (cfs)			with Andover (cfs)			Full Buildout (cfs)		
	2-yr	10-yr	100-yr	2-yr	10-yr	100-yr	2-yr	10-yr	100-yr
AND-1	76	165	379	33	57	98	33	57	98
J-1	112	258	628	60	117	291	59	108	204
J-3	114	270	673	65	131	337	64	121	241
J-4	116	276	695	68	138	357	66	127	260

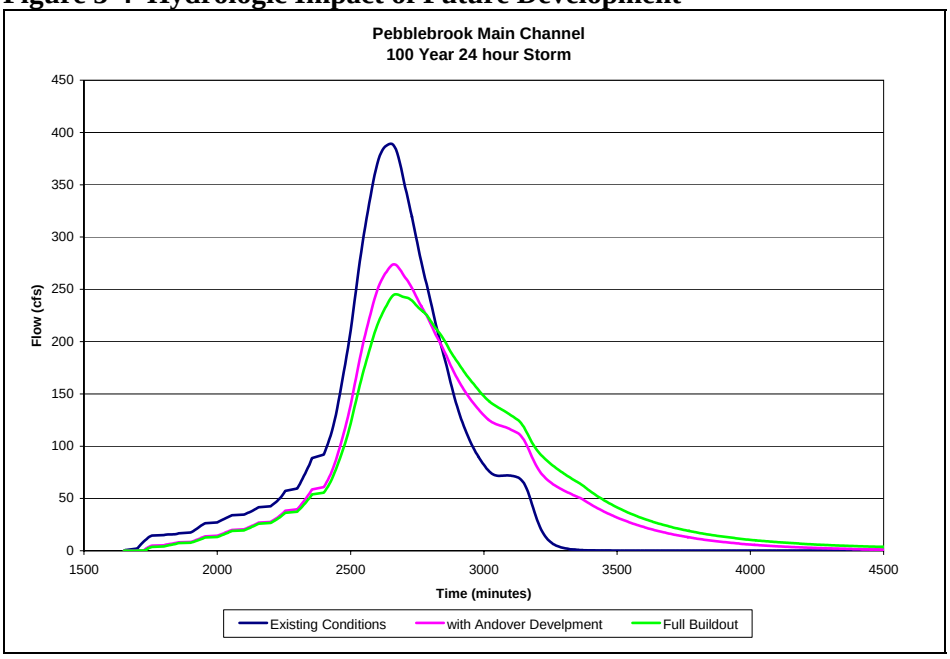
3.3 Future Development: Stormwater Detention Policy Evaluation

Current detention standards require control of 100-year and 10-year storms. For a given site, the 100-year post-development peak rate of runoff must be restricted to the 10-year pre-development peak rate. The 10-year post-development flow must be restricted to the 2-year pre-development peak rate.

The effectiveness of this policy was evaluated by using future land use runoff curve numbers in undeveloped or partially developed subbasins. Hypothetical detention facilities were modeled at the downstream end of these subbasins to represent current detention requirements (control of the 100-year and 10-year post-development flows to 10-year and 2-year pre-development rates, respectively).

The results of this analysis are illustrated in Figure 3-5, which compares the entire watershed under existing conditions (blue), existing conditions with the Andover Development (magenta), and the "full build-out" conditions with current detention standards (green). The flow vs. time graphs (runoff hydrographs) represent the 100-year 24-hour storm flow.

Figure 3-4 Hydrologic Impact of Future Development



The hydrologic analysis shows that current detention standards will be effective in controlling peak flow rates and corresponding flood elevations. However, these hydrographs also illustrate the impact of urbanization on the *volume* and *duration* of stormwater runoff. Under developed conditions, peak flow is reduced but it takes longer for flows to recede.

Urbanization can alter the geometry and stability of stream channels. Larger and more frequent discharges that accompany watershed development cause downstream channels to enlarge, by widening, down cutting, or a combination of both.

3.4 Andover Development Evaluation

As part of the study, the proposed Andover stormwater detention ponds were reviewed. The *Andover Overall Drainage Report* submitted to Hamilton County was reviewed in order to compare conclusions of the report to the HEC-HMS model and to evaluate the effectiveness of the proposed on-site detention at Andover. The review of the report focused mainly on the eastern portion of Andover that drains to the Pebblebrook Main Channel.

Table 3-5 shows the pre-developed discharge at Moontown Road given in the *Andover Overall Drainage Report* and the calculated discharge from the HEC-HMS model.

Table 3-5 Pre-developed Flow Calculations for Andover Basin

Storm Event	Andover Report	Pebblebrook Study (HEC-HMS)
2-year	108 cfs	76 cfs
10-year	208 cfs	165 cfs

The difference in the pre-developed discharge is a likely due to differences in rainfall depth and distribution sources. The Andover report used the NRCS Type II 24-hour rainfall distribution. This study used the Huff distribution third quartile. The NRCS Type II distribution tends to yield higher peak flows than the Huff distribution. The second difference is the rainfall depths used. The Andover report referenced National Weather Service (NWS) Technical Paper (TP-40) and this report used Bulletin 71 rainfall depths. TP-40 is an older set of rainfall data but is the most current 24-hour rainfall data published by the NWS. Bulletin 71 rainfall data are newer and generally more widely recognized in Midwestern states. For a 100-year 24-hour duration rainfall, Bulletin 71 lists 6.8 inches of rain in central Indiana. TP-40 lists 5.5 inches for the same event in central Indiana.

Post-developed runoff rates were calculated using a composite CN value taken from the Andover report. A single detention pond was used in the HEC-HMS model to simulate the 9 ponds proposed in the Andover development. Although modeling one pond will not simulate the dynamics of the 9 ponds in Andover, it provides an overall verification of the required storage volume and peak discharge rates.

During our HEC-HMS modeling of the Andover development, it was found that the *Andover Overall Drainage Report* had omitted several offsite drainage areas that would impact the proposed Andover detention system. This issue was brought to the attention of Stoeppelwerth & Associates and revisions were made to the calculations originally submitted to Hamilton County. Table 3-6 lists post-development flows from the original *Andover Overall Drainage Report* , the modified discharges given to Clark Dietz (on 11/29/2004), and HEC-HMS post-development discharges for the site.

Table 3-6 Post-developed Flow Calculations for Andover Basin

Storm Event	Original Andover Report	Modified Andover Report	Pebblebrook Study (HEC-HMS)
10-year	31 cfs	66 cfs	57 cfs
100-year	36 cfs	87 cfs	98 cfs

The modified Andover discharges are higher but are still below the County's detention standard. During the 100-year event the Andover Ponds would discharge to the Pebblebrook Southern Branch at a peak of 98 cfs, which is less than the pre-developed 10-year discharge rate of 165 cfs as calculated by this study. During the 10-year event the Andover Ponds would discharge to the Pebblebrook Southern Branch at a peak of 57 cfs, which is less than the pre-developed 2-year discharge rate of 76 cfs as calculated by this study.

The resulting release rates per acre for the Andover basin are 0.32 cfs/acre for the 100-year storm event and 0.19 cfs/acre for the 10-year storm. Similar, or more restrictive, release rates should be used for other undeveloped portions of the watershed (primarily the area drained by the George Booth drain).

3.5 Problem Identification

Pebblebrook Place Culverts

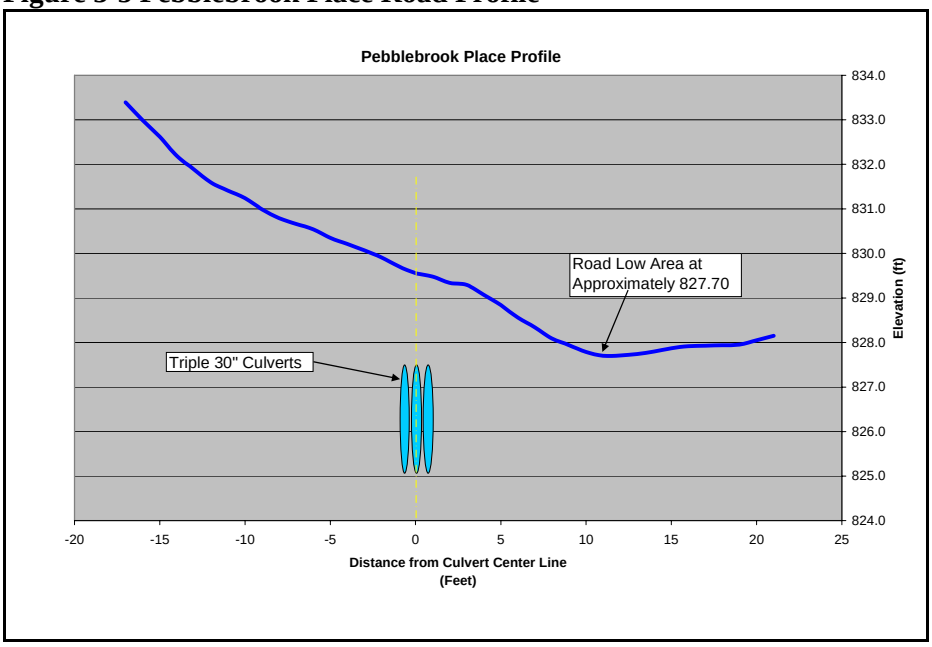
The existing hydraulic capacity of the triple 30-inch culvert crossing under Pebblebrook Place was analyzed using HY8, a computer model produced by the Federal Highway Administration (FHWA) to calculate culvert hydraulic capacity. The current capacity of the triple 30-inch culverts at Pebblebrook Place is approximately 80 cfs. The existing conditions 2-year flow is 116 cfs (see table 3-4). Hence, the triple 30-inch culverts have less than a 2-year capacity. Photographs below show Pebblebrook Main Channel overtopping the roadway during a recent storm event. The photographs were taken on January 5th, 2005.

