Patuxent River Hydrologic Study at Laurel, Maryland

Submitted to:

City of Laurel
Department of Public Works
Laurel, Maryland

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1.0 INTRODUCTION

The objective of this hydrologic study of the Patuxent River for the City of Laurel was to update the hydrologic model for revising the August 19, 1985 FEMA Flood Insurance Study (FIS) for the City of Laurel. The study was initiated as part of the 1990 Interagency Agreement #101011 managed by the Maryland Department of the Environment (MDE), previously the Maryland Department of Natural Resources (DNR).

This report will be utilized by the City of Laurel for watershed management and also as a planning tool relative to land use impacts. The Patuxent River watershed that was investigated extends from the headwaters downstream to the Patuxent Environmental Science Center (PESC). The drainage area analyzed was 150 mi^2. The study area, shown in Figure 1, included the WSSC reservoirs at Brighton Dam and Rocky Gorge Dam.

The hydrologic analysis was simulated for the existing watershed conditions and reflects the flooding potential of the community at the time this study was completed. A calibration analysis was performed for a significant storm that occurred May 5-6, 1989. The results were calibrated to observed flood hydrographs at three USGS stream gages in the upper watershed, and measured water levels at WSSC's two reservoirs upstream of the City of Laurel. The models were developed with methods that are physically based which allow for flexibility in simulating a variety of storm events. This flexibility was verified by recreating the substantially larger flood resulting from Tropical Storm Agnes on June 21-22, 1972. Verification results were matched against observed peaks at two USGS gages located at Unity and Cattail Creek at Roxbury Mills, as well as measured water levels at WSSC's two reservoirs upstream of the City of Laurel.

Peak- flood discharges were estimated for the 2-, 10-, and the 100-yr storms. The results were then used to develop the 100-yr floodplain within the City of Laurel to revise the Flood Insurance Rate Map (FIRM) Panel 240053 0001 D, dated August 19, 1985. The 1% annual chance (100-yr) flood has been adopted by FEMA as the base flood for flood plain management purposes. The FIRM identified the flood-prone areas of the Patuxent River and is being revised due to detailed floodplain analyses and improved topographic mapping.

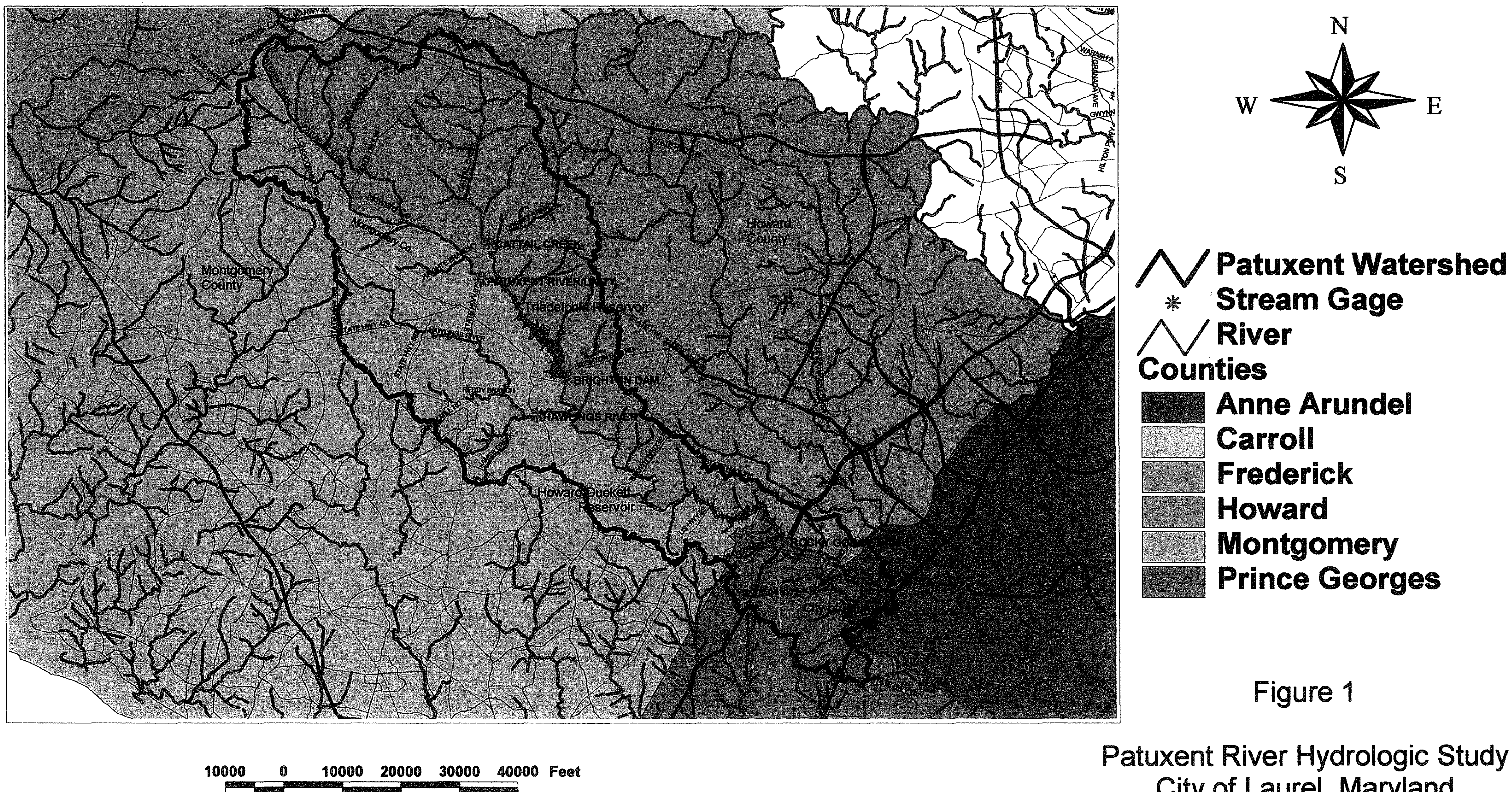

Figure 1

Patuxent River Hydrologic Study
City of Laurel, Maryland

Location Map

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2.0 WATERSHED DESCRIPTION

2.1 Location of Watershed

The Patuxent River watershed is one of the primary drainage systems in Howard, Prince George's, Montgomery, and Anne Arundel counties. At its confluence with the Chesapeake Bay, the drainage area is 936 mi^2. The Patuxent River drainage area to just south of Laurel is 150 mi^2 and predominantly lies within the Piedmont Physiographic region of Maryland. Two tributaries of the Patuxent River, Crow and Bear Branches, are located within the City of Laurel.

River discharge near the City of Laurel is monitored at USGS stream gage 01592500, located immediately downstream of the Rocky Gorge Dam. The stream gage was installed in 1945 prior to the construction of the upstream WSSC dams. The drainage area to the gage is 132 mi^2.

2.2 Climate

The City of Laurel is located in the mid-Atlantic region and experiences the four seasons. The climate is influenced by the Chesapeake Bay and the Atlantic Ocean to the east, and to a lesser extent the Appalachian Mountains to the west. The winter weather is primarily cold and dry, continental-polar winds from the west and northwest, but occasionally maritime-tropical winds from the south and southwest bring warm, often humid air to the region. During the summer, the dominance of these two air masses is reversed. Annual precipitation averages about 40 inches per year. The annual mean daily temperature is 16°C, with a daily annual maximum of 22°C and a minimum of 7°C. Annual temperature extremes vary from -21°C to 38°C.

2.3 Land Use

The land uses within the upper watershed range from forest to agriculture to a variety of interspersed low-density residential areas. The stream valleys of the upper Patuxent River and its headwater tributaries are heavily forested and thickly vegetated. A ridge marking the fall line between the Atlantic Coastal Plain and the hills of the Piedmont Plateau defines the western edge of the developed area of Laurel. Within the city limits the land uses consist of a variety of medium- to high-density residential areas and commercial development. The stream valley of the Patuxent River within the City of Laurel is predominantly wooded and located in park and residential areas, and to a lesser degree commercial areas. In addition, there are several stream crossings over the Patuxent River in the city limits.

2.4 WSSC Reservoirs

The Patuxent River has two large reservoirs located upstream that are managed by the Washington Suburban Sanitary Commission (WSSC). The first is T. Howard Duckett Reservoir at Rocky Gorge Dam, immediately upstream of the City of Laurel. The other is the Tridelphia Reservoir at Brighton Dam located about 13 river miles upstream of the Rocky Gorge Dam.

The Rocky Gorge Dam, built in 1954, has a total length of 840 feet at its crest. The drainage area is 132 mi^2. The outlet works have an under-sluice valve with a centerline elevation of 167 feet, three turbines with an approximate capacity of 50 cfs each, and seven Tainter-type spillway gates with crest elevations of 270 feet. The design capacity of the spillway is 57,700 cfs for a pool elevation of 288 feet.

The Brighton Dam, built in 1943, has a total length of 995 feet at its crest. The drainage area is 79 mi^2. The outlet works have silt valves with a centerline elevation of 365 feet, two turbines with an approximate capacity of 75 cfs each, and thirteen Tainter-type spillway gates with crest elevations of 270 feet.

3.0 EXISTING HYDROLOGIC CONDITIONS

3.1 Hydrologic Analysis Methodology

The HEC-1 Flood Hydrograph Package was chosen to analyze the Patuxent River hydrology. The model provides flexibility in choosing various methods to estimate runoff and routing, and ease to simulate various rainfall distributions. Due to the upstream reservoirs, three HEC-1 hydrologic models were developed. The three models were for the drainage to: (1) Brighton Dam, (2) Rocky Gorge Dam, and (3) City of Laurel. With the exception of the two reservoirs, the models assumed there are no stormwater management structures or stream crossings that provide significant retention of a 100-yr storm event.

The initial analysis was performed using the SCS method to estimate the runoff volume and runoff distribution. This method requires estimates of Runoff Curve Numbers (RCN) and Time-of-Concentration (Tc). However, the calibration analysis, as described in Section 4.0, indicated that the SCS's methods might not be suitable for the Patuxent River watershed. The calibration required significant increases (approximately three-fold) of Tc values to reduce the simulated peak discharges and to attempt to match the observed hydrographs. At issue are the underlying assumptions of the SCS method, which are described in the following sub-sections. The Holtan Loss Rate method was used to estimate rainfall abstraction and the Kinematic Wave method was used to route runoff through the sub-basin. Theses techniques were chosen to replace the SCS's methods, and require additional parameter estimation.

3.2 Hydrologic Parameter Estimation

The initial parameter estimated was the watershed above the City of Laurel, and then the break-down into sub-basins. For each subbasin hydrologic parameters were estimated for the Loss Rate and Routing methods. The estimation was enhanced by the use of a Geographic Information Systems (GIS) model.

3.2.1 GIS Hydrologic Analysis

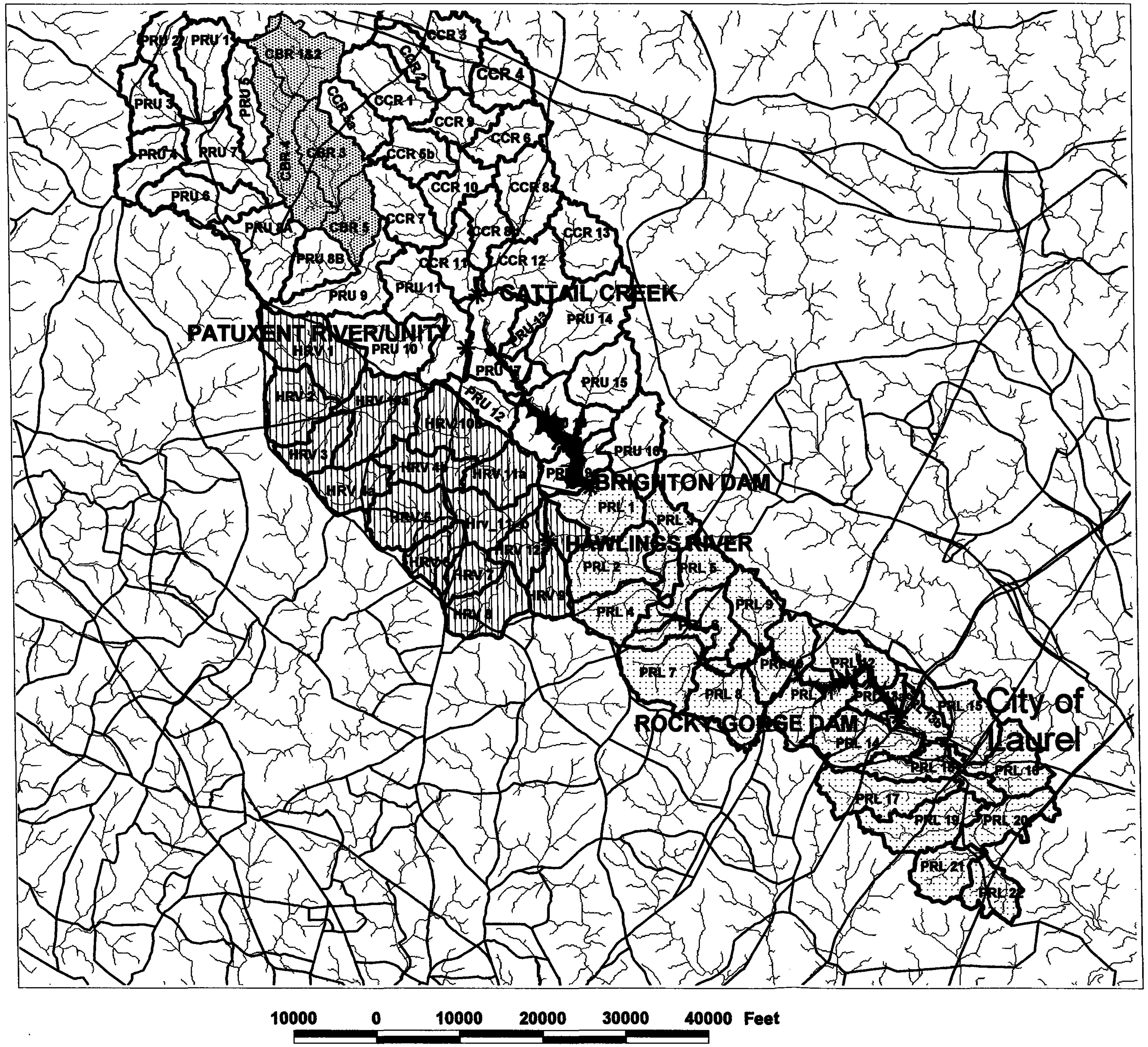
The Maryland State Highway Administration (SHA) GIS-Hydro interface expedited and standardized parameter estimation. The application runs under ArcView GIS (ver. 3.1) along with the Spatial Analyst extension. The GIS-Hydro databases contain information on elevation, land uses, and hydrologic soils for the State of Maryland in a raster format consisting of 100' by 100' cells.

Topography, base mapping, and stream network data originated from USGS digital mapping. The digital elevation model (DEM) was based on the topographic mapping from USGS Quadrangles (1:24000).

The existing land use was developed from vector data provided by the Maryland Office of Planning. The land uses were broken into thirty types of land uses that cover un-developed areas, agricultural, residential, commercial, and other classifications.

3.2.2 Sub-basin Drainage Areas

The goal was to create sub-basins with average drainage areas of 2 mi^2 in order to provide sufficient resolution in simulating the hydrology. Computations were automated by utilizing the GIS-Hydro interface to expedite the sub-basin delineations. Figure 2 shows the watershed and sub-basin delineations, and Figure 3 shows the HEC-1 model schematic.



