Tennessee Valley Authority, 1101 Market Street, LP 5A, Chattanooga, Tennessee 37402-2801

April 17, 2008

10 CFR 52.75

Mr. Joseph M. Sebrosky
U.S. Nuclear Regulatory Commission
Two White Flint North T7E18
11545 Rockville Pike
Rockville, MD 20852-2738

In the Matter of)
Tennessee Valley Authority)

Docket Numbers 52-014 and 52-015

NUCLEAR REGULATORY COMMISSION (NRC) – BELLEFONTE NUCLEAR PLANT (BLN) – REVIEW OF BLN APPLICATION - WHITE PAPER - HYDROLOGIC ANALYSIS DESCRIPTION

Reference Letters:

1. Letter from Ashok Bhatnagar (TVA) to Mr. R. William Borchardt (NRC), Application for Combined License for BLN Units 3 and 4, dated October 30, 2007.
2. Letter from Jack Bailey (TVA) to Mr. R. William Borchardt (NRC), TVA Plan for Addressing NRC-Identified Issues Regarding BLN's Hydrology Calculation, March 14, 2008.

The purpose of this letter is to transmit to the NRC the (BLN) whitepaper entitled, "Hydrologic Analysis Description." This whitepaper completes the corresponding commitment for a whitepaper as described in the March 14, 2008, TVA letter to NRC (Reference Letter 2). The contents of the whitepaper will be further discussed with the NRC in a public meeting on April 25, 2008.

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TVA has developed the enclosed Hydrology Analysis Description to aide the NRC in their review of the probable maximum flood (PMF) calculation contained in the BLN Units 3&4 Combined License (COL) application (Reference Letter 1).

The whitepaper describes the methods, procedures, and programs (computer codes) used to determine the design basis flood level at the BLN site. TVA's model uses a total of nine computer codes to develop the inputs and outputs that produce elevation and discharge hydrographs at the BLN site including the Simulated Open Channel Hydraulic (SOCH) Code.
This whitepaper also discusses consideration of flooding from both severe hydrometeorological conditions and seismic activity to meet the criteria set forth in Regulatory Guide 1.59 Appendix A which has been replaced by American National Standards Institute (ANSI) ANSI/ANS-2.8-1992.

This submittal is intended to aide the NRC reviewer in understanding the SOCH model and expedite the review of the corresponding application sections. Further efforts are in progress to support the remaining commitments included in the March 14, 2008, TVA letter to NRC (Reference Letter 2). TVA intends to update the whitepaper to address NRC comments and questions during the week of May 26, 2008. These efforts are expected to support the June 2008 NRC site visit.

TVA respectfully requests NRC comments within two weeks of receipt of this letter. If there are any questions regarding this application, please contact Phillip Ray at 1101 Market Street, LP 5A, Chattanooga, Tennessee 37402-2801, by telephone at (423) 751-7030, or via email at pmray@tva.gov.

Sincerely,

Andrea L. Sterdis
Manager, New Nuclear Licensing and Industry Affairs,
Nuclear Generation Development & Construction

Enclosure
cc: See page 3.

cc (Enclosure):

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Bellefonte Nuclear Plant

White Paper

Hydrologic Analysis Description

Tennessee Valley Authority

April 17, 2008

Executive Summary

The Tennessee Valley Authority (TVA) developed the method of analysis, procedures and computer programs needed to determine the design basis flood levels for nuclear plant sites in the 1970s. Determination of maximum flood levels included consideration of the most severe flood conditions that can reasonably be predicted to occur at a site as a result of both severe hydrometeorological conditions and seismic activity. This process was followed to meet criteria set forth in Nuclear Regulatory Guide 1.59.

Early reviews of the process were completed with NRC staff in 1974 and again in 1978. These reviews consisted of an overall discussion of the hydrologic process/procedures followed to determine inflows to the TVA reservoir system as well as the stream course model development and calibration used to route flood events through the system. At that time there were no standard computer programs (codes) available that would handle unsteady flow and dam failure analysis. As a result of this early work and method development TVA developed a runoff and stream course modeling process for the TVA reservoir system that provided the basis for currently licensed plants (Sequoyah Nuclear Plant, Watts Bar Nuclear Plant and Browns Ferry Nuclear Plant). The Bellefonte Nuclear Plant (BLN) Unit 1 and Unit 2 Final Safety Analysis Report (FSAR) was also based on this process.

The BLN Unit 3 and Unit 4 Combined Operating License Application (COLA) was submitted using data and analysis that was determined for the original BLN FSAR (Unit 1 and Unit 2) and was documented in a 1998 reassessment. The 1998 reassessment of calculations was completed to document the earlier work and to evaluate the impact of dam safety modifications that had been completed by TVA. TVA's dam safety program started in 1982 to ensure consistency with Federal Guidelines for Dam Safety and similar efforts at other Federal agencies. In 1998, the analysis process and documentation was brought under the nuclear quality assurance process for the first time. Prior to this time the hydrologic analysis portion of the FSARs which meet criteria set forth in Regulatory Guide 1.59 were completed by the Water Management organization within TVA. While the Water Management organization operated under a quality assurance process it was not at the same level of detail as that followed by nuclear quality assurance.

The quality assurance audit conducted by NRC staff in early 2008 raised several questions related to past work regarding design basis flood level determinations when TVA's nuclear organization was not able to readily produce supporting materials for the review. While there is supporting data and analysis available to document the work, it is stored in file books and on microfilm stored in both Knoxville and Chattanooga. For a basin of nearly 24,000 square miles (mi^2) above the BLN site, with 21 TVA dams located above it, 45 watershed sub-basins to determine inflows to the system and several hundred miles of stream reaches to route all the flood events necessary to determine the controlling flood event, producing and documenting supporting data and analysis is a major task.

There also is the need to update, validate and document all of the inputs and computer codes. This White Paper describes (1) what the original codes computed, (2) application of those codes to determine design basis flood levels for the plant sites, and (3) the updates planned to further streamline and validate the computer code applications. TVA is currently assimilating supporting data for both the runoff and the stream course model. The nine (9) computer codes used in the reanalysis will be validated using current quality assurance procedures. Each of the sub-basins for the runoff model will be checked and validated with more recent storms where appropriate. The geometry used in the stream course model will be updated as appropriate with current bathymetry data and calibrated with more recent flood events such as the 2003 flood.

Once the runoff and stream course model is updated a reassessment of potential controlling flood events will be simulated with elevation and discharge information determined for BLN. This will include the controlling Probable Maximum Flood (PMF) and the seismically induced dam failure events. TVA will provide a comparison between information provided with previous computer codes and the current validated codes.

TVA remains confident that the process and computer code(s) application meet criteria set forth in Regulatory Guide 1.59 (Appendix A replaced by American National Standards Institute ANSI/ANS-2.8-1992) and accurately predicts the expected PMF and seismically induced flood levels at BLN.

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1. Purpose

The purpose of this White Paper is to describe the methodology, procedures, and programs (computer codes) used to determine the design basis flood level at the Bellefonte Nuclear Plant (BLN) site. This includes consideration of flooding from both severe hydrometeorological conditions and seismic activity to meet the criteria set forth in Regulatory Guide 1.59 Appendix A which has been replaced by American National Standards Institute (ANSI) ANSI/ANS-2.8-1992. There are a total of nine computer codes used to develop the inputs and outputs that produce elevation and discharge hydrographs at the BLN site. Eight of these codes provide input either directly or indirectly to the Simulated Open Channel Hydraulic (SOCH) code, which was used to determine elevation and discharge hydrographs at BLN. A list of the eight input codes to SOCH include: UNITGRPH, THIESSEN, FLDHYDRO, TRBROUTE, CHANROUT, DBREACH, CONVEYANCE, and WEIGHTED WIDTH. For each code there will be a discussion of its purpose, inputs, outputs and calibration as appropriate. This White Paper will also describe the overall application of these codes and the flood routing sequence required to determine elevation and discharge hydrographs at BLN to meet criteria set forth in ANSI/ANS-2.8-1992. This White Paper also describes the updates planned to further streamline and validate the computer code applications.

2. Background

The Tennessee Valley Authority (TVA) was formed in 1933 as a multipurpose federal corporation responsible for managing a range of programs in the Tennessee River Valley for the use, conservation, and development of the water resources related to the Tennessee River. In carrying out this mission, TVA operates a system of dams and reservoirs with associated facilities, shown in Figure 1. As directed by the TVA Act, TVA uses this system to manage the water resources of the Tennessee River for the purposes of navigation, flood control, power production and consistent with those purposes, for a wide range of other public benefits.

BLN is located on the right bank of Guntersville Reservoir at Tennessee River mile (TRM) 391.5. The site comprises approximately 1,500 acres between the Town Creek embayment and the Tennessee River, which is the major flooding source for the site. At the BLN site, the Tennessee River drains an area of 23,340 square miles (mi^2). The Tennessee River drainage area for the entire basin is shown Figure 2. Guntersville Dam is located downstream of the site at TRM 349.0. The drainage area at Guntersville Dam is 24,450 mi^2.

The Tennessee River basin drainage area covers 40,910 mi^2 and is divided into two distinct regions. One region is approximately 21,400 mi^2 upstream of Chattanooga, Tennessee, east of the Cumberland Mountains; and the other is about 19,500 mi^2 downstream of Chattanooga. The drainage area lies mostly in the state of Tennessee with parts in six other states—Kentucky, Virginia, North Carolina, Georgia, Alabama, and Mississippi as shown on Figure 2.