Looking southwest at Pebblebrook Place during January 5th, 2005 storm event

Looking northeast at Pebblebrook Place during January 5th, 2005 storm event

As seen above in the photographs, another problem is the profile of the roadway. Water overtops the road east of the culverts since the elevation of the roadway is considerably lower. The low spot in the roadway is approximately at the same elevation as the crown of the triple 30-inch culverts and allows very little surcharging of the culverts prior to roadway overtopping. A larger culvert replacing the existing at the same inverts would increase the crest elevation of the pipe. Figure 3-5 shows the roadway profile at the culvert crossing of Pebblebrook Place. The culverts are centered at the zero-foot mark on the horizontal axis with an approximate invert of 825.1 feet.

Figure 3-5 Pebblebrook Place Road Profile



Due to construction of upstream detention ponds, peak flows will decrease as the watershed is developed with the County's detention policy. However, even with the reduction in flow at the full buildout with detention scenario, the triple 30-inch culverts would still have less than 10-year capacity. The triple culvert barrel configuration is also prone to clogging with debris, which further reduces capacity.

<u>Moontown Road Improvements</u>

As indicated in Section 2.3, problems where identified at the southern branch of Pebblebrook Main Channel where it crosses Moontown Road. In this area there is significant channel sedimentation partially blocking the 36-inch culvert crossing at Moontown Road. There have been numerous reports of runoff overtopping the road at this location. The primary causes of the overtopping are:

- Sedimentation at the downstream end of the culvert
- Undersized culvert (inadequate hydraulic capacity)

The drainage area tributary to this culvert is from the Andover subbasin. Hydraulic calculations of the culvert using HY8 modeling software show that the existing 36-inch culvert has a capacity of approximately 40 cfs. Flow calculations using HEC-HMS (existing conditions) show 379 cfs for a 100-year storm event at the culvert (Moontown Road). The flow will be greatly reduced due to detention facilities once the Andover Development is complete. Upon full development of Andover, the 100-year peak flows will decrease to 98 cfs, which is still more than twice the capacity of the 36-inch culvert. Therefore, the culvert should be replaced.

3.6 Solutions

Proposed solutions were developed for the Pebblebrook Place and Moontown Road culvert crossings. Cost estimates were developed for each solution and include a construction cost estimate, a construction contingency of 15%, and non-construction costs of 20% (for design, construction engineering and inspection, and administration). Land acquisitions costs are not included. The following sections present the solutions for these two problem areas.

Pebblebrook Place Culverts

The triple 30-inch culverts at Pebblebrook Place require replacement, as they are undersized and prone to clogging. A key factor in sizing a replacement culvert is to select a design flow. Typically, the County would like to see culverts along regulated drains with significant drainage areas to have the capacity to handle the 100-year storm. At Pebblebrook Place, the existing conditions 100-year flow is 695 cfs. Because of the topography at this culvert crossing, a large clear span bridge would be required to provide this level of capacity. This would not be cost-effective given the limited amount of traffic on this residential street.

Increasing the roadway elevation to minimize flooding is ***not recommended*** due to the proximity of residences at 1098 and 1094 Pebblebrook Drive. As shown in Figure 3-5 the Pebblebrook Place Road profile has an overtopping elevation of 827.70 feet, which is approximately the same elevation as the crown of the existing culverts. Increasing the roadway elevation would increase the headwater and risk flooding of upstream residences. The two referenced properties (1098 and 1094 Pebblebrook Drive) have water entry elevations of 830 +/- feet. Therefore, solutions at this location ***should not*** include raising the existing pavement (overtopping) elevation.

As mentioned previously, as this watershed develops, the new upstream detention facilities will significantly reduce flows at Pebblebrook Place. For example, once the Andover development is constructed, the 100-year flow at Pebblebrook Place will drop to 357 cfs. Under the full buildout of the watershed with detention scenario, the 100-year flow will drop to 260 cfs.

Box culverts were analyzed using the HY8 program to determine the culvert size necessary for flows of 260 cfs and 357 cfs. The results of this analysis are presented in Table 3-7. The analysis showed that a 12-foot wide by 4-foot high concrete box culvert would be required to pass 260 cfs without roadway overtopping. To safely pass 357 cfs, a 17-foot wide by 4-foot high concrete box culvert would be required. The preliminary opinion of total project cost for the 12-foot by 4-foot and the 17-foot by 4-foot box culverts are $132,000 and $182,000, respectively (see Appendix A for cost details).

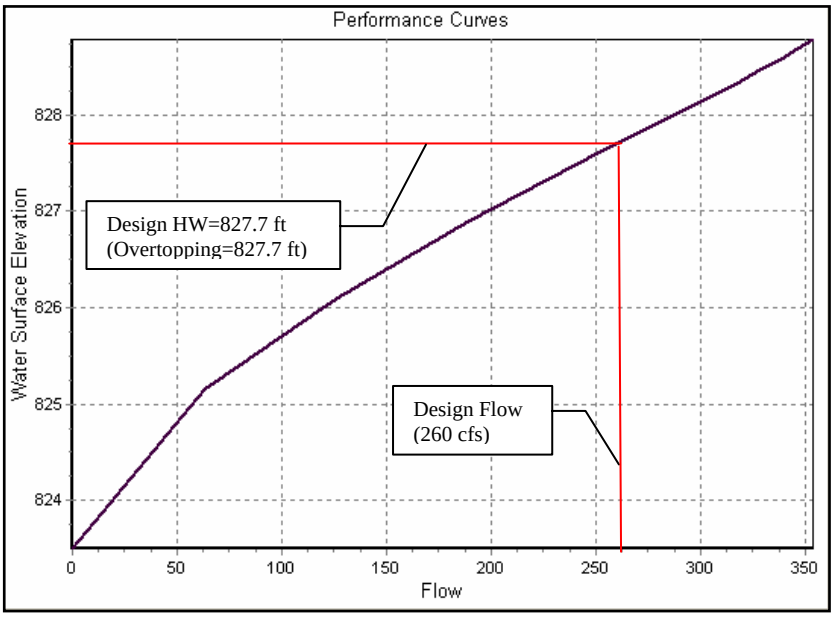
Table 3-7 Proposed Culvert Overtopping Evaluation

Design Flow	12' x 4' Box Culvert		17' x 4' Box Culvert	
	Headwater Elevation (ft)	Overtopping Depth (ft)	Headwater Elevation (ft)	Overtopping Depth (ft)
260 cfs (full development)	827.7	0	827.0	-0.7
357 cfs (with Andover)	828.2	0.50	827.6	-0.1
695 cfs (existing)	828.8	1.1	828.7	1.0

Until the upstream watershed is fully developed with detention, the 12-foot by 4-foot box culvert will provide less than 100-year capacity. Under existing conditions, the new culvert will nearly pass a 10-year storm (276 cfs vs. 260 cfs). After the Andover development is in place, the culvert will nearly pass a 50-year storm (271 cfs vs. 260 cfs). Hence, this culvert will significantly improve performance under current conditions, and ultimately, will provide a 100-year level of protection. The 12-foot by 4-foot box culvert will be much less prone to the debris clogging problems that the existing triple 30-inch culverts experience.

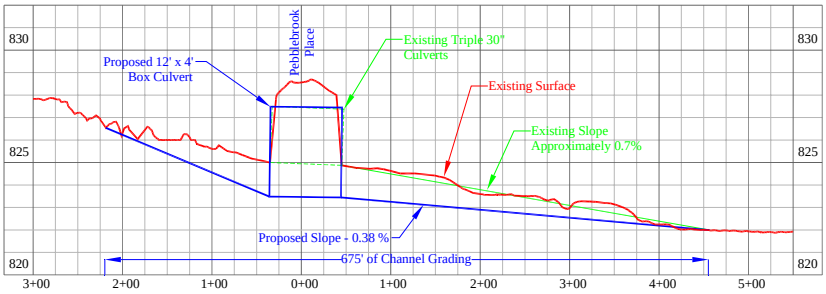
The 17-foot by 4-foot box culvert that was analyzed would provide additional protection until the watershed is fully developed. However, the additional cost of approximately $50,000 (over that of a 12-foot by 4-foot culvert) does not appear to be warranted given the relatively low overtopping depth (0.5 feet with Andover). Neither culvert size would be capable of passing the existing conditions 100-year flow of 695 cfs. Overtopping depths would be approximately 1.0 feet for both culvert sizes. Figure 3-6 illustrates the rating curve from HY8 showing the capacity of the proposed 12-foot by 4-foot culvert.

Figure 3-6 HY8 performance curve of the recommended 12-foot by 4-foot culvert



The proposed culvert has a higher rise than the existing 30-inch culverts and would need to be approximately 1.5 feet lower to fit under the roadway. To accomplish this, the channel would need to be re-graded and lowered upstream and downstream of the culvert. This is possible, as the slopes of the stream in the vicinity of Pebblebrook Place are fairly steep. Figure 3-7 illustrates the impact of the proposed culvert on the channel profile. The installation of the culvert, as proposed, should not require any significant changes to the Pebblebrook Place roadway profile.