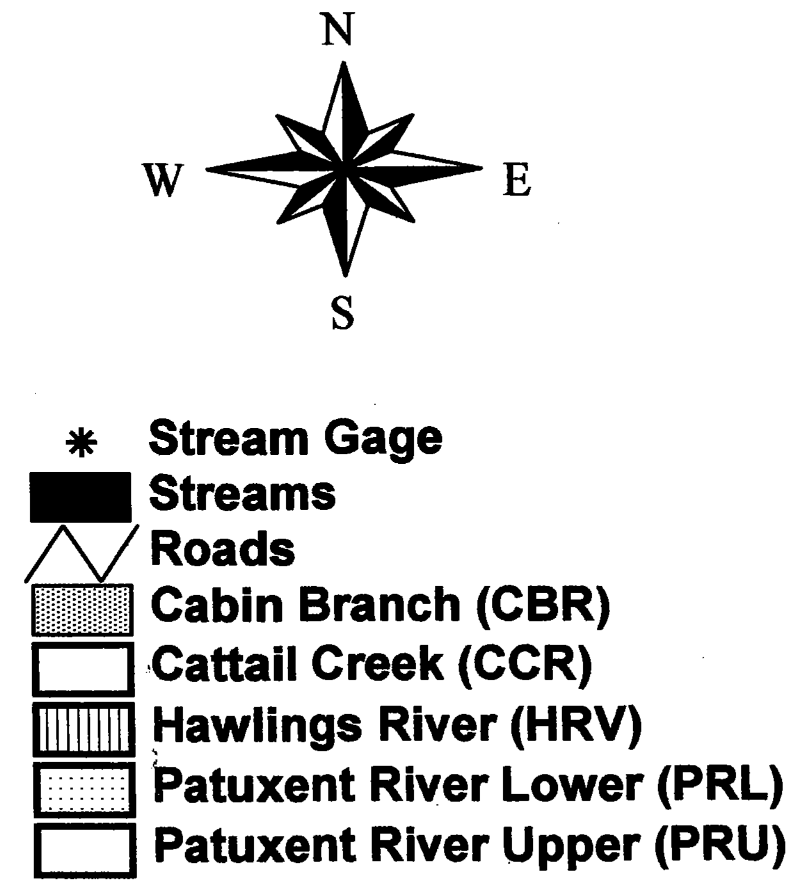


Figure 2

Patuxent River Hydrologic Study
City of Laurel, Maryland

Watershed Map

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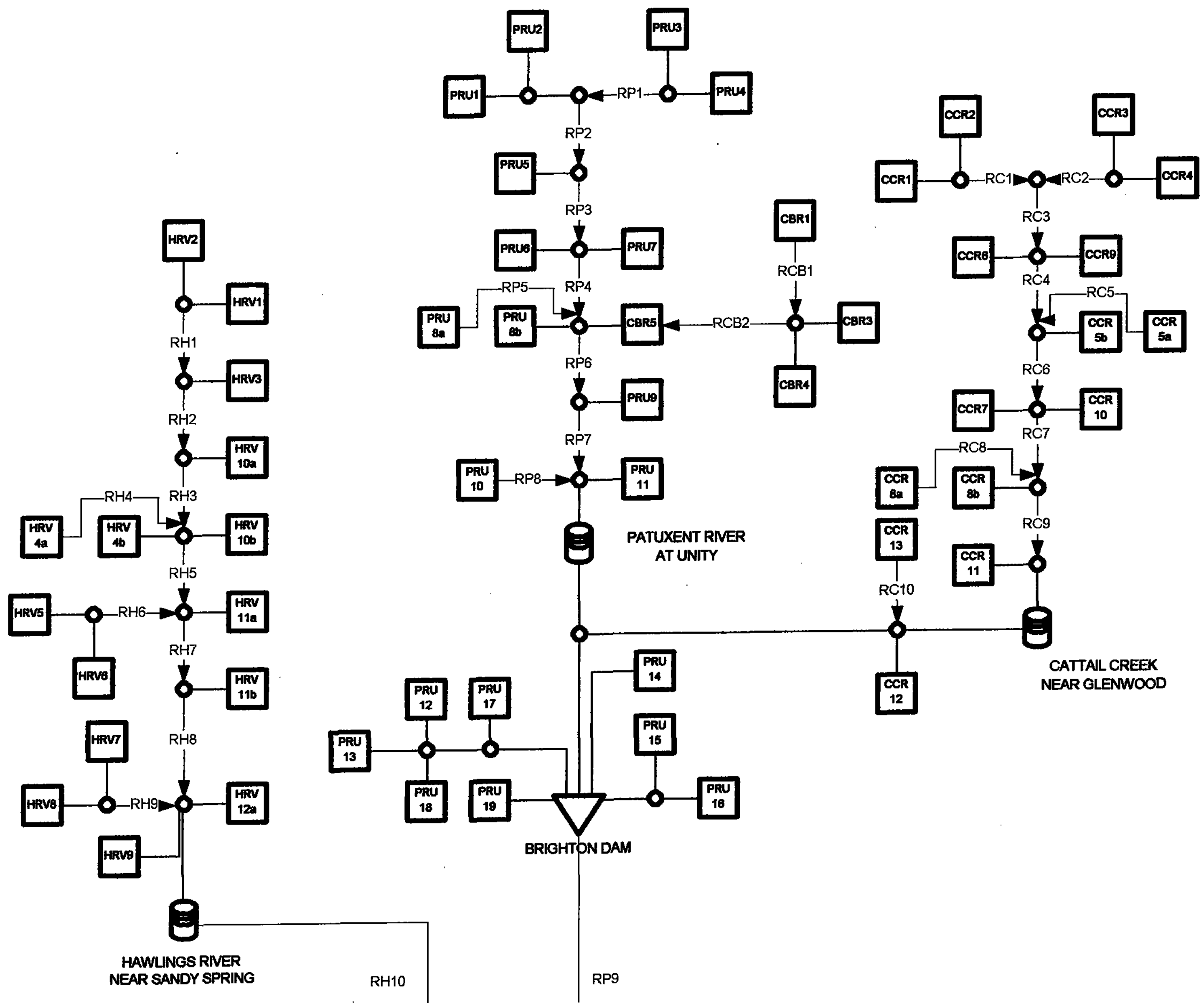
Figure 3: HEC-1 Hydrologic Model Schematic



3.2.3 Loss Rate Parameters

The initial method for estimating the rainfall abstraction was the SCS method. This method is a popular method that is used widely in the State of Maryland. The SCS method uses the RCN as a "lumped" parameter for rainfall abstraction. However, the method does not account for potential soil infiltration capacity or the rainfall intensity distribution. Other available methods that account for these paramtaters are the Green-Ampt and Holtan methods. The Holtan method was chosen because the parameters could be estimated by expanding the GIS-Hydro application, and it is the only method that accounts for the redistribution of soil moisture.

For each sub-basin the Holtan method requires soil infiltration rate, imperviousness, and soil moisture capacity to estimate rainfall abstraction. The parameters were estimated based on soil survey reports, planimetric mapping, GIS-Hydro, and field observations. Table 1 describes and summarizes these parameters.

Table 1 – Summary of Holtan Loss Rate Parameters

Parameter	Sub-basin Estimates	Source	Comments
Soil Infiltration rate (Fc)	0.20 (in/hour)	Howard, Montgomery, and P.G. County Soil Surveys (SCS, NRCS) and Technical Paper on Rain-Runoff Approaches (Khine, 1992)	Initial Fc estimates ranged from 0.2 to 0.6 based on SCS soil descriptions. However, all soil groups were dominated by silt-loam so Fc was set as a constant based on calibration results
Available Soil Moisture (SA)	0.96 (in)	Howard, Montgomery, and P.G. County Soil Surveys (SCS and NRCS)	SA was the average capacity based on the soil texture for the top 6" of soil and is dependant on Fc and antecedent moisture conditions (AMC)
Vegetative Growth Index (GI)	0.7-1.0	Technical Paper on Rain-Runoff Approaches (Khine, 1992)	The watershed is heavily influenced by agricultural vegetation, which is planted in spring and matures in summer thus GI = 0.7. For watersheds with mature perennial vegetation use GI = 1.0
Basal Area (BA)	0.27-0.66 (in/hour)	Technical Paper on Rain-Runoff Approaches (Khine, 1992)	BA was estimated based on land uses providing variance between sub-basins (developed areas were typically 0.1 and vegetated areas were 0.5 to 0.9)
Percent Imperviousness (%I)	0-38 (%)	TR-55 Urban Hydrology Manual (SCS)	%I was based on land uses to reflect areas that are impermeable to water, usually from man-made structures. (developed areas range from 25-100% and vegetated areas are 0%)

3.2.4 Routing Parameters

The initial method for hydrograph development was the SCS unit hydrograph. This method sets the peaking factor of the dimensionless unit hydrograph, and the sub-basin is defined by the Tc estimate. The calibration analysis indicated that a four-fold increase of Tc estimates were required to match the simulated results with the observed peak discharges and hydrograph shapes. Generating these Tc values would require elevated roughness coefficients for the overland and shallow concentrated flows that may be beyond acceptable levels.

The two possible approaches to replacing the SCS unit hydrograph are developing a unit hydrograph based on observed flood hydrographs, or implementing the Kinematic Wave/Muskingum-Cunge routing method. Development of a unit hydrograph requires an estimate of the peaking factor, however, this parameter varies due to different sub-watershed characteristics, rainfall duration and distribution, and flood stages within the stream valley. A physically based approach, such as the Kinematic Wave/Muskingum-Cunge routing method, provides the ability to respond to different types of rainfall events by transforming the runoff into the discharge hydrograph.

The Kinematic Wave/Muskingum-Cunge routing method was selected for the development of the runoff hydrograph, which was defined by three flow paths: overland, collector channel, and main channel. The Kinematic Wave portion estimates the overland flow runoff distribution. The Muskingum-Cunge routing method was used for both the collector and main channels within the sub-basin. This method was also used for the main-stem routing for upstream hydrographs through a sub-basin, but was refined by using an 8-point channel section. These methods use lengths, slopes, and Manning's roughness coefficients, and also account for contributing drainage areas and channel shapes.

The parameters were estimated based on topographic mapping, GIS-Hydro, and field measurements. Field reconnaissance was performed in May 1999, which closely resembles conditions during May 1989. The roughness coefficients were partially based on recent field reconnaissance (summer 1999) that closely resemble the field conditions of the May 1989 storm event. A general observation was that the collector/main channels within the sub-basins and the main-stem channels were heavily vegetated from ground cover to above flood-stage levels. Appendix A includes plots of typical stream cross-sections and pictures of typical channels and overbanks. Supporting documentation and reasoning behind selection of Manning's Roughness coefficients is included in Appendix B. These parameters are summarized in Table 2.

Table 2 – Summary of Routing Parameters

Parameter	Range	Source	Comment
Flow Lengths	400-16,600 (ft)	USGS Quad maps and GIS-Hydro model	Overland flow was set to 400', sub-basin channels were averaged, and main-stem channels were measured
Slopes	0.006-0.25 (ft/ft)	USGS Quad maps and GIS-Hydro model	Overland flow and subbasin channels were averaged, and main-stem channels were measured
Manning's roughness coefficient ("n")	0.05-0.2	Technical Paper on Watershed Characteristics (Khine, 1998) and Field Reconnaissance	"n" was initially estimated on best judgement and field observations, but was varied during calibration. This was the most sensitive parameter
Contributing Area	0.029-0.4 (mi^2)	USGS Quad maps and GIS-Hydro model	This was computed by multiplying length by 800 ft width (overland flow enters from both sides)
Channel Shape: Subbasins	TRAP	Technical Paper on Watershed Characteristics (Khine, 1998) and Field Reconnaissance	Channels within the sub-basins were approximated as trapezoidal with gradual side-slopes
Channel Shape: Main Stem Routing	8-point X-section	Field Reconnaissance and USGS Quad maps	For the main channels the representative floodplain and channel dimensions were surveyed

3.3 Reservoir Operations

The Tridelphia and Howard T. Duckett reservoirs at Brighton and Rocky Gorge dams, respectively, are regulated by WSSC for water supply by maintaining a normal reservoir elevation. Though they are not intended to be flood control structures, WSSC maintains a minimum three-foot freeboard as a factor of safety to provide flood mitigation. The available storage based on this freeboard is on the order of an inch of watershed runoff, which can store the majority of the smaller and more frequent storm events. In general, large but infrequent storms are minimally subdued by the reservoirs and are passed through the dams, however, minor attenuation occurs. Appendix C contains relevant storage information for the reservoirs.

WSSC's Systems Control Division coordinates the releases of flood discharges. Decisions are based on hydrologic conditions monitored 24-hours a day. WSSC performs real-time monitoring of the water levels at both reservoirs, at stream gages just downstream of both reservoirs, and at bridges of U.S. Rt. 1-North and Georgia Avenue (upstream of Brighton Dam, known as Unity Gage). Recently the WSSC has implemented continuous real-time monitoring at four rain gages in the watershed and at both reservoirs. The information from the preceding 24 hours is displayed on their Supervisory Control and Data Acquisition (SCADA) computer system at the WSSC Control Center located near the City of Laurel.

WSSC's current policy is to maintain three feet of freeboard at each reservoir for flood mitigation. At the time of the Agnes Flood, WSSC's policy was to provide 2 feet of freeboard. When a storm event registers an inch of rainfall at any of the rain gages, the reservoirs' volumes are reviewed and WSSC estimates the potential storm runoff (based on the soil antecedent conditions and forecasted rainfall) and the time of concentration of the storm to reach the reservoirs. The data are entered into their reservoir management program to estimate the number of gates to open and the required openings to pass the storm runoff. WSSC attempts to provide a 2 to 3 hour delay in the peak discharge to aid the City of Laurel in implementing their Flood Emergency Plan.

Due to the influence of the upstream dams, a hydrologic analysis must consider the influence of the reservoirs. The outflow hydrographs and peak discharges can be estimated using observed reservoir records and engineering equations presented in the "Engineering Report on Tridelphia and Duckett Reservoirs" (WSSC, 1972). WSSC tracks the reservoir levels, spillway gates, and other releases on 30-minute intervals which are documented on WSSC's "Record of Water Released" forms for both reservoirs (see Appendix D for examples). The release hydrographs from the dams are then used as boundary conditions for the downstream HEC-1 models.

4.0 HYDROLOGIC MODEL CALIBRATION ANALYSIS

4.1 Calibration Analysis Methodology

The goal of the calibration process was to minimize the differences between the simulated and observed hydrographs by adjusting the watershed parameters within acceptable limits. The model was calibrated for a storm event that occurred May 5th to early May 6th in 1989. The storm provided about 4 inches of rainfall over a 12 hour period. This event generated significant runoff that lead the WSSC to release rising flood waters through the spillways of both dams.

The storm event was simulated in the HEC-1 model using the observed rainfall distribution and then compared against recorded hydrograph data from three USGS stream gages. The calibration analysis was a two-part process. The hydrograph generated from the initial data estimates served as a reference point for subsequent parameter tuning. Simulated and observed stream flow hydrographs were compared and analyzed for any similarities and differences in the following key factors, which are listed in priority:

- total runoff volume (in)
- peak storm discharge (cfs)
- timing of peak storm discharge (hrs)
- general hydrograph shape

Adjustments were made to the initial estimates of routing and loss rate parameters to attain the best match between the simulated and observed hydrographs in these four key factors. Calibration was considered complete when the simulated peak discharge and runoff volume were within 10% of the observed values from the May 1989 storm.

4.2 Rainfall Distribution

A hyetograph was developed for the storm precipitation that began late May 5th to early May 6th from 30-minute rainfall data collected at the Brighton Dam rain gage. The 4.2 inches of total rainfall fell over a 12-hour period, but predominantly was concentrated in the last 8 hours. Appendix D contains the rainfall records logged on WSSC's Brighton Dam forms. The rainfall distribution was consistent with other local rain gages. The rainfall total was also consistent with the daily rainfall measured at rain gages within/nearby the drainage area. The storm event was simulated uniformly over the watershed. In reality, however, the storm may have been staggered over the watershed.

4.3 Storm Flow Hydrograph Analysis

Three USGS stream gages with recorded stream hydrograph data from the May 1989 storm are located in the watershed, one on the Patuxent River main-stem (known as Unity gage) and the other two on the tributaries called Cattail Creek (near Glenwood) and Hawlings River (near Sandy Spring). See Figure 1 for the gage locations. The drainage areas for the Unity and Cattail gages are 35 mi^2 and 23 mi^2, respectively, and account for about 75% of the drainage into Brighton Dam. The Hawlings River gage has a drainage area of 27 mi^2. Together, these three drainage areas comprise over half of the Patuxent River watershed above the City of Laurel.

Runoff was estimated by separating the base flow from the observed hydrographs. The inflection points on the rising and receding hydrograph limbs identified the base flow. The base flows between these points were linearly interpolated. The estimated observed storm runoff totals for each gage were 1.36 inches at Patuxent River near Unity, 1.9 inches at Cattail Creek, and 1.7 inches at Hawlings River.

4.4 Calibration Analysis at USGS Stream Gages

The initial HEC-1 models of stream flow at the Cattail Creek, Hawlings River, and Patuxent River at Unity gages used estimates of the hydrologic parameters described in Section 3. The initial parameter estimates did not produce an adequate match to the observed hydrographs. Review of the results produced by these simulations showed that improvements would require:

- Using Muskingum/Cunge flow routing for both collector and main channels within sub-basins in order to improve replication of the peak flow, and where possible implementing the 8-point channel for the sub-basin and for the routing between sub-basins
- Changing Fc values to a constant 0.2 in/hr to match the silt-loam soil type consistently present in the top six inches of soil (effective depth of soil) since varying the rate caused sporadic results
- Using a higher GI to reflect less agricultural land use and more mature vegetation (i.e. tree stands, lawns, open space, etc.), particularly in the upper Patuxent River watershed

Application of the physically based Holtan Loss Rate method, the three-part flow path basin routing, and the Muskingum-Cunge main-stem routing afforded flexibility in estimating the peak flow, timing of the peak, runoff volume, and general hydrograph shape. The following is a stepwise description of the final calibration process.

First, the loss rate parameters were modified to improve the estimate of runoff volume. One inch of rainfall occurred in the preceding five days, thus the AMC condition was determined to be Group II. The AMC was used in conjunction with soil infiltration rate to estimate the available soil moisture (Khine, 1992). By modifying soil infiltration, the simulated volume approached the observed volume, but the peak flow was also influenced. The growth index was raised for the upper sub-basins of the Patuxent River to simulate additional abstraction. Once the runoff volume was within 10% of the target value, the calibration efforts were focused to match the peak flow and the timing of the peak flow. A general observation was that the loss rate was dictated by the losses controlled by vegetation as much as by soil infiltration.

The shape of the simulated hydrograph was most sensitive to changes in the various flow paths. Although length is a major component of the flow path, it was considered a set variable. Surface conditions, particularly overland flow and shallow concentrated flow, impacted the overall travel time. Ranges for Manning's roughness values in each of the three sections of flow were based on field observations. The watershed contained portions of dense tree stand mixed with heavy underbrush, which impedes and controls overland and shallow concentrated portions of the flow path even in high flow conditions. Streambeds were typically made up of cobbles and boulders, but the stream banks were heavily vegetated. Stream meandering was also prevalent in the upper three sub-watersheds. Main channels were lined with tall, heavy grasses and overhanging trees. The land surface and channel conditions caused the Manning's roughness estimates to reach the upper limits (e.g. approximately 0.8 for the overland, 0.4 for collector, and 0.1 for the channel).

Roughness values were reduced from the initial estimates for the main-stem channels (characterized as 8-point sections), in order to reduce peak flow and reduce the time to peak flow. The main-stem channel roughness values ranged from 0.06 to 0.1 and overbank roughness values varied from 0.17 to 0.2 to account for variations in streambed/streambank material and vegetation. Within the sub-basins, roughness for the collector channel was 0.40 and for the main channel was 0.20. The channel roughness influenced both peak timing and hydrograph shape. Justification of the coefficient selection is provided in Appendix B.