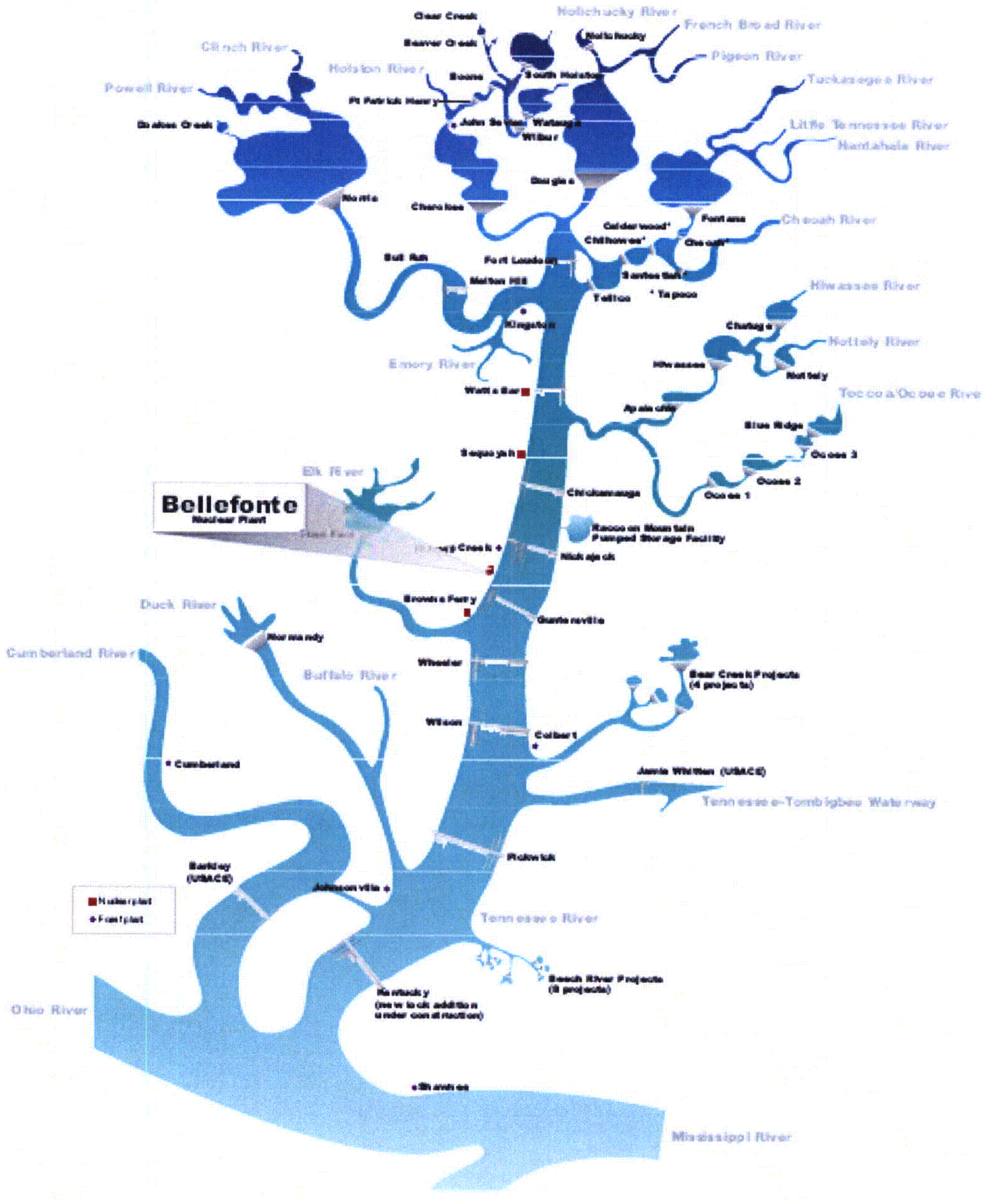


Figure 1. TVA's Water Control System

Tennessee River System

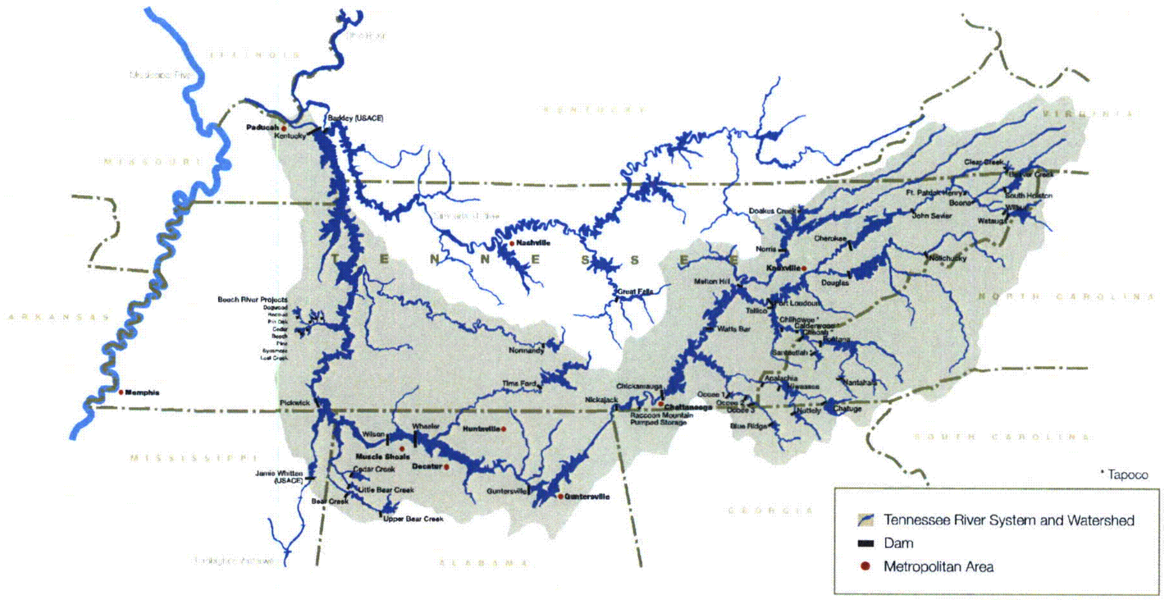


Figure 2. Tennessee River Basin Drainage Area

The eastern half of the Valley includes the slopes of the Blue Ridge and Great Smoky Mountains, where an abundant growth of timber covers the ground. The watershed is about 65 percent forested with much of the mountainous areas being 100 percent forested. The western half of the Valley is less rugged, with substantial areas of flat or rolling land occurring in middle Tennessee and along the western edge.

The total river fall from the maximum reservoir surface at Watauga Dam (highest elevation of the reservoir system) to the minimum tailwater surface at Kentucky Dam (lowest elevation on the system) is 1,675 feet in 828.6 miles. The Tennessee River has a fall of 515 feet in 579.9 river miles from the top of the Fort Loudoun Dam spillway gates to the minimum tailwater elevation at Kentucky Dam. The mainstream (Tennessee River) fall is gradual, except in the Muscle Shoals area of Alabama, where a drop of 100 feet is found in a stretch of less than 20 miles.

The climate of the watershed is humid temperate. Mean annual rainfall over the Tennessee River Basin amounts to about 51 inches, varying during the past 118 years of recordkeeping between a low of 31 inches in 2007 and a high of 65 inches in 1973. The heaviest concentrations of rainfall occur in certain mountainous areas along the headwaters of the tributaries, where mean annual rainfall reaches over 90 inches. In portions of the French Broad, Clinch, and Holston Valleys, the mean annual rainfall is as low as 40 inches.

Rainfall occurs relatively evenly throughout the year. The lowest monthly rainfall average of 3.0 inches occurs in October. The highest monthly average is 5.4 inches in March, with December and January a close second with an average of about 4.8 inches. The average rainfall and runoff by month are shown in Figure 3.

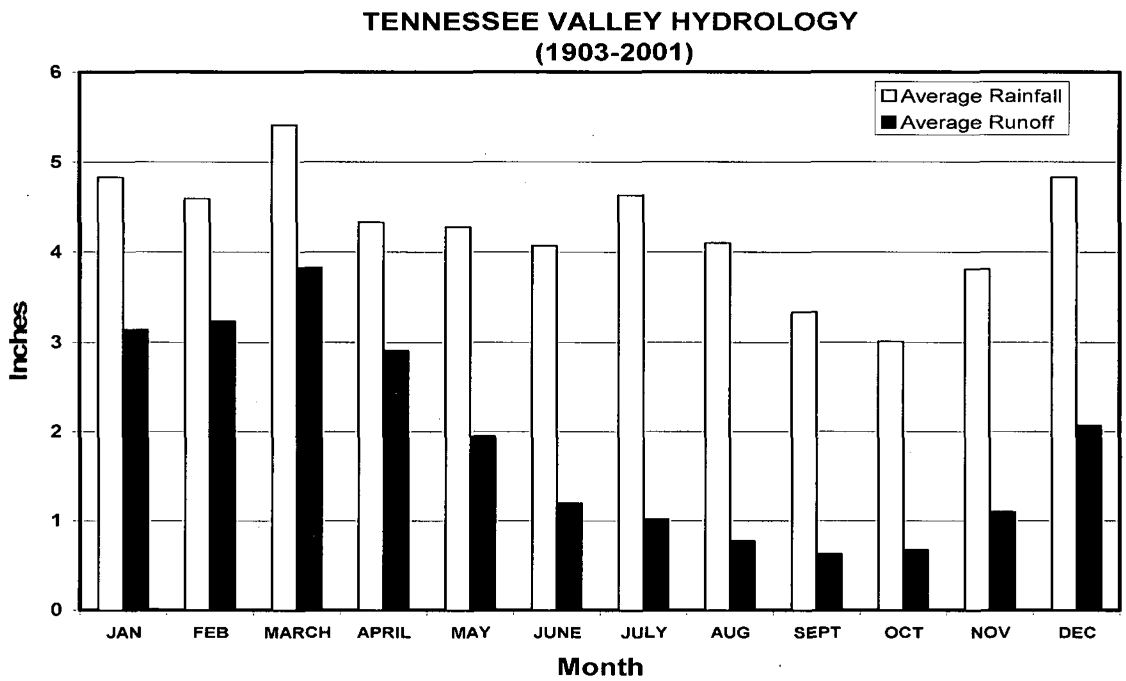


Figure 3. Average Rainfall and Runoff (Source – TVA data)

The major flood-producing storms at the BLN site are the result of winter frontal system events, which occur in the months of January through early April. A review of historical floods in the Tennessee Valley indicated that a high percentage of the storms were typically 9-day events resulting from the movement of frontal systems across the region.

A primary purpose of the TVA water control system is flood reduction, as defined by the TVA Act of 1933, with particular emphasis on reducing flood levels at Chattanooga. The BLN site is located 72.7 miles below Chattanooga and benefits from this system and the watershed configuration.

There are currently 21 reservoirs in the TVA system upstream from BLN, 14 of which have substantial reserved flood detention capacity during the primary flood season. Table 1 provides a list of available flood detention capacity for these 14 TVA projects above BLN. The remaining seven (7) TVA projects have no reserved flood storage. In addition there are six dams owned by the Aluminum Company of America (ALCOA) which can contribute to flood reduction, but do not have dependable reserved flood detention capacity.

The flood detention capacity reserved in the TVA system varies seasonally, with the greatest storage available during the January through March flood season. The system flood detention capacity above BLN varies from 4.2 inches of runoff on January 1 to 4.1 inches of runoff on March 15, decreasing to 1.3 inches of runoff during the summer and fall.

Table 1. Available Flood Detention Capacity

Project	Flood Storage January 1 (acre - feet)	Flood Storage March 15 (acre – feet)	Flood Storage Summer (acre – feet)
Tributary			
Boone	75,800	48,200	12,900
Chatuge	62,600	62,600	13,900
Cherokee	749,000	749,000	118,000
Douglas	1,080,000	1,020,000	238,000
Fontana	514,000	514,000	73,400
Hiwassee	206,000	206,000	35,000
Norris	1,113,000	1,113,000	512,000
Nottely	61,600	61,600	12,300
South Holston	253,000	220,000	106,000
Tellico	120,000	120,000	32,000
Watauga	153,000	153,000	109,000
Main River *			
Fort Loudoun	111,000	111,000	30,000
Watts Bar	379,000	379,000	165,000
Chickamauga	345,000	345,000	115,000
Total	5,223,000	5,102,400	1,572,500

* Note – Nickajack (not shown) is run-of-river project with zero flood storage.

Reservoir operating guidelines are implemented as prescribed operating ranges of reservoir levels throughout the year. TVA represents these guidelines in graphs called guide curves, which show the reservoir levels for navigation, flood control, recreation, and other operating objectives. Guide curves also depict the volume of water available to TVA for hydropower generation and other beneficial uses.

Guide curves for mainstem and tributary reservoirs have different characteristics. Mainstem guide curves typically allow for a much smaller range of reservoir elevation change. Tributary guide curves include a larger change in reservoir elevations over the annual cycle and usually include a discretionary operating zone (the area between the flood guide and Minimum Operations Guide [MOG]). Because guide curves specify certain periods for raising or lowering the reservoirs, they substantially affect seasonal releases in tailwater areas downstream of the dams. Each project has its own guide curve.

These project-specific guide curves are based on original project allocations and subsequent modifications, many years of historical flows, flood season conditions, and experience with project and reservoir system operations. Reservoir operations based on the guide curves maintain project storage volume available for flood control within the watershed at any given time of year, as well as the amount of stored water needed to meet other purposes such as year-round navigation, power generation, reservoir recreation, water quality, waste assimilation, and other environmental resource considerations. Figures 4 and 5 show generic tributary and mainstem reservoir guide curves.

