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Hydrologic Analysis and Model Development for Cannonsville Reservoir[1]

Emmet M. Owens
Department of Civil and Environmental Engineering
Syracuse University, Syracuse, NY 13244

Rakesh K. Gelda and Steven W. Effler
Upstate Freshwater Institute
P.O. Box 506, Syracuse, NY 13214

James M. Hassett
Environmental Resource and Forest Engineering
SUNY College of Environmental Science and Forestry
1 Forestry Drive, Syracuse, NY 13210

ABSTRACT

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The hydrology of Cannonsville Reservoir, a water supply and flow augmentation reservoir for the City of New York, is analyzed. Measurements of components of the hydrologic budget of the reservoir over the entire 30 years of operation are reviewed. The largest tributary is gauged near its discharge to the reservoir; all others are ungauged. Water surface elevation and outflow, in the form of spillway flow, direct releases for flow augmentation, and drinking water withdrawal are monitored. Multiple uses of the reservoir and variations in runoff common to the region result in strong seasonal and interannual variations in hydrology. Spillway flow typically occurs in late winter and spring, and is rare in summer and early fall. Releases, to meet minimum flow requirements in the Delaware River downstream, generally occur only in summer and early fall. Drinking water withdrawal is relatively uniform throughout the year. On average, spill, dam releases, and withdrawal for the water supply have represented 40%, 32%, and 28%, respectively, of the outflow of the reservoir over the 30 years. Reservoir water surface elevation is generally greatest in the spring and lowest in the fall. Reductions in maximum depth of 40% and in storage volume of 75% have occurred in dry years. A hydrologic model was developed to maintain a hydrologic balance and estimate surface inflows from the ungauged portion of the watershed. While estimates based on a simple ratio of ungauged flow to gauged flow may be used for annual averages, a more complex budget calculation based on a 10-day averaging period was used to provide a time series of daily average ungauged inflow. The average annual flushing rate for the 30 years was 2.6 y^{-1} while the range was 1.9 to 3.6 y^{-1}. Use of the entire 30 years hydrologic record is recommended to support forecasting with hydrothermal and water quality models.

Key Words: hydrologic budget, reservoir operation, flushing rate, hydrologic balance.

Establishment of a hydrologic budget is fundamental to support effective management of any lake or reservoir. Simulation models of water quantity (e.g., Johnson et al. 1991) and quality (e.g., Chapra 1997, Chapra and Reckhow 1983, Thomann and Mueller 1987) that are based on the principle of conservation of mass have become important tools to support such management. The credibility of these models depends critically on the accuracy of measurements and estimates of major sources/sinks. In addition, the determination of material loading (Heidtke and Auer 1992, Longabucco and Rafferty 1998, Stepczuk et al. 1998a) for water quality modeling depends on accurate

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determination of inflow from major sources of water and constituents.

Wide seasonal, interannual, and longer term variations in inflows and outflows that are common for many lakes are magnified and modified for many reservoirs due to outflow requirements. Perhaps the most important and conspicuous feature which contrasts lakes and reservoirs is the relatively large temporal variations in water storage and associated water surface elevation (WSE) that is often observed in reservoirs. Analysis of historical data for Cannonsville Reservoir (1989-1995) conducted by Effler and Bader (1998) established empirical relationships between WSE and the duration of stratification and selected features of water quality. Shorter periods of stratification and degraded water quality were associated with increased drawdown. These findings indicate that there may be a water quality cost when watershed runoff cannot match outflow that is required to meet the intended use of a reservoir. Some water quality effects may be mediated by the changes in features of the stratification regime, as these changes are known to influence various common measures of water quality (Orlob 1983, Owens and Effler 1989, Stefan et al. 1976, Stauffer and Lee 1973). Reservoir hydrology may also be an important consideration in the selection of conditions to be used in testing of hydrothermal and water quality models, and in application of the models to forecast water quality conditions as a part of the evaluation of management alternatives.

Our objective is to review and analyze the entire 30-year hydrologic record of Cannonsville Reservoir, including all available measurements. In addition, we develop and test a simple reservoir hydrologic model that maintains a hydrologic balance for the reservoir for the 30-year period at a time scale consistent with the needs of dynamic hydrothermal (Gelda et al. 1998, Owens 1998) and water quality (Doerr et al. 1998, Owens et al. 1998) models to support testing, hindcasting and forecasting. Estimates of inflows from the ungauged portion of the reservoir's watershed are developed as part of this analysis. Approaches for incorporation of the hydrologic record and the hydrologic model in conducting testing and forecasting with hydrothermal and water quality models are discussed.

Hydrologic Setting of Cannonsville Reservoir

Overview

Cannonsville Reservoir is located 190 km northwest of New York City near Deposit, NY (Fig. 1). The reservoir is owned and operated by the New York City Department of Environmental Protection (NYCDEP) as a supply of drinking water for the City and to augment flow in the Delaware River downstream of the reservoir. The reservoir has been in operation since 1966.