Table 3 - Summary of Hydrologic Parameters for Calibrated HEC-1 Models

	Area (sq mi)	Length (ft)			Slope (ft/ft)			Percent Impervious (%)	Growth Index (GI)	Basal Area (BA)	Contributing Area	Permeability (Fc)	Available Soil Moisture	Manning's Roughness					
														Main Stem Channel			Subbasin Network		
		Overland	Collector	Main Channel	Overland	Collector	Main Channel							Left Overbank	Main Stem	Right Overbank	Overland	Collector	Main
CCR1	2.30	400	1175	10800	0.1160	0.0297	0.0137	7	0.7	0.370	0.034	0.2	0.96				0.8	0.4	0.2
CCR2	1.37	400	2860	4700	0.2319	0.0209	0.0126	11	0.7	0.354	0.082	0.2	0.96				0.8	0.4	0.2
CCR3	2.41	400	1228	10700	0.1853	0.0266	0.0129	11	0.7	0.398	0.035	0.2	0.96				0.8	0.4	0.2
CCR4	1.78	400	1093	6000	0.1310	0.0269	0.0131	9	0.7	0.515	0.031	0.2	0.96				0.8	0.4	0.2
CCR5a	1.64	400	1114	7700	0.2086	0.0316	0.0196	7	0.7	0.468	0.032	0.2	0.96				0.8	0.4	0.2
CCR5b	1.69	400	1410	9000	0.1640	0.0280	0.0109	10	0.7	0.504	0.040	0.2	0.96				0.8	0.4	0.2
CCR6	1.62	400	1567	6600	0.2086	0.0256	0.0129	7	0.7	0.483	0.045	0.2	0.96				0.8	0.4	0.2
CCR7	2.05	400	1442	7600	0.1310	0.0233	0.0168	8	0.7	0.422	0.041	0.2	0.96				0.8	0.4	0.2
CCR8a	2.34	400	1356	9200	0.1860	0.0235	0.0111	10	0.7	0.448	0.039	0.2	0.96				0.8	0.4	0.2
CCR8b	1.31	400	955	6900	0.0926	0.0417	0.0124	6	0.7	0.512	0.027	0.2	0.96				0.8	0.4	0.2
CCR9	1.36	400	1920	10400	0.2086	0.0273	0.0047	9	0.7	0.508	0.055	0.2	0.96	0.2	0.1	0.2	0.8	0.4	
CCR10	1.62	400	793	10500	0.2319	0.0347	0.0084	10	0.7	0.524	0.023	0.2	0.96	0.2	0.1	0.2	0.8	0.4	
CCR11	1.36	400	1322	8600	0.1970	0.0298	0.0065	6	0.7	0.464	0.038	0.2	0.96	0.2	0.1	0.2	0.8	0.4	
CCR12	2.38	400	1619	10600	0.0693	0.0325	0.0115	10	0.7	0.440	0.046	0.2	0.96	0.2	0.1	0.2	0.8	0.4	
CCR13	2.00	400	1356	4600	0.1160	0.0239	0.0107	16	0.7	0.366	0.039	0.2	0.96				0.8	0.4	0.2
HRV1	3.02	400	1435	11700	0.0460	0.0201	0.0090	2	0.7	0.436	0.041	0.2	0.96				0.8	0.4	0.2
HRV2	2.06	400	1508	6600	0.0200	0.0154	0.0100	5	0.7	0.429	0.043	0.2	0.96				0.8	0.4	0.2
HRV3	1.32	400	1086	6500	0.0495	0.0248	0.0108	5	0.7	0.491	0.031	0.2	0.96				0.8	0.4	0.2
HRV4a	1.60	400	2233	5200	0.0141	0.0176	0.0044	15	0.7	0.345	0.064	0.2	0.96				0.8	0.4	0.2
HRV4b	1.60	400	1122	10400	0.0354	0.0196	0.0119	27	0.7	0.455	0.032	0.2	0.96				0.8	0.4	0.2
HRV5	2.24	400	1167	7800	0.0500	0.0183	0.0090	2	0.7	0.518	0.033	0.2	0.96				0.8	0.4	0.2
HRV6	1.26	400	1880	4900	0.0212	0.0267	0.0118	38	0.7	0.405	0.054	0.2	0.96				0.8	0.4	0.2
HRV7	1.12	400	1575	2800	0.0141	0.0236	0.0143	38	0.7	0.269	0.045	0.2	0.96				0.8	0.4	0.2
HRV8	1.89	400	1346	6100	0.0100	0.0199	0.0121	28	0.7	0.412	0.039	0.2	0.96				0.8	0.4	0.2
HRV9	1.19	400	1113	5500	0.0200	0.0272	0.0149	4	0.7	0.538	0.032	0.2	0.96				0.8	0.4	0.2
HRV10a	2.96	400	1294	14300	0.0100	0.0173	0.0041	6	0.7	0.441	0.037	0.2	0.96	0.19	0.08	0.19	0.8	0.4	
HRV10b	2.44	400	1124	10500	0.0200	0.0252	0.0070	5	0.7	0.539	0.032	0.2	0.96	0.19	0.08	0.19	0.8	0.4	
HRV11a	1.62	400	1344	8300	0.0600	0.0173	0.0041	7	0.7	0.584	0.039	0.2	0.96	0.19	0.08	0.19	0.8	0.4	
HRV11b	1.80	400	1738	8300	0.0400	0.0252	0.0070	6	0.7	0.528	0.050	0.2	0.96	0.19	0.08	0.19	0.8	0.4	
HRV12a	1.30	400	1738	8000	0.0354	0.0291	0.0017	7	0.7	0.584	0.050	0.2	0.96	0.19	0.08	0.19	0.8	0.4	
HRV12b	0.77	400	1444	8000	0.0354	0.0291	0.0017	7	0.7	0.584	0.041	0.2	0.96	0.19	0.08	0.19	0.8	0.4	
PRU1	2.04	400	1467	9400	0.2553	0.0368	0.0133	1	1.0	0.544	0.042	0.2	0.96				0.8	0.4	0.15
PRU2	1.83	400	1169	9300	0.1320	0.0318	0.0123	4	1.0	0.578	0.034	0.2	0.96				0.8	0.4	0.15
PRU3	1.43	400	1444	4700	0.1970	0.0380	0.0216	2	1.0	0.416	0.041	0.2	0.96				0.8	0.4	0.15
PRU4	1.76	400	1000	7100	0.2319	0.0427	0.0203	7	1.0	0.557	0.029	0.2	0.96				0.8	0.4	0.15
PRU5	1.72	400	1140	9800	0.1310	0.0349	0.0191	2	1.0	0.487	0.033	0.2	0.96				0.8	0.4	0.15
PRU6	2.21	400	863	11200	0.1640	0.0345	0.0179	2	1.0	0.401	0.025	0.2	0.96				0.8	0.4	0.15
PRU7	2.39	400	1282	16700	0.2319	0.0515	0.0081	5	1.0	0.543	0.037	0.2	0.96	0.17	0.06	0.17	0.8	0.4	
PRU8a	2.57	400	1512	10800	0.0233	0.0291	0.0082	0	1.0	0.484	0.043	0.2	0.96	0.17	0.06	0.17	0.8	0.4	
PRU8b	1.63	400	1473	7800	0.1393	0.0289	0.0067	0	1.0	0.659	0.042	0.2	0.96	0.17	0.06	0.17	0.8	0.4	
PRU9	2.40	400	1627	9200	0.1640	0.0330	0.0121	0	1.0	0.548	0.047	0.2	0.96	0.17	0.06	0.17	0.8	0.4	
PRU10	2.60	400	919	10700	0.0212	0.0181	0.0107	7	1.0	0.318	0.026	0.2	0.96				0.8	0.4	0.15
PRU11	3.35	400	1522	15000	0.1160	0.0386	0.0072	3	1.0	0.562	0.044	0.2	0.96	0.17	0.06	0.17	0.8	0.4	
PRU12	1.14	400	1517	3300	0.0071	0.0236	0.0124	1	1.0	0.409	0.044	0.2	0.96				0.8	0.4	0.15
PRU13	0.74	400	1120	2900	0.0330	0.0165	0.0190	11	1.0	0.489	0.032	0.2	0.96				0.8	0.4	0.15
PRU14	3.50	400	1335	11400	0.0660	0.0307	0.0129	8	1.0	0.495	0.038	0.2	0.96				0.8	0.4	0.15
PRU15	2.07	400	1364	7100	0.0926	0.0311	0.0141	13	1.0	0.549	0.039	0.2	0.96				0.8	0.4	0.15
PRU16	2.28	400	962	8300	0.0980	0.0358	0.0095	15	1.0	0.515	0.028	0.2	0.96				0.8	0.4	0.15
PRU17	2.77	400	1000	33700	0.1400	0.0314	0.0020	16	1.0	0.488	0.029	0.2	0.96				0.8	0.4	0.15
PRU18	2.74	400	1092	30000	0.0990	0.0221	0.0011	29	1.0	0.522	0.031	0.2	0.96				0.8	0.4	0.15
PRU19	1.53	400	1350	12300	0.0071	0.0206	0.0075	32	1.0	0.528	0.039	0.2	0.96				0.8	0.4	0.15

Table 3 - Summary of Hydrologic Parameters for Calibrated HEC-1 Models (continued)

	Area (sq mi)	Length (ft)			Slope (ft/ft)			Percent Impervious (%)	Growth Index (GI)	Basal Area (BA)	Contributing Area	Permeability (Fc)	Available Soil Moisture	Manning's Roughness					
														Main Stem Channel			Subbasin Network		
		Overland	Collector	Main Channel	Overland	Collector	Main Channel							Left Overbank	Main Stem	Right Overbank	Overland	Collector	Main
CBR1	2.43	400	1382	7700	0.0980	0.0231	0.0111	2	1.0	0.399	0.040	0.2	0.96				0.8	0.4	0.15
CBR3	2.05	400	1138	8800	0.0926	0.0281	0.0183	3	1.0	0.403	0.033	0.2	0.96				0.8	0.4	0.15
CBR4	2.94	400	1346	18600	0.0700	0.0388	0.0090	2	1.0	0.442	0.039	0.2	0.96	0.17	0.06	0.17	0.8	0.4	
CBR5	1.58	400	893	7500	0.0693	0.0320	0.0096	2	1.0	0.669	0.026	0.2	0.96	0.17	0.06	0.17	0.8	0.4	
PRL1	2.16	400	1150	13900	0.0226	0.0386	0.0062	9	1.0	0.655	0.033	0.2	0.96	0.17	0.06	0.17	0.8	0.4	
PRL2	2.36	400	1592	10900	0.0212	0.0274	0.0069	8	1.0	0.725	0.046	0.2	0.96	0.17	0.06	0.17	0.8	0.4	
PRL3	1.74	400	1160	8700	0.1160	0.0355	0.0124	7	1.0	0.639	0.033	0.2	0.96	0.17	0.06	0.17	0.8	0.4	
PRL4	1.80	400	1208	7300	0.0100	0.0355	0.0171	10	1.0	0.636	0.035	0.2	0.96				0.8	0.4	0.2
PRL5	1.76	400	1450	10500	0.0460	0.0338	0.0128	15	1.0	0.614	0.042	0.2	0.96				0.8	0.4	0.2
PRL6	1.95	400	1417	8900	0.0460	0.0449	0.0111	16	1.0	0.571	0.041	0.2	0.96				0.8	0.4	0.2
PRL7	2.29	400	1181	10400	0.0467	0.0358	0.0177	10	1.0	0.647	0.034	0.2	0.96				0.8	0.4	0.2
PRL8	2.37	400	1367	11800	0.0980	0.0378	0.0092	12	1.0	0.671	0.039	0.2	0.96				0.8	0.4	0.2
PRL9	1.53	400	1950	5200	0.0980	0.0279	0.0208	9	1.0	0.554	0.056	0.2	0.96				0.8	0.4	0.2
PRL10	2.36	400	1738	8600	0.1619	0.0458	0.0111	14	1.0	0.563	0.050	0.2	0.96				0.8	0.4	0.2
PRL11	2.26	400	1800	12100	0.0693	0.0398	0.0081	10	1.0	0.653	0.052	0.2	0.96				0.8	0.4	0.2
PRL12	1.83	400	1129	9100	0.1853	0.0207	0.0105	25	1.0	0.524	0.032	0.2	0.96				0.8	0.4	0.2
PRL13a	0.90	400	1550	5100	0.1310	0.0422	0.0140	18	1.0	0.628	0.044	0.2	0.96				0.8	0.4	0.2
PRL13b	0.90	400	1550	6200	0.1310	0.0422	0.0026	18	1.0	0.628	0.044	0.2	0.96	0.12	0.06	0.12	0.8	0.4	
PRL14	2.29	400	1353	13700	0.0926	0.0213	0.0115	31	1.0	0.470	0.039	0.2	0.96				0.8	0.4	0.2
PRL15	2.26	400	1742	8900	0.0980	0.0236	0.0148	37	1.0	0.461	0.050	0.2	0.96	0.12	0.05	0.08	0.8	0.4	
PRL16	1.92	400	2000	6900	0.0926	0.0256	0.0095	46	1.0	0.343	0.057	0.2	0.96	0.12	0.05	0.08	0.8	0.4	
PRL17	2.34	400	1406	12600	0.1386	0.0297	0.0151	46	1.0	0.343	0.040	0.2	0.96				0.8	0.4	0.2
PRL18	1.76	400	2338	8700	0.0660	0.0268	0.0121	37	1.0	0.436	0.067	0.2	0.96				0.8	0.4	0.2
PRL19	2.26	400	1005	10200	0.1970	0.0184	0.0077	31	1.0	0.476	0.029	0.2	0.96				0.8	0.4	0.2
PRL20	1.63	400	1255	10900	0.1160	0.0273	0.0060	11	1.0	0.720	0.036	0.2	0.96	0.12	0.05	0.12	0.8	0.4	
PRL21	1.47	400	2229	4900	0.2093	0.0161	0.0114	29	1.0	0.519	0.064	0.2	0.96				0.8	0.4	0.2
PRL22	1.27	400	1460	3800	0.0693	0.0221	0.0095	24	1.0	0.574	0.042	0.2	0.96				0.8	0.4	0.2

After calibrating these variables, slight adjustments to the 8-point channel enabled fine-tuning of the general hydrograph shape. Only the channel section was adjusted to account for varying channel shapes. Table 3 summarizes the final sub-basin parameters.

Comparison of the simulated versus observed hydrographs at the Patuxent River/Unity, Cattail Creek, and Hawlings River gages are shown in Figures 4 to 6, respectively. Also represented in these figures is the observed May 1989 storm hyetograph divided into abstraction and runoff. Table 4 summarizes the calibration results. Appendix E contains the HEC-1 model input and output for the calibration simulation. The following describes some of the unique calibration characteristics of each gage's watershed.

Table 4 – Summary of Calibration Results

Cattail Creek	Observed	3516	13.00	1.95
	Simulated	3815	13.17	1.66
	Deviation (%)	8.5	1.3	14.9
Patuxent River @ Unity	Observed	4635	12.33	1.36
	Simulated	4876	13.17	1.38
	Deviation (%)	5.2	6.8	1.4
Hawlings River	Observed	3513	10.33	1.72
	Simulated	3652	9.67	1.60
	Deviation (%)	4.0	6.4	7.2

The Hawlings River model required the least modifications. There is significant agricultural land use in the watershed, thus the Growth Index was set to 0.7 to account for seasonal vegetation. Manning's roughness for the channel and overbank were 0.08 and 0.19, respectively. The Cattail Creek watershed also contains significant agricultural land use and utilized a Growth Index of 0.7. Channel and overbank roughness values were set to 0.1 and 0.2, respectively.

Although Patuxent River at Unity watershed had a similar soil texture, it contained less agricultural land and possibly greater forest cover compared to the Cattail Creek and Hawlings River watersheds. For this reason, it did not follow the same calibration pattern. Soil permeability was held consistent to those of the other two watersheds, but Growth Index was increased to 1.0 to account for perennial vegetation. Manning's roughness for the channel and overbank were set to 0.06 and 0.17, respectively, for the entire reach of the upper Patuxent River.

Although the observed and simulated hydrographs do have slight deviations for each of the gages, the results are considered good. In fact, the rising limbs matched very closely and the peak timings differed by less than 10 %. As Table 4 shows, the runoff volume simulated at Cattail Creek was slightly outside the 10% error bounds.

Figure 4: Calibration Hydrographs: Patuxent River at Unity

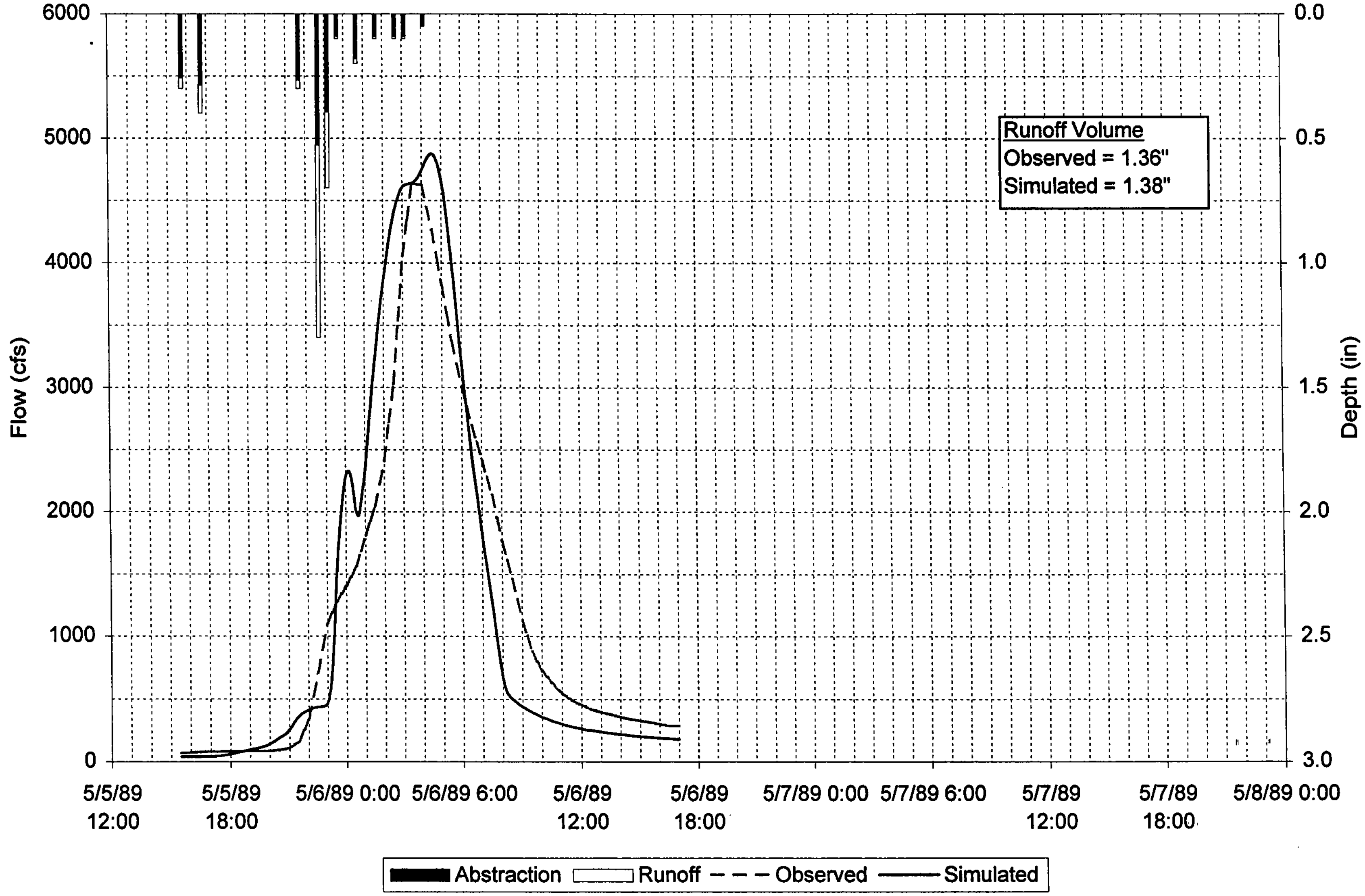


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Figure 5: Calibration Hydrographs: Cattail Creek