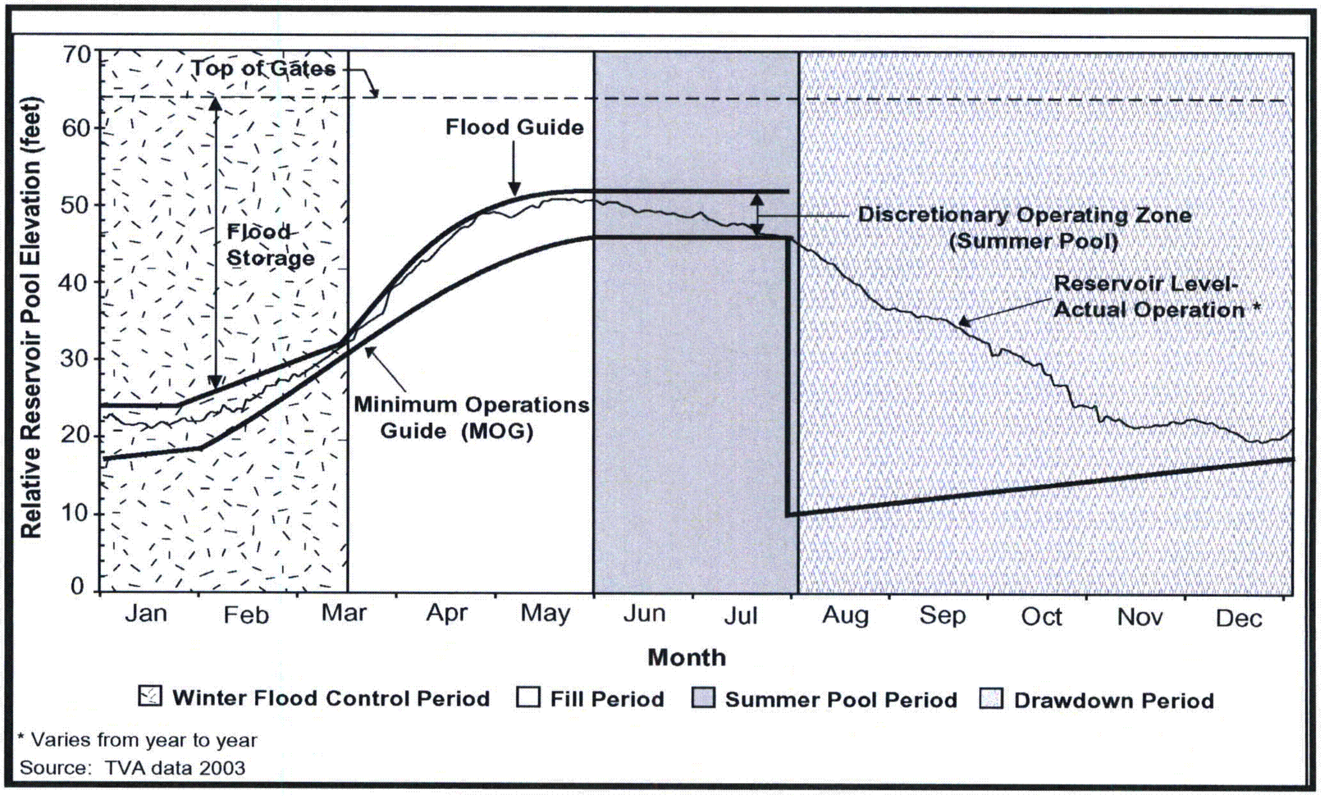


Figure 4. Generic Tributary Reservoir Guide Curve

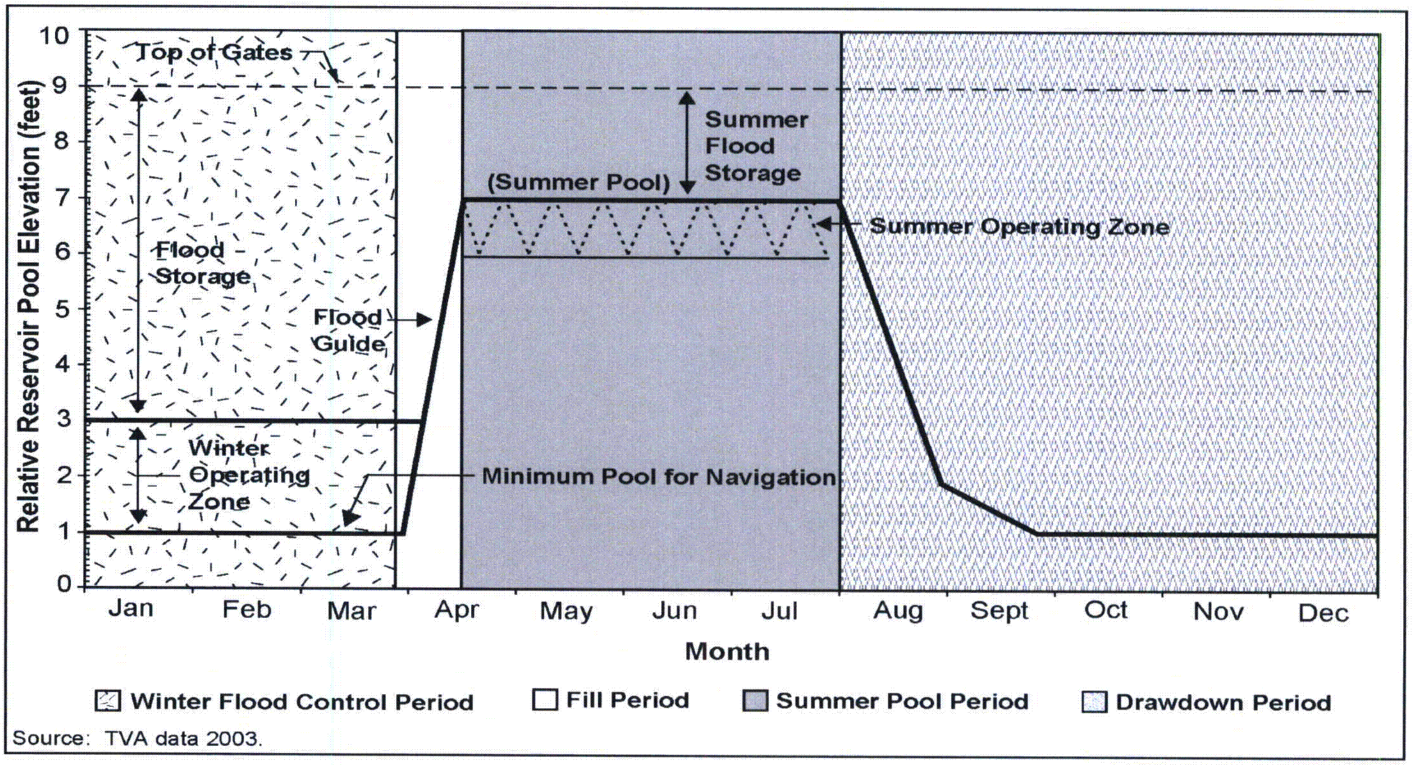


Figure 5. Generic Mainstem Reservoir Guide Curve

TVA operating guidelines must be flexible enough to respond to unusual or extreme circumstances in the system that are beyond TVA's control. The most important of these is variation in rainfall and runoff, at times resulting in low inflow conditions (droughts) or high inflow conditions (floods) that substantially increase the difficulty in meeting the multiple needs of the system.

The tributary reservoirs provide a significant portion of the system's flood storage; their reservoir pool levels may vary substantially over the annual cycle.

To achieve multiple reservoir system elevations, the guide curve must include operational flexibility. Managing the tributary reservoir levels within a discretionary operating zone creates this flexibility. The lower limit of this zone is the MOG. When a reservoir is at or below its MOG, only minimum flows are released.

The upper limit of the discretionary operating zone is the flood guide. Reservoir levels generally are not allowed to exceed this limit because the flood guide controls the minimum amount of flood storage available in a reservoir. By limiting reservoir elevations to a level equal to or lower than the flood guide, TVA is assured that flood storage necessary to minimize flood risk is available for use. Occasionally, temporary fill to higher levels occurs when high flows are regulated, and lower levels may occur for power generation emergencies.

The generic guide curve for a mainstem reservoir (Figure 5) shows that the schedules for drawdown and fill are somewhat similar to those for a tributary reservoir. The drawdown for a mainstem reservoir is generally much smaller than that for a tributary reservoir because of the difference in reservoir characteristics. All mainstem projects have a seasonal fluctuation zone, which is followed to the extent practicable.

- **January-March**. Reservoir elevations are lowest from January through late March, the period of highest runoff and flood risk, as shown on Figure 5. Pools are maintained within a 1- to 2-foot winter operating zone to the extent possible, except when regulating high flows. The bottom of this winter regulating zone is the lowest elevation to which the reservoir is drawn while still meeting minimum navigation depth requirements.

- **April**. From late March through the middle of April, reservoir elevations are raised to the summer pool level as runoff and system demands allow.

- **Mid-April through Late Summer**. Reservoirs are maintained at summer operating levels until seasonal drawdown begins. Normal operation includes a band of reservoir fluctuations, called the summer operating zone. Fluctuations of reservoir levels in this zone are used for power generation; and for mosquito control operations on Chickamauga, Guntersville, Wheeler, and Pickwick Reservoirs.

  Occasionally, temporary fills to higher levels occur when high flows are regulated, and lower levels may occur for power generation emergencies.

- **Fall Drawdown**. Reservoir elevations are lowered to the winter operating level beginning at various dates through summer and fall.

Appendix A provides the guide curves for each of the tributary and mainstem reservoirs above BLN. The maximum, minimum, and median levels are also shown on these guide curves.

Flood control above BLN is provided principally by the 11 tributary reservoirs. Tellico Dam is counted as a tributary project because it is located on the Little Tennessee River although, because of a canal connection with Fort Loudoun Reservoir, it also functions as a mainstem dam. On March 15, near the end of the flood season, these tributary reservoirs provide a minimum of 4,267,400 acre-feet of detention capacity, equivalent to 3.4 inches of runoff on the 23,340 mi^2 drainage area above BLN. This is 84% of the total flood storage available above Nickajack Reservoir (Nickajack has no flood storage). The three main river reservoirs—Fort Loudoun, Watts Bar and Chickamauga provide an additional 835,000 acre-feet of flood storage which is equivalent to 0.7 inches of runoff making a total of 5,102,400 acre-feet of detention capacity, equivalent to 4.1 inches of runoff above BLN.

The reservoir system and its operation is an important element in understanding the studies conducted to determine maximum elevation and discharge hydrographs at the BLN site. Computer codes were used to develop, calibrate, and verify the runoff and stream course models needed to compute elevations and discharges at the BLN site. These computer codes will be described in this White Paper and will be used to simulate a series of flood events through the reservoir system. Computer codes used in the original analysis were single purpose codes for a main frame computer.

3. Summary of Computer Codes

Computer Code	Description
UNITGRPH	Computes unit hydrograph for each sub area from historical flood data.
THIESSEN	Determines weighting factors for average basin rainfall.
FLDHYDRO	Determines inflow from unit hydrographs and rainfall
TRBROUTE	Routing of hydrograph from one point to another using different routing procedures
CHANROUT	Determines routing method coefficients
DBREACH	Determines earth embankment failure time based on soil type and period of overtopping during a flood
CONVEYANCE	Determines cross sectional area (A) and composite hydraulic radius ($R^{2/3}$) for SOCH geometry
WEIGHTED WIDTH	Determines equivalent weighted width (B) to account for reservoir volume in SOCH geometry
SOCH	One dimensional unsteady flow model that computes elevation, discharge, and average velocity at selected locations.

4. Runoff and Stream Course Models

The drainage area for the runoff model used to determine Tennessee River flood hydrographs at BLN (Unit1 and Unit 2 FSAR) included 50 unit areas and covers the total watershed above Guntersville Dam which is located downstream of BLN. For the update and verification, the runoff model will be divided into 45 unit areas (6 small sub-areas will be combined into 1). Unit hydrographs as determined by the UNITGRPH code are used to compute flows from unit areas. The unit area flows determined by the flood hydrograph code (FLDHYDRO) are combined with appropriate time sequencing or channel routing procedures using the TRBROUTE code to compute inflows into the most upstream reservoirs which in turn are routed through the reservoirs using the TRBROUTE code. Resulting outflows are combined with additional local inflows and carried downstream using appropriate time sequencing or routing procedures (TRBROUTE code), including unsteady flow routing (SOCH code). Figure 6 shows unit areas of the watershed upstream from Guntersville Dam.