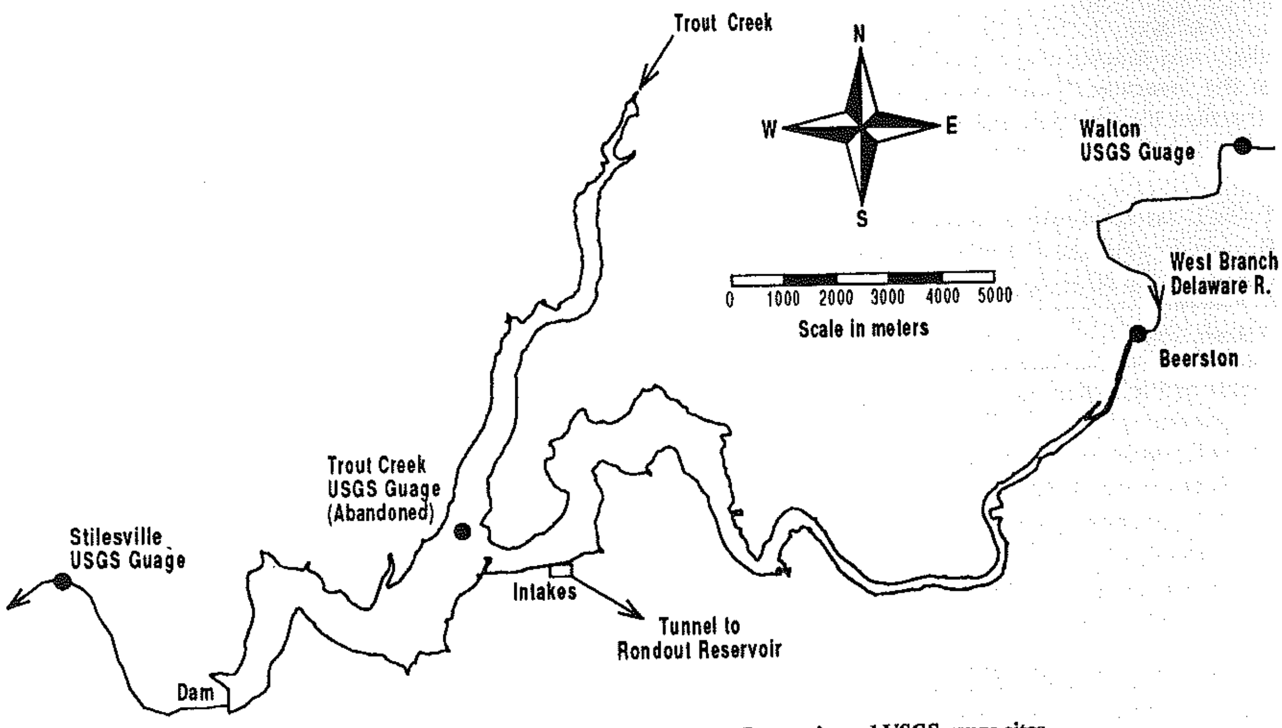

Figure 1.–Map showing major tributaries to and withdrawals from Cannonsville Reservoir, and USGS gauge sites.

The primary tributary to the reservoir is the West Branch of the Delaware River (WBDR, Fig. 1), which drains 79% of the total reservoir watershed (Table 1). Trout Creek, the second largest tributary drains about 5% of the reservoir watershed and enters the reservoir at the upstream end of its other "arm." The remaining surface inflow enters from smaller streams distributed around the perimeter of the basin. Water leaves the reservoir by one of three pathways: flow over the dam's spillway (designated "spill"), which occurs in an uncontrolled manner when the WSE of the reservoir exceeds the crest elevation (350.6 m NGVD); drinking water withdrawals; and releases at the base of the dam to the downstream portion of WBDR. Drinking water withdrawal occurs at a mid-reservoir intake structure and flows through a tunnel to Rondout Reservoir. Water can be withdrawn at depths of 10, 20 or 37 m below the spillway crest elevation, although the middle intake is most often used.

The release works at the dam on the western end of the reservoir (Fig. 1) are a series of pipes of varying diameter located at the base of the earth-fill dam. Releases are controlled by a valve on each pipe, which may be closed or fully open. The releases at the dam are regulated to meet the downstream release requirement mandated by New York State law, and together with similar releases from Pepacton and Neversink Reservoirs, to augment flow further downstream in the Delaware River.

Bathymetry/Hypsographic Data

A geographical information system (GIS) methodology was employed to model the physical features of the reservoir (Gorokhovich et al. 1996). Preconstruction topographic maps (USGS 1964 7.5-minute quadrangles) were digitized to determine underwater contours of the Cannonsville basin. The digitized maps were then converted into a 3-meter grid structure using conversion and interpolation software. The grid structure was used to generate hypsographic data (Fig. 2) and also to provide geometric data for hydrothermal (Gelda et al. 1998) and water quality (Doerr et al. 1998) models.

Table 1.–Areas of selected portions of the Cannonsville Reservoir watershed. Data obtained from NYCDEP GIS database (Gorokhovich et al. 1996)

Description	Drainage Area, km²
Cannonsville Reservoir	
-at dam	1178
-reservoir surface area	19.3
-reservoir watershed	1159
West Branch Delaware River	
-at Walton	860
-at Beerston	916
Trout Creek	
-old Cannonsville gauge	128
-at reservoir	55.3

Hydrologic Data

NYCDEP provided data on daily average rates for the three outflows and WSE for the period 1966-1995, and daily precipitation measured at the dam from 1982-1995. Flow rates for the spillway and each release pipe are determined from rating curves established by NYCDEP. Flow in the pipeline to Rondout Reservoir is determined by direct instrumentation. WSE is measured with a float gauge at the dam.

The United States Geologic Survey (USGS) provided daily average streamflow for gauges on WBDR at Walton (period of record 1951 to 1995); on Trout Creek at Cannonsville (1951-1962, abandoned in 1963 prior to filling of the reservoir); and on WBDR at Stilesville (1989-1995), located downstream of the dam (Fig. 1). The USGS considers the flow record for Walton "good" (indicating that 95% of daily discharges are within 10% of the true value) for most of the year and "poor" (indicating that 95% of the daily discharges have error in excess of 25%) during the winter months of some years due to ice problems. The Trout Creek and Stilesville records are considered "good."

The accuracy of the measurements of reservoir spill and dam releases can be checked by comparing the summation of these to the streamflow measured at Stilesville, located about 2 km downstream from the dam (Fig. 1). There are no identifiable (from USGS

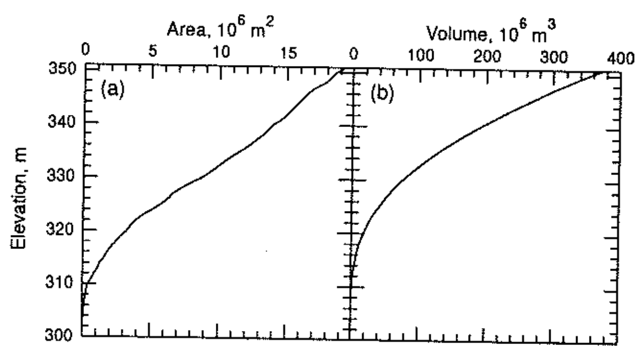

Figure 2.–Cannonsville Reservoir hypsographic data: (a) horizontal area as a function of reservoir water surface elevation, and (b) volume as a function of reservoir water surface elevation. Derived from 3-meter grid digital elevation model (see Gorokhovich et al. 1996); elevation datum is NGVD.

7.5-minute topographic maps) streams entering the river between the dam and the Stilesville gauge. The high degree of agreement between the summation of the two reservoir outflows and the downstream flow, on an annual basis for 6 years (Table 2), supports the accuracy of these measurements of outflow. The percent difference ranged from ~ 3.7 to 10.2; the average was 5.3%. The root mean square (rms) difference between the summed reservoir outflow and the downstream flow, each averaged over a 10-day period, was 5.8% for this 6-year period. These differences are less than the modest levels of uncertainty in the flow measurements. Independent checks on measurements of other of the components of the hydrologic budget (e.g., water supply withdrawals, WSE, inflows from the watershed) are not available. The accuracy of measurements of other hydrologic components is supported by the degree of balance of the subsequently presented hydrologic budget calculations and interrelationships demonstrated between the components.