Figure 3-7 Profile of the Pebblebrook Main Channel at Pebblebrook Place



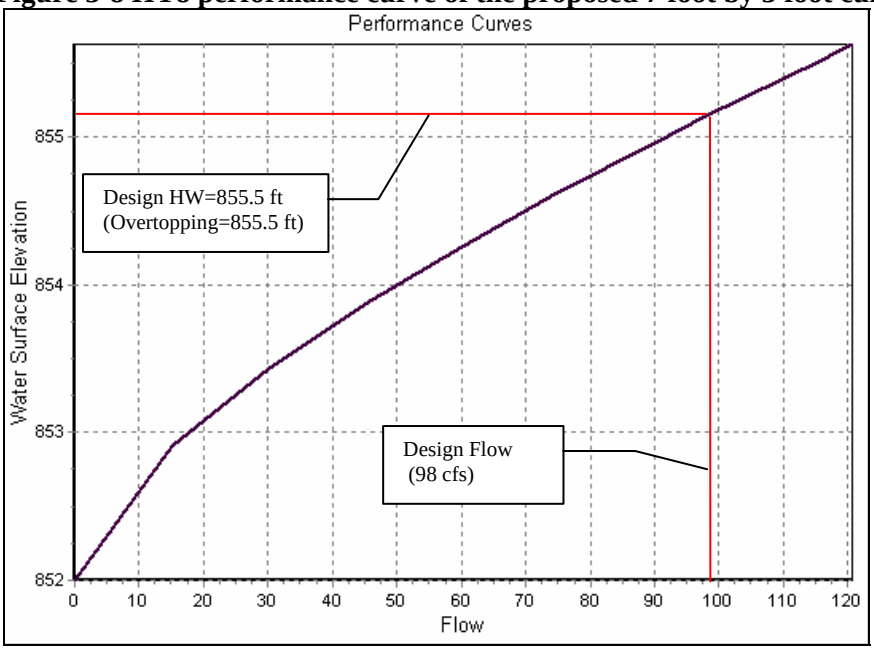
As shown in the profile, 675 feet of channel grading would be required. The channel grading allows the 12-foot by 4-foot concrete box culvert to be set low enough to allow adequate cover for the culvert wall thickness and pavement cross section.

Appendix A shows a detailed total construction cost opinion for the improvements at Pebblebrook Place. The preliminary opinion of total project cost for this improvement is $132,000.

<u>Moontown Road Improvements</u>

The existing 36-inch culvert at Moontown Road needs to be replaced. The 100-year flow (as computed by the HEC-HMS model) was selected for design of this culvert. Given Andover is currently under construction, the 100-year flow with Andover detention in place was used as the basis for design. This flow is 98 cfs. Using the HY8 program, a 7-foot by 3-foot concrete culvert was found to provide adequate capacity to pass 98 cfs without overtopping. Figure 3-8 is a rating curve from HY8 showing the capacity of the proposed culvert.

Figure 3-8 HY8 performance curve of the proposed 7 foot by 3 foot culvert



In addition to the replacement of the culvert at Moontown Road, downstream channel improvements would be required to utilize the available capacity of the proposed culvert. Approximately 1850 feet of channel grading/cleanout would need to me preformed downstream of Moontown Road to the Pebblebrook Golf Course. This channel improvement would eliminate adverse tailwater conditions for the new culvert and provide improved drainage in this channel reach, which is currently silted in and overgrown with vegetation. The channel should be 5-foot wide with 3:1 side slopes with a minimum depth of 2 feet.

Appendix A shows a detailed total construction cost opinion for the improvements at Moontown Road. The preliminary opinion of total project cost for this improvement is $186,000.

3.5 Summary – Pebblebrook Main Channel

It is recommended that the improvements listed below be performed in order to eliminate safety concerns due to flooding, maintenance problems, and structural problems associated with water overtopping the roadway. These improvements include:

<u>Capitol Improvements</u>

- Pebblebrook Place Improvements
  - ⇒ New 12-foot by 4-foot concrete box culvert
  - ⇒ Approximately 675 feet of channel regrading
  - ⇒ $132,000
- Moontown Road Improvements
  - ⇒ New 7-foot by 3-foot concrete box culvert
  - ⇒ Approximately 1850 feet of channel regrading
  - ⇒ $186,000

<u>Other Recommendations</u>

- Future watershed developments should have detention policy release rates at least as restrictive as those provided in the Andover Development, which are as follows:
  - 0.32 cfs/acre for a 100-year storm event
  - 0.19 cfs/acre for a 10-year storm event
- The Andover development should re-submit the drainage report for detention basin sizing using rainfall depths and Huff time distribution listed in the following publication:
  - "Bulletin 71 - Rainfall Frequency Atlas of the Midwest" (Midwestern Climate Center and Illinois State Water Survey, 1992).
- The Pebblebrook Main Channel should be periodically inspected in areas where minor blockages and bank erosion where noted in the field reconnaissance.

4.0 PEBBLEBROOK SOUTH WATERSHED

4.1 Watershed Description

The Pebblebrook South Watershed drains approximately 0.5 square miles of mostly residential land use. The remainder of the watershed consists of agricultural and recreational (Pebblebrook Golf Course) land use. The watershed contains 11 detention ponds, some of them interconnected, that drain to the Pebblebrook South Channel. Figure 4-1 on the following page illustrates the watershed subbasin boundaries, the layout and connectivity of the existing detention ponds, and a pond numbering system used in this report. All ponds eventually drain into Pond 3 which drains into Pebblebrook South Channel. Pond 10 at the Ashfield Development and Pond 1 flows east into Pond 2. Pond 2 flows east into Pond 3. Pond 11 discharges to a storm sewer on Pebblebrook Boulevard. that flows to Pond 3. Ponds 5, 6, 7, 8, and 9 drain into Pond 4 which drains into Pond 3.

Pond 2 looking North

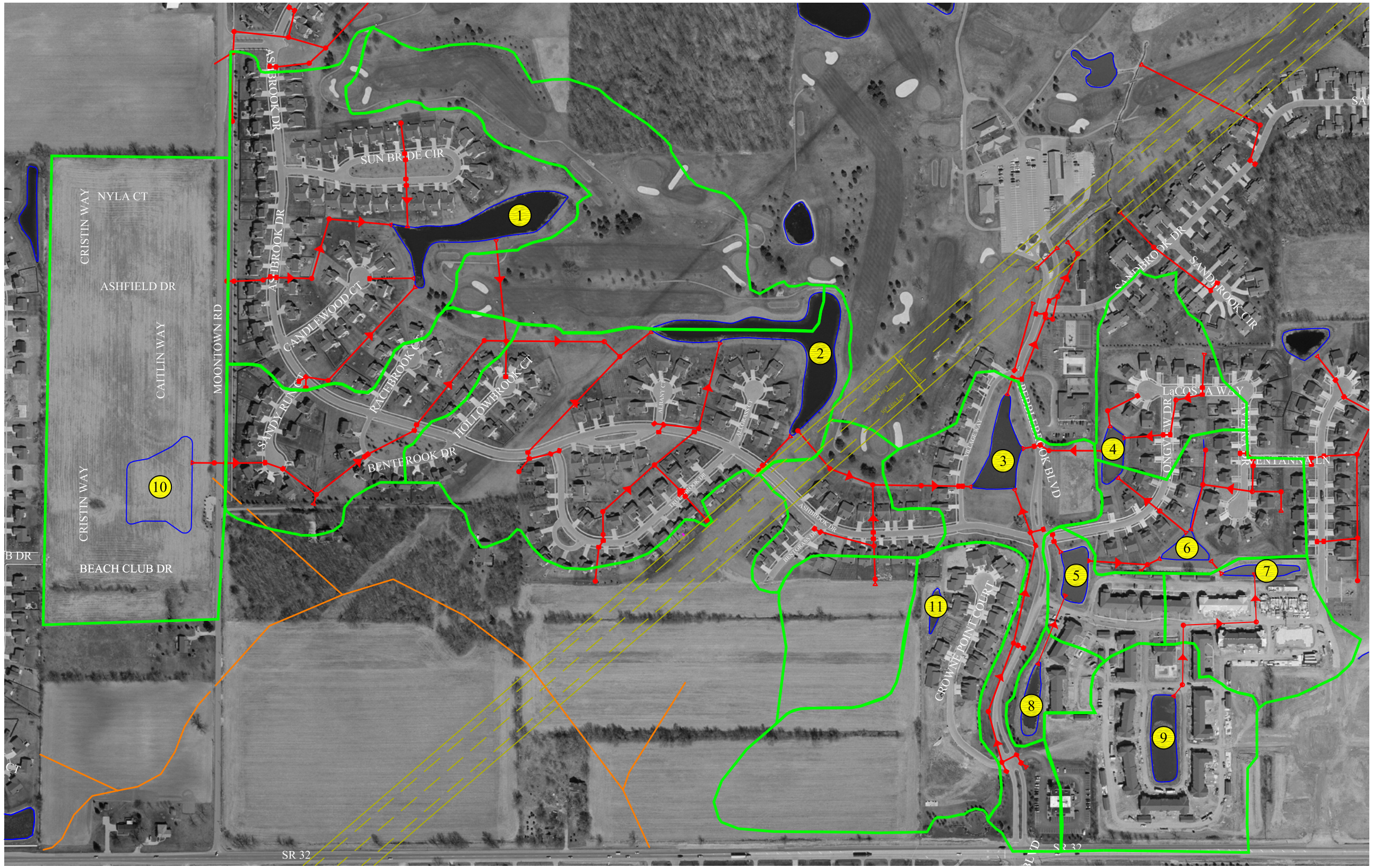
Pond 3 looking North

4.2 Problem Identification

The Labor Day 2003 rainfall dropped more than 7 inches of rain in the vicinity of the Pebblebrook Neighborhood, causing ponds to overtop. Ponds 2 and 3 have been identified by residents to have the most severe problems. When Pond 2 overtops its banks, the overflow is directed behind homes on the north side of Ashbrook Drive and south of the pond to a low area on Ashbrook. This occurred during Labor Day 2003 and has been reported to come close to overtopping during other major storm events. The following pictures were obtained from a resident who experienced flood damage during the Labor Day 2003 storm.