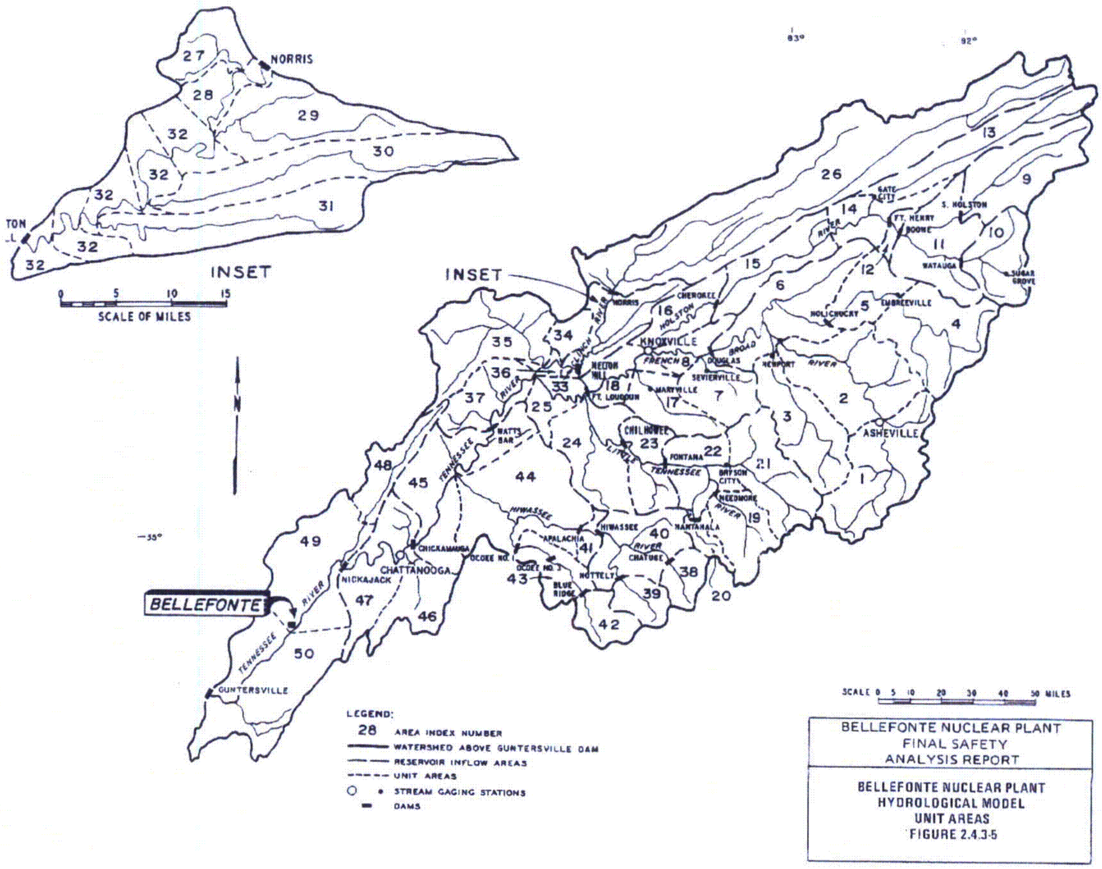


Figure 6. Watershed Unit Areas
(BLN Unit 1 and Unit 2 FSAR)

4.1 Runoff Model

The watershed runoff model has evolved over many years. It has been used by TVA for siting nuclear plants and for dam safety studies. The UNITGRPH code was used to develop unit hydrographs for each area. Maximum historical floods were used to develop the unit hydrographs for the unit areas where this information was available. The historic flood information was obtained from (1) recording stream gages, (2) tributary dam headwater elevation, discharge, and storage relationships which permitted calculation of the flood and (3) estimates on ungaged watersheds where historic flood information upstream and downstream of the watershed was available to make estimates.

For those unit areas where flood hydrographs were not available, synthetic unit hydrographs were developed. These synthetic unit hydrographs were developed based on relationships from similar watersheds relating the unit hydrograph peak flow to the drainage area size and time to peak in terms of watershed slope and length developed from unit hydrograph parameters computed where discharge data was available.

The historical floods used in developing the unit hydrographs were large out of bank events throughout the channels in the basin. Therefore, use of the runoff model is considered to be adequate to predict Probable Maximum Flood (PMF) flows. This conclusion is based in part upon studies by others and unpublished work by TVA that indicates the assumption of linearity in unit hydrographs is valid when they are developed from large, out of bank floods produced by major storms. These unit hydrographs will duplicate such storms.

4.1.1 Unit Hydrographs

A unit hydrograph, with a volume of one inch of runoff, for each unit area where historical flood information was available was developed using the UNITGRPH code from an analysis of two or more of the largest floods of record. The historic flood flows used came directly from stream gages or were computed from reservoir headwater and discharge records. Reverse reservoir routing was used to obtain flood inflows at the tributary reservoirs from reservoir headwater elevation and discharge records. Flood flows for ungaged areas where discharge data was available upstream and downstream of the ungaged area were determined using routing procedures, which included time sequencing, channel routing (CHANROUT and TRBROUTE code), or unsteady flow routing (SOCH code). Average basin rainfall and its time distribution were determined from rainfall records using the Thiessen method.

The UNITGRPH code, which makes use of matrix algebra to determine the best fit unit graph from a single or a series of complex floods using statistical curve fitting techniques, was used to determine the unit hydrograph (Reference 1). Verification of the adopted unit hydrograph for each unit area was made for its ability to duplicate historic floods.

An example of the unit hydrograph developed for the 468 mi^2 watershed above Watauga Dam is shown on Figure 7. The two large floods of March 1963 and March 1965 were used in the unit hydrograph development. Single-unit hydrographs were developed for each flood and combined to produce a composite. The composite unit hydrograph was used to duplicate the 1963, 1965, and the March 1973 floods. The verification of the three floods is shown on Figures 8, 9, and 10, respectively.

The procedure defined above was used to calibrate and verify each of the original 50 unit areas that make up the river basin above BLN. An updated verification of the unit hydrographs will be made by use of the HEC1 code with more recent flood events as appropriate. An explanation will be provided for any differences identified between the original work and the updated unit hydrographs.