Dynamics of Reservoir Hydrology/Operations

Variations in the magnitude of inflows and the requirements for water supply and downstream releases combine to make the hydrology (including WSE) of Cannonsville Reservoir highly dynamic on an interannual (Fig. 3), as well as seasonal (Fig. 4), basis. Strong interannual variations in the WBDR flow have occurred over this period (Fig. 3a). The highest annual average flow (~ 27 $m^3 \cdot s^{-1}$, 1977) was about 2.5 times the lowest (~ 11 $m^3 \cdot s^{-1}$, 1980). Inflow was generally higher in the 1970s than in the 1980s (Fig. 3a).

The magnitudes of dam releases have also varied greatly from year-to-year over the period of operation

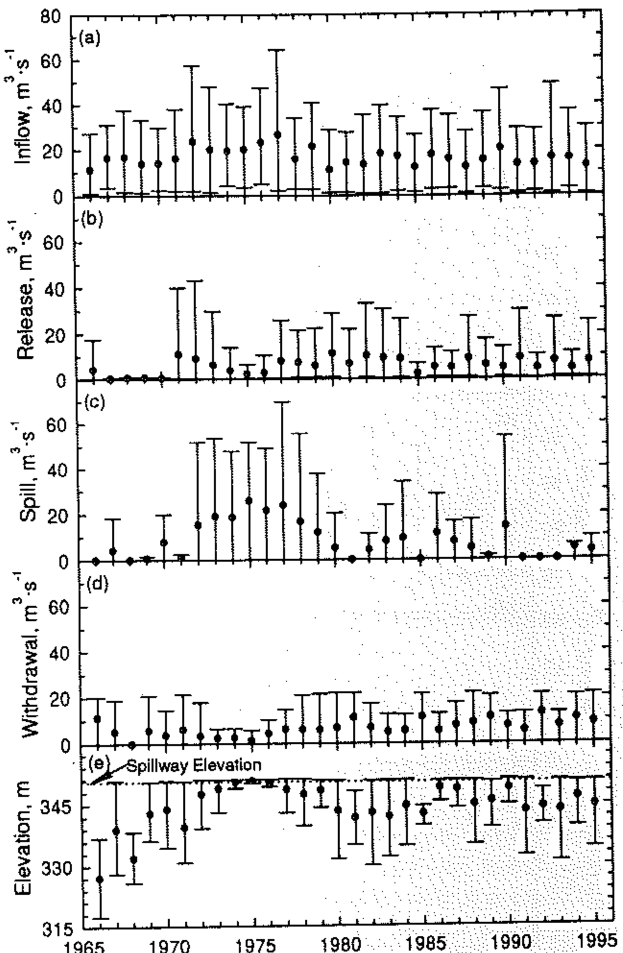

Figure 3.–Statistics of hydrologic features of Cannonsville Reservoir for the period 1966-1995 evaluated separately for each year: (a) WBDR flow at Walton, (b) dam release, (c) spillway discharge, (d) drinking water withdrawal, and (e) reservoir water surface elevation. Each plot shows the mean (filled circle), and 10 and 90 percentiles (horizontal lines below and above).

of the reservoir (Fig. 3b). Though an inverse relationship between annual dam release and inflow at Walton is indicated, it was not found to be strong. In part, this reflects interannual differences in operation of the reservoir, such as the use of this particular reservoir rather than the other two reservoirs available to meet flow augmentation obligations. Spill has been an important export pathway for the reservoir (Fig. 3c) during years in which inflow was relatively high (Fig. 3a). The reservoir has been used more for water supply (withdrawals via the tunnel) over the last 15 years of its operation (Fig. 3d); the average withdrawal rate for the 1981-1995 interval (~ 8 $m^3 \cdot s^{-1}$) was nearly twice as great as for 1966-1980 ($p = 0.05$). Variations in dam releases (Fig. 3b) and spill (Fig. 3c) within individual years have usually been substantially greater than those for water supply withdrawal (Fig. 3d).

Table 2.–Comparison of USGS (gauge at Stilesville) and NYCDEP (sum of spill and dam release) flow volumes (10^6 m^3) downstream of Cannonsville Dam.

Water Year	USGS	NYCDEP	% Difference
1989	283	266	6.1
1990	602	572	5.0
1991	633	568	10.2
1992	131	136	-3.7
1993	659	629	4.6
1994	260	266	-2.5
1989-94	2572	2436	5.3

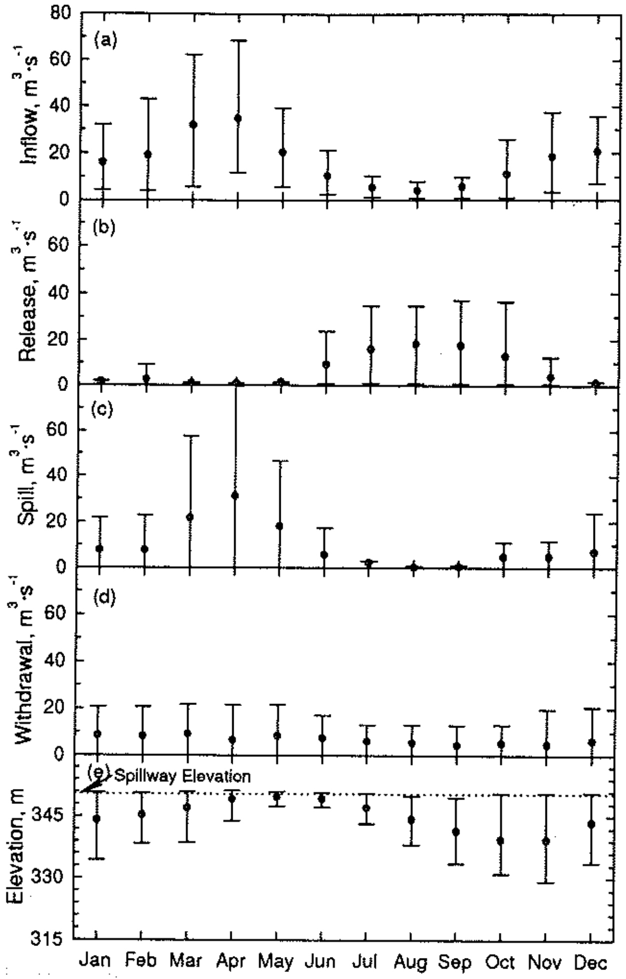

Figure 4.–Statistics of hydrologic features of Cannonsville Reservoir for the period 1966-1995 evaluated separately for each month: (a) WBDR flow at Walton, (b) dam release, (c) spillway discharge, (d) drinking water withdrawal, and (c) reservoir water surface elevation. Each plot shows the mean (filled circle), and 10 and 90 percentiles (horizontal lines below and above).