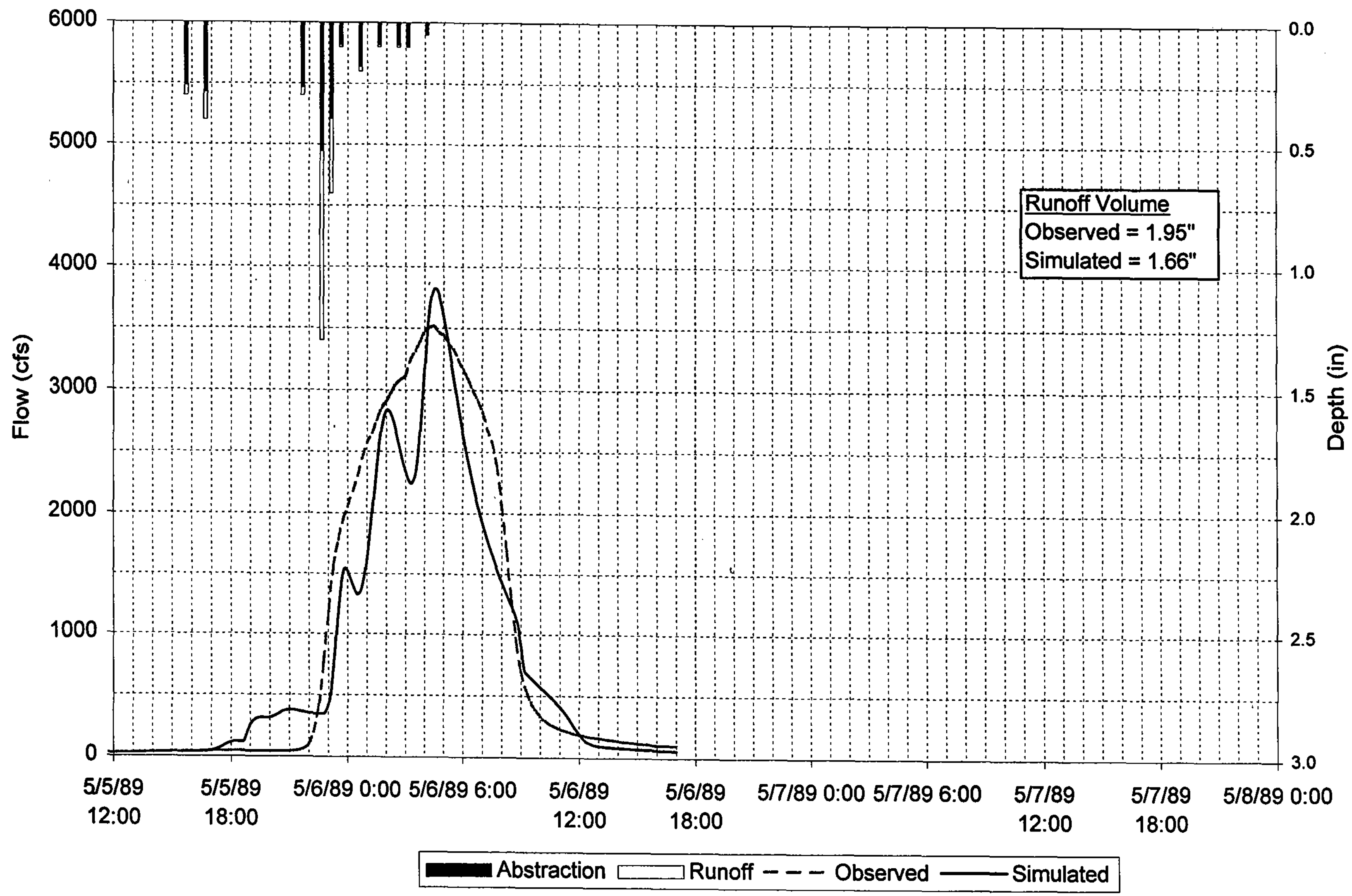
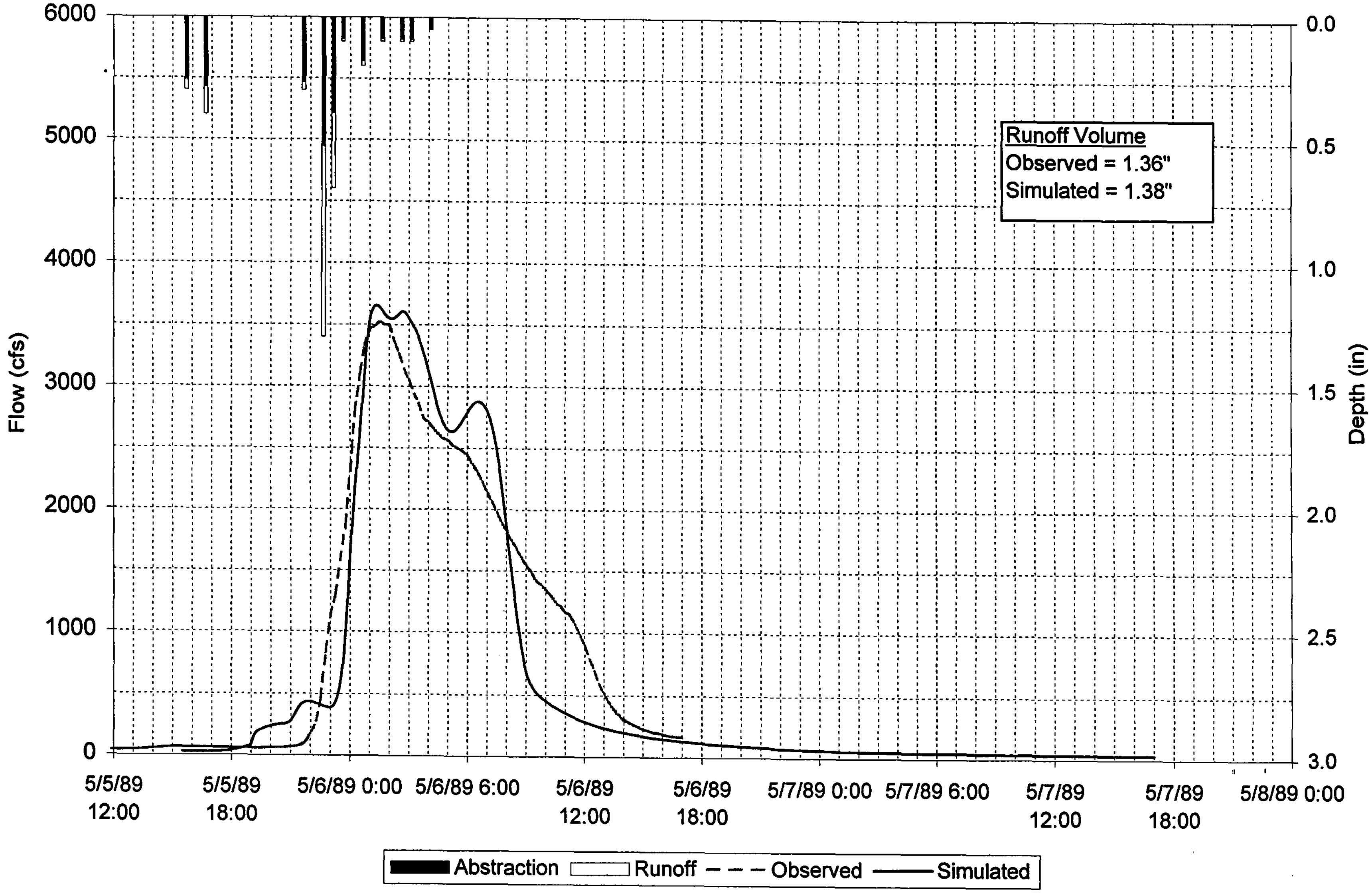


Figure 6: Calibration Hydrographs: Hawlings River



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4.5 Comparison Analysis at WSSC Reservoirs

Following the calibration of the watershed parameters, the next step was to simulate the inflow at WSSC's dams. Records of operation containing the observed reservoir pool elevations at Brighton Dam and Rocky Gorge Dam for the May 1989 storm (see Appendix D) along with WSSC's engineering equations for the spillway gates were used to calculate the release hydrographs. Figure 7 shows the May 1989 HEC-1 simulated storm inflow and the calculated outflow hydrographs at Brighton Dam, and also include the reservoir flood elevations.

The water levels were also used to estimate the change in reservoir storage (see Appendix C) during this storm event. Based on this information, the estimated storm inflow from the observed records was calculated by the equation:

$$I = O + \Delta S/\Delta t$$

where:
- I = estimated inflow to the reservoir (cfs)
- O = calculated outflow from the reservoir based on spillway data (cfs)
- ΔS = calculated change in the reservoir storage based on spillway data (ft^3)
- Δt = time increment (sec)

A good match occurred between the simulated HEC-1 and the estimated inflow peak discharges. This provided sufficient evidence that the calculated release hydrograph from Brighton Dam based on WSSC's data could be used as the starting condition for the downstream HEC-1 model to the Rocky Gorge Dam.

The next HEC-1 model included the calibrated Hawlings River watershed and the drainage along the Patuxent River main-stem between the dams. Figure 8 shows the May 1989 HEC-1 simulated storm inflow and the calculated outflow hydrographs at Rocky Gorge Dam.

Similar to the Brighton Dam, the water levels were used to estimate the storm inflow based on WSSC's records. The results provided a sufficient match between the simulated and estimated peak discharge inflow so that the calculated release hydrograph from Rocky Gorge Dam could be used as a starting condition for the downstream HEC-1 model through the City of Laurel.

The final HEC-1 model included the smaller tributaries and the drainage along the Patuxent River main-stem downstream of the Rocky Gorge dam. Because there was no calibration performed for the lower Patuxent River drainage, the results of the upstream calibration were used in developing the downstream hydrologic model. Table 3 includes a summary of these parameters. The simulated hydrograph at the City of Laurel is shown in Figure 9. This figure also shows the hydrograph for a continuous watershed simulation without considering the dampening effects of the reservoirs for the May 1989 storm event.