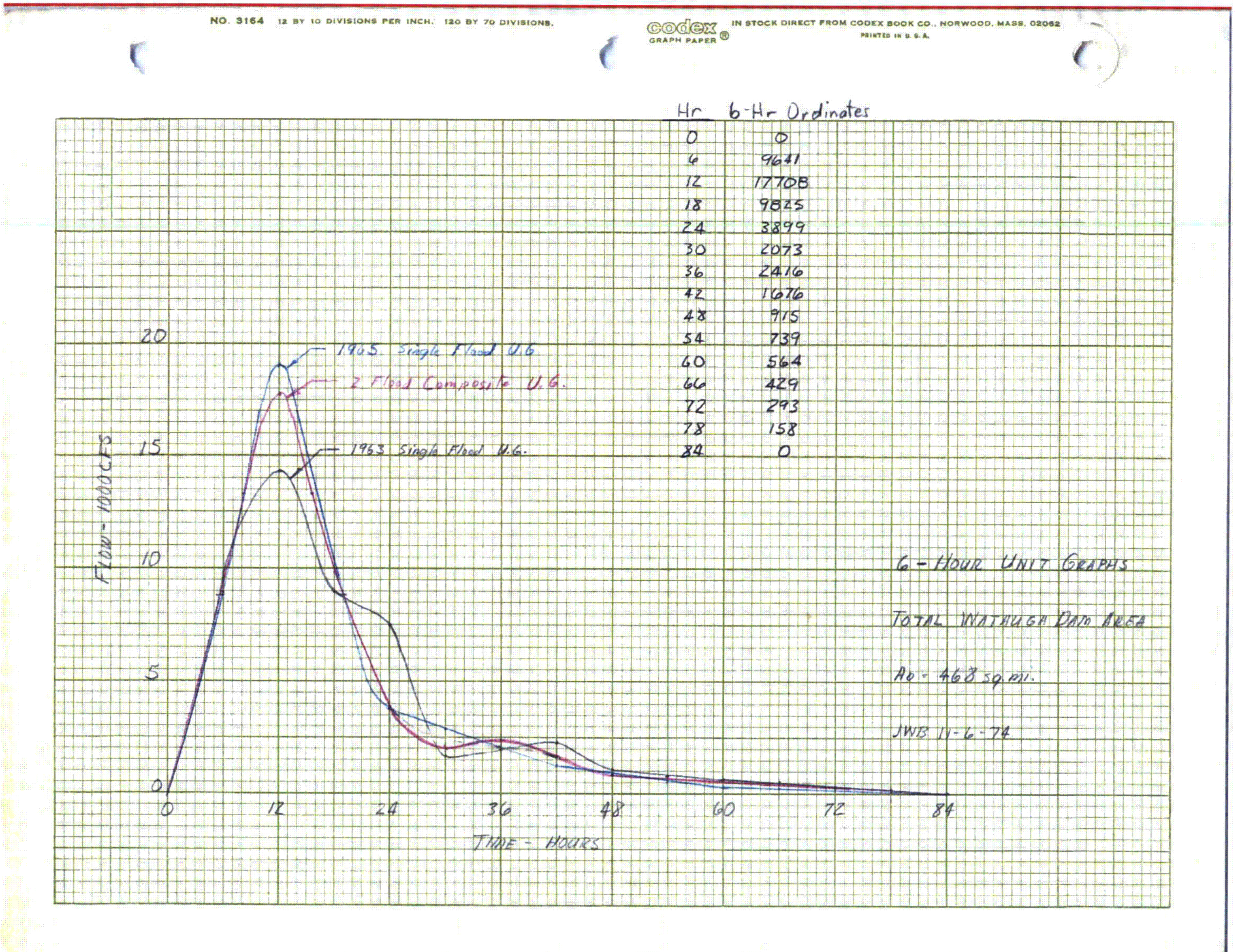


Figure 7. Example Unit Hydrograph Area above Watauga Dam

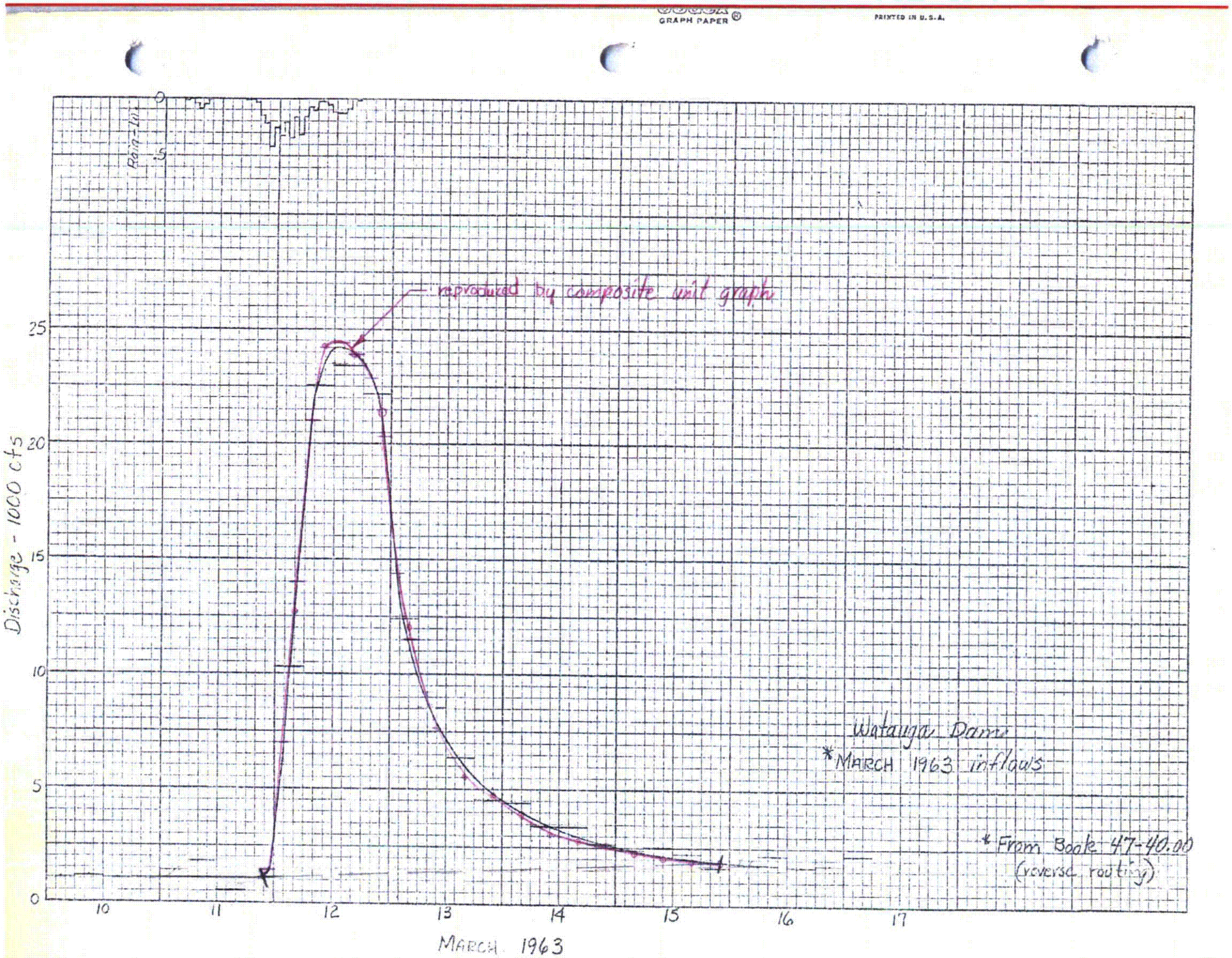


Figure 8. 1963 Flood Unit Hydrograph Area above Watauga Dam

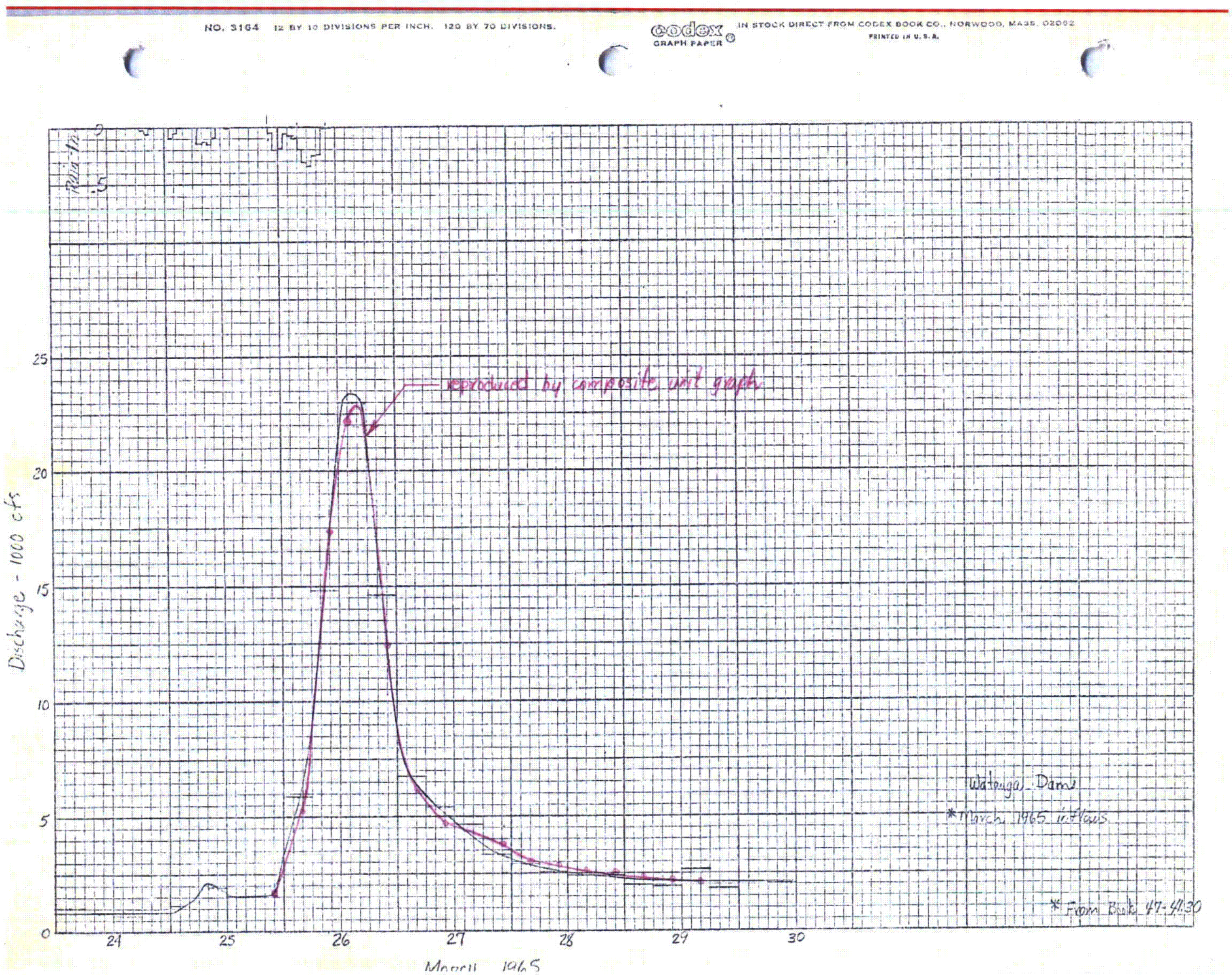


Figure 9. 1965 Flood Unit Hydrograph Area above Watauga Dam

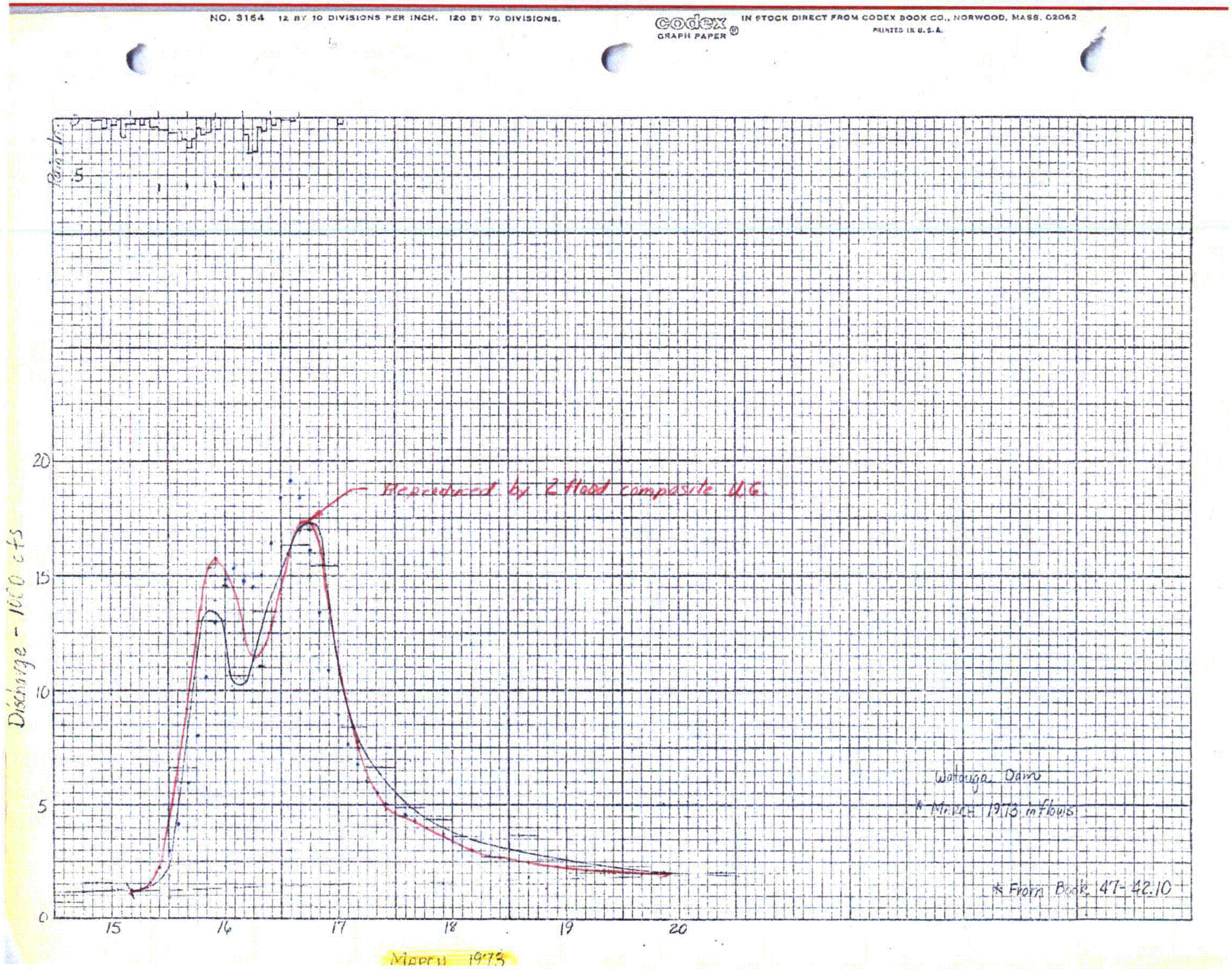


Figure 10. 1973 Flood Unit Hydrograph Area above Watauga Dam

4.1.2 Flood Hydrographs

4.1.2.1 Historic Floods

The unit hydrographs were used to compute flows from each of the 50 unit areas using the FLDHYDRO code. The flows were then combined with the TRBROUTE code with time sequencing or Muskingum channel routing procedures to compute inflows into upstream tributary reservoirs. The Goodrich semi-graphical method and flat pool storage assumptions were used to route the flows through the tributary reservoirs. The resulting outflows, together with additional local inflows at selected locations, were input to the stream course unsteady flow model (SOCH code) used on the main river.

The historic floods of March 1963 and March 1973 were used to verify the runoff model. An example of how the flows were combined above Douglas Dam is shown schematically on Figure 11. Computed flows from the six unit areas above Douglas Dam were combined by Muskingum or lag routing using the TRBROUTE code to obtain inflows into the reservoir. The Muskingum routing method involved an iterative process to determine routing coefficients thus lag routing was used in most cases and was considered conservative. The lag routing would translate the hydrograph to the point of interest downstream with no attenuation. The verification for the March 1963 and March 1973 floods at Douglas Dam is shown on Figures 12 and 13, respectively. Also, the verification at Cherokee Dam from the combination of the seven unit areas above Cherokee Dam are shown on Figures 14 and 15 for the March 1963 and March 1973 floods, respectively.