Wide variations in WSE occur in Cannonsville Reservoir between years and within years (Fig. 3e), as a result of the combined effects of natural variations in inflow and the operational demands for water (dam releases and withdrawals for water supply). The reservoir remained more nearly full in high runoff years such as the mid-1970s, 1986, and 1990. Variations in WSE of 20 m within a year have been common (Fig. 3e). The volume of the reservoir is reduced to about 20% of its full capacity by a drawdown of 20 m (330 m MSL; see Fig. 2).

Strong seasonal variations in WBDR streamflow at Walton prevail, with the highest inflow and interannual variations in inflow occurring in spring (Fig. 4a), associated to a large extent with rainfall and snowmelt. The minimum usually occurs in July, August, or September. Inflow increases to an intermediate level in late fall and winter. In strong contrast to inflow, most of the dam release occurs in late summer and early fall (Fig. 4b). The seasonal variation of the spill (Fig. 4c) has tracked that of inflow (Fig. 4a), being greatest in spring. Spill has occurred rarely in August and September. Water supply withdrawals (Fig. 4d) have been more seasonally uniform than the other outflows or inflow. The seasonal decrease in WSE in early fall (Fig. 4e) has lagged behind the seasonal decrease in inflow, reflecting the influence of reservoir storage. The reservoir is most often full in April, May, and June. The greatest magnitude and variability in drawdown has occurred in the September-November interval (Fig. 4e). The WSE tends to increase through winter to full capacity in spring.

Based on daily average outflow for the 30-year record, partitioned according to the three components (Fig. 5), the greatest outflow occurs in spring, and mostly via spill. Dam release has been the dominant mode of outflow from the reservoir for the June-October interval. The greater uniformity of water supply withdrawal is apparent. On average, spill, dam releases, and withdrawal for the water supply have represented 40%, 32%, and 28%, respectively, of the total outflow from the reservoir over the 30 years of operation.

To support an analysis of the long-term variation of gross measures of reservoir hydrology, the average values of the WBDR streamflow at Walton, water surface elevation, and total outflow for the June-September period of each year of the historical record were determined. Using a simple ranking as a measure of probability of occurrence, cumulative probability distributions for these three quantities were constructed (Fig. 6). Rankings for 1994 and 1995 are highlighted as these years are the focus of testing of a nutrient – phytoplankton model for the reservoir (Doerr et al.

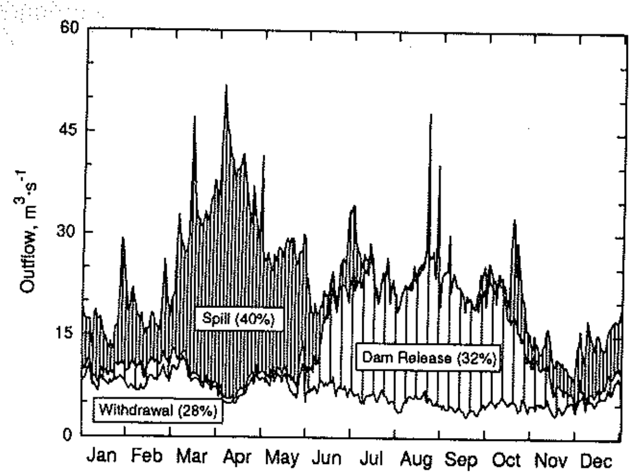

Figure 5.–Variation over the year of the three components of reservoir outflow. Data were averaged from daily values for the period of record (1966 - 1995).

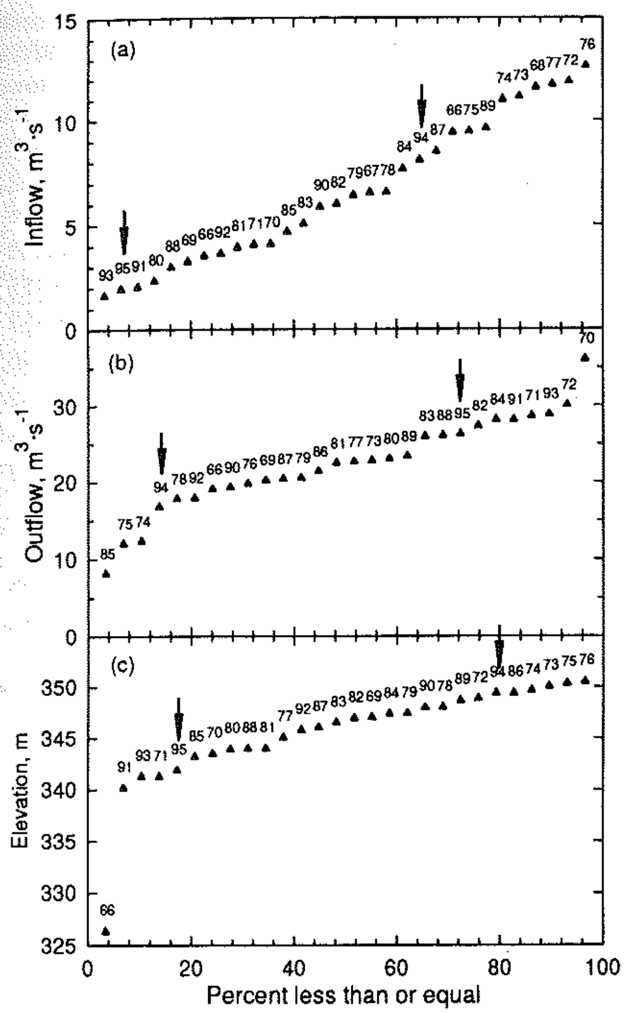

Figure 6.–Cumulative probability distributions for hydrologic features of Cannonsville Reservoir for the June-September interval of the 1966-1995 period: (a) inflow from WBDR, (b) total reservoir outflow (sum of spill, withdrawals, and releases), and (c) water surface elevation.

