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Stormwater study for the yellis tract watershed for the city of bethlehem, July 1986, 10p.

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STORMWATER STUDY FOR THE

YELLIS TRACT WATERSHED

by

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for

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IHL-107-86

1. INTRODUCTION

The City of Bethlehem asked the writers to assess possible changes in stormwater runoff patterns from the Yellis Tract Watershed as a result of developing existing meadows and modifying the detention pond system. The watershed is in north Bethlehem bordered roughly by Holy Savior Cemetery, Nazareth Pike, and the Monocacy Creek (see Figure 1). The Soil Conservation Service (SCS) TR-20 Computer Program for Project Formulation Hydrology was used to determine the response of the watershed to possible modifications.

Basically, three major cases were studied:

1) The pre-existing, minimally developed basin, circa 1955.

2) The existing watershed conditions with current land use and development with limited detention available along the main swale. The swale becomes a well defined channel north of the Pine Top Trail.

3) Developing the remainder of the basin and increasing detention along the main swale.

The central questions answered by this study are:

1) Has development increased peak flows from the pre-existing situation to the present?

2) What will be the effect of future development on runoff?

3) Can stormwater runoff be managed by providing detention, especially in the main swale? Specifically, can flows in downstream channels be contained within the existing banks?

This report contains a brief description of the model and its inputs, and then highlights the results of this study. Another report submitted to the City Engineer entitled "Background Report: Computations and Data for the Yellis Tract Stormwater Study" (herein referred to as the Background Report) contains a detailed listing of the input data for the model as well as detailed model outputs.

2. THE MODEL

The TR-20 Computer Program for Project Formulation Hydrology uses the procedures described in the SCS National Engineering Handbook, Section 4, Hydrology (NEH-4) except for the reach flood routing procedure which is described in Appendix G & H of the TR-20 manual. The TR-20 model is useful in generating hydrographs and routing runoff through stream reaches and detention storage. It can be used to readily assess effects of land use and detention changes. The TR-20 input data characterizes the drainage areas, reaches, and detention structures. The drainage areas require an area, a time-of-concentration, and a curve number, (which is an index of runoff potential determined by the soil type, slope, and land use). The reaches are modeled using information such as reach geometry, slope, and roughness characteristics. Detention structures require elevation, discharge, and storage volume data. The model is formulated as a series of contributing subareas, reaches, and structures that route the stormwater through the watershed. A flow chart schematic of the overall flow through the watershed is shown in Figure 2. Corresponding areas and structures are shown in Figure 1.

3. THE BASIN

The watershed is composed of 0.6 square miles of suburban residential land as shown in Figure 1. Its soils are Washington, Conestoga, and Clarksburg Silt Loams with an SCS hydrologic soil group designation B. Slopes generally range between 3 and 8% (see NEH-4 and Northampton County Soils Report). Based upon SCS procedures, curve numbers (CN's) were chosen for these combinations of land use, soil type and for average antecedent moisture conditions. Times of concentration (t_c) for each subarea were chosen using the Upland Method (see NEH-4). The watershed was sub-divided into twelve subbasins according to their drainage characteristics as shown in Figure 1 (see Background Report for specific values of CN's and t_c).

Swales, gutters, and sewers carry stormwater from the upper subareas of the watershed to the structures on the Yellis tract. A channel north of Pine Top Trail then carries runoff to the Monocacy Creek. This channel has a capacity of carrying approximately 120 cubic feet per second within its banks over a majority of its length.

The structures within the watershed consist of:

a) depression resulting from a sinkhole south of Macada Road (designated as structure 10 (S10) on Figure 1),
b) detention pond at the corner of Johnston Drive & Powder Mill Road (S7),
c) detention pond south of Redfern Lane (S1),
d) detention pond north of Redfern Lane (S6),
e) detention pond south of Pine Top Drive (S2),
f) culvert at Pine Top Trail (S4) and
g) culvert at Biery's Bridge Road (S5).