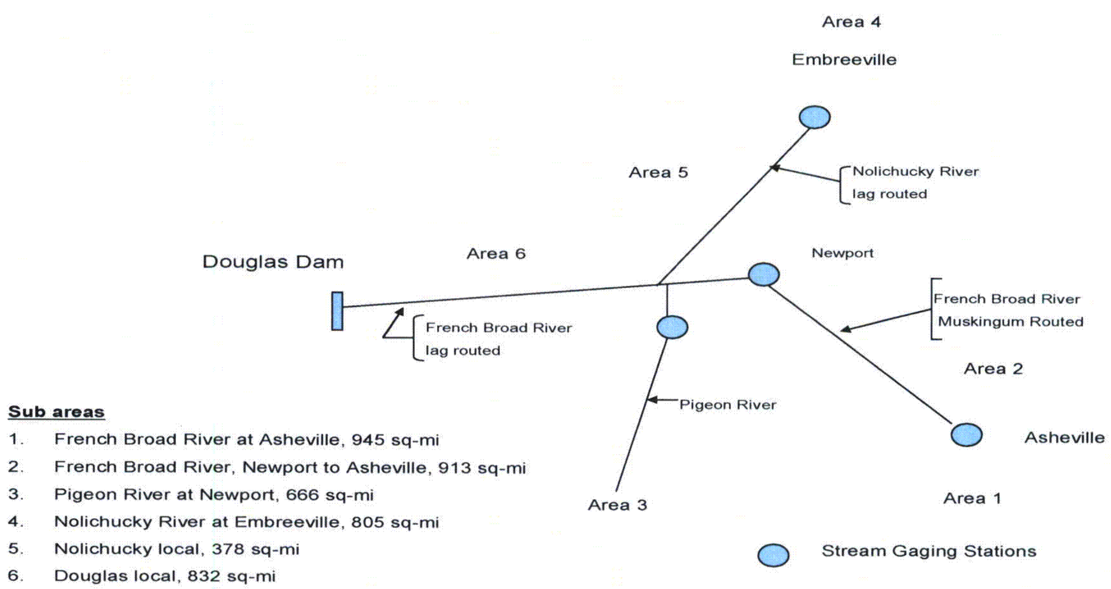


Figure 11. Flow Schematic – Flows Combined Above Douglas Dam

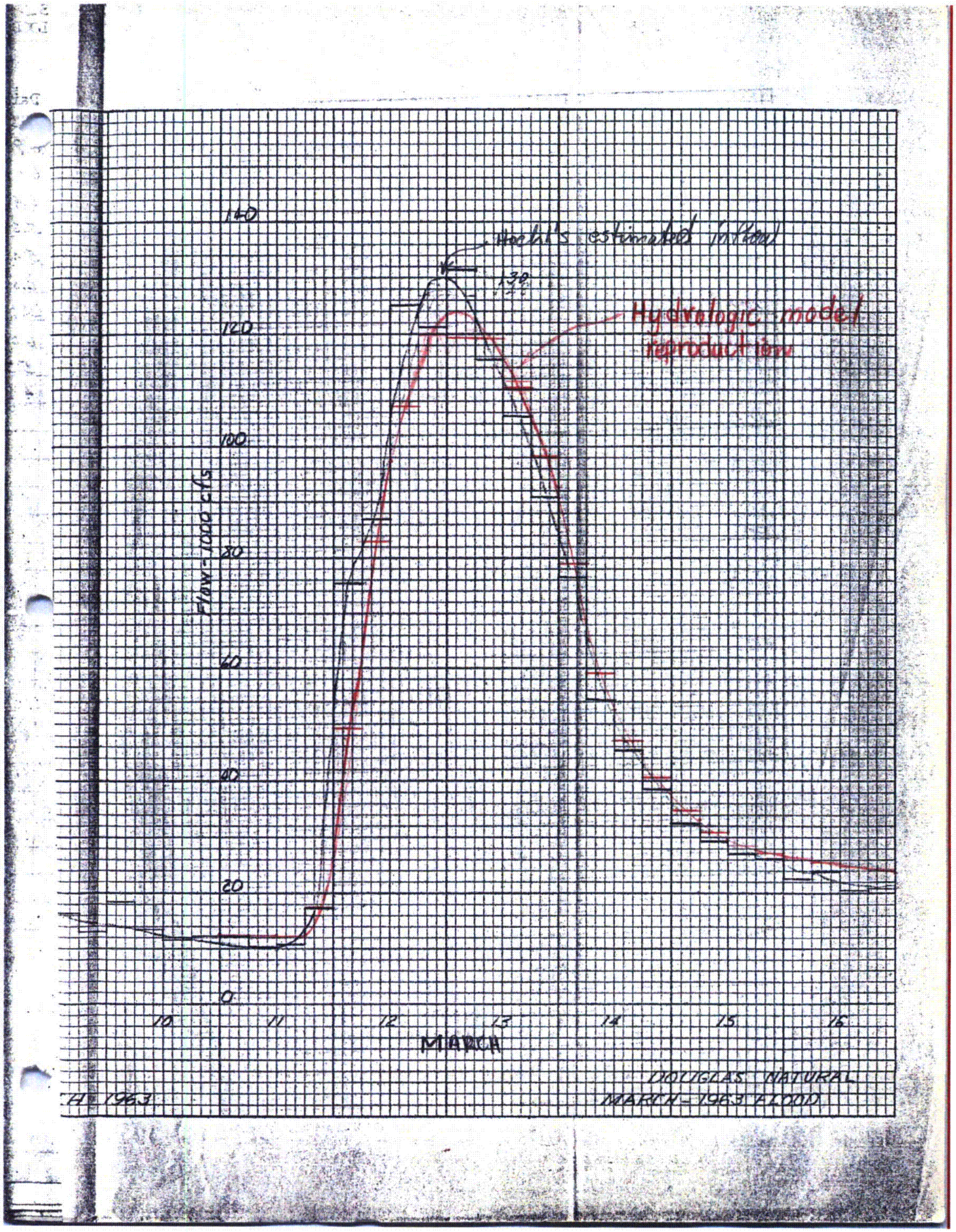


Figure 12. March 1963 Flood, Douglas Dam

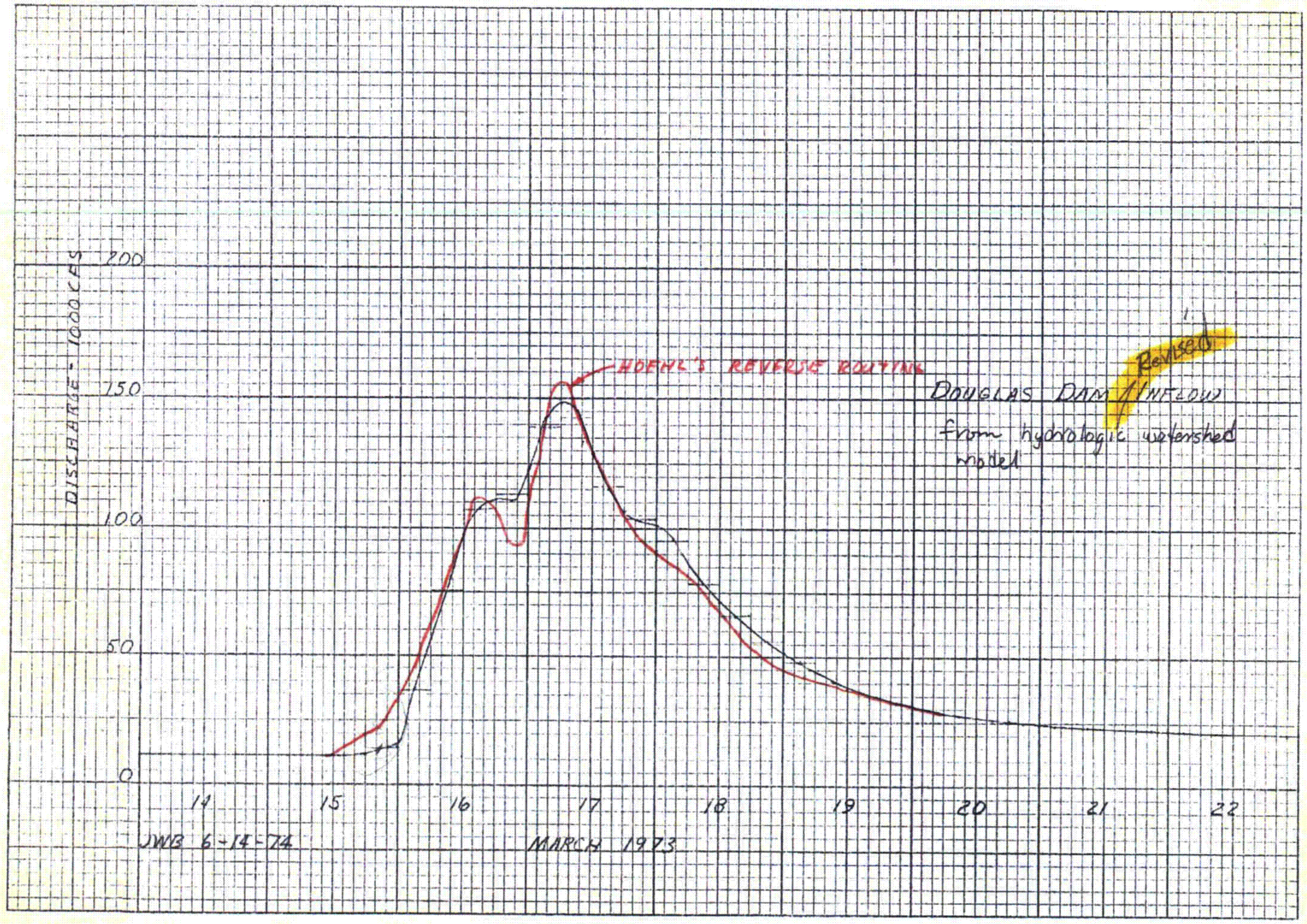


Figure 13. March 1973 Flood, Douglas Dam

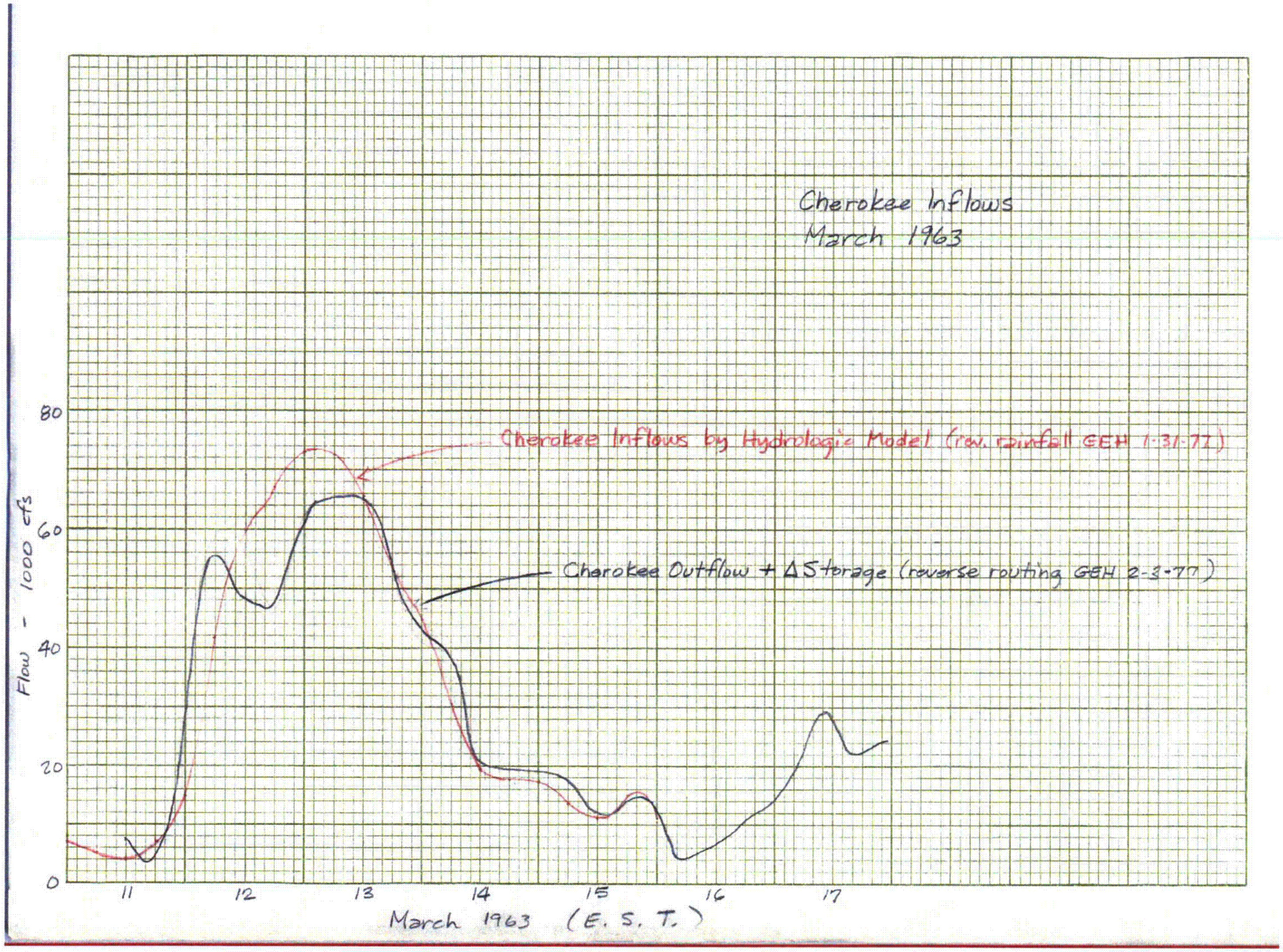


Figure 14. March 1963 Flood, Cherokee Dam

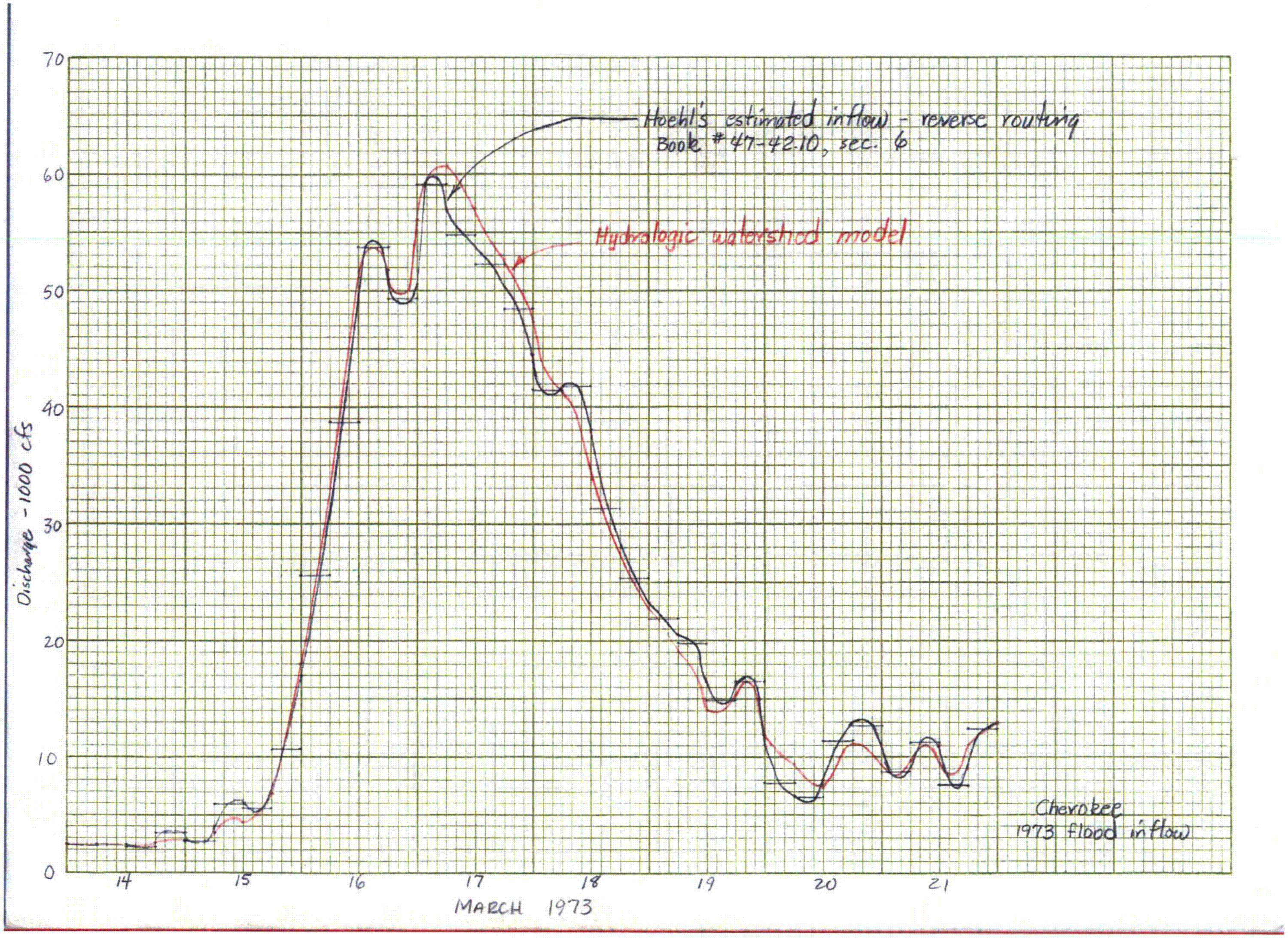


Figure 15. March 1973 Flood, Cherokee Dam

4.1.2.2 Probable Maximum Flood (PMF)

The criteria set forth in ANSI/ANS-2.8-1992 were followed in determining the PMF. The PMF was determined from consideration of all potentially critical areal and seasonal variations of probable maximum precipitation (PMP) on the watershed above BLN.

A PMF computation involves selection of a sequence of meteorological and hydrologic events. These include the principal storm rainfall, antecedent storm rainfall, time and areal distribution of rainfall, infiltration loss rates, and the hydrograph determination.

4.1.2.2.1 Probable Maximum Precipitation (PMP)

For the Tennessee Valley region, studies by TVA have shown that major floods are typically caused by a pair of storms with a 3-day dry interval between them. For the Tennessee Valley region, antecedent storm rainfall depths vary from 15 to 50 percent of PMP depending on storm duration, location and size of watershed, and season of occurrence.

PMP was defined for TVA by the U.S. Weather Bureau in Hydrometeorological Report (HMR) No. 41 (Reference 2). This report defines depth-area-duration characteristics, seasonal variations, and antecedent storm potentials.