1998). The year 1995 was unusually dry; only 1993 had less inflow from WBDR to the reservoir (Fig. 6a). By contrast, 1994 was moderately wet; the cumulative inflow was in the 60th percentile. Although a dry year, outflow in 1995 was in the 75th percentile for the period of operation (Fig. 6b). Substantial dam releases were needed in this year to augment flow further downstream in the Delaware River. The combination of low inflow and elevated outflow in 1995 resulted in the fifth lowest average WSE (Fig. 6c) for the period of record. In sharp contrast, 1994 was in the 15th percentile of outflow (Fig. 6b) and 80th percentile of WSE. The widely divergent hydrologic conditions that occurred during these 2 years provide a robust test of related hydrologic, hypsographic, and material loading (Longabucco and Rafferty 1998) conditions for model simulations of water quality (Doerr et al. 1998). Further, the 8-year period of monitoring (1988-1995) that supported testing of hydrothermal models (Gelda et al. 1998, Owens 1998) and the development of empirical water quality – WSE relationships (Effler and Bader 1998) should be considered generally representative on a hydrologic basis, as they encompassed a wide range of hydrologic conditions (Fig. 6).

Reservoir Hydrologic Models

Annual Budget with Simple Inflow Model

A reservoir hydrologic budget analysis begins with the following water balance:

$$\frac{dV}{dt} = Q_{IG} + Q_{IU} + Q_G - Q_O + A_s\,(P - E) \qquad (1)$$

where V is reservoir volume (storage), t is time, Q_{IG} is the gauged surface inflow, Q_{IU} is the ungauged surface inflow, Q_G is the net groundwater inflow/outflow, Q_O is the total outflow (sum of drinking water withdrawal, spill, and dam releases), A_s is the surface area of the reservoir, P and E are the precipitation directly on to, and the evaporation from, the reservoir's water surface, respectively.

Simplifying assumptions are made to apply Eq. 1 to Cannonsville Reservoir. First, since there is no evidence of substantial direct exchange between the reservoir basin and the groundwater system (Soren 1963), it is assumed that Q_G is small and can be neglected. Second, it is assumed that the $(P-E)$ term is small, relative to the other terms in Eq. 1, and can be neglected. The validity of this assumption was checked by comparing daily measurements of precipitation at the Cannonsville dam and daily estimates of evaporation calculated as part of the hydrothermal models for the reservoir (Gelda et al. 1998, Owens 1998). Evaluations for several years indicate that the term $A_s(P-E)$, averaged over a year, accounts for less that 2% of the average annual outflow. Since this is well within the range of error for the measured waterflows, the assumption is judged to be appropriate. The water balance equation thus simplifies to

$$\frac{dV}{dt} = Q_{IG} + Q_{IU} + Q_O \qquad (2)$$

For computational purposes, daily average values of V (based on WSE and hypsographic data, Fig. 2), Q_{IG} (Walton gauging station) and Q_O (NYCDEP) are available from the historical record. Integration of Eq. 2 provides an expression for the change of storage over the time interval as given by

$$\Delta V = \Delta t \Sigma \, (Q_{IG} + Q_{IU} - Q_O) \quad (3)$$

A value of $\Delta t = 24$ hours allows the summations to proceed using daily average values of the flow rates, with all values known except Q_{IU}. Calculations based on Eq. 3 are used here to mimic the manner in which water budget, or more generally mass balance calculations, are carried out in mechanistic hydrothermal and water quality models. An initial condition (value of V) is specified at the start of the simulation period, and values of V are computed for subsequent times based on specified values of Q_{IG}, Q_{IU}, and Q_O.

A simple model for Q_{IU} investigated here is given by

$$Q_{IU} = f \, Q_{IG} \quad (4)$$

where f is a dimensionless factor. This equation implies ungauged surface inflow is a constant fraction of the gauged surface inflow, which further implies similarity between characteristics of the gauged and ungauged watersheds (slope, soil type, land use, etc.) and precipitation received by the gauged and ungauged watersheds. If the watersheds are in fact similar, f should be roughly equal to the ratio of the gauged and ungauged watershed areas. Dissimilarities in watershed characteristics may indicate that some other, but still relatively constant, value of f may be appropriate.

Paired streamflow measurements of WBDR at Walton and Trout Creek at Cannonsville (Fig. 1), made over the period 1951-62, were used to test this inflow model. The annual average streamflows of WBDR at Walton and of Trout Creek represent Q_{IG} and Q_{IU}, respectively, and f was calculated for each year and compared to the ratio of the watershed areas (0.149, Table 1). Calculated f values range from 0.125 to 0.158, with an average value of 0.141 and a standard deviation of 0.0099 (Table 3). The analysis further suggests that Trout Creek inflows can be predicted most accurately on an annual basis using $f = 0.141$, which is only about 5% less than the ratio of watershed areas. Modest differences in precipitation between the WBDR and Trout Creek could explain the variability in f and, to a lesser extent, the small deviation of the average value from the ratio of the areas of the watersheds.

The daily inflow from two ungauged portions of the reservoir watershed, the Trout Creek watershed and the ungauged portion of the WBDR watershed, were estimated using Eq. 4. The mouth of Trout Creek is well upstream of the location of the abandoned USGS gauge (Fig. 1). Based on the ratio of watershed areas (Table 1), the f value for the Trout Creek inflow to Cannonsville Reservoir is 0.064. Trout Creek inflows

Table 3.–Average annual flows ($m^3 \cdot s^{-1}$) for Trout Creek at Cannonsville and West Branch Delaware River (WBDR) at Walton, 1951 to 1962, and their ratio. Flows measured by USGS.