According to city procedures, the design storm for this basin

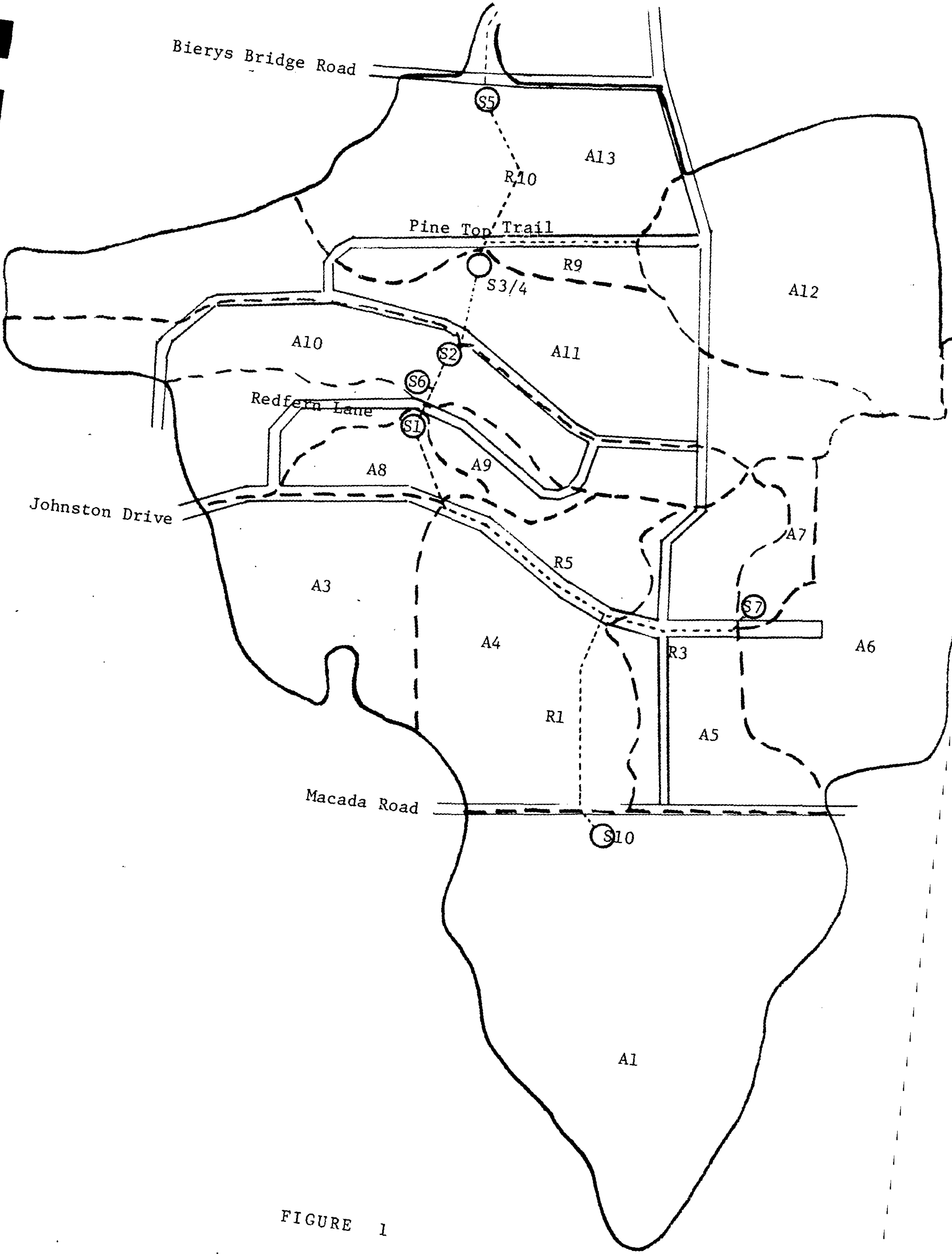
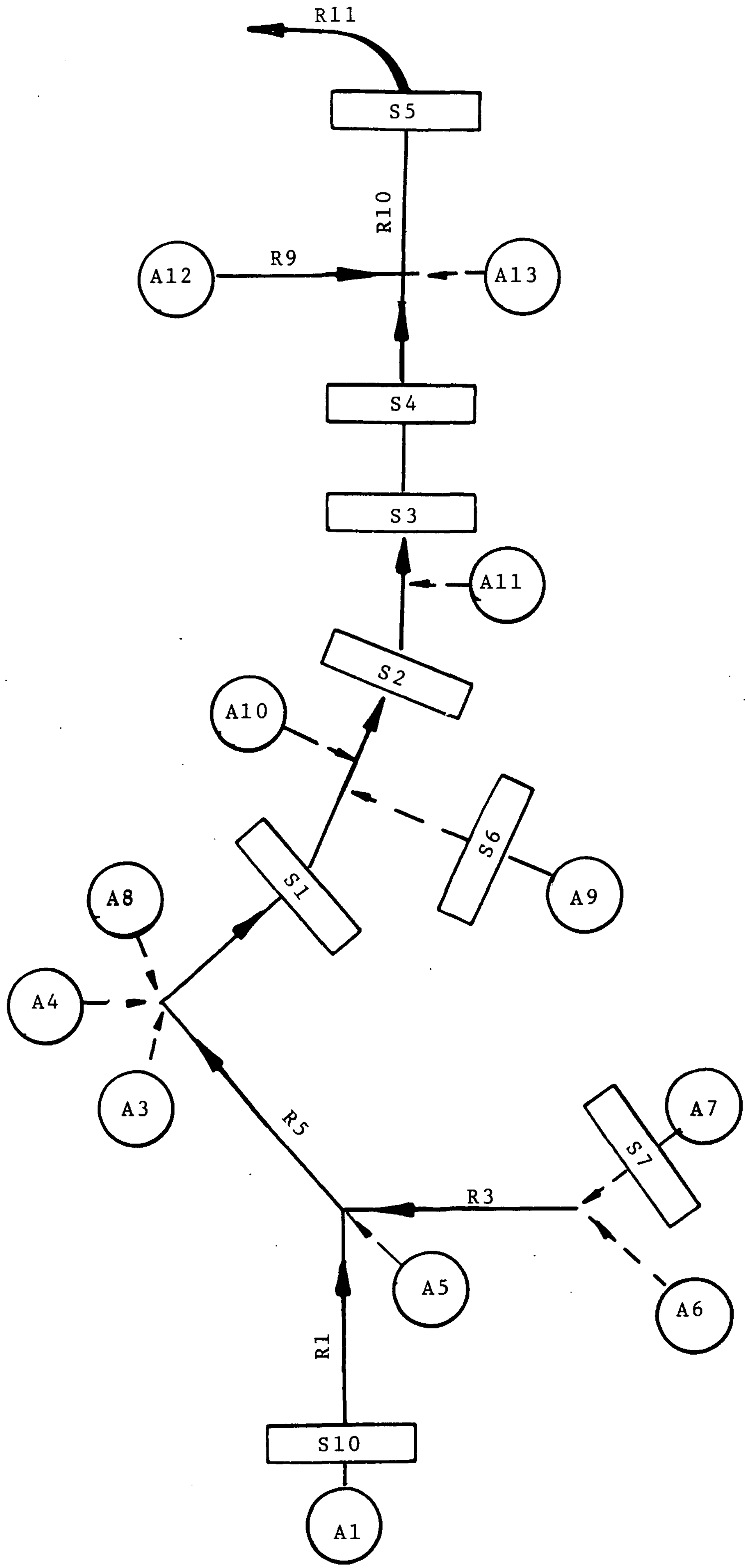