Figure 7: Calibration Hydrographs: Brighton Dam Inflow and Outflow

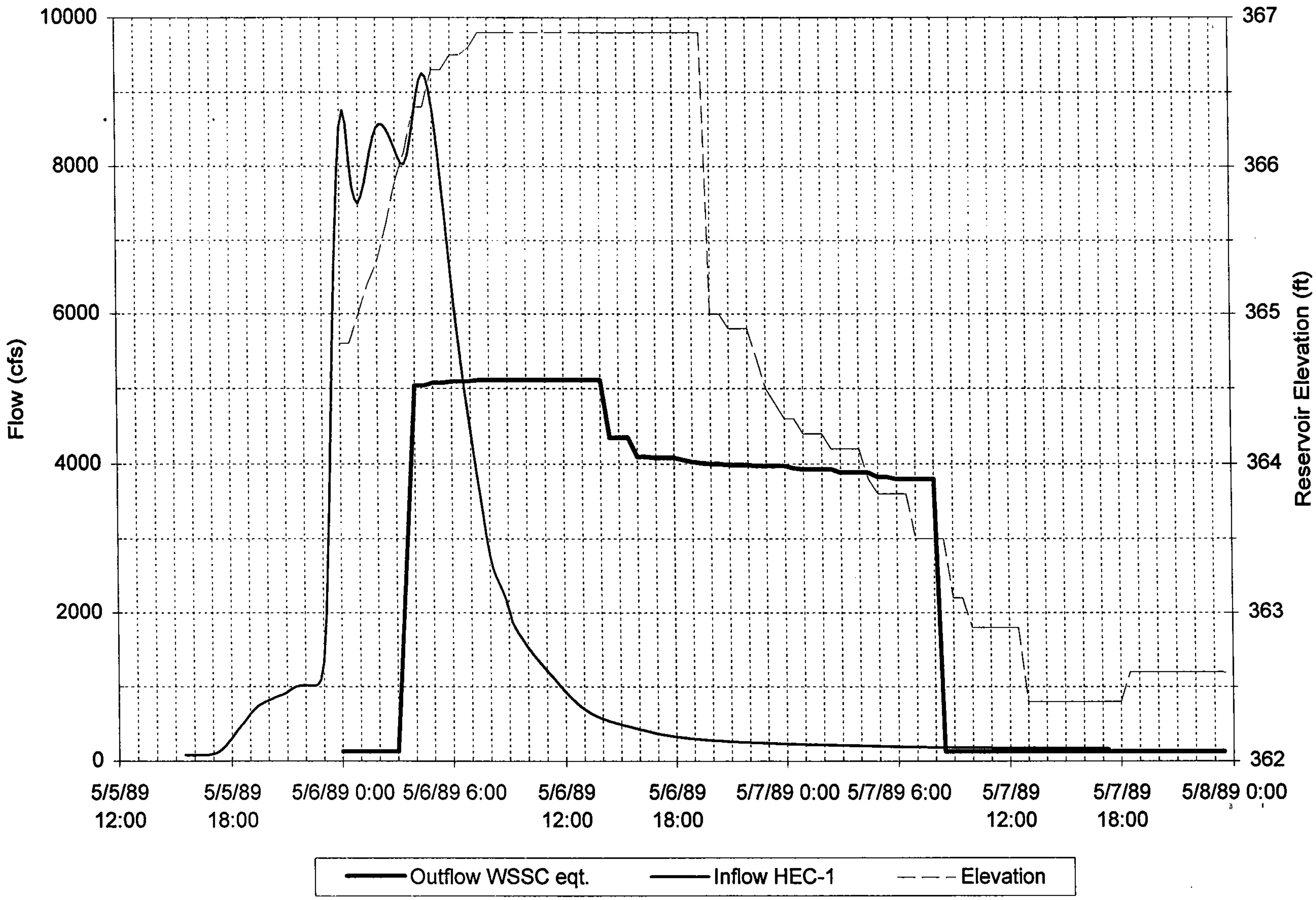

Figure 8: Calibration Hydrographs: Rocky Gorge Dam Inflow and Outflow

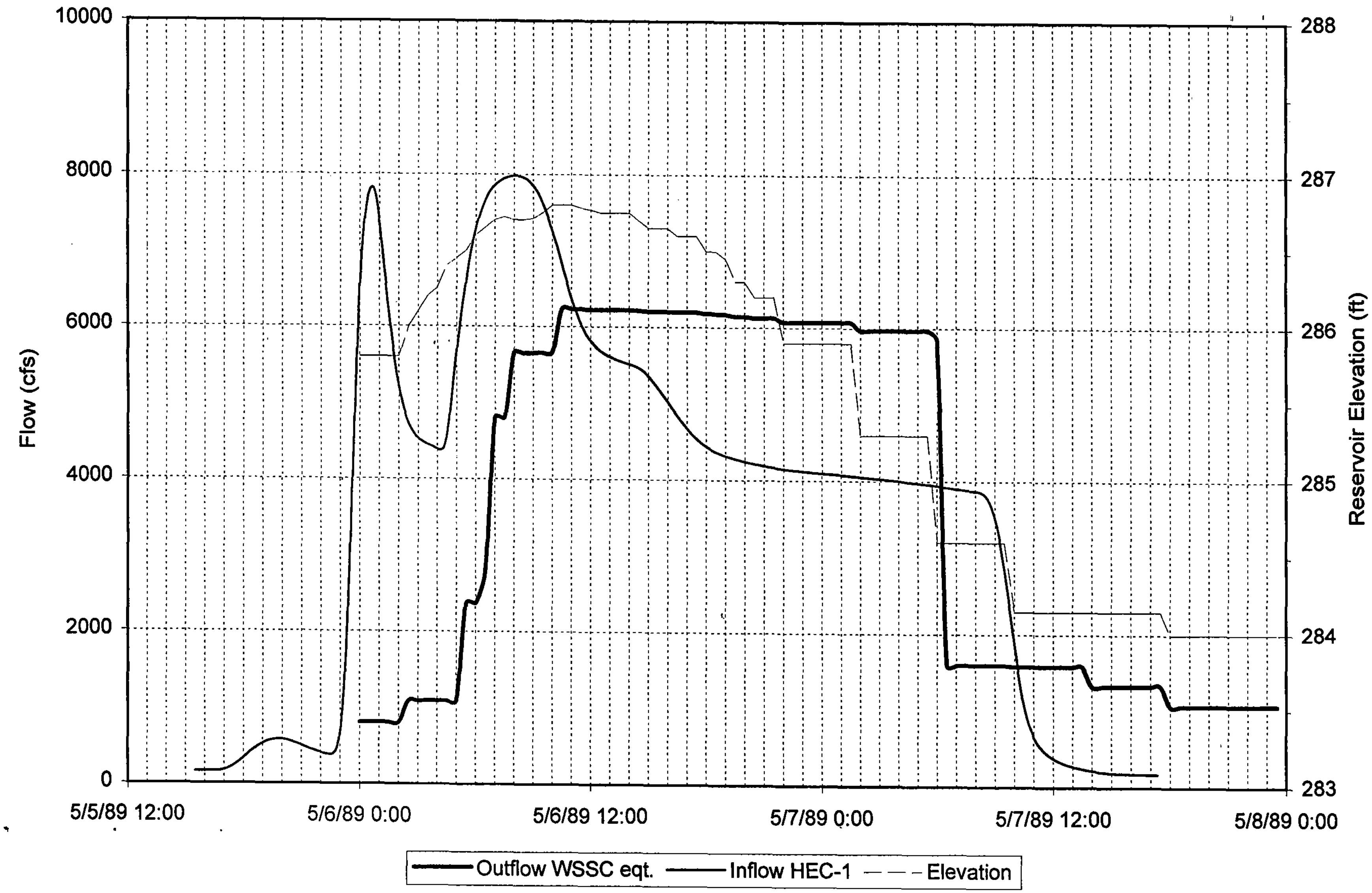

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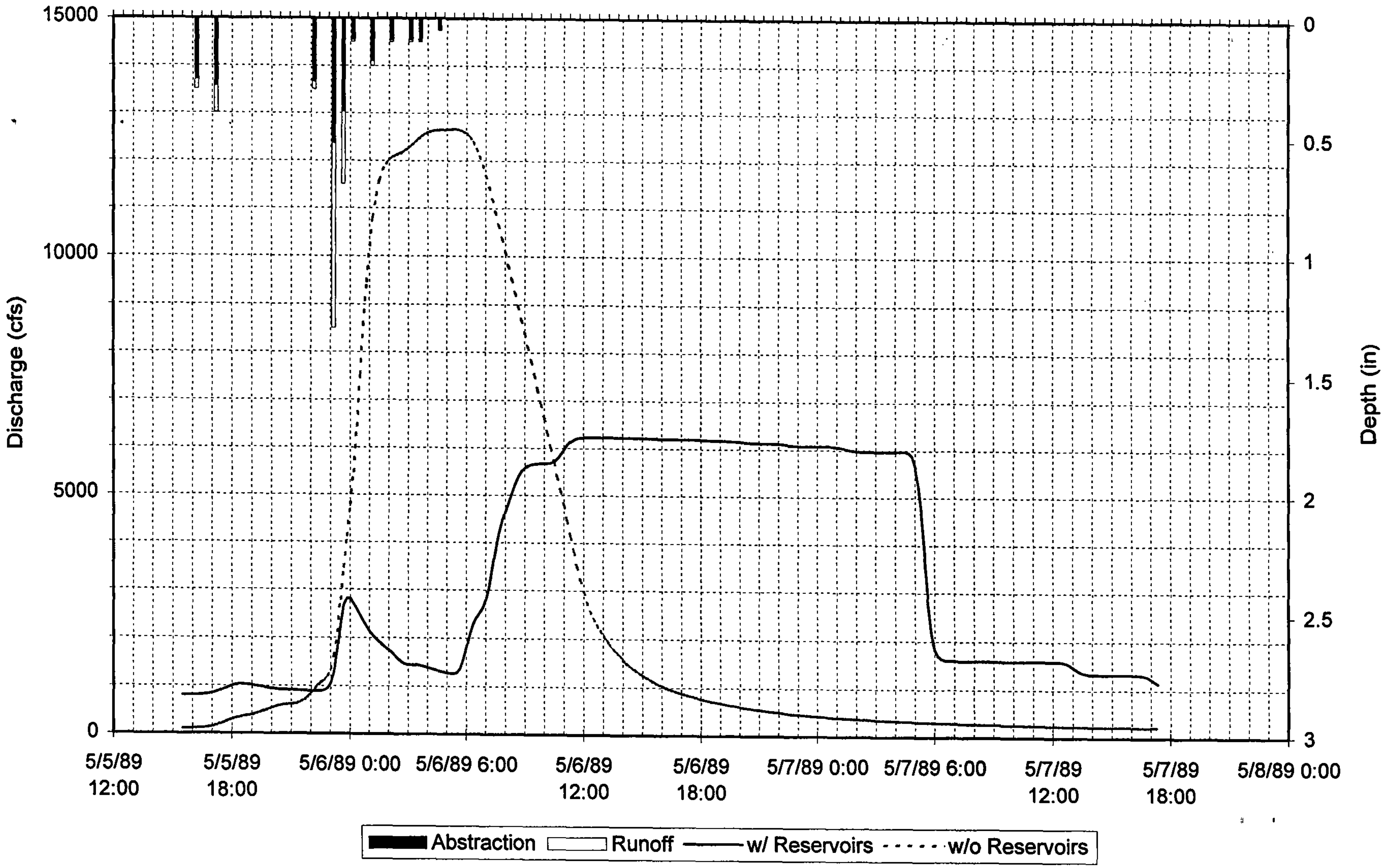
Figure 9: Calibration Hydrographs: Comparison at City of Laurel



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4.6 Parameter Sensitivity Analysis

To test the influence of each individual hydrologic parameter on the model results, a sensitivity analysis was performed. This analysis aids in defining how uncertainty in estimating parameters affects the conclusions of the hydrologic analyses. The calibrated HEC-1 model for the Patuxent River at Unity was the subject of the sensitivity analysis. The parameters that were judged to be the most significant source of uncertainty in the Holtan Loss Equation were soil infiltration (Fc) and vegetative infiltration rate (GI·BA), and in the Muskingum-Cunge routing method was the Manning's roughness coefficient (n).

Soil infiltration

The soil infiltration rate was varied uniformly by ± 25% for all sub-basins. Avail soil moisture was changed accordingly based on field capacity (AMCII). This caused slightly less than 20% change in the simulated peak discharge for both the increase and decrease respectively, but slightly more than 20% change in the simulated runoff. The results are shown in Figure 10. A change in soil infiltration rate produces a significant change in runoff volume and peak flow, but insignificant variance in time to peak.

Vegetative Infiltration (GI·BA)

Another loss rate sensitivity simulation was performed for the combined factor of Growth Index and Basal Area, which describe the influence of vegetation on infiltration rate. Figure 11 shows the results when this factor was varied by ± 25%. The changes produced about 6% variance in peak discharge and about 10% in runoff for both the increase and decrease respectively, but had little effect on time to peak and overall shape of the hydrograph.

Manning's Roughness Coefficients

The Manning's roughness coefficient was a sensitive parameter that impacted the timing and the peak discharge of the hydrograph. Two separate cases were studied when testing this parameter.

First the roughness coefficients were varied by ±25% for only the main-stem channel routing. These results are shown in Figure 12. Varying the roughness values changed the peak discharge by less than 10% and the timing by less than one half-hour. The time to peak was shortened for reduced values, but lengthened for increased values.

Next the roughness coefficients were varied by ±25% for both the main-stem channel routing and sub-basin's main channel. These results are shown in Figure 13. Varying the roughness values changed the peak discharge by less than 15% and the timing by about one half-hour. The time to peak was shortened for reduced values, but lengthened for increased values.

Inter-basin Routing

An alternative simulation was to identify the effect of having no inter-basin routing in the models. This caused the peak discharge to be increased three-fold and the time to peak to be shortened by about 4 hours. This indicates that the upstream stream valley attenuates the Patuxent River hydrograph.

Figure 10: Sensitivity Hydrographs: Soil Infiltration / Available Soil Moisture

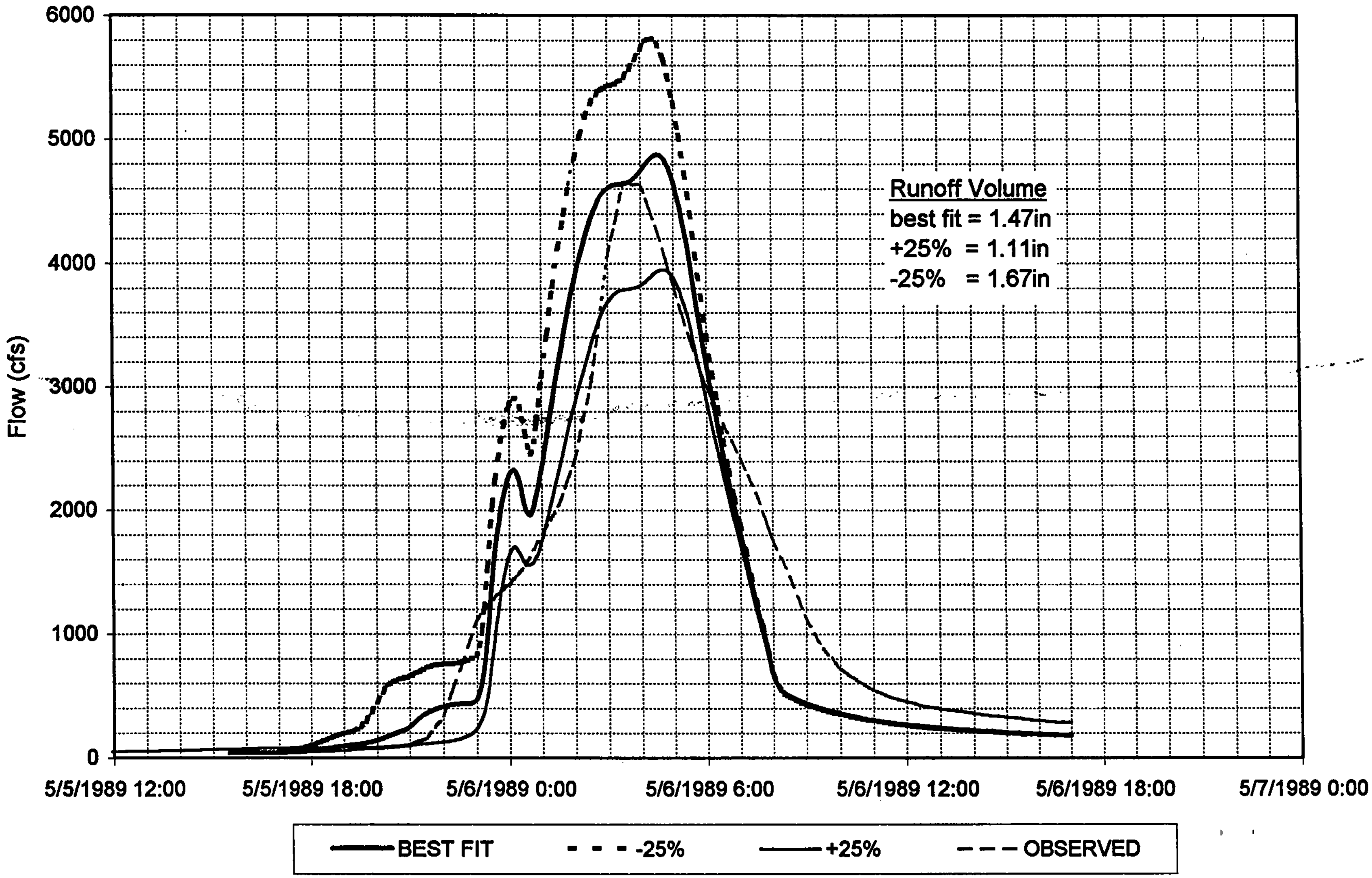

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Figure 11: Sensitivity Hydrographs: Vegetative Infiltration (GI,BA)

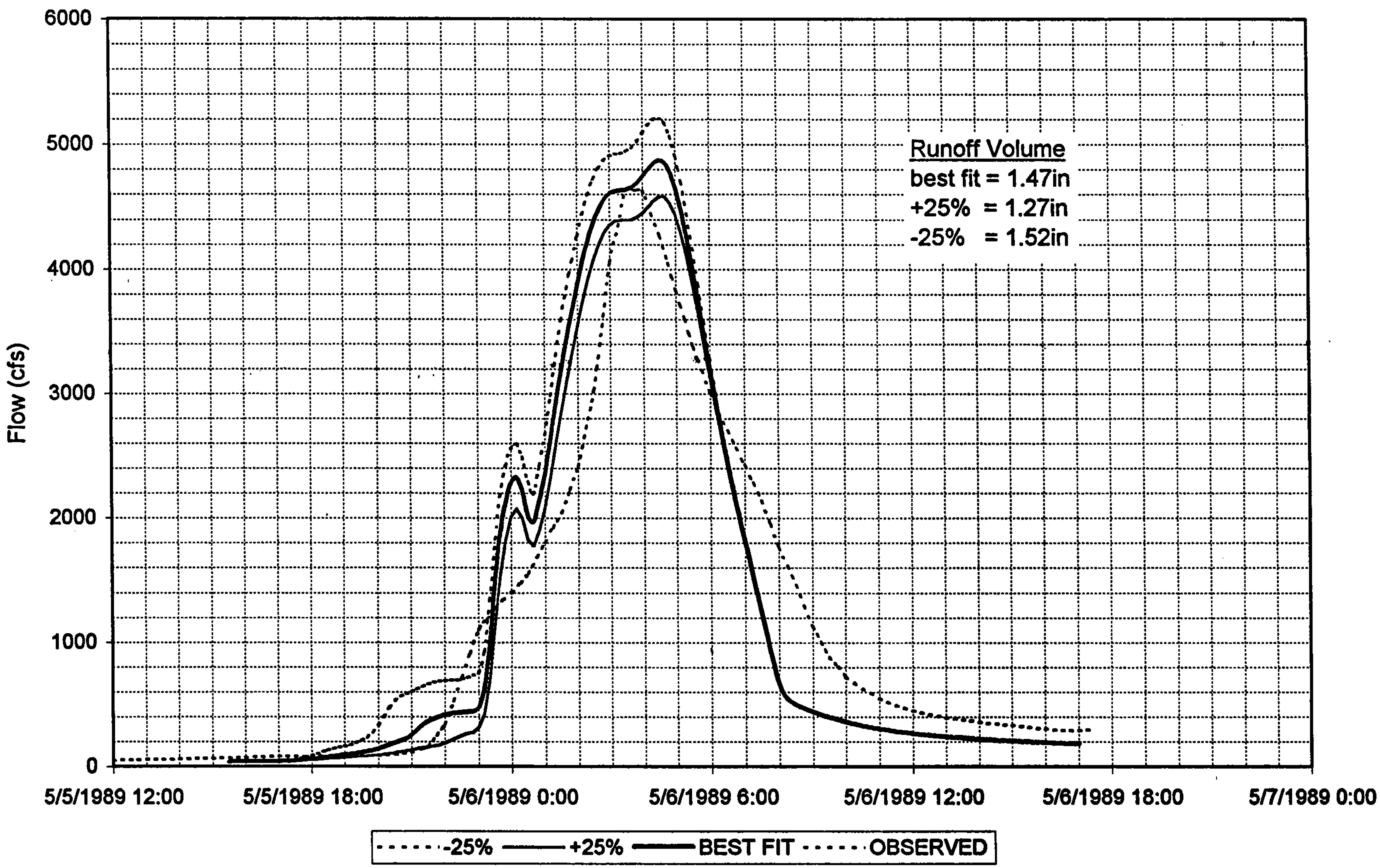

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Figure 12: Sensitivity Hydrographs: Main-Stem Channel Roughness