Year	Trout Creek	WBDR	f
1951	2.54	19.58	0.130
1952	2.18	16.41	0.133
1953	2.12	17.03	0.125
1954	2.27	16.49	0.137
1955	2.64	18.14	0.146
1956	3.03	20.01	0.152
1957	1.84	12.69	0.145
1958	2.72	19.24	0.142
1959	2.51	17.03	0.148
1960	2.43	17.63	0.138
1961	2.21	13.97	0.158
1962	1.82	11.96	0.152
1951-62	28.3	200	0.141

were calculated by

$$Q_{TC} = 0.064 \, Q_W \quad (5)$$

where Q_{TC} is the flow in Trout Creek at the shoreline of the reservoir, and Q_W is the measured daily average flow at Walton. In addition, the drainage area of the WBDR at Beerston (Fig. 1, the location of the mouth of WBDR) is 6.5% larger than the drainage area at Walton (Table 1). Thus inflows at Beerston (Q_B) were calculated by

$$Q_B = 1.065 \, Q_W \quad (6)$$

The simple inflow model (Eq. 4) was further applied to predict the inflow from the entire ungauged portion of the watershed for recent years of reservoir operation. Data for the period from 1979 through 1995 were analyzed, with f being determined in two ways. First, a value of f was determined for each year to achieve a yearly water balance. Annual estimates for f were determined by combining Eqs. 3 and 4, which yields

$$f = \frac{\Delta V + \Delta t \Sigma Q_o - \Delta t \Sigma Q_{IG}}{\Delta t \Sigma Q_{IG}} \quad (7)$$

where the summations are made over an entire year using daily average flows at Walton, and ΔV is the change in storage volume over the year. The numerator on the right hand side of Eq. 7 represents the integrated ungauged inflow volume computed from the water balance (Eq. 3); the ratio of this to the integrated gauged inflow yields a value of f for the period (Table 4).

An average value for f=0.381 was calculated for the 17 years of record, which is 9% higher than expected based on the ratio of watershed areas alone (299/860 = 0.348, see Table 1). In addition, the range of values was substantial, from 0.269 to 0.454 (Table 3). The coefficient of variation for the values of f determined according to Eq. 7 (0.138) is about twice the value obtained when considering just the WBDR and Trout Creek flow measurements (0.069, Table 3), consistent with the introduction of additional sources of variation or error inherent in the calculation of f according to Eq. 7. This additional variability or error is associated with the measurements of the components of Q_o and WSE, and the calculation of V (includes uncertainty in WSE and hypographic data; Fig. 2), which were not considered in the earlier calculations (Table 2).

To investigate the use of f=0.381 in long-term water balances, a continuous simulation of reservoir storage was made. The measured storage volume on the first day of 1979 was used to initialize the calculation of V in Eq. 3. A daily time step was used and Q_{IU} was computed using Eq. 4. Significant errors in computed storage volume occurred, with computed end-of-year storage values as much as 30% too low (at the end of 1991; Fig. 7). The root mean square (rms) error in computed daily reservoir storage was 15% of the observed storage (Table 5). Errors in the computed reservoir volume of this magnitude are unacceptable for the purposes of hydrothermal and water quality modeling (e.g., Chapra 1997, Thomann and Mueller 1987). Thus, the use of Eq. 4 to generate daily estimates of Q_{IU} with a constant value of f was rejected for support of these types of models.

A similar calculation was done to evaluate the use of annual, as opposed to long-term average, values of f. This calculation used f=0.396 to predict daily reservoir storage during 1995 (Table 3). The calculated end-of-year storage matched the observed value but there were significant errors over the year (Fig. 8), with rms and maximum errors of 3.4% and 8.4% (Table 5). Errors of this magnitude are also deemed unacceptable for these dynamic models.

Table 4.–Annual water budget calculations for Cannonsville Reservoir. Change in storage ΔV and outflow volume $\Delta t\sum Q_o$ from NYCDEP records; inflow volume $\Delta t\sum Q_{IG}$ from USGS records for WBDR at Walton; ungauged inflow volume $t\sum Q_{IU}$ computed from $\Delta V + \Delta t\sum Q_o - \Delta t\sum Q_{IG}$; f is ratio of ungauged to gauged inflow volume. All volumes are 10^6 m^3.

Year	ΔV	$\Delta t\sum Q_o$	$\Delta t\sum Q_{IG}$	$\Delta t\sum Q_{IU}$	f
1979	180	743	676	246	0.364
1980	-223	735	355	157	0.444
1981	58	572	451	180	0.398
1982	-89	671	421	161	0.382
1983	92	705	576	221	0.384
1984	-29	739	542	168	0.311
1985	114	421	386	149	0.385
1986	83	690	557	215	0.387
1987	-3	636	496	137	0.276
1988	-160	705	390	155	0.397
1989	115	568	481	202	0.420
1990	55	837	644	248	0.385
1991	-156	690	421	113	0.269
1992	70	542	421	191	0.454
1993	-77	837	534	226	0.423
1994	99	618	500	216	0.433
1995	-64	615	394	156	0.396
1979-95	64	11321	8245	3141	0.381

Refined Hydrologic Budget for Tributary Daily Flows

The goal of the refined hydrologic budget is to estimate the time series of daily ungauged inflows Q_{IU} to maintain a hydrologic balance (Eq. 3), given measurements of gauged inflows Q_{IG}, the total outflow from the reservoir Q_o, and the storage volume V. For purposes of this simulation, Q_{IG} is redefined as the inflow from both WBDR (at Beerston) and Trout Creek (determined from flows at Walton from Eqs. 5 and 6) and Q_{IU} is the inflow from the remaining 188 km^2 of the watershed.

This model is based on the concept that the change in storage volume over any time period of N consecutive

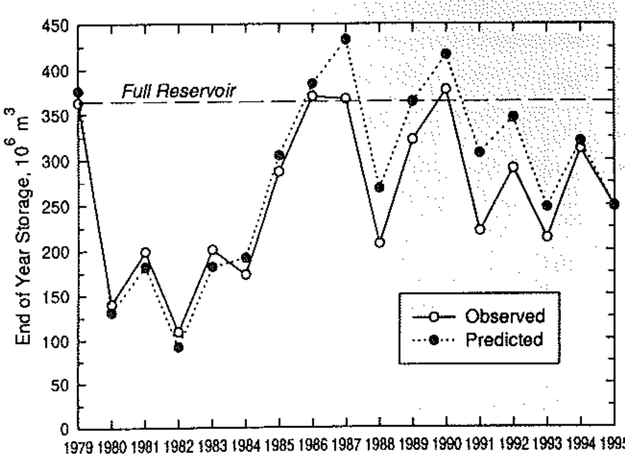

Figure 7.–Computed and measured end-of-year storage in Cannonsville Reservoir for 1979-1995. Computed results obtained using daily flow data and time step, with f = 0.381.

Table 5.–Root mean square and maximum errors in predicted daily reservoir storage using three methods.