Flooding of Ashbrook Drive during the Labor Day 2003 storm event.

Flooding along a home on the north side of Ashbrook Drive during the Labor Day 2003 storm event.



SCALE: NTS

LEGEND

- # Pond Number
- Flow Arrow
- Basin Boundary

Figure 4-1
Pebblebrook South Watershed

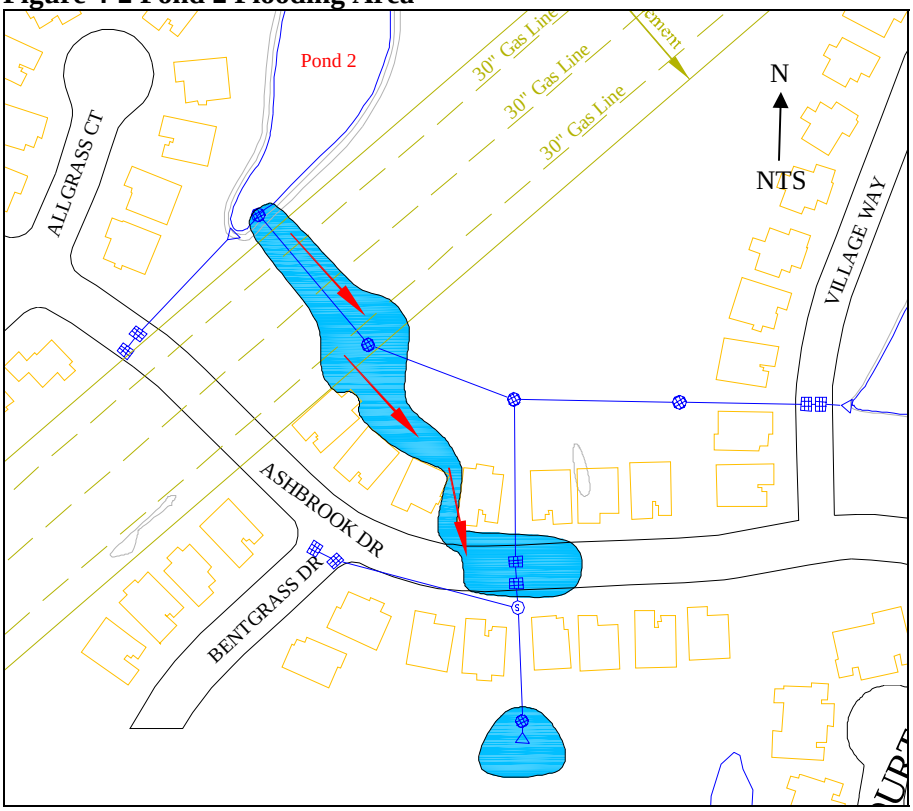
Clark Dietz

Flooding along a home on the north side of Ashbrook Drive during the Labor Day 2003 storm event.

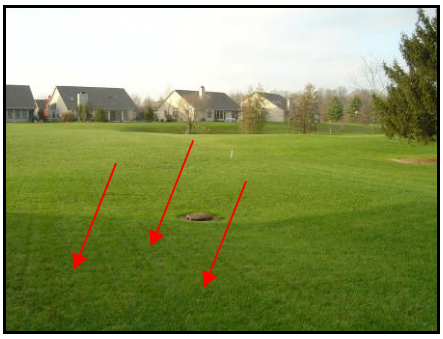
Flooding outside of a home during the Labor Day 2003 storm event.

The path of the overtopping water is adjacent to and on the north side of single-family residential structures on Ashbrook Drive. Flooding entered these homes during this storm event when Pond 2 overtopped and breached these homes, causing flood damage. Figure 4-2 shows a plan view of the approximate area where the water flows (indicated with red arrows) when Pond 2 overtops.

Figure 4-2 Pond 2 Flooding Area



The following pictures show the path the water would take when overtopping out of Pond 2.



Looking towards Pond 2 where the overtopping occurs

Looking towards homes on the north side of Ashbrook where flooding occurs

4.3 Hydrologic/Hydraulic Analysis

<u>XP-SWMM Model Development</u>

XP-SWMM2000 (Version 9.0), produced by XP Software, was used to analyze existing detention facilities in the Pebblebrook South Watershed. XP-SWMM is a dynamic (unsteady) flow model that performs both hydrologic and hydraulic analyses and can more accurately account for unsteady flow conditions associated with detention facilities. The model is based on the EPA Stormwater Management Model (SWMM), which has been in continuous use since approximately 1970. XP-SWMM offers a graphical user interface and detailed model output.

XP-SWMM2000 was used on the Pebblebrook South Watershed to simulate and evaluate the existing detention facilities in the Pebblebrook neighborhood. Figure 4-3 illustrates the XP-SWMM interface for the detention facility modeling. In this XP-SWMM analysis, hydrographs are generated using the same hydrologic methodology utilized by the HEC-HMS model. The runoff hydrographs are routed through links and nodes that represent the storm sewers and ponds in the Pebblebrook neighborhood. Storm sewers and ponds where modeled based on as-built drawings, GIS information, and site visits. The following table lists existing pond storage volumes, 100-year design water surface elevations (from as-built plans), and approximate pond bank elevations.

Figure 4-3 XP-SWMM schematic from model

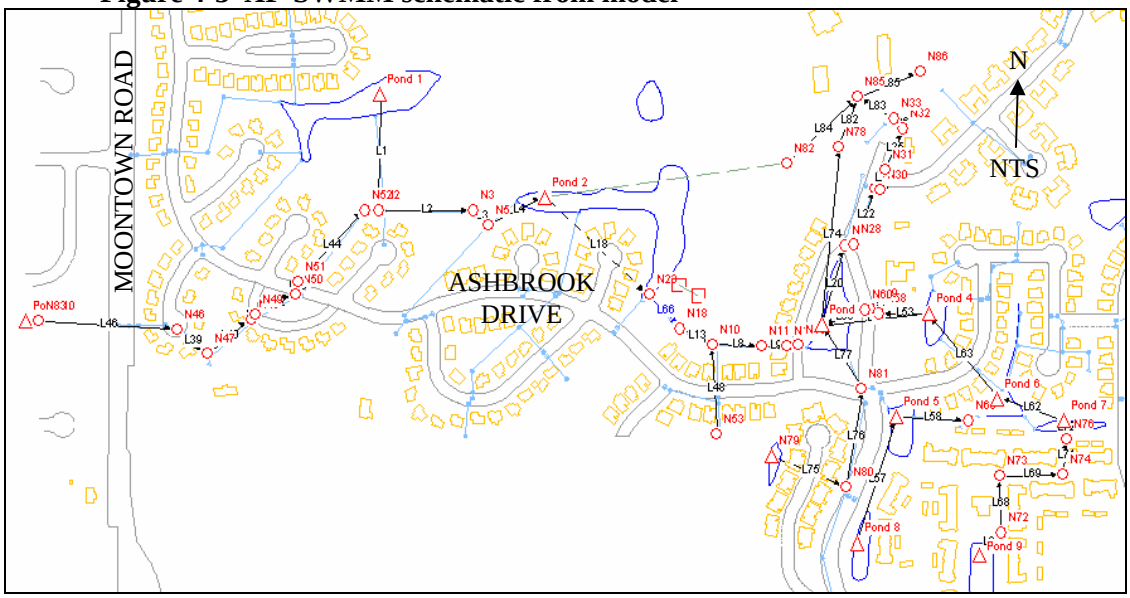


Table 4-1 Existing Pond Parameters

Pond ID	Available StorageVolume (Acre-Feet)	100-Year Design Peak Water Surface Elevation (feet)
1	6.9	854.54
2	10.0	843.85
3	2.2	834.77
4	1.1	837.20
5	1.2	837.34
6	1.9	837.33
7	0.8	837.33
8	1.2	836.93
9	2.8	837.37
10	11.6	855.14
11	1.7	NA*

*As-built data not available for this area

The watershed was delineated into 15 subbasins (PB-1 through PB-13) and Ashfield (existing development west of Pebblebrook). Land use and runoff characteristics were determined for each subbasin within the watershed using the same methodology described in Section 3.2. Figure 4-3 illustrates the basin delineation with respect to the ponds and storm sewers. Table 4-2 lists the basin characteristics used in the XP-SWMM model.