Bierys Bridge Road
S5
A13
R10
Pine Top Trail
R9
S3/4
A12
A10
S2
A11
S6
Redfern Lane
S1
A9
A8
Johnston Drive
A7
R5
A3
S7
A4
R3
A6
R1
A5
Macada Road
S10
A1

FIGURE 1


R11
S5
R10
A12
R9
A13
S4
S3
A11
S2
A10
S6
S1
A8
A9
A4
A3
R5
A7
S7
R3
A5
A6
R1
S10
A1
FIGURE 2

is the 24-hour, 25-year storm using the SCS Type II cumulative rainfall distribution. Simulations were also conducted for 24-hour SCS Type II storms with recurrence intervals of 2 and 10 years.

A complete listing of the data for the basin is found in the Background Report.

4. TEST CASES

Many simulations were performed and the input/output of each is contained in the Background Report. Presented here is a selection of seven simulations that allow answers to the relevant questions that need resolution. These selected simulations are summarized in Table 1.

Table 1: TEST CASE DESCRIPTIONS

CASE	DESCRIPTION
I	Pre-existing Conditions Curve Numbers Equal 60; No S6 or S7; S1, S2, S4, S5 remain but with smaller storage volumes to represent roads that would impede flow; subarea 1 is 10% of existing size.
IIa	Existing Conditions Curve Numbers equal 70 in all areas except subareas 6 and 3, which are 60; structures 1, 2, 4, 5, 6, 7, and 10 included (see Figure 1).
IIb	Developed Subarea 6 Existing Condition with subarea 6 Curve Number changed from 60 to 70.
IIc	Removed S7 Existing Condition (Case IIa) without detention at S7.
IIIa	Increased Storage at S2. Expanded the available storage at S2 (only change from Case IIa).
IIIb	Increased Storage at S2 and S3. Expanded storage at S2 (Case III a) and created storage at S3.
IIIc	More Storage at S2 and S3. Doubled the storage at S2 from case IIIa with storage at S3 from Case IIIb.

5. RESULTS

Following are the results of the test cases. Peak outflows from selected structures and reaches are listed as well as the time-to-peak. The Background Report has a complete listing of the hydrographs at all of the structures and reaches. The results presented here are most pertinent to the questions asked in the Introduction. Flows are presented in cubic feet per second, and time of peaks are in hours since the beginning of rainfall of the design storm. The results are for the 25-year storm.

CASE I	LOCATION				
PRE-EXISTING	S10	S1	S2	S3/4	R10
Peak Flow	78 cfs	99	116	206	299
Time of Peak	(10.10 hrs)	(10.22)	(10.03)	(10.06)	(10.13)

CASE II	A6	S7	S1	S2	S3/4	R10
a. EXISTING Peak Flow	47	7	226	299	392	467
Time of Peak	(10.08)	(10.15)	(10.11)	(10.08)	(10.06)	(10.08)
b. DEVELOP A6 Peak Flow	71	7	240	326	423	495
Time of Peak	(10.06)	(10.15)	(10.05)	(10.05)	(10.04)	(10.07)
c. REMOVE S7 Peak Flow	47	14	226	306	402	476
Time of Peak	(10.08)	(9.97)	(10.11)	(10.07)	(10.05)	(10.08)

CASE III	INFLOW S2	OUTFLOW S2	S3/4	R10
a. INCREASED STORAGE S2 Peak Flow	330	279	361	440
Time of Peak	(10.03)	(10.16)	(10.07)	(10.09)
b. INCREASED STORAGE S2 & S3 Peak Flow	330	279	363	441
Time of Peak	(10.03)	(10.16)	(10.07)	(10.09)
c. DOUBLED STORAGE AT S2 Peak Flow	330	252	328	404
Time of Peak	(10.03)	(10.22)	(10.09)	(10.10)

6. DISCUSSION

A. Comparison of Pre-Existing and Existing Simulations (Case I vs IIa)

The effect of development from the pre-existing to the existing simulation is shown by comparing Cases I and IIa. For the 25-year storm, flow in the reach north of Pine Top Trail (R10) shows a 56% increase with development and the inclusion of some structures. At S3/4, the increase in peak flow is 91%, and at S1, 127%. The results indicate that development of the watershed from 1955 to present (1986) had a significant effect on the peak runoff. Note that the capacity of the existing channel, R10, is exceeded by the design storm for the pre-existing watershed condition.

B. Comparison of Development in Subarea 6 to the Existing Case (Case IIb vs IIa); and Comparison of Removal of S7 to the Existing Case (Case IIc vs IIa).

For future development in Subarea 6, Case IIb vs IIa shows a 51% increase in discharge from that area, but further downstream the effects are attenuated to 7% at S1, 8% at S3/4, and 6% in R10. The effect of this development is significant just downstream, but it is less noticeable in its impact at S1, S3/4, and R10.

Case IIc vs case IIa illustrates the effect of removing S7. Flow increased 88% just below that structure. At S1, there is no difference, 2% at S3/4, and 2% at R10 which are considered negligible. It is emphasized that S7 has a significant impact on flows just downstream (on Johnston Drive), but its effect is minimal further downstream. The lack of flow increase at S1 is due to the timing of peak flows arriving from S7 compared to other contributing areas.