Prediction method	rms error, percent	Maximum error, percent
Constant f=0.381; 17 years	15	39
Constant f=0.396; 1995	3.4	8.4
Refined method; 17 years	0.34	0.93

days must be balanced by the sum of inflows and outflows over that period. It is assumed that, through previous application of this same procedure, the ungauged inflow for the first N-1 days of an N-day period have been determined and the goal is to determine Q_{IU} on the last day of this period. An estimate of the ungauged flow can be obtained from

$$(Q_{IU})_i = \frac{1}{\Delta t}(V_{i+1} - V_{i-N+1}) + \sum_{j=i-N+1}^{i}(Q_O)_j - \sum_{j=i-N+1}^{i}(Q_{IG})_j - \sum_{j=i-N+1}^{i-1}(Q_{IU})_j + \frac{1}{\Delta t}E_{i-N} \qquad (8)$$

where the subscript i refers to the value of the quantity on the i^{th} day of the calculation, and E_i is the cumulative error in the water budget since the start of the simulation, and is calculated from

$$E_i = V_{i+1} - V_1 + \Delta t\sum_{j=1}^{i}(Q_O)_j - \Delta t\sum_{j=1}^{i}(Q_{IG})_j + \Delta t\sum_{j=1}^{i}(Q_{IU})_j \qquad (9)$$

The error E is not a true measure of the total error in the water balance calculation, but rather has a more narrow definition. It is known that the quantities V, Q_O, and Q_{IG} on the right side of Eq. 8 cannot be measured without error. At times, these errors are of such a magnitude that the computed value of $(Q_{IU})_i$ is negative. This result is of course not physically realistic; a negative value of $(Q_{IU})_i$ is not allowed and is replaced in the time series by zero. When this is done, a nonzero value of the error E results. The term E_{i-N} in Eq. 8 corrects the water budget on subsequent days of the calculation for the insertion of zero values of $(Q_{IU})_i$ in the series.

The water balance calculation described by Eqs. 8 and 9 proceeds from i = 1 to the number of days in the time series. The water budget calculations move ahead one day at a time. For the special case $i < N$ (the first days of the calculation period), the ungauged flow is estimated from a special form of Eq. 8 given by

$$(Q_{IU})_i = \frac{1}{\Delta t}(V_{i+1} - V_1) + \sum_{j=1}^{i}(Q_O)_j - \sum_{j=1}^{i}(Q_{IG})_j - \sum_{j=1}^{i-1}(Q_{IU})_j \qquad (10)$$

The quantity N represents the number of days over which a water balance is imposed on the reservoir. Ideally N should be made as small as possible in order to minimize the amount of averaging that is introduced into the calculation. However, as N is decreased, the amount of error calculated by Eq. 9 generally increases. Thus, some amount of averaging is introduced into the water budget calculation. A value of N= 10 days was used here, as this is arguably a lower bound for the temporal resolution of nutrient-phytoplankton models.

Due to variability in the measurements of storage volume V and the other terms in the hydrologic budget, and that negative values of ungauged inflows $(Q_{IU})_j$ are not allowed, the resulting values of $(Q_{IU})_j$ can be somewhat "noisy" (fluctuating between zero and nonzero values) and, at times, inconsistent with the measured flow at Walton. For this reason, a moving average of period N days is computed. The resulting smoothed estimate of the ungauged inflow $\overline{(Q_{IU})_i}$ is computed from

$$\overline{(Q_{IU})_i} = \frac{1}{N}\sum_{j=i-N/2}^{i+N/2}(Q_{IU})_j \qquad (11)$$

A calculation of reservoir storage was performed using Eqs. 8 through 11, with 1995 as a test year. The model predictions for January through March provide an example of how the model calculations proceed (Fig. 9). The "raw" estimates of $(Q_{IU})_i$ are computed from Eq. 8 without removing negative values. The

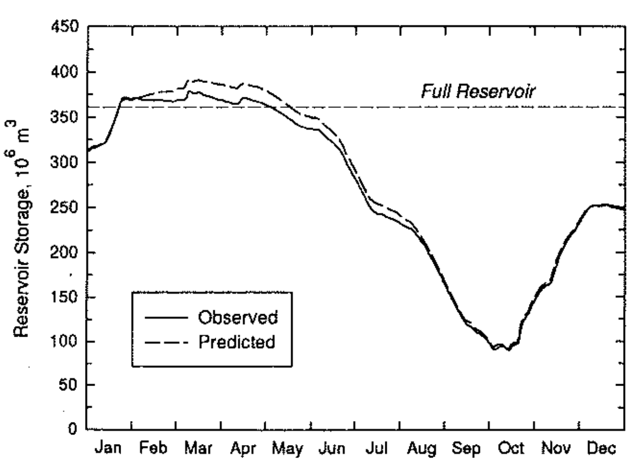

Figure 8.–Computed and measured daily storage in Cannonsville Reservoir for 1995. Computed results were obtained using daily flow data and time step, with f = 0.396.

negative values during the month of February indicate that the inflow from WBDR and Trout Creek computed by Eqs. 5 and 6 (with no inflow from the remainder of the watershed) is larger than the combined change in storage and outflow. This is a clear indication that there are errors in the measured components of the water budget over this period. During the winter months, the accuracy of USGS streamflow measurements at Walton can be reduced significantly due to ice at the gauge site. When the "raw" values of $(Q_{TU})_i$ are adjusted by setting negative values to zero (Fig. 9a), non-zero errors (Eq. 9) in the water budget calculation are introduced (Fig. 9b). The errors in the water budget occurring in February are removed by the middle of March by reducing the "raw" values of $(Q_{TU})_i$ (Fig. 9a). The smoothing operation described by Eq. 11 reduces some of the oscillations that are present in the corrected values of $(Q_{TU})_i$ (Fig. 9c). Use of the resulting time series of ungauged inflow to predict reservoir storage volume (according to Eq. 3) in 1995 results in a high degree of match with the observations; the rms and maximum errors in reservoir storage were 0.34% and 0.93%, respectively (Table 5). This is a major improvement over the previous simple inflow model. Thus the smoothed time series $(\overline{Q_{TU}})_i$ has been used in hydrologic, hydrothermal (Gelda et al. 1998, Owens 1998), and water quality (Doerr et al. 1998, Stepczuk et al. 1998b, Owens et al. 1998) model calculations.