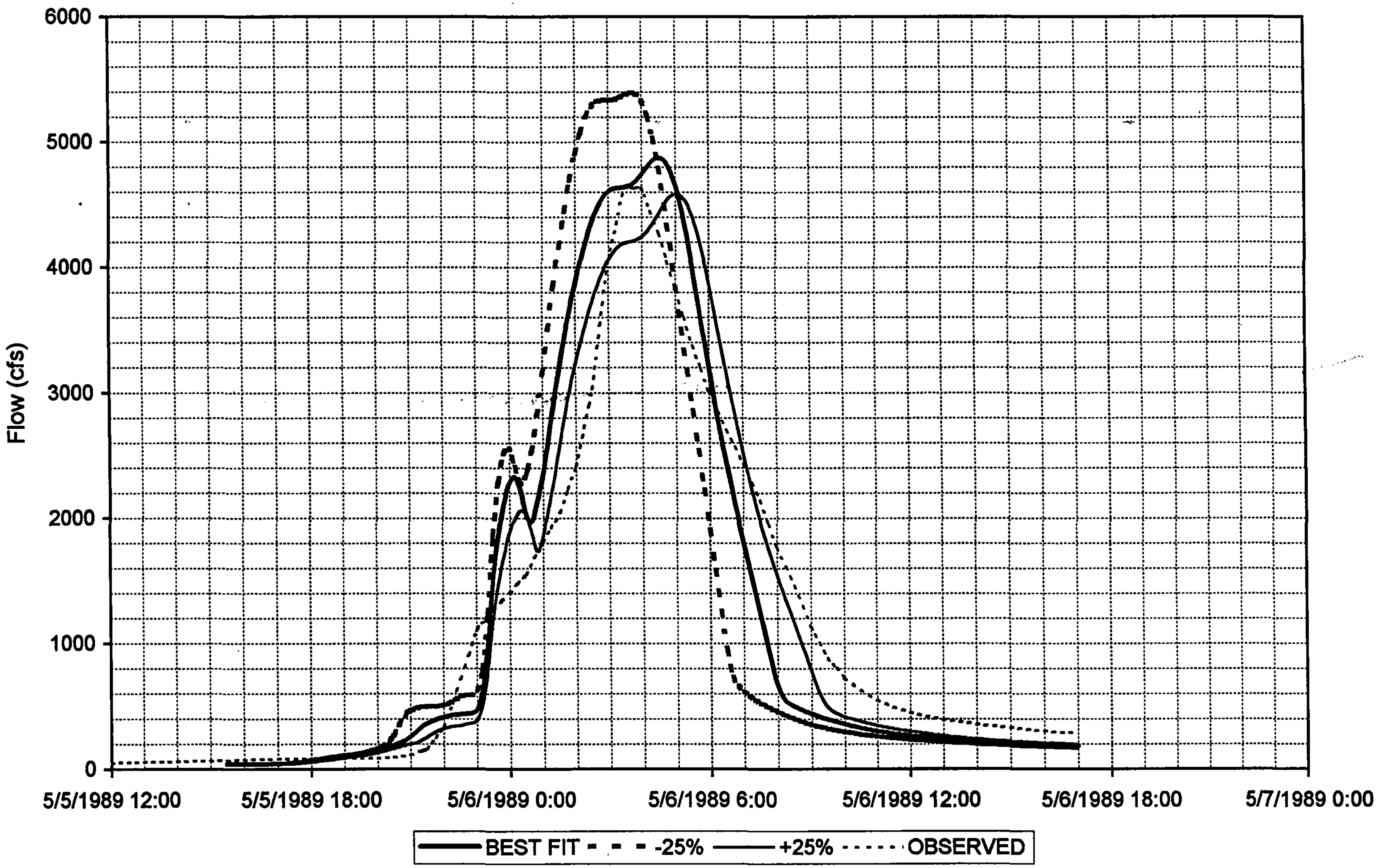

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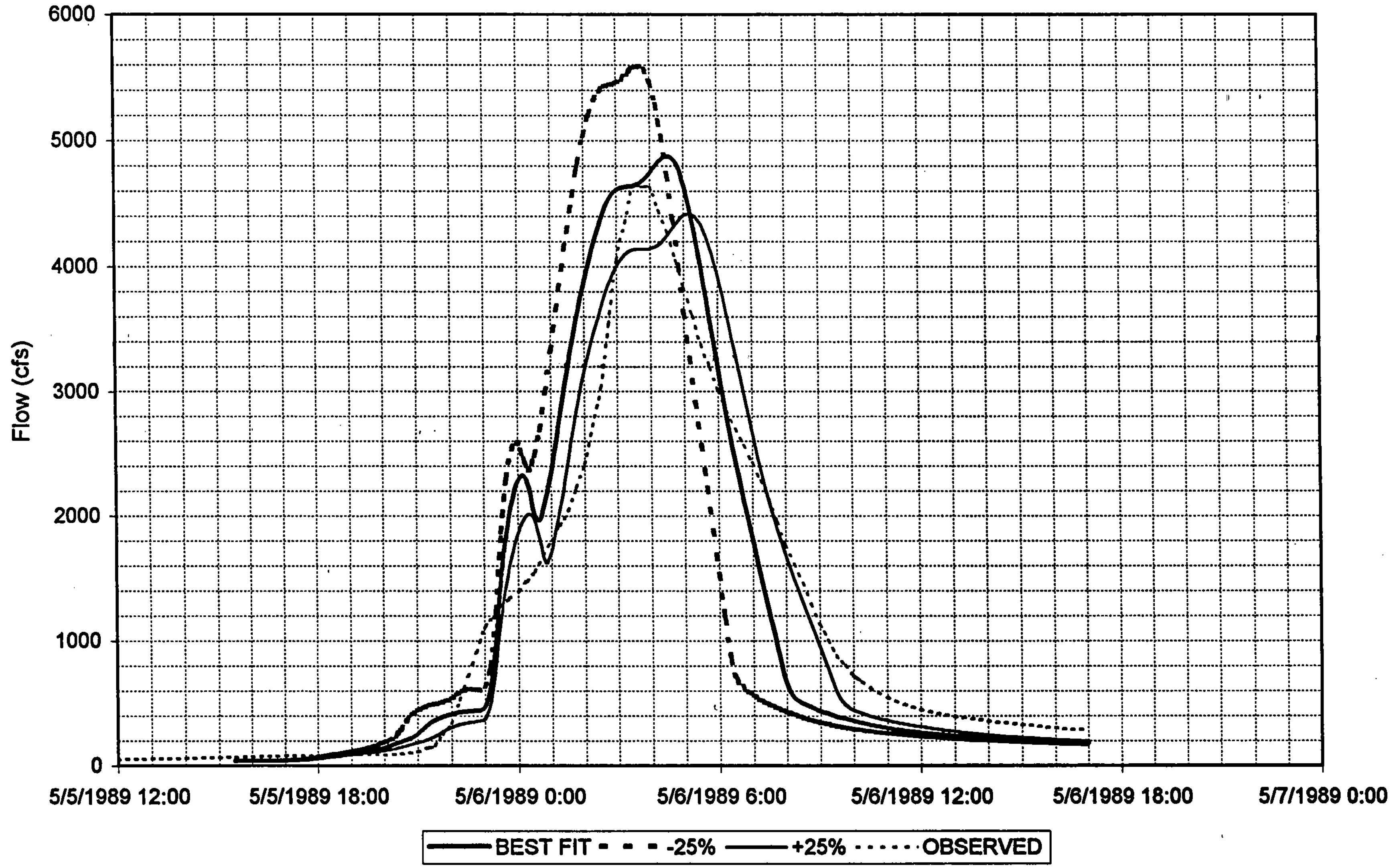
Figure 13: Sensitivity Hydrographs: Main-Stem and Main Channel Roughness



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5.0 HYDROLOGIC MODEL VERIFICATION ANALYSIS

5.1 Verification Analysis Methodology

A verification analysis provides a test of the calibrated hydrologic model by simulating a different storm event and comparing the results to observed flood data. The goal of the verification process was to recreate, with sufficient accuracy, the observed peak flows at the USGS stream gages as well as the flood levels at the reservoirs. The model calibration was performed for a May 1989 storm event with a return interval on the order of a 10- to 20-yr flood. The model verification was performed for the June 1972 event, which is the largest flood of record in the City of Laurel and at the upstream reservoirs.

Matching the Agnes Storm provides confidence in the HEC-1 model performance over for large storm events. The verification was performed to match peak discharges at the USGS gages located on the Patuxent River at Unity (01591000), and Cattail Creek at Roxbury Mills (01591500). These gages provide excellent comparison, because the existing watersheds are predominantly undeveloped which is similar to the conditions circa-1972. Verification was considered complete when the simulated peak discharges were within ±10% of the observed values.

5.2 Rainfall Distribution

The Agnes Storm was an extreme event with intense and heavy rainfall. The USGS's Post-Flood Report cited that the Patuxent River headwaters experienced nearly a total 13" of rain for this storm. However, the lower parts of the watershed near the City of Laurel received less than 6". Rainfall isohyetal maps of Maryland for the Agnes Flood were reviewed, including the 24-hr maximum rainfall amounts, as shown in Figure 14. A general observation was the rainfall was below 8" to the east of I-95, but rainfall was in excess of 8" to the west. Also, the upper Patuxent River watershed was aligned in the path of the highest rainfall totals observed in Maryland.

The 24-hr rainfall for the upper Patuxent River watershed was set to 9.36", which was recorded at Brighton Dam as shown in the "Record of Water Released" forms. Based on these records, the majority of this rainfall fell over a 15-hr period. As Figure 14 shows, rainfall totals from the Agnes storm varied across the state, however the Brighton Dam rainfall is consistent with the path of the peak rainfall over this watershed. The storm event was simulated uniformly over the watershed in the HEC-1 models based on Figure 14.

A hyetograph was developed from early June 21st to June 22nd, 1972 based on the rainfall measurements at Brighton Dam. The rainfall data was recorded over the duration of the storm; however, measurements were not taken on consistent intervals. This data was used to interpolate cumulative rainfall totals at one-hour intervals as shown in Figure 15. In order to ensure this distribution was reasonable, it was compared to 30-minute rainfall data compiled from three rain gages within the path of the peak rainfall between Washington, D.C. and the City of Baltimore. Table 5 summarizes the rainfall amounts at the nearby gages. Notice that the peak intensity at Brighton Dam fell within the range of peak intensities at the three adjacent gages thus the data was considered reasonable.

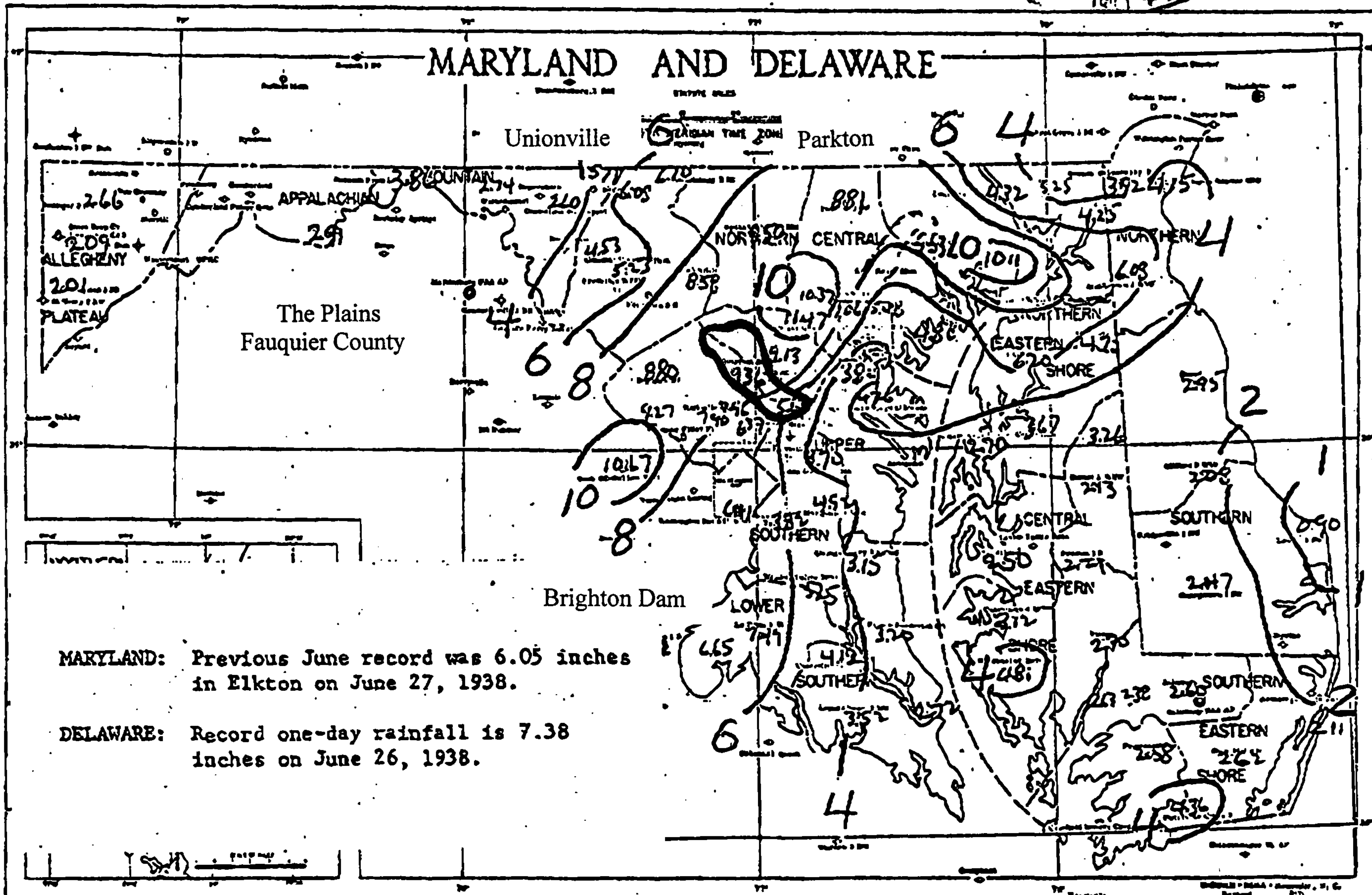

Figure 14: 24-hr Agnes Rainfall Isohyetal Map

Figure 15: Agnes Storm Cumulative Rainfall Distributions

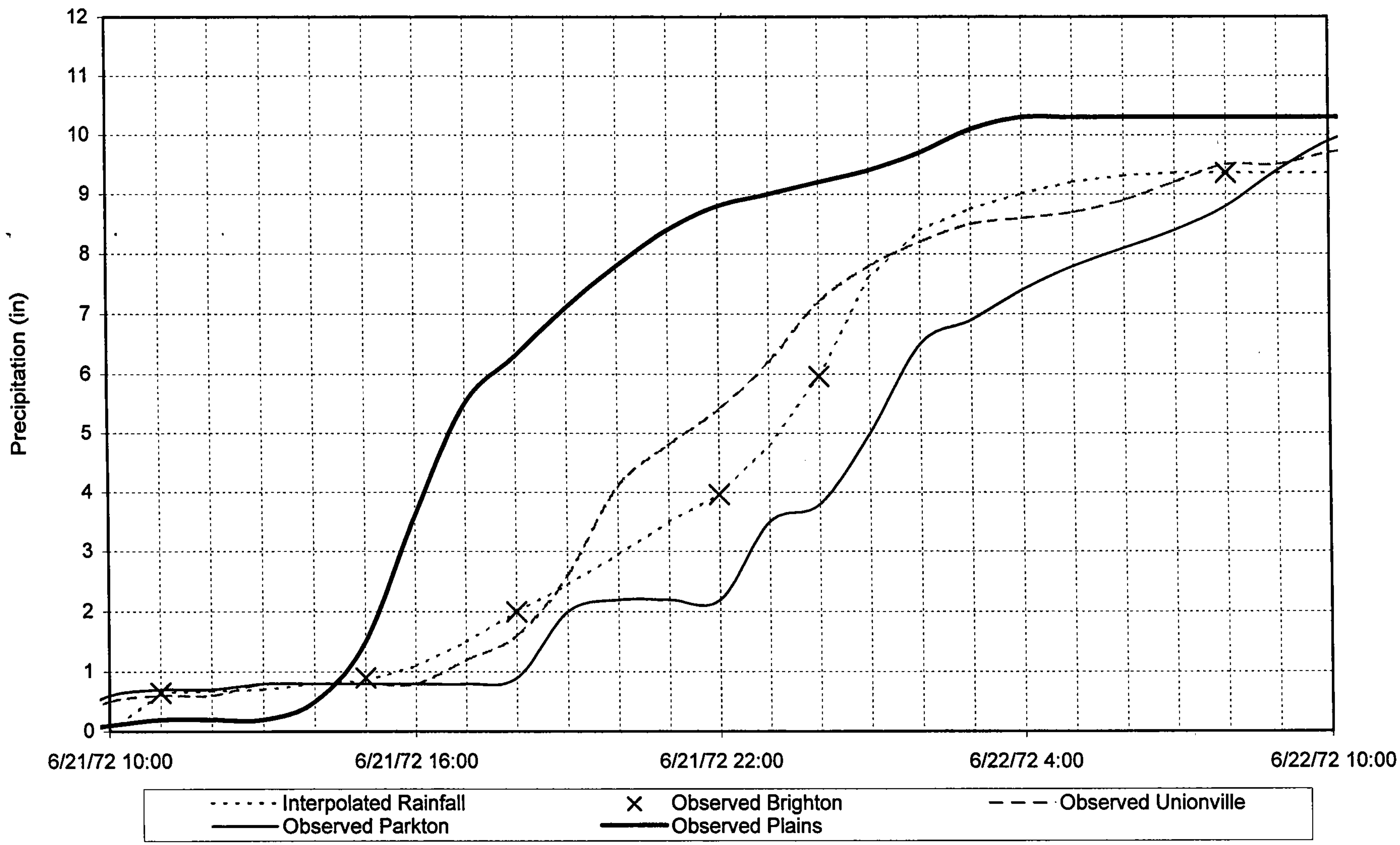

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Table 5 – Agnes Storm Rainfall

State	Rain Gage	Peak Intensity (in/hr)	Max 24-hr Total (in)
MD	Brighton Dam	**1.64**	**9.36**
	Unionville	1.5	9.6
	Parkton	1.5	10.3
VA	Fauquier	2.1	10.3

Review of daily totals indicated that the five days preceding the storm collected 0.28 inches of rainfall. In addition. the first 0.22" of the Agnes Storm rainfall was assumed to be infiltrated in order to reach initial soil conditions near field capacity (AMC Group II). Therefore, the total rainfall entered into the HEC-1 model was reduced from 9.36" to 9.14".

5.3 Storm Flow Hydrograph Analysis

Two USGS stream gages were analyzed that are located in the watershed during the June 21-22, 1972 storm. The first was on the Patuxent River (known as Unity gage - 01591000), and the second was on the Cattail Creek (01591500), just downstream of the current Cattail Creek gage (01591400). The drainage areas for the Unity and Cattail gages are 35 mi^2 and 27 mi^2, respectively, and account for about 75% of the drainage into Brighton Dam. According to the USGS, the Unity gage was not in service the week prior to the Agnes Storm (approximately June 17 to June 24) due to a mechanical failure. The Cattail Creek gage at Roxbury Mills was taken out of full-time service in 1956. Therefore, streamflow hydrograph data was not available within the watershed. Fortunately, the USGS used high-water marks at the Unity and Cattail Creek gages to extrapolate the peak flow from established rating curve tables. High water marks were also used at the CSX railroad bridge to calculate the peak flow in the City of Laurel due to the washout of the gage downstream of Rocky Gorge Dam. The USGS's historical notes calculated the flow to be about 27,100 cfs, however, the published peak was set to 26,000 cfs based on a reference from WSSC. Table 6 summarizes the verification results. The estimated peak discharges at the gages for the Agnes Storm were considered acceptable and provided confidence in the model's prediction of large storm events.

5.4 Comparison Analysis at WSSC Reservoirs

Similar to the calibration, an inflow and outflow comparison at the WSSC reservoirs was performed. Records of operation listing the observed reservoir pool elevations at Brighton Dam and Rocky Gorge Dam (see Appendix F) along with WSSC's engineering equations for the spillway gates were used to calculate the release hydrographs. The records of operation are fairly comprehensive at both dams. However, the Brighton Dam Taintor gate operations from June 22 at 5:00 AM to June 22 at 3:30 PM were not recorded. In order to complete the recession limb of the outflow hydrograph, it was necessary to estimate the likely gate operations.

Table 6 - Summary of Verification Results

Sub-watershed	USGS Gage #	Drainage Area (mi^2)	Qpeak (cfs)			Runoff (in)		
			Observed	HEC-1	Deviation (%)	Observed	HEC-1	Deviation (%)
Cattail Creek at Roxbury Mill Rd.	01591500	27.7	12000	12885	7		1.38	
Patuxent River at Unity	01591000	34.8	14500	16412	13		1.66	
Brighton Dam Inflow	-	78.9	-	32671		4.36	4.75	9
Brighton Dam Outflow using WSSC Equation	01591610	78.9	17607	17607		-		
Hawlings River	01591700	27.0	-	10080		-	-	-
Rocky Gorge Dam Inflow	-	132.4	-	31140		4.47	4.3	4
Rocky Gorge Dam Outflow using WSSC Equation	01592500	132.4	26000	27860		-	-	
Patuxent River at City of Laurel	-	150.5	-	28736		-	-	-

According to the WSSC's Water Operations Division, one motor will close one gate two links in a half hour period, under normal conditions. Recession limb baseflow was set to the mean daily flow of June 23, which was taken from USGS historical data. The remaining boundary conditions were determined from the positions of the gates at 5:00 AM and 3:30 PM. From this, the remainder of the outflow hydrograph was reconstructed. Figure 16 shows the June 1972 hydrographs for the HEC-1 simulated storm inflow, the calculated outflow at Brighton Dam, and the reservoir flood elevations.