Figure 4-4 Schematic showing the basin delineation

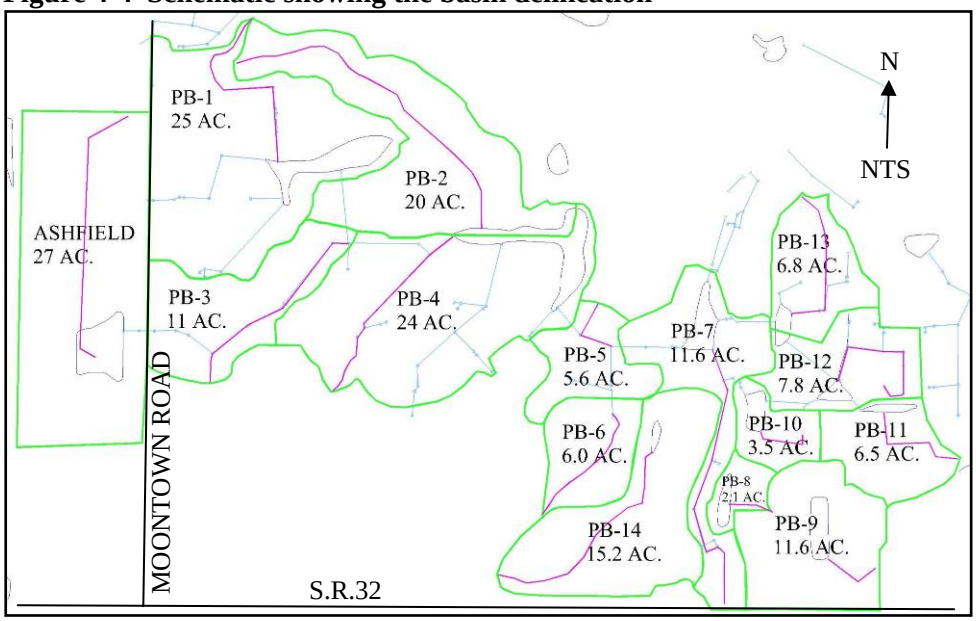


Table 4-2 Subbasin Hydrologic Parameters

Subbasin ID	Area (sq. mi.)	Time of Concentration (min)	Lag Time (min)	Curve Number
Ashfield	0.042	39	23	79
PB-1	0.039	43	26	80
PB-2	0.031	79	48	69
PB-3	0.017	24	14	78
PB-4	0.038	27	16	79
PB-5	0.007	23	14	76
PB-6	0.009	44	27	79
PB-7	0.009	35	21	78
PB-8	0.018	3	2	91
PB-9	0.013	18	11	90
PB-10	0.005	12	7	89
PB-11	0.010	16	9	89
PB-12	0.012	19	11	82
PB-13	0.011	28	17	78
PB-14	0.023	62	37	76

Once basins were delineated and hydrologic characteristics were determined, a model of the existing Pebblebrook neighborhood was created. First, the pipe system was input, which included invert elevations, pipe diameters, roughness coefficients, ground elevations, and outfall types. Storage nodes were used where detention facilities existed. Subbasin runoff characteristics and design rainfall events were assigned to runoff nodes using the same hydrology methods as explained in Section 3.2.

The 100-year storm was used as a benchmark event to which proposed improvements should be designed. Table 4-3 compares the high water elevations

for the 100-year (design) storm event from the as-built drawings and the 100-year storm event calculated by XP-SWMM model of the existing system.

Table 4-3 Existing Pond Parameters

Pond ID	100-Year Design Elevation (feet)	100-Year XP-SWMM Model Elevation (feet)	Difference XP-SWMM vs. Design HW (feet)
1	854.54	854.44	-0.10
2	843.85	845.27	1.42
3	834.77	834.84	0.07
4	837.20	836.63	-0.57
5	837.34	836.83	-0.51
6	837.33	836.83	-0.50
7	837.33	836.89	-0.44
8	836.93	836.82	-0.11
9	837.37	837.80	0.43
10	855.14	855.65	0.51
11	NA*	839.83	----

*As-built data not provided for this area

As shown in Table 4-3 the XP-SWMM model generally predicts similar 100-year peak water surface elevations, as compared to the design elevations. However, at Pond 2, the XP-SWMM model predicts a 100-year peak water surface elevation 1.42 feet higher than the design elevation. This is consistent with the observed flooding during the Labor Day 2003 event.

Several other ponds were also computed to be higher than the design high water elevation, though the difference was generally less than 0.5 feet. Pond 3 would have spilled over the roadway, downstream to the entrance drive to Pebblebrook Golf Course (also reported during the Labor Day 2003 event). This pond was also found to be overflowing during wet weather in early January 2005. Photos of the pond during this recent flood event are shown below. Pond 9 and 10 were higher than the design highwater, through no flooding problems were reported during the Labor Day 2003 event. Figures 4-5 and 4-6 show hydrographs for Ponds 2 and 3.

Looking northwest towards Pond 3 where the overtopping occurred on January 5th, 2005

Looking north towards Pond 3 where the overtopping occurred on January 5th, 2005

Figure 4-5 Pond 2 Water Surface Elevations: 100-year Storm Event

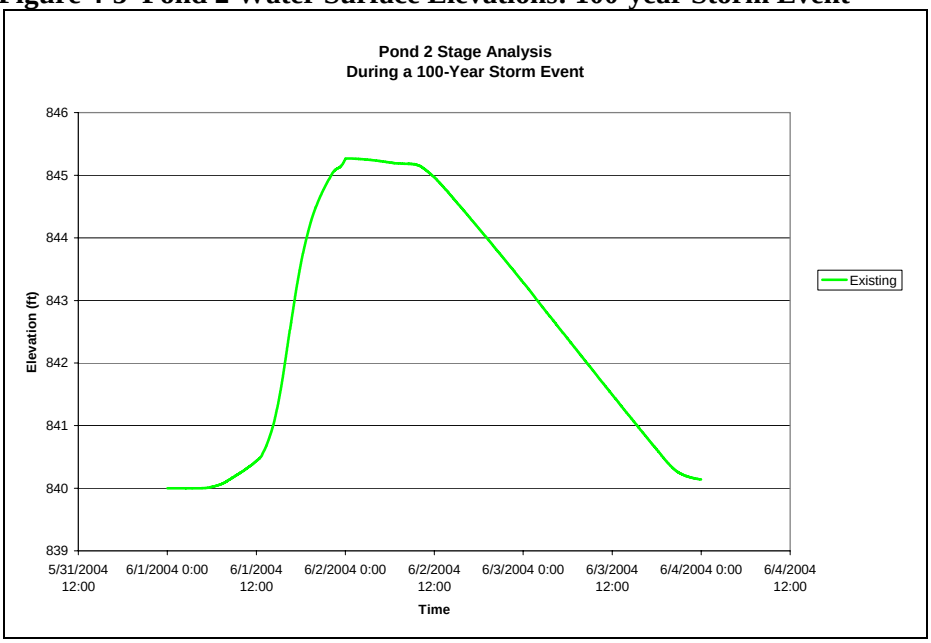
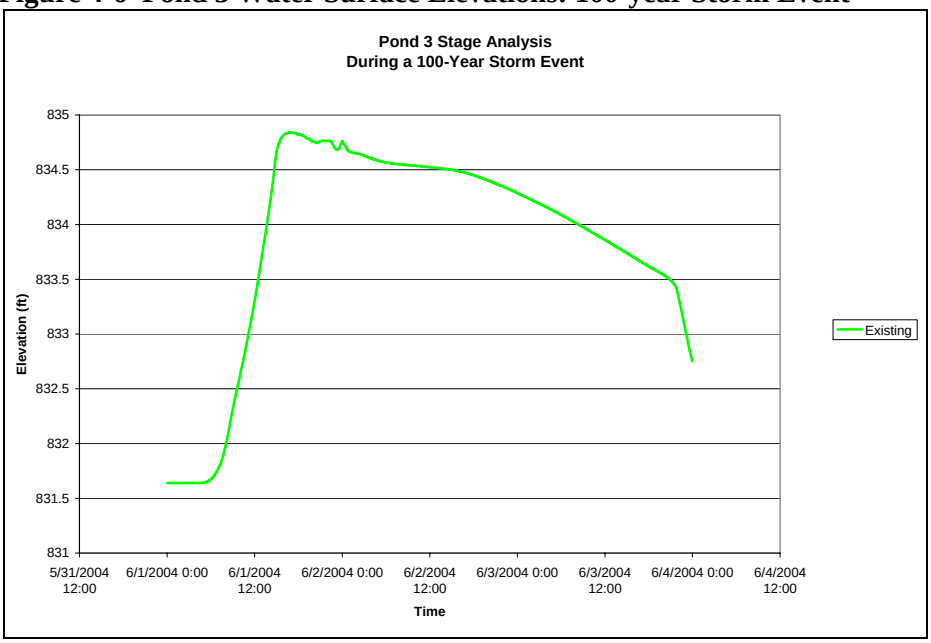


Figure 4-6 Pond 3 Water Surface Elevations: 100-year Storm Event



Rainfall data from a rain gauge in northern Marion County were used to create a rainfall hydrograph of the Labor Day 2003 storm. The rainfall hyetograph was input into the model to confirm reports during the actual storm. Ponds 2 and 3 both overflowed as well as other ponds in the system. Since the Labor Day 2003 storm was larger than a 100-year storm it was not used to design improvements in this report. Detention facilities are typically designed for 100-year storm events. However larger storm events should have a safe overflow system that would prevent flooding to homes.

4.4 Alternative Solutions

Alternative solutions to the flooding problems at Ponds 2 and 3 were evaluated. Pond 2 flooding was considered more severe than Pond 3 flooding because of the potential for Pond 2 overflow to cause property damage. While Pond 3 overtops more frequently, it does not damage homes because its overflow path is safely directed to a ditch to the north and eventually to the Pebblebrook South Channel. However, the overtopping of Pond 3 would still not be desirable since the water overtopping the roadway can cause a safety hazard to motorists and can damage the roadway. Overtopping of Pond 3 also creates an unsafe condition in that the edge of the pond, where the water normally becomes deeper, is obscured by the shallow flooding that occurs when it overtops.