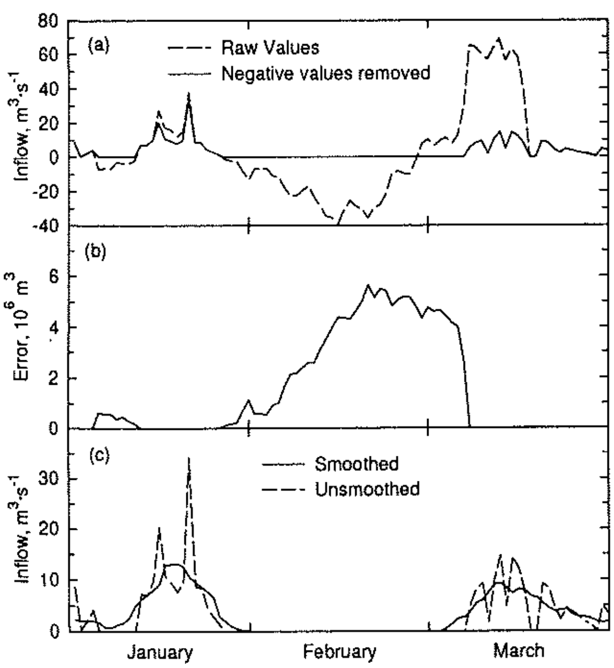

Figure 9.–Predictions of the refined hydrologic model for January through March 1995: (a) raw and corrected (negatives values removed) values of $(Q_{TU})_i$; (b) values of the error E_i associated with the corrected values of $(Q_{TU})_i$; and (c) the corrected and smoothed values of $(Q_{TU})_i$.

Management Perspectives

Flushing Rates and Response Times

The flushing rate of a lake or reservoir is a fundamental hydrologic characteristic with important implications for water quality management. The instantaneous completely mixed flushing rate is given by the ratio of outflow rate to storage volume, Q_o/V. Integrating this rate over time yields F, the number of flushes which occur over the period of integration, as given by

$$F = \int \frac{Q_o}{V}\, dt \tag{12}$$

Annual flushing rates were determined for years in the interval 1969-1995. Cannonsville Reservoir has flushed at a rather rapid rate, with an average of 2.6·y^{-1} and a range of 1.9 (1985) to 3.6 (1977; Fig. 10). Annual values of F were only weakly correlated ($r = 0.33$) with the annual average WSE values (as an inverse relationship).

The response time for a completely mixed system to come into equilibrium with a change in external loading of a conservative substance is a function of F as given by

$$\%\ \text{of steady state} = 100\ (1 - e^{F\Delta t}) \tag{13}$$

where Δt is the time period considered. Though this expression is an imperfect representation (e.g., stratified systems are not completely mixed), it remains a valuable diagnostic representation. For $F = 2.6 \cdot y^{1}$, approaching the lower bound of observed rates (Fig. 10), the reservoir would reach 93% of the steady-state concentration within 1 year of a change in loading, and 99% in 2 years. The percentages are higher for larger values of F, or for non-conservative substances, as long as substantial sediment release (feedback) does not occur (Chapra 1997). Thus, Cannonsville Reservoir would respond rapidly (1 to 2 years) to systematic reductions in phosphorus loading because of the high flushing rate of the impoundment and low release of phosphorus from the sediments (Effler and Bader 1998, Erickson and Auer 1998).

Support of Mechanistic Models

Hydrologic conditions are central features of the overall suite of forcing conditions for mechanistic hydrothermal and water quality models with respect to heat transfer, flushing and material loading/export. The primary concerns for support of model hindcasting are accuracy and the robustness of the cases to be tested

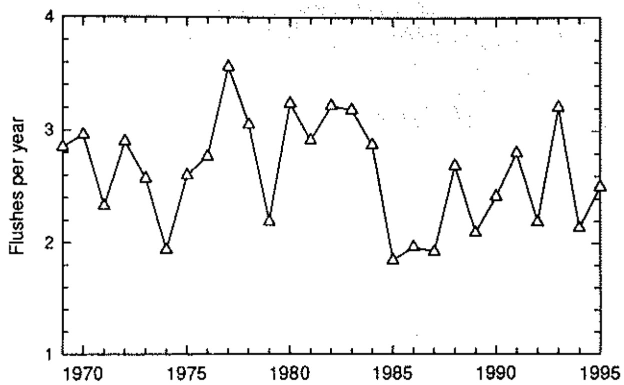

Figure 10.–Computed values of the annual number of flushes (*F*) in Cannonsville Reservoir over the period 1969-1995.

with the model. The accuracy of the various hydrologic measurements (and the hydrographic data) contributing to the hydrologic budget of Cannonsville Reservoir appears to be good based on: 1) direct comparison of certain components to independent measurements, 2) the consistency of inflow measurements from two different watersheds with the ratio of their drainage basin areas, and 3) the performance of overall hydrologic budget analyses. The imbalances in budget calculations encountered were well within the accuracy commonly associated with hydrologic measurements. Thus, these calculations of the hydrology of the reservoir can be used in mechanistic hydrothermal (Gelda et al. 1998, Owens 1998) and water quality (Doerr et al. 1998, Stepczuk et al. 1998b, Owens et al. 1998) models of the reservoir, presented subsequently in this issue, do not introduce substantial error or uncertainty.

The primary concern for model forecasting is the use of forcing conditions that are representative of a desired case, most often described as the critical case. Commonly, this has involved the selection of an infrequently occurring event or condition. Perhaps the best known specification of hydrologic conditions in water quality modeling is the use of the 10-year return period drought streamflow in the analysis of stream and river assimilative capacity (Thomann and Mueller 1987). While the return frequency of annual average components of the hydrologic budget have been identified here (Fig. 6), it is unclear what particular combinations of these components represent conditions which are critical to water quality. The selection of other natural forcing conditions, such as material loading and meteorology, must also be made.

The entire 30-year historical record of hydrologic as well as other ambient environmental forcing conditions may be used as the basis for forecasting of reservoir water quality in the analysis of water quality management programs. These conditions are inherently representative of the hydrology of the reservoir and include the effect of regional meteorological and reservoir operation. A similar approach has been used previously in hydrothermal model analyses for perturbed systems (Effler and Owens 1985, Owens and Effler 1989). This approach takes advantage of the richness of the record, (Figs. 3 through 5) and it eliminates the potentially arbitrary process of selection critical conditions. Distributions (based on 30 years) of predicted hydrothermal/water quality conditions can be generated by driving the entire record through the models (see Owens et al. 1998). Such distributions describe the responses of the reservoir to documented hydrologic conditions including operations. The effects of operational strategy can be evaluated by retaining the 30 years of inflows, but modifying outflows according to specified management scenarios. Critical hydrologic conditions can be identified objectively from review of the multiple-year predictions. Many managers will find such multiple-year predictions attractive, as a representation of the variability to be expected due to natural variations in ambient forcing conditions, and as a basis to evaluate the extent to which year-to-year differences in measured water quality reflect systematic changes.

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