Two storms with three possible isohyetal patterns (map showing contours of equal precipitation) and seasonal variations described in HMR Report No. 41 were examined to determine which would produce maximum flood levels at BLN. One storm would produce PMP depths on the 21,400 mi^2 watershed above Chattanooga. Two potentially critical isohyetal patterns are presented in HMR Report No. 41 for this storm. The storm critical to this study is a March storm with the "downstream pattern" shown on Figure 16 along with the maximum 6-hour storm depths. Figure 17 shows the total rainfall amounts for the 72-hour storm.

Another storm described in HMR Report No. 41 would produce PMP depths on a 7,980 mi^2 watershed centered in the Tennessee Valley below the major tributary dams. The isohyetal pattern for the 7,980 mi^2 storm is shown on Figure 18 along with the maximum 6-hour storm depths. The pattern is not orographically fixed and can be moved parallel to the long axis northeast and southwest along the Tennessee Valley. The storm was centered at Bulls Gap, Tennessee, 50 miles northeast of Knoxville, shown on Figure 18.

Potential storm amounts differing by seasons were analyzed in sufficient number to ensure that the March storms would be the controlling events. In addition different storm centerings were investigated to ensure that the most critical position was used.

Both the 21,400 mi^2 and 7,980 mi^2 storms were 9-day events. A 3-day antecedent storm was postulated to occur 3 days prior to the 3-day PMP storm in all PMF determinations. Depths equivalent to 40 percent of the main storm were used for the antecedent storms in both the 21,400 and 7,980 mi^2 storms with uniform areal distribution as recommended in HMR Report No. 41.

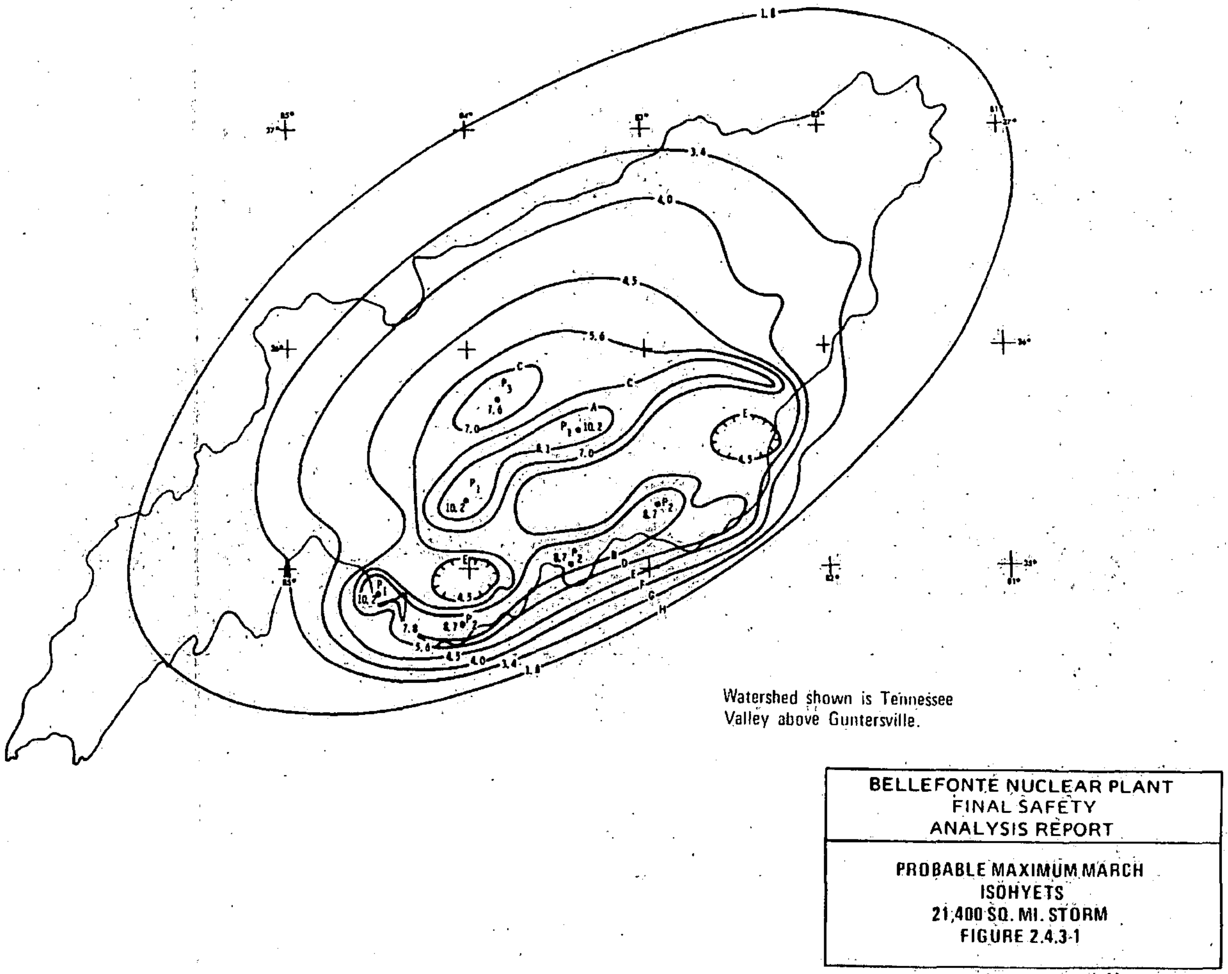


Figure 16. 21,400 Mi^2 Storm – 6-Hour Totals
(Source BLN Unit 1 and 2 FSAR)

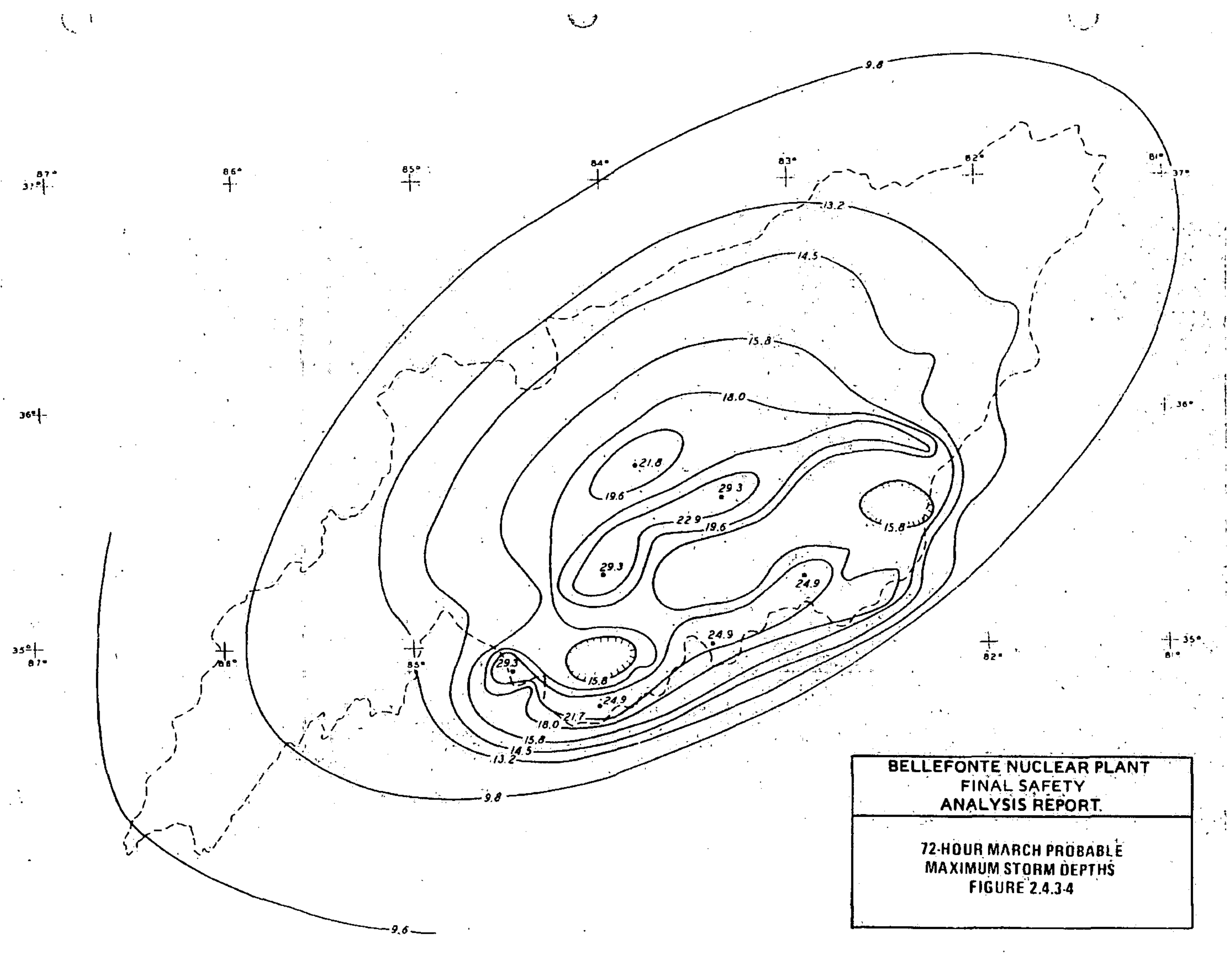


Figure 17. 21,400 Mi^2 Storm – 72-Hour Totals (Source BLN Unit 1 and Unit 2 FSAR)