C. Comparison of Increasing Storage in the Main Swale to the Existing Case (Cases IIIa through IIIc vs IIa)

Cases IIIa through IIIc show the effect of increased storage at S2 and S3. A simulation was conducted using a reasonably obtainable increase in storage at S2 (Case IIIa). Compared to the existing condition, this increased storage at S2 reduces the peak outflow by 7% at S2, 8% at S3/4, and 6% in Reach R10 (Case IIIa vs IIa). Coupling the increased storage at S2 with increased storage at S3 gives a 6% decrease in peak flow in R10 (Case IIIb vs IIa).

For illustration purposes another simulation (Case IIIc) was conducted with a much greater storage at S2 than the reasonable storage used in Cases IIIa and IIIb. This simulation with a doubled storage at S2 and the Case IIIb storage at S3 yield a 14% reduction in R10 compared to the existing Case IIa. These extensive modifications of Case IIIc may not be physically possible and do not yield significant peak flow reductions at the main swale/channel locations considered.

7. CONCLUSIONS AND RECOMMENDATIONS

Conclusions

It is evident from this study that the natural ability of the existing swale/channel system to transmit water out of the basin has been overwhelmed by the combination of increased upstream development, possible inclusion of additional drainage area, and encroachment on the swale/channel system. It is also evident that stormwater runoff downstream of Pine Top Drive cannot be successfully managed by the simulated expansion of main swale detention facilities alone. The transition from the predevelopment condition to the present state has been accompanied by a 50 to 125 percent increase in the peak runoff for the design storm (24-hour, 25-year storm with the SCS Type II distribution).

The types of mitigation options considered here fall into three general categories:

1. Upstream detention
2. Main swale detention
3. Increased flow capacity of downstream channels

The conclusions based on the simulations are presented under these three headings.

1. Upstream Detention

Simulations conducted as part of this study indicate that small detention ponds scattered throughout the basin have a significant effect just downstream of that area and a lesser effect further downstream. Thus, the proposed development in one area (Subarea 6) would significantly increase flow immediately downstream (Johnston Drive) but would have a small impact further downstream. If developed, a detention facility immediately downstream of this area would be beneficial.

The existing structure S7 at the corner of Johnston Drive and Powder Mill Road is effective in controlling stormwater flows and should not be removed from service. Although small, S7 significantly reduces the 25-year design storm peak flow along Johnston Drive.

2. Main Swale Detention

Although reducing the design storm peak flows by 10-20% proposed improvements to the swale detention basin S2 will not solve the problem of flooding downstream. Less than a 15% reduction in design storm peak flow would result from a significant expansion of S2, which is extensive and may not be economically feasible (see Case IIIc).

3. Increased Flow Capacity of Downstream Channels

The reach downstream of Pine Top Trail has a bankfull capacity of approximately 120 cubic feet per second over a majority of its length. Simulations indicate that the capacity of the channel will be almost achieved by the two-year storm for the existing watershed conditions and would have been greatly exceeded by the 25-year storm for the pre-exisitng watershed conditions. As a result, even extensive detention facilities will not prevent overtopping of the channel banks from storms with return periods of 25 years. Additional mitigation would be achieved by increasing the capacity of the channel.

<u>Recommendations</u>

All stormwater mitigation techniques must be analyzed in terms of their economic benefits and costs. The recommendations made below, while all are technically feasible, must be investigated in terms of their relative costs and benefits. It is the judgement of the writers that the channel improvements will provide the most benefit for the least cost whereas swale detention facilities, although useful, may not achieve the desired mitigation effect for a reasonable cost. The following mitigation methods are presented in order of importance and should be investigated further from a cost stand point:

1. Improvements to the culvert under Pine Top Trail and to the channel between Pine Top Trail and Biery's Bridge Road.

2. Detention facilities for all newly developed areas off the main swale.

3. Improved detention facilities between Redfern Lane and Pine Top Drive (S2).

Thus, it is recommended that main swale detention only be considered in conjunction with channel improvements and upstream detention.