A simple check was used to estimate the runoff volume of the inflow hydrograph and the runoff volume of the outflow hydrograph. Though the reservoir at Brighton Dam probably stored minor amounts of the storm runoff, the difference between the runoff estimates was less than 10%. The calculated release hydrograph from Brighton Dam was then used as the starting condition for the downstream HEC-1 model to the Rocky Gorge Dam. Figure 17 shows the June 1972 hydrographs for the HEC-1 simulated storm inflow, the calculated outflow at Rocky Gorge Dam, and the reservoir flood elevations. Likewise, the calculated release hydrograph from Rocky Gorge Dam could be used as a starting condition for the downstream HEC-1 model through the City of Laurel. See Figure 18 for the Anges Flood hydrograph at the City of Laurel and the runoff hyetograph. Appendix G contains the Agnes Storm verification hydrologic model results.

Figure 16: Verification Hydrographs: Brighton Dam Inflow and Outflow

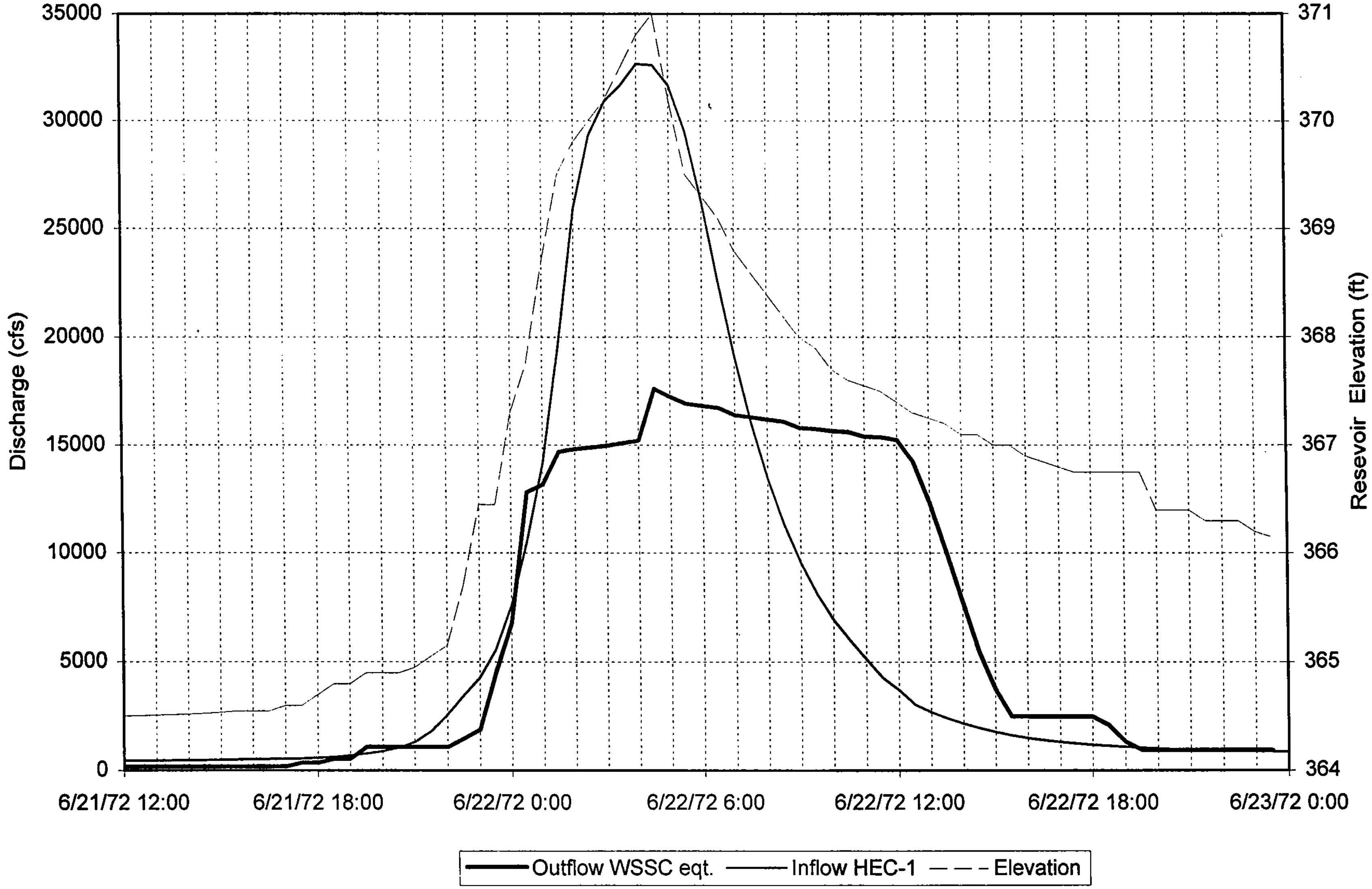

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Figure 17: Verification Hydrographs: Rocky Gorge Dam Inflow vs. Outflow

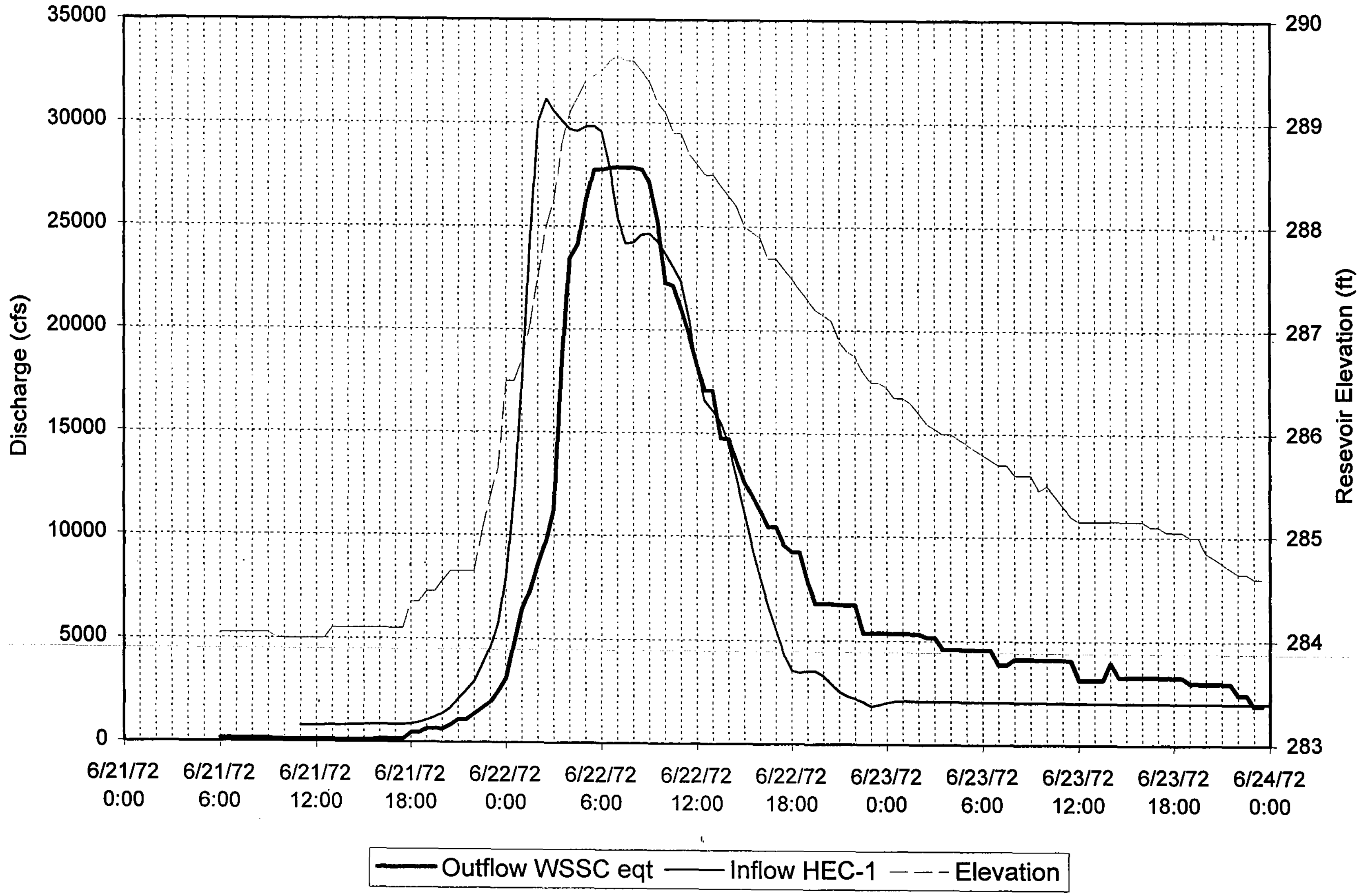

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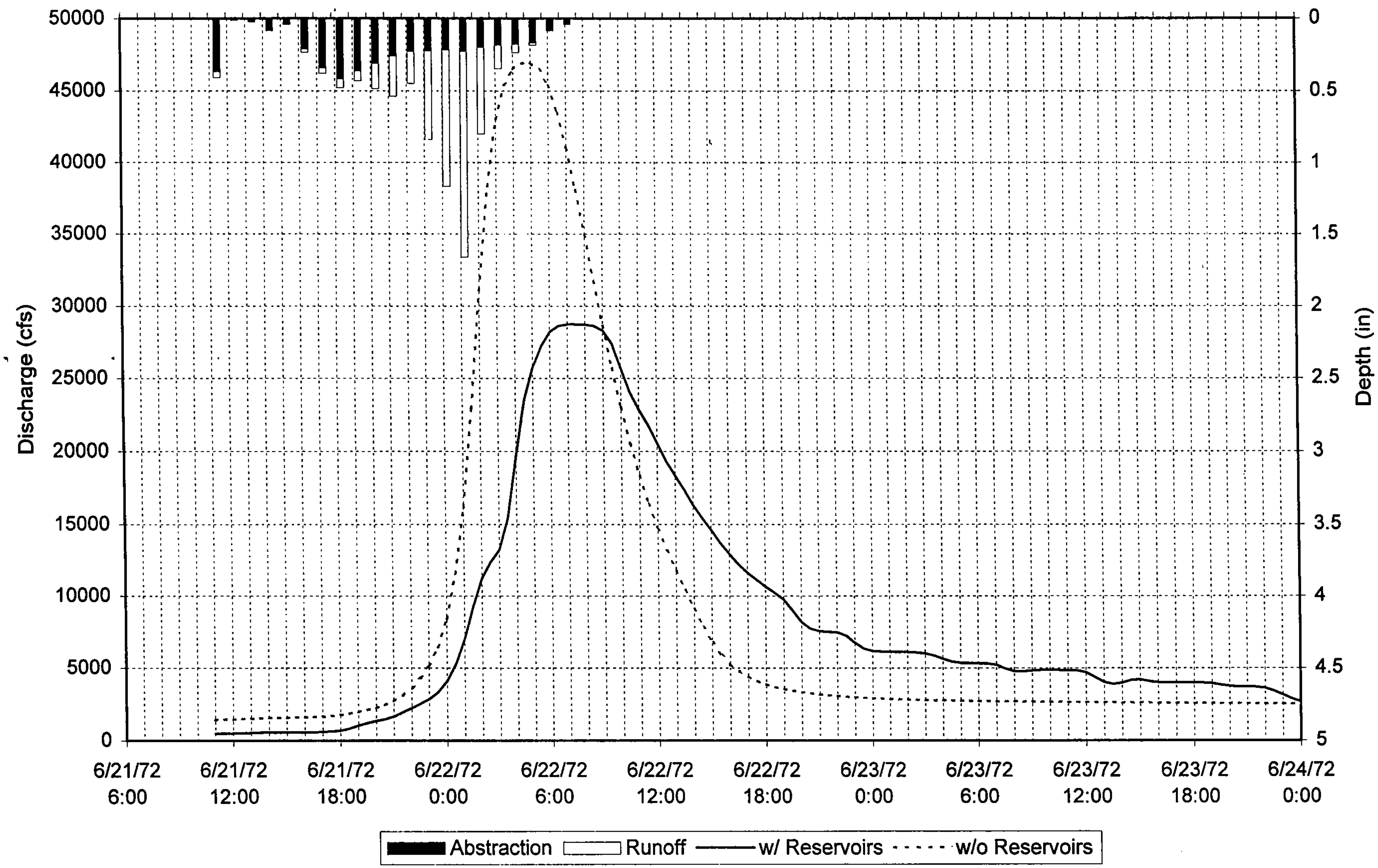
Figure 18: Verification Hydrographs: Comparison at City of Laurel



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6.0 EXISTING WATERSHED DESIGN STORM ANALYSIS

6.1 Design Storm Events

The calibrated HEC-1 hydrologic model was used to simulate the design storm events for predicting the 10-, 50-, 100-, and 500-yr flood discharges. The simulation was for existing watershed conditions and assumes AMC Group II. The SCS 24-hr rainfall distribution for Montgomery and Howard Counties is used for the design storm events, and these rainfall totals are summarized in Table 7.

Table 7 – Design Storm 24-hr Rainfall Totals

	10-yr	50-yr	100-yr	500-yr
24-hr rainfall (in)	5.1	6.3	7.2	9.1

6.2 WSSC Reservoirs Analysis

The hydrologic analysis of the Patuxent River upstream of the City of Laurel is complex due to the existence of two large reservoirs on the main-stem. The WSSC's Systems Control Division manages flood discharges by continuously tracking hydrologic conditions. The WSSC performs real-time monitoring of the water levels at both reservoirs, discharge at stream gages just downstream of both reservoirs, and rainfall at four rain gages within the watershed and at both reservoirs. Each of these factors in combination governs the release of water.

Estimating the design storm flood peaks requires simulating the current dam operations. WSSC's typical operating procedures for frequent rainfall events provide significant flood mitigation, however, the objectives of the gate operations for large and in frequent storms, such as a 100-yr event, are different. For this study the goal was to simulate WSSC's response to the design storm events that would prevent over-topping of the Taintor gates and provide minimal attenuation within the reservoir. The projected gate operations also must be within the logistics of WSSC's staff and facility. Variables in this process include the number of operators located at the dam during a storm, advanced notice of storm path and magnitude, and response time of controllers and operators which dictates the timing of gate openings.

A meeting was held with the WSSC Systems Controller to review assumptions of these variables and discuss the approach in simulating the design storms through the dams. The results were:

- The controller are informed in advance of storm to allow opening of gates to avoid overtopping of Taintor gates (the top of the closed gates at Brighton Dam is 366.3' and at Rocky Gorge Dam is 286.3').
- The design storms provided uniform rainfall over the entire watershed to the City of Laurel
- There are four operators on-site at Brighton Dam and three operators on-site at Rocky Gorge Dam at the start of a storm event (Brighton Dam has four motors and Rocky Gorge Dam has three motors)
- Gates are opened by an even number of links (2 links = 6 inches)
- If necessary a gate can be opened at a maximum rate of 14 links within a 30 minute period
- A motor can be moved and a gate opened within a 30 minute period
- Every gate will be opened to increase the overtopping elevation of the Taintor gates
- Gates will be closed as the in-flow recedes, but the reservoir elevation will not overtop the Taintor gates

To account for the effects of the reservoirs, the design storm events were simulated in the hydrologic models using three alternatives:

1. No reservoirs (continuous hydrologic simulation)
2. 0' freeboard at Brighton and Rocky Gorge Dams
3. 3' of freeboard at Brighton and Rocky Gorge Dams

Alternative 1 simulates the watershed as a continuous system where storm flows are routed along the main-stem without the influence of the reservoirs. This is considered a conservative approach because the reservoirs and dams provide partial attenuation of large storm events. This run was accomplished by simply removing the reservoir outflows. However, the development of the HEC-1 model excluded routing steps through the river reaches inundated by the reservoirs, because it was assumed that flow entering the reservoir immediately affects the reservoir level. Results for the simulated 100-yr discharges without the reservoirs were compared against other methods for estimating the 100-yr discharges. These methods are listed below in their priority for comparison:

- Flood frequency analyses at stream gages within the watershed by the USGS or by using HEC's Flood Frequency Analysis (HEC-FFA)
- USGS regional regression curves
- FEMA Flood Insurance Studies
- Flood frequency analyses at stream gages near the watershed with similar characteristics by the USGS or by using HEC-FFA

The HEC-FFA utilized annual flood peaks. The results of gages within the watershed provide specific results for the Patuxent River. The regression analysis provides a means of estimating discharges at ungaged drainage points. The current Flood Insurance Study (FIS) for the City of Laurel cited a frequency analysis of the stream gage just downstream of Rocky Gorge Dam. The analysis of gages with similar characteristics in the Patuxent River watershed provides comparative results. Table 8 summarizes the results of these comparisons.

The comparisons were very good for the upper watershed to Brighton Dam. USGS regression estimates were consistently less than the HEC-FFA results. However, the two were within ≈10% of each other (except the Unity gage). Only two gages in this watershed contain significant periods of record (>50 years). Of these two gages, only the Patuxent River at Unity gage (59 years) monitors an uncontrolled watershed. An extreme event occurring in September of 1971 washed out the Unity gage. In order to attain the peak flow, established rating tables were extrapolated beyond standard USGS limits (limit = 1inch of extrapolation on rating curves). This value strongly influences the statistics of this drainage area to the point where the 100-yr estimate is nearly double the estimates of the regression equation and the HEC-1 model. Appendix H contains the results of the HEC-FFA.

It should also be noted that the peak discharge at the confluence of Brighton Dam outflow and the Hawlings River dictates the peak discharge at Rocky Gorge Dam. This is because the upper watershed is a multi-order stream network, while first-order streams feed the main-stem downstream of Brighton Dam. The downstream drainage from the confluence of the Hawlings and Patuxent Rivers to the Rocky Gorge Dam passes prior to the peak at the confluence, which is routed through the stream valley with minor additional runoff. This phenomenon also occurred during the May 1989 calibration and the June 1972 Agnes verification simulations.