Pond 3 is the lowest pond in the Pebblebrook South Watershed. As mentioned previously, the other 10 ponds in this watershed all eventually drain through Pond 3. This results in a large volume of runoff being routed through the pond, which is why it fills up and floods more frequently (events greater than a 2-year storm cause flooding). The flooding at Pond 3 has no impact on the flooding on Pond 2 because Pond 3 is much lower than Pond 2. Pond 3 overtops at an elevation of approximately 834.5 feet. The outlet elevation from Pond 2 is at 840.0 feet, well

above the flood elevation in Pond 3. Pond 2 is simply undersized for the amount of runoff it receives during a 100-year event.

Improvements to Pond 2 will provide some relief to the flooding at Pond 3 as less runoff will be routed through Pond 3. The following sections present alternative solution to the pond flooding. Potential solutions to the pond flooding require that the ponding elevations be reduced. This is accomplished through:

- Increasing the available storage at the pond
- Increasing the outflow capacity from the pond (larger outfall)
- A combination of additional storage and outflow capacity

The evaluation begins with Pond 2 (the upstream pond) as any improvements to this pond should be factored in the solutions for the Pond 3 (the downstream pond).

Pond 2 Solutions

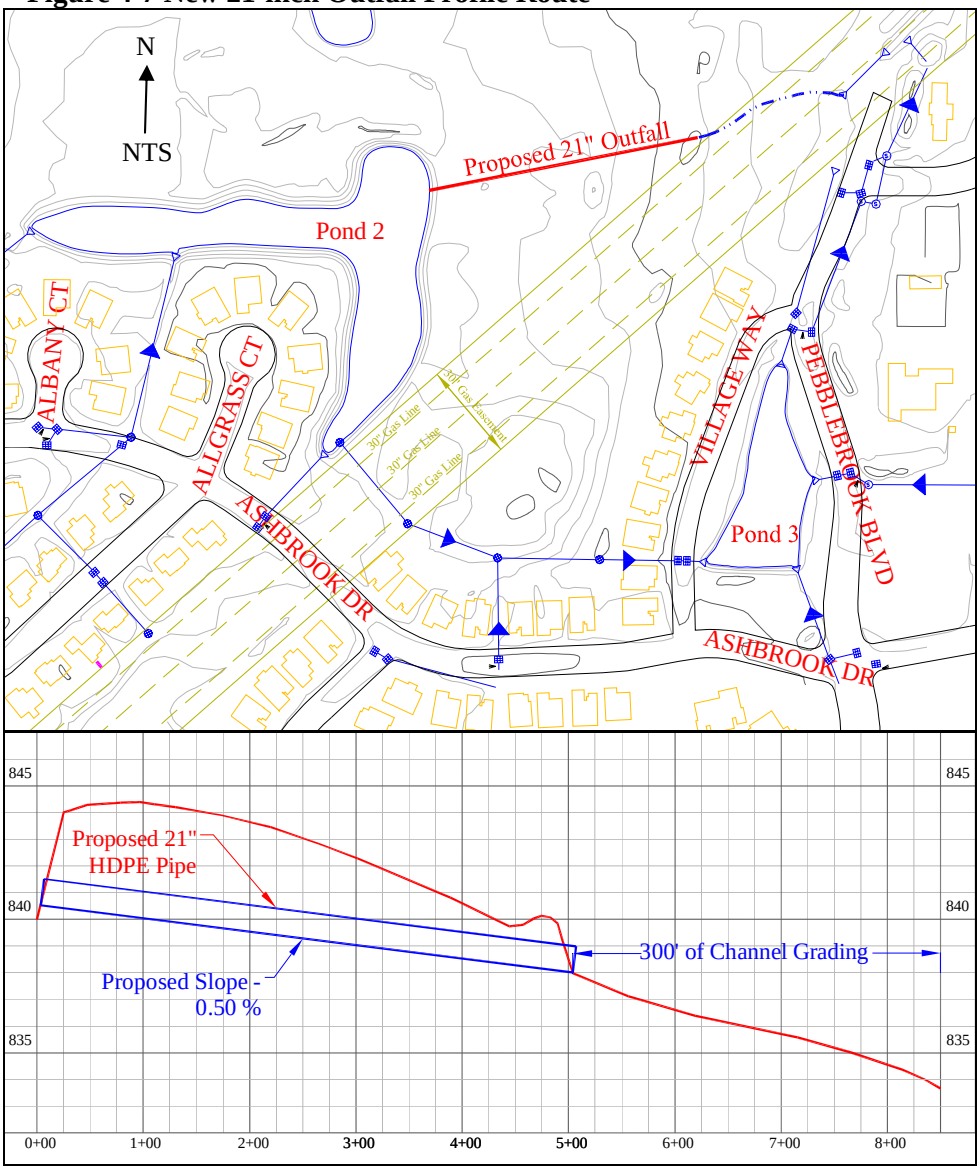
Alternative 1

Alternative 1 includes increasing the storage volume of Pond 2 to reduce peak flood elevation. This alternative is was found to be unfeasible due to site constraints. The pond is surrounded by golf course fairways, residential homes and a large gas utility easement. There is no room for expansion of this pond. A construction cost opinion for this alternative was not estimated since expansion of the pond is not feasible.

Alternative 2

Alternative 2 includes construction of a new outfall from Pond 2. The new 21" outfall would be constructed northeasterly, across the golf course (fairway #2) and discharge to the open channel that begins near the golf course entrance. The new outfall would have to "daylight" into an open swale for the last 300 feet due to elevation constraints. This outfall route is shown on Figure 4-7.

Figure 4-7 New 21-inch Outfall Profile Route

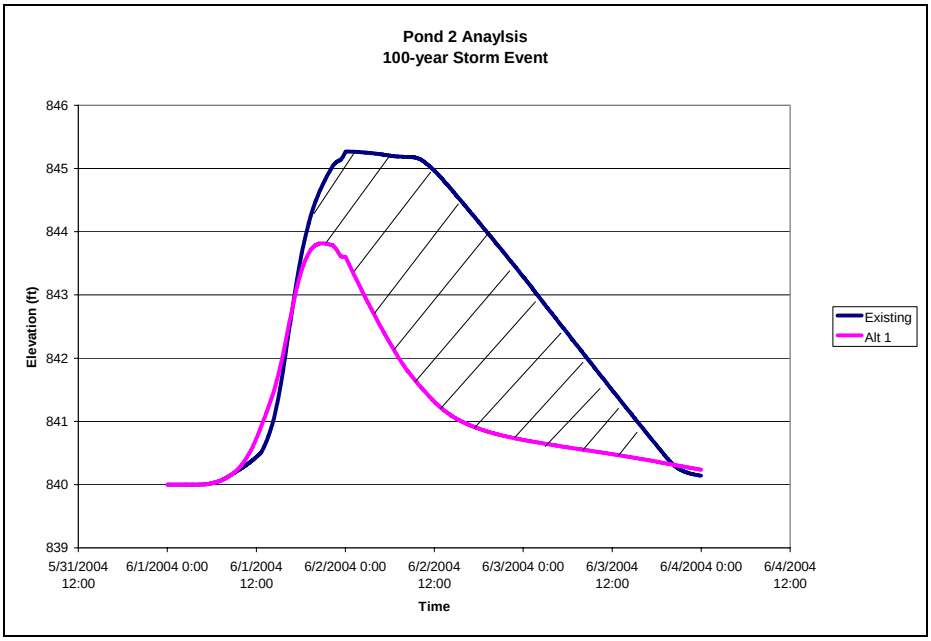


To prevent low flows from smaller storms from discharging to the open swale and causing potential excess vegetation growth, the invert of the new 21-inch outfall would be set 0.5 feet above the existing 12-inch outfall. Hence, low flows would discharge through the existing 12-inch outlet, drain through Pond 3 and then through the piped outlet system to the Pebblebrook South channel. To limit the amount of flow that is discharged to Pond 3, a 5-inch restrictor plate would be installed on the existing 12-inch outlet pipe from Pond 2.

The resulting stage hydrograph for this alternative is shown in Figure 4-8. The blue hydrograph shows the existing conditions, with the pond elevation exceeding 845 feet (overtopping occurs at about elevation 845.5 feet). The red hydrograph shows the pond with the new 21-inch outlet. As shown, the peak pond elevation would drop to approximately 843.8 feet, its original design elevation of 844.0 feet. This figure also illustrates the large volume of water than will no longer be routed

through Pond 3 (see hatched area on Figure 4-8). Bypassing this flow away from Pond 3 via the new 21-inch outfall from Pond 2 will reduce the frequency and duration of flooding at Pond 3.