Table 8 – Comparison of 100-yr Discharges (cfs) without WSSC Reservoirs

Location	USGS ID	Drainage Area (mi^2)	HEC-1	USGS Regression[2]	Flood Frequency[3]	Years of Record	FEMA FIS[4]
Cattail Creek at confluence with Patuxent near Roxbury Mills Rd.	01591500	27.7	9954	8718	9820	12	-
Cattail Creek near Glenwood	01591400	22.9	9562	7259	8500	19	-
Patuxent River near Unity	01591000	34.8	12,776	8937	23,700	53	15,500
Hawlings River near Sandy Spring	01591700	27.0	8219	8297	9210	19	-
Patuxent River d/s Brighton Dam	01591610	78.9	23,621	14,409	NA	16	-
Patuxent River d/s Rocky Gorge Dam	01592500	132.4	31,877	18,978	NA	52	22,000
Patuxent River @ CSX railroad crossing in City of Laurel	-	137.9	34,242	19,421	-	-	-
Patuxent River u/s of Little Patuxent River confluence	-	150.5	34,208	20,378	-	-	24,000
Little Patuxent River near Guilford[1]	01593500	38.0	12,825	9376	10,500	66	-

[1] Contiguous watershed within same land uses and physiographic region as Patuxent River therefore used HEC-1 model results with identical drainage area
[2] USGS Report #95-4154 (Dillow, 1996)
[3] Stream gage statistical analysis using HEC-FFA or reported by USGS in Prof Pap#95-4154 (Dillow, 1996)
[4] City of Laurel FEMA Flood Insurance Study (Community # 240053) dated August 19, 1985

Alternatives 2 and 3 accounted for potential flood mitigation by simulating the dams simultaneously with no freeboard and with 3' freeboard. Procedures were developed to account for the process WSSC utilizes for managing large runoff events. Specifically, WSSC's spillway management computer program was used to estimate the maximum potential gate openings at the two dams for passing the design storm. The parameters the program requires are:

1. Total storm runoff for the watershed to Rocky Gorge Dam
2. Storm runoff duration
3. Time from start of rainfall for runoff to reach reservoirs
4. Starting reservoir elevations
5. Discharge through valves and hydroelectric units

The program used the inputs above and an assumed unit hydrograph to estimate a runoff hydrograph. If the reservoir cannot retain this runoff, the hydrograph is used to estimate the gate openings necessary to pass the storm. For the design storms, inputs 1 to 3 are estimated based on the results of Alternative 1. Input 4 is based on the assumed freeboard. Input 5 was assumed to equal baseflow at the beginning of the event.

An iterative process was required to obtain a reasonable arrangement of gate openings based on WSSC's procedures and the projected gate-opening configuration. The most sensitive assumption was the fastest gate-opening rate. Historical records show gates being opened ≥2 links in a half-hour period. Appendix I contains the proposed spillway gate openings for the 100-yr design storm.

Proposed gate openings were estimated for each storm event under Alternative 2 (0' freeboard), however, the focus was on the 100-yr analysis. The WSSC spillway management computer program was used to provide an initial estimate of the number of gates to open and the number of links to open each gate. The WSSC program underestimated the required gate openings since the model simulated excessively high reservoir levels. The gates were opened more rapidly to meet the flood reservoir routing goals.

Based on the HEC-1 model inflow hydrograph and the proposed gate openings, the reservoir levels and the outfall hydrographs from the dams were estimated interactively using WSSC's engineering equations. The water levels were estimated in a stepwise fashion by the change in reservoir storage (difference between outflow and inflow) at each time step, which in turn provides a reservoir elevation. As discussed previously, the following governing equation is used.

$$\Delta S = \left[\frac{\frac{I_2 - I_1}{2} - \frac{O_2 - O_1}{2}}{t_2 - t_1} \right]$$

where: I = estimated inflow to the reservoir (cfs)
O = calculated outflow from the reservoir based on spillway data (cfs)
ΔS = calculated change in the reservoir storage (ft^3)
Δt = time increment (sec)

6.3 Design Discharge Simulations

The various design storms were simulated in HEC-1 for the three different alternatives. The HEC-1 model used 15-minute time intervals to provide reasonable results and sufficient resolution to perform the water level estimation. The simulations indicated that passing a storm on the scale of a 100-yr event required gate openings to be rapidly opened to a larger extent than typical WSSC operations. For example, gates were typically opened 2 to 5 links per half hour at one gate. In extreme events, gates have been opened greater than 10 links per half-hour at one gate.

Table 9 compares the design discharges estimated for the three alternatives. The results for the zero and three foot freeboard scenarios show that attenuation occurs at Brighton Dam based on the proposed gate openings. This is due to the limitations of opening gates based on the current configuration of the hoists and WSSC's typical operating procedures. Increasing the freeboard provides the potential for increased attenuation.

6.4 Design Discharge Summary

Figures 19 and 20 show the 100-yr hydrographs at Brighton Dam and Rocky Gorge Dam. Figure 21 shows the 100-yr hydrographs through the City of Laurel and also the runoff distribution simulated in the HEC-1 model. Appendix J contains the HEC-1 model results.

The estimated peak 100-yr discharges for the existing watershed conditions downstream of the Rocky Gorge Dam will be used in the calibrated Patuxent River HEC-2 hydraulic model to estimate the 100-yr flood elevations through the City of Laurel. The results of the existing conditions will be used for revising the FEMA Flood Insurance Rate Maps (FIRM) for the City of Laurel.

Table 9 –Summary of Existing Watershed Design Discharges

HEC-1 Code	Location	Drainage Area (mi^2)	10-yr[1] (cfs)	50- yr[1] (cfs)	100- yr[1] (cfs)	500- yr[1] (cfs)
CL6B	D/S of Rocky Gorge Dam with no reservoirs	132.4	14,310	22,474	34,499	53,378
	D/S of Rocky Gorge Dam with reservoirs @ 0' freeboard	132.4	12,459	17,675	29,419	43,678
	D/S of Rocky Gorge Dam with reservoirs @ 3' freeboard	132.4	8,277	15,174	26,258	41,717
RL8	Patuxent River @ CSX railroad crossing in City of Laurel with no reservoirs	137.9	14,298	22,266	33,988	52,573
	Patuxent River @ CSX railroad crossing in City of Laurel with reservoirs @ 0' freeboard	137.9	12,399	17,563	29,339	43,242
	Patuxent River @ CSX railroad crossing in City of Laurel with reservoirs @ 3' freeboard	137.9	8,296	14,977	26,234	41,452
CL8	Patuxent River at MD State Route 198 with no reservoirs	139.8	14,250	22,168	34,054	53,164
	Patuxent River at MD State Route 198 with reservoirs @ 0' freeboard	139.8	12,432	17,603	29,379	43.294
	Patuxent River at MD State Route 198 with reservoirs @ 3' freeboard	139.8	8,324	15,011	26,267	41,504
CLAU	Patuxent River u/s of Little Patuxent River confluence with no reservoirs	150.5	14,380	22,246	34,208	53,164
	Patuxent River u/s of Little Patuxent River confluence with reservoirs @ 0' freeboard	150.5	12,508	17,680	29,419	45,256
	Patuxent River u/s of Little Patuxent River confluence with reservoirs @ 3' freeboard	150.5	8,893	15,066	26,361	41,628

1 – based on three feet of freeboard at WSSC's reservoirs

Figure 19: Existing 100-yr Hydrographs: Brighton Dam Inflow and Outflow

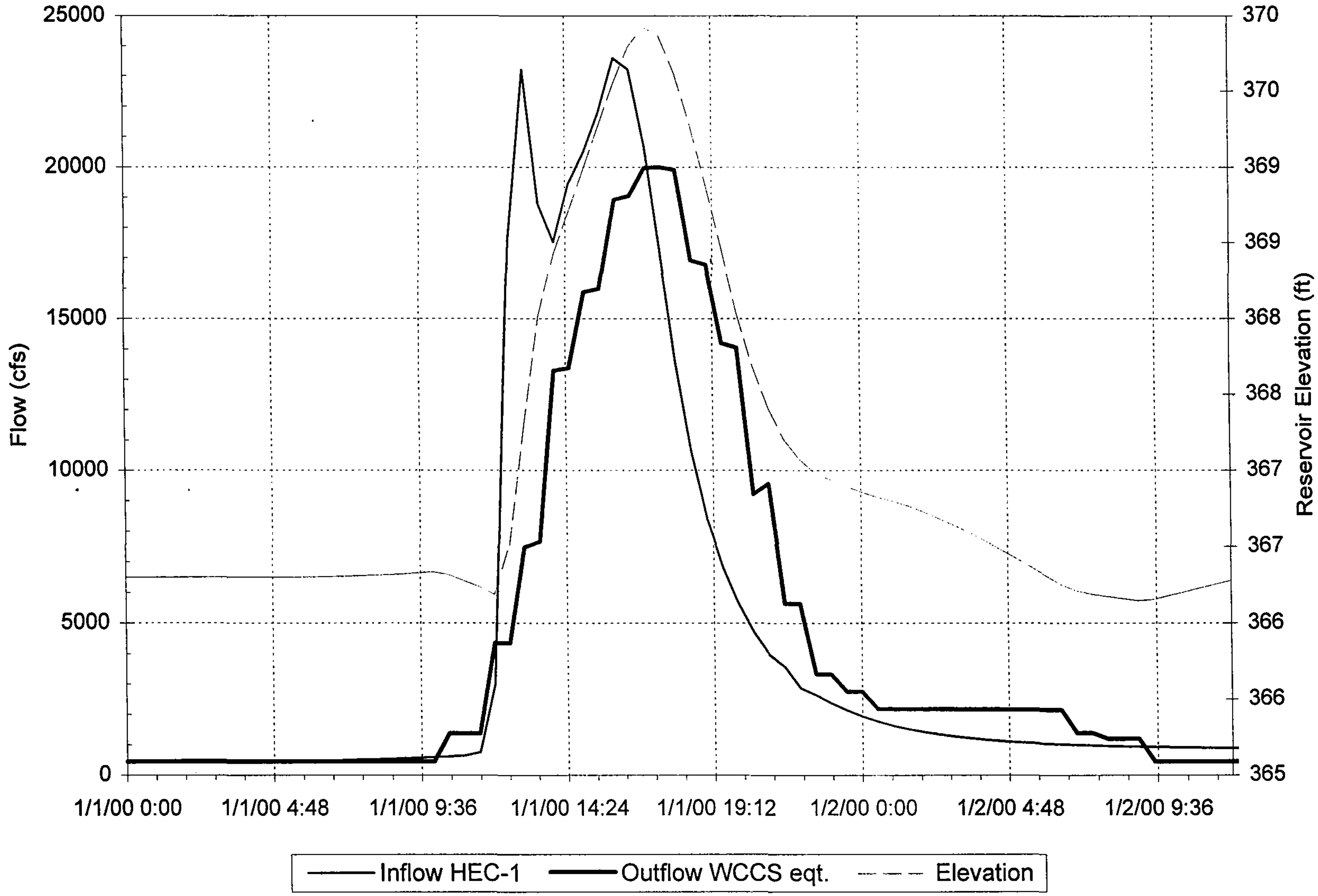

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Figure 20: Existing 100-yr Hydrographs: Rockyg Gorge Dam Inflow and Outflow

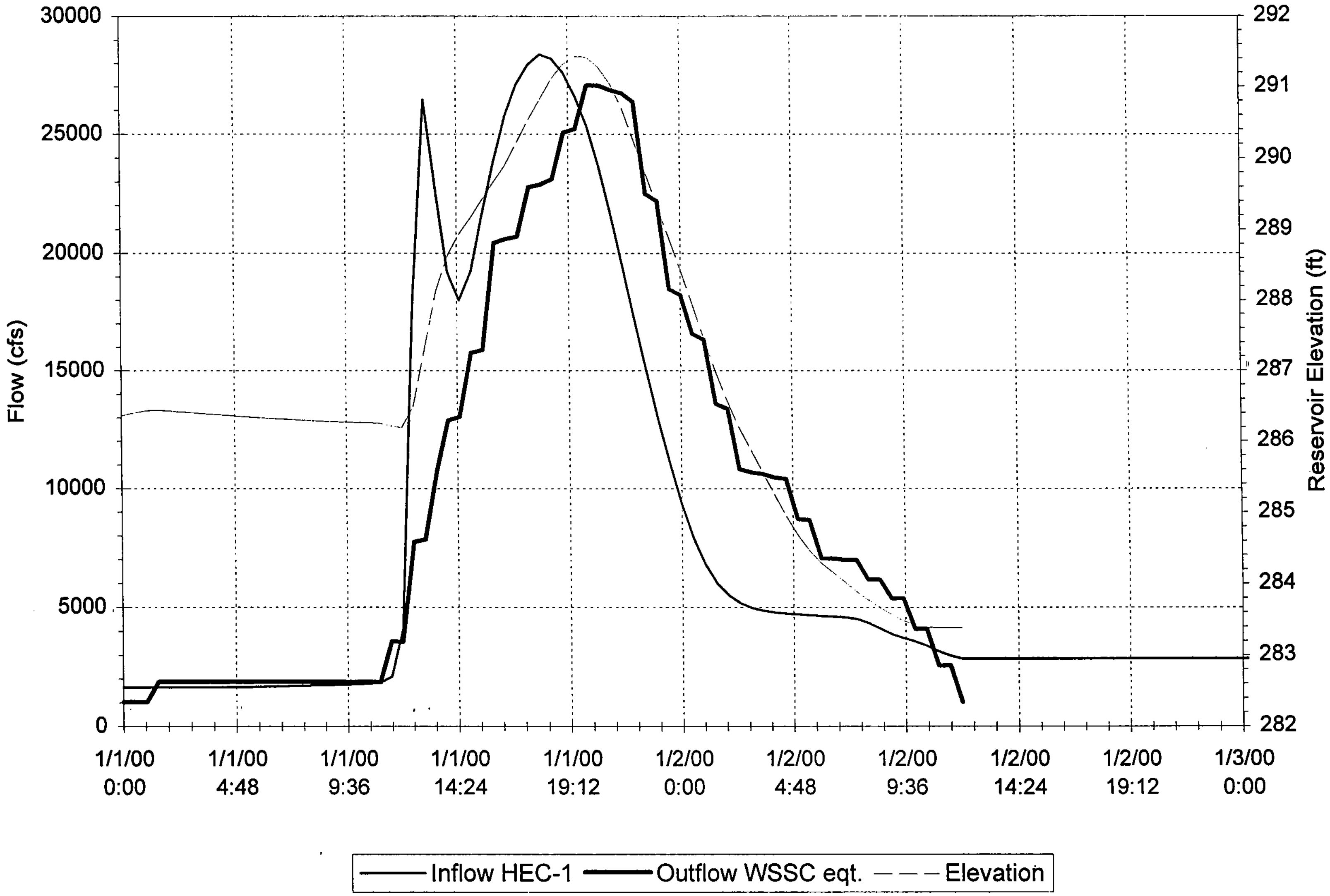

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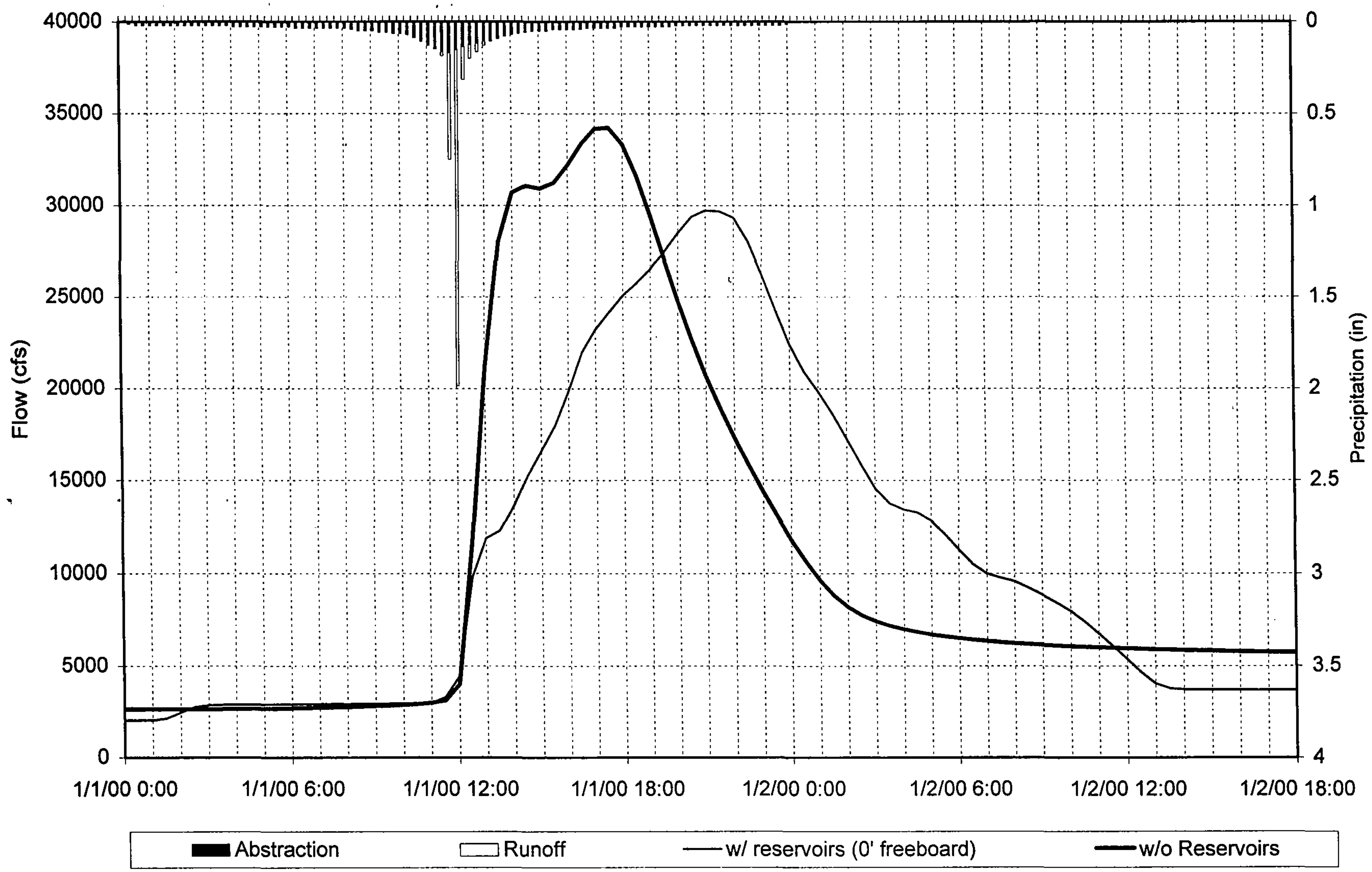
Figure 21: Existing 100-yr Hydrographs: Comparison at City of Laurel



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