Figure 4-8 Graph showing Pond 2 stages

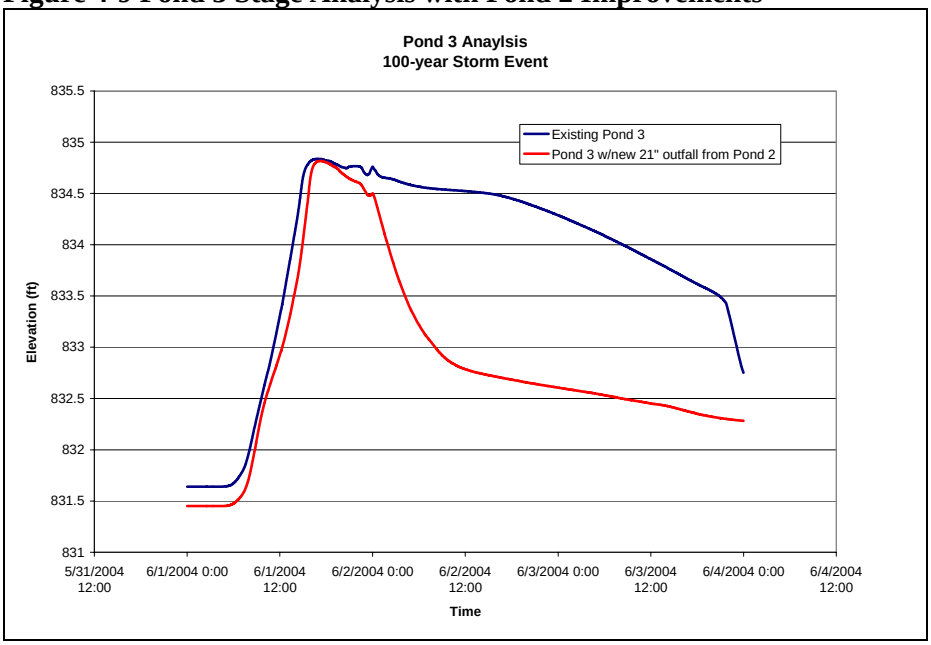


The preliminary opinion of total project costs for Alternative 1 is $76,000. Appendix A contains the detail of this cost estimate.

<u>Pond 3 Solutions</u>

Improvements to Pond 3 assume that the new 21-inch outfall from Pond 2 will be constructed. The Pond 2 outfall by itself would have a very positive impact to the performance of Pond 3. The frequency of flooding would be reduced from a 2-year event to somewhere between a 10-year and 25-year event. During a 100-year storm, Pond 3 would still flood, though the duration of flooding would be significantly shorter. This is illustrated in Figure 4-9, which shows the existing Pond 3 stage hydrograph (shown in blue), along with the proposed hydrograph, which includes the new 21-inch outfall from Pond 2 (shown in red). The peak elevation would be about the same, but the stage would drop quickly (within a few hours) down below the flood elevation of approximately 834.5 feet.

Figure 4-9 Pond 3 Stage Analysis with Pond 2 Improvements

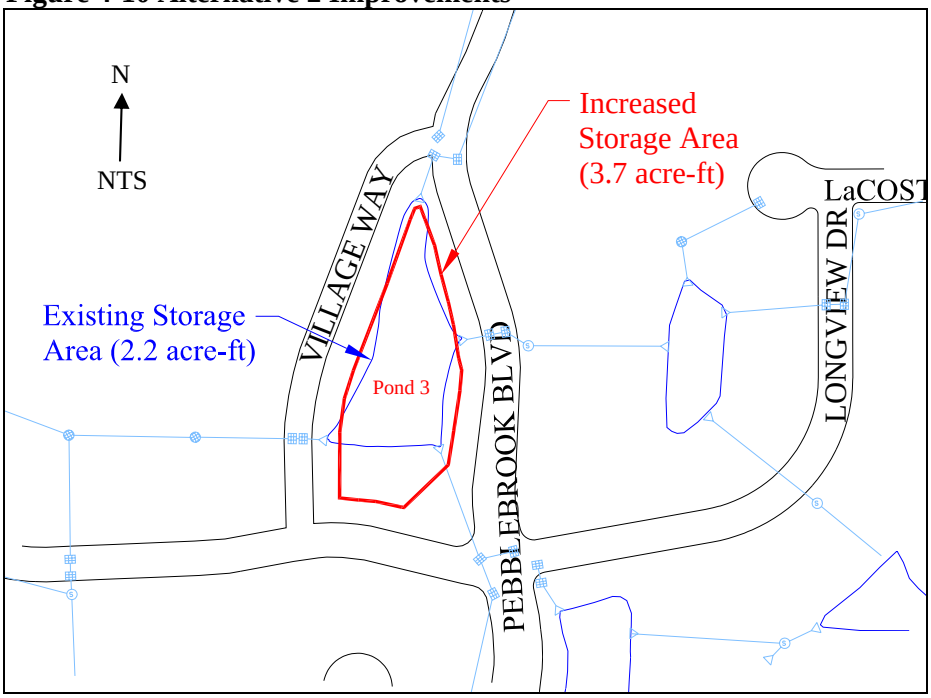


To prevent any flooding during the 100-year storm, further improvements would be needed. Alternatives considered, including additional storage, increased outlet size from Pond 3, and a combination of the two. These alternatives improvements are outlined below, followed by an evaluation of their effectiveness using the XP-SWMM model.

Alternative 1

Alternative 1 includes an increase in the storage volume of Pond 3. Figure 4-10 shows the area where Pond 3 could be expanded under this alternative. It could be expanded from 2.2 acre-feet to 3.7 acre-feet. However, an existing force main and air-release valve are located on the south side of the existing pond and would have to be relocated. This could be accomplished during the excavation of the additional storage.

Figure 4-10 Alternative 2 Improvements



The preliminary opinion of total project costs for Alternate 1 is estimated at $200,000. Appendix A contains the detail of this cost estimate.

Alternative 2

Alternative 2 includes an increase of the size of the Pond 3 outfall from 15-inch to 24-inch. The two segments of storm sewer downstream of Pond 3 (about 100 lineal feet) would need to be replaced with 24-inch sewer. This improvement would also require the replacement of an inlet and an end section.

Appendix A shows a detailed total construction cost opinion for Alternative 2 improvements. The total construction cost opinion for these improvements is estimated at $13,000.

Alternative 3

Alternative 3 includes the additional storage plus increasing the size of the outfall to Pond 3 (from 15-inch to 24-inch).

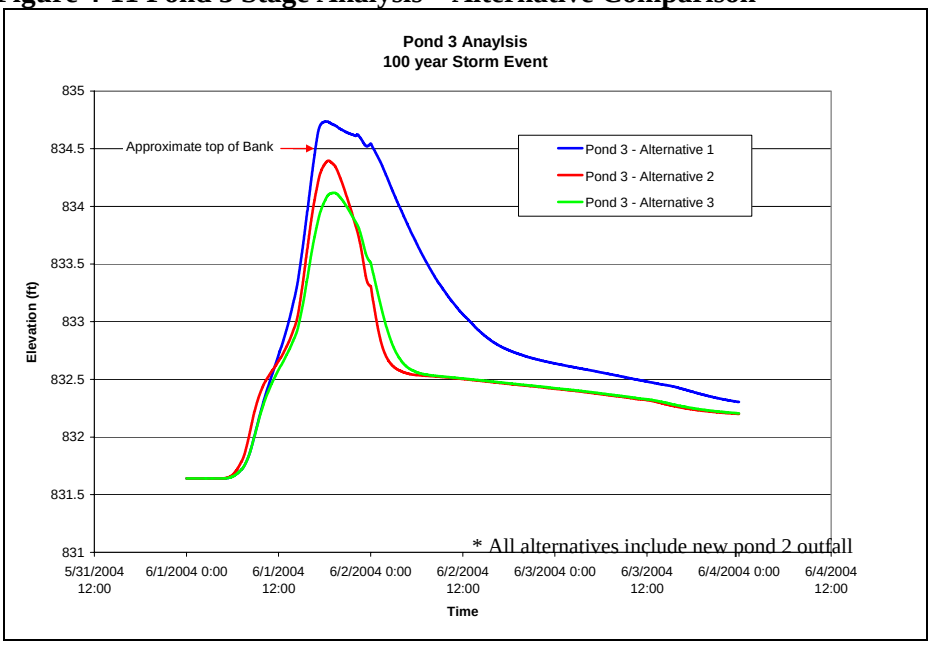
Appendix A shows a detailed total construction cost opinion for Alternative 3 improvements. The total construction cost opinion for these improvements is estimated at $215,000.

XP-SWMM Evaluation of Alternatives

The XP-SWMM model was used to determine the effectiveness of the three improvement alternatives for Pond 3. Again, all three alternatives assume the new 21-inch outlet from Pond 2 will be constructed. The results of this analysis are

shown on Figure 4-11. The figure shows the stage hydrographs for each of the Pond 3 alternatives. The roadway downstream of the pond begins to flood at about elevation 834.5 feet.

Figure 4-11 Pond 3 Stage Analysis – Alternative Comparison

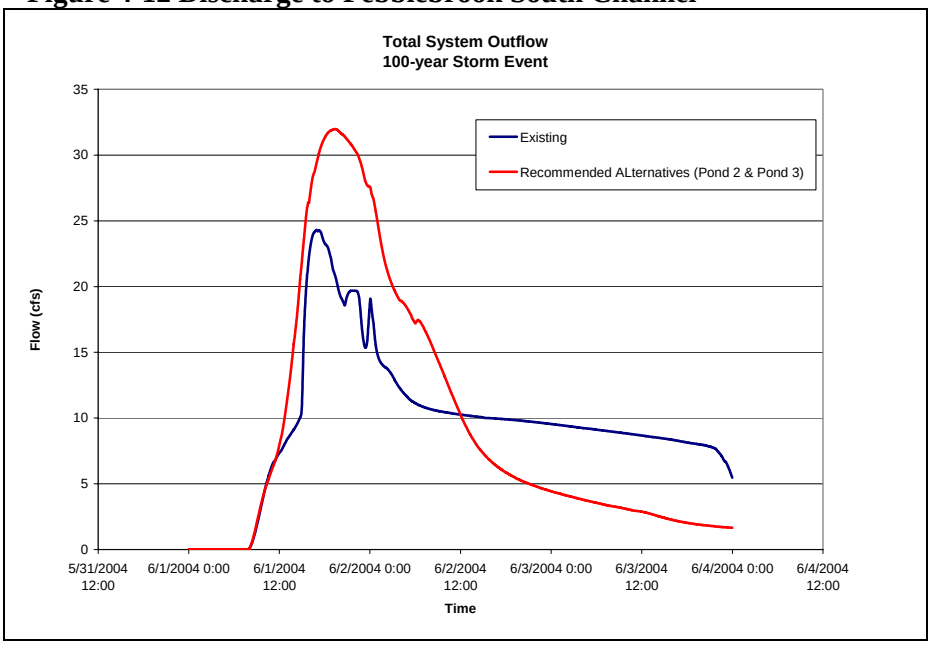


Alternative 1 (blue hydrograph), which includes increasing the storage at Pond 3, has little impact on flooding due to the large contribution of runoff from all of the other ponds in the Pebblebrook South Watershed. Alternative 2 (red hydrograph), which involves increasing the size of the Pond 3 outlet pipe (from 15-inch to 24-inch) lowers the elevation of Pond 3 below the roadway flooding elevation. Alternative 3 (green hydrograph), which includes both the storage increase and upsizing of the outlet pipe, provides further, though minor (0.3 feet), reduction in the peak 100-year stage.

Based on the XP-SWMM analysis of the Pond 3 alternatives, it appears the most cost effective option is to increase the size of the outlet pipe (Alternative No. 2). Expanding the pond storage provides very little additional flood reduction benefits for the added cost.

The downstream impact from both the new 21-inch outfall from Pond 2 and the upsizing of the outlet pipe from Pond 3 was also analyzed. The results of this analysis are shown in Figure 4-12. The figure shows the hydrograph downstream of Pond 3, where it discharges into the Pebblebrook South channel at the golf course entrance. The blue hydrograph shows existing conditions (no improvements) and the red hydrograph shows proposed conditions (new 21-inch outfall from Pond 2 and upsized outlet pipe from Pond 3).

Figure 4-12 Discharge to Pebblebrook South Channel



As a result of the Pond 2 and Pond 3 improvements, the net peak flow from the Pebblebrook South Watershed will increase from 24 cfs to 32 cfs during a *100-year* storm event. Though this represents an increase, when looked at an overall watershed basis, the series of ponds in the Pebblebrook South Watershed provide an excellent level of control. The total drainage area tributary to the Pebblebrook South channel is 322 acres. Under existing conditions, this equates to a release rate of 0.08 cfs/acre. Under proposed conditions, this would increase to 0.1 cfs/acre. These release rates are very restrictive when compared to most communities and provide a high level of stormwater management control.

The flow increase from smaller storms was also analyzed. During the 2-year event, the peak flow from the Pebblebrook South Watershed will increase from 9.0 cfs to 10.4 cfs. During a 10-year event the peak flow would increase from 14.7 cfs to 22.3 cfs.

The only downstream structures are a CMP culvert (300 feet of 49-inch by 33-inch CMP arch pipe) and a golf cart crossing bridge. Impacts to these structures would be minor and would occur during heavy rain events, when the golf course is not utilized.

To further minimize any downstream impacts, no new upstream areas should be allowed to discharge into the Pebblebrook South Watershed. Also, the remaining undeveloped area in the watershed (north of SR 32 and west of Pebblebrook Boulevard) should have the same restrictive release as the overall Pebblebrook South Watershed (0.08 cfs/acre for the 100-year event).

4.5 Summary – Pebblebrook South Watershed

It is recommended that the improvements listed below be performed in order to eliminate property damage and safety concerns due to flooding of Ponds 2 and 3 in the Pebblebrook South Watershed.

Capitol Improvements

- Pond 2 Improvements
  - ⇒ New 21-inch outfall across golf course
  - ⇒ Approximately 300 feet of channel grading
  - ⇒ Place 5-inch restrictor plate on existing 12-inch outfall
  - ⇒ $76,000

- Pond 3 Improvements
  - ⇒ Upsize outlet sewer size from 15-inch to 24-inch
  - ⇒ $13,000

Other Recommendations

- No new drainage areas should be added to the Pebblebrook South Watershed. Areas draining to the Fred Hines drain should continue to discharge as the do currently.

- Any new developed areas within the Pebblebrook South Watershed including basins 6 and 14 should be required to provide detention that restricts the 100-year release rate to 0.08 cfs/acre.

Appendix A – Construction Cost Opinions

OPINION OF PROBABLE CONSTRUCTION COST

HAMILTON COUNTY

PEBBLEBROOK MAIN CHANNEL - PEBBLEBROOK PLACE IMPROVEMENTS (12'x4')

ITEM	QUANTITY	UNIT	UNIT COST	TOTAL
12' x 4' Concrete Box Culvert	60	LF	$900	$54,000
Channel Grading	675	LF	$50	$33,750
Pavement Restoration	100	SY	$50	$5,000
Mobilization and Demobilization	1	LSUM	$5,000	$5,000
Construction Total				$97,750
Construction Contingency (15%)				$14,700
Non-Construction Costs (20%)				$19,600
Total Project Cost				**$132,000**

OPINION OF PROBABLE CONSTRUCTION COST

HAMILTON COUNTY

PEBBLEBROOK MAIN CHANNEL - PEBBLEBROOK PLACE IMPROVEMENTS (17'x4')

ITEM	QUANTITY	UNIT	UNIT COST	TOTAL
17' x 4' Concrete Box Culvert	60	LF	$1,500	$90,000
Channel Grading	675	LF	$50	$33,750
Pavement Restoration	100	SY	$50	$5,000
Mobilization and Demobilization	1	LSUM	$6,000	$6,000
Construction Total				$134,750
Construction Contingency (15%)				$20,200
Non-Construction Costs (20%)				$27,000
Total Project Cost				**$182,000**

OPINION OF PROBABLE CONSTRUCTION COST

HAMILTON COUNTY

PEBBLEBROOK MAIN CHANNEL - MOONTOWN ROAD IMPROVEMENTS

ITEM	QUANTITY	UNIT	UNIT COST	TOTAL
7' x 3' Concrete Box Culvert	40	LF	$400	$16,000
Channel Grading	1850	LF	$50	$92,500
Channel Restoration/Stabilization	1	LSUM	$20,000	$20,000
Pavement Restoration	50	SY	$50	$2,500
Mobilization and Demobilization	1	LSUM	$7,000	$7,000
Construction Total				$138,000
Construction Contingency (15%)				$20,700
Non-Construction Costs (20%)				$27,600
Total Project Cost				**$186,000**

OPINION OF PROBABLE CONSTRUCTION COST

HAMILTON COUNTY

PEBBLEBROOK SOUTH CHANNEL - POND 2 - ALTERNATIVE 2

ITEM	QUANTITY	UNIT	UNIT COST	TOTAL
21" RCP Pipe	500	LF	$55	$27,500
Site Restoration	1	EA	$10,000	$10,000
5" x 5" restriction	1	EA	$2,000	$2,000
Channel Grading	300	LF	$50	$15,000
Rip Rap	2	CYS	$75	$150
Mobilization and Demobilization	1	LSUM	$2,000	$2,000
Construction Total				$56,650
Construction Contingency (15%)				$8,500
Non-Construction Costs (20%)				$11,300
Total Project Cost				**$76,000**

OPINION OF PROBABLE CONSTRUCTION COST

HAMILTON COUNTY

PEBBLEBROOK SOUTH CHANNEL - POND 3 - ALTERNATIVE 1

ITEM	QUANTITY	UNIT	UNIT COST	TOTAL
10" Forcemain	400	LF	$40	$16,000
Air-Release Valve	1	EA	$140	$140
Excavation for Basin	6000	CY	$20	$120,000
Site Restoration	1	LSUM	$5,000	$5,000
Mobilization and Demobilization	1	LSUM	$7,000	$7,000
Construction Total				$148,140
Construction Contingency (15%)				$22,200
Non-Construction Costs (20%)				$29,600
Total Project Cost				**$200,000**

OPINION OF PROBABLE CONSTRUCTION COST

HAMILTON COUNTY

PEBBLEBROOK SOUTH CHANNEL - POND 3 - ALTERNATIVE 2

ITEM	QUANTITY	UNIT	UNIT COST	TOTAL
24" RCP line	100	LF	$60	$6,000
New Inlet	1	EA	$2,000	$2,000
Pavement Restoration	10	SY	$50	$500
Site Restoration	1	LSUM	$1,000	$1,000
Mobilization and Demobilization	1	LSUM	$0	$0
Construction Total				$9,500
Construction Contingency (15%)				$1,400
Non-Construction Costs (20%)				$1,900
Total Project Cost				**$13,000**

OPINION OF PROBABLE CONSTRUCTION COST

HAMILTON COUNTY

PEBBLEBROOK SOUTH CHANNEL - POND 3 - ALTERNATIVE 3

ITEM	QUANTITY	UNIT	UNIT COST	TOTAL
10" Forcemain	400	LF	$40	$16,000
Air-Release Valve	1	EA	$140	$140
Excavation for Basin	6000	CY	$20	$120,000
24" RCP line	100	LF	$60	$6,000
New Inlet	1	EA	$2,000	$2,000
Pavement Restoration	10	SY	$50	$500
Site Restoration	1	LSUM	$6,000	$6,000
Mobilization and Demobilization	1	LSUM	$8,000	$8,000
Construction Total				$158,640
Construction Contingency (15%)				$23,800
Non-Construction Costs (20%)				$31,700
Total Project Cost				**$215,000**