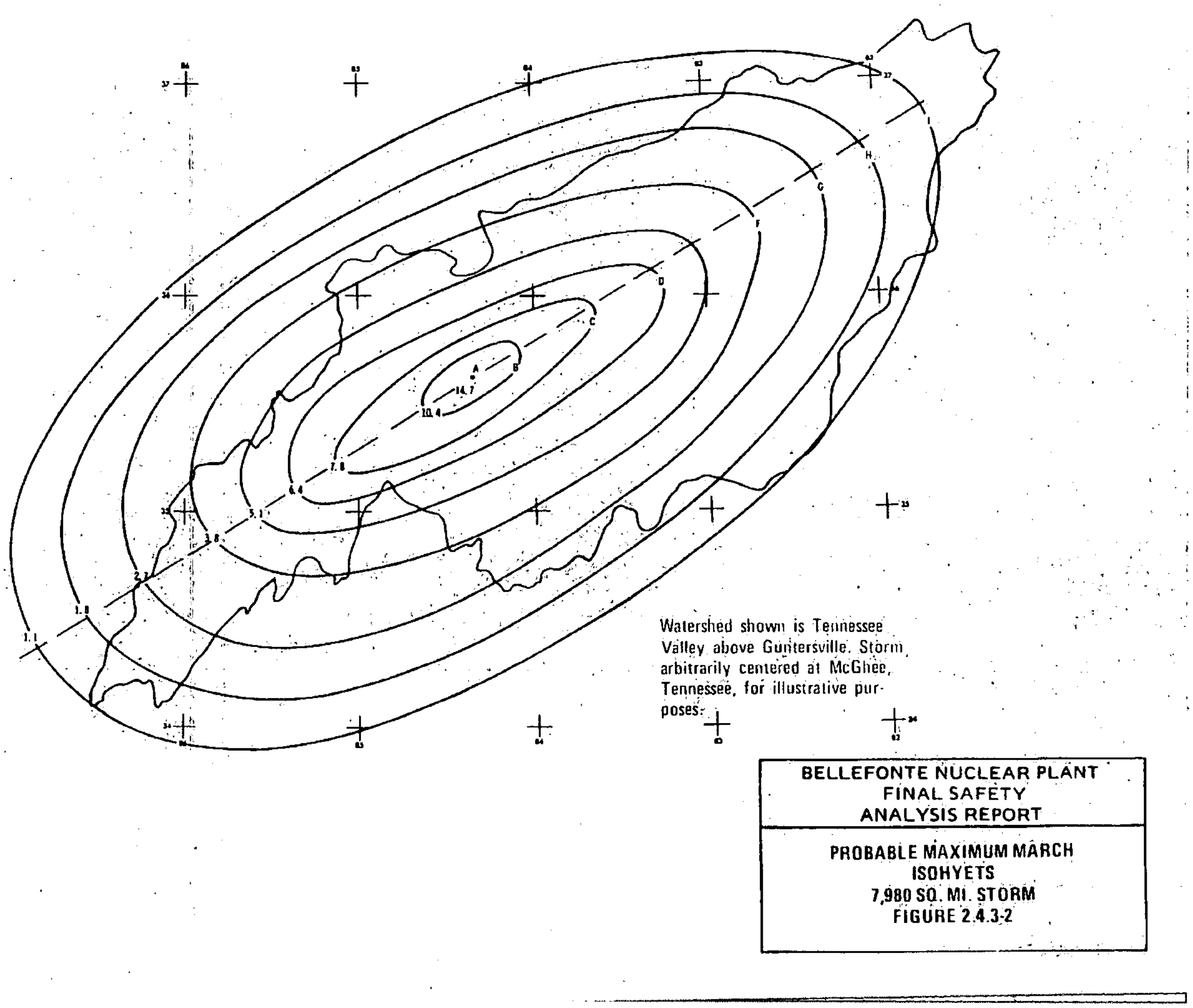


Figure 18. 7,980 Mi^2 Storm – 6-Hour Totals (Source BLN Unit 1 and Unit 2 FSAR)

A standard time distribution pattern was adopted for all storms based upon major observed storms transposable to the Tennessee Valley and distributions used by other Federal agencies. The adopted distribution is shown on Figure 19. Studies made to define warning times for the flood protection plans at Watts Bar Nuclear Plant (WBN) and Sequoyah Nuclear Plant (SNP) upstream show that alternatively placing the maximum 24-hour precipitation on the first, second, or third day results in comparable flood levels at the BLN site.

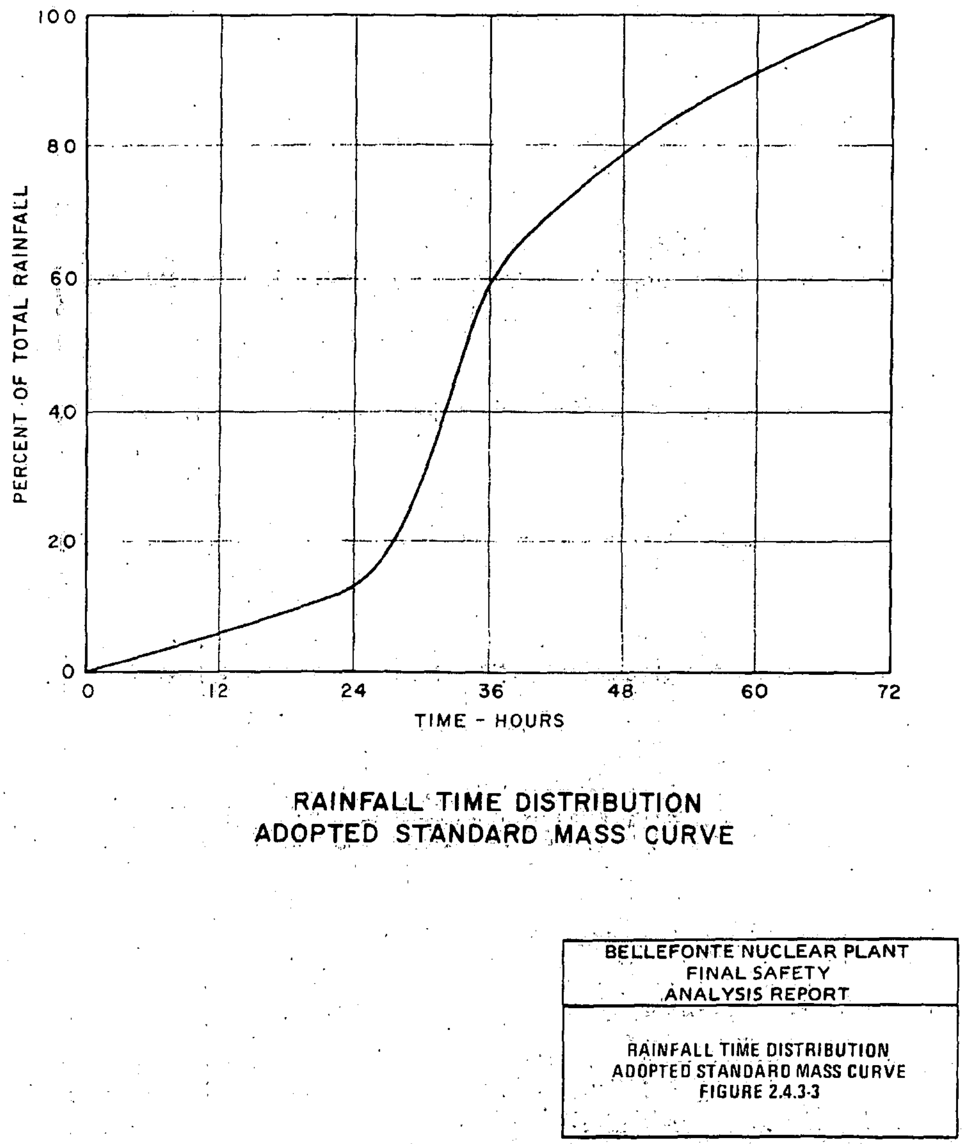


Figure 19. Rainfall Time Distribution
(Source BLN Unit 1 and Unit 2 FSAR)

4.1.2.2.2 Precipitation Losses

Precipitation losses are estimated with multivariable relationships used in the day-to-day operation of the TVA system. These relationships, developed from a study of storm and flood records, relate the amount of precipitation excess (and hence the precipitation loss) to the rainfall, the week of the year, an antecedent precipitation index (API), and geographic location (Reference 3). The relationships are such that the loss subtraction from rainfall to compute precipitation excess is greatest at the start of the storm and decreases to no subtraction when the storm rainfall totals from 7 to 16 inches. Precipitation losses become zero in the later part of extreme storms.

For this study, median moisture conditions as determined from past records were used to determine the API at the start of the storm sequence. Because the antecedent storm is so large, variations in adopted initial moisture conditions will not affect the precipitation excess computed for the main storm. The precipitation loss in the critical probable maximum storm is 2.24 inches, amounting to 35 percent of rainfall, for the 3-day antecedent storm and 1.76 inches, 11 percent of rainfall, for the 3-day main storm. These compare with observed precipitation losses of 2.9 inches in the 1973 flood. Table 2 displays the API, rain, and precipitation excess for each of the 50 subwatersheds of the hydrologic model.

Table 2. Probable Maximum Storm Rainfall and Precipitation Excess

		Antecedent Storm		Main Storm	
Unit Area No.	Area	Rain, Inches	P_e,[a] Inches	Rain, Inches	P_e,[b] Inches
1	Asheville	6.44	2.99	17.40	14.72
2	Newport, French Broad	6.44	4.04	18.50	16.51
3	Newport, Pigeon	6.44	4.04	19.30	17.31
4	Embreeville	6.44	4.04	15.10	13.11
5	Nolichucky Local	6.44	4.04	15.50	13.51
6	Douglas Local	6.44	4.86	17.10	15.88
7	Little Pigeon River	6.44	4.04	20.90	18.91
8	French Broad Local	6.44	4.19	18.60	16.81
9	South Holston	6.44	4.52	12.30	10.70
10	Watauga	6.44	4.04	13.30	11.31
11	Boone Local	6.44	4.04	14.10	12.11
12	Fort Patrick Henry	6.44	4.86	14.40	13.18
13	Gate City	6.44	4.86	12.30	11.08
14	Surgoinsville Local	6.44	4.86	14.60	13.38
15	Cherokee Local below Surgoinsville	6.44	4.86	15.80	14.58
16	Holston River Local	6.44	4.52	17.10	15.50
17	Little River	6.44	4.04	21.50	19.51
18	Fort Loudoun Local	6.44	4.04	17.60	15.61
19	Needmore	6.44	2.99	21.20	18.52
20	Nantahala	6.44	2.99	21.50	18.82
21	Bryson City	6.44	2.99	19.10	16.42
22	Fontana Local	6.44	2.99	20.70	18.02
23	Little Tennessee Local - Fontana to Chilhowee Dam	6.44	2.99	24.00	21.32
24	Little Tennessee Local - Chilhowee to Tellico Dam	6.44	4.04	21.00	19.01
25	Watts Bar Local above Clinch River	6.44	4.04	15.80	13.81
26	Norris Dam	6.44	4.86	13.80	12.58
27	Coal Creek	6.44	4.52	14.60	13.19
28	Clinch Local	6.44	4.52	14.90	13.49
29	Hinds Creek	6.44	4.52	15.30	13.89
30	Bullrun Creek	6.44	4.68	15.70	14.29

Table 2 (Continued). Probable Maximum Storm Rainfall and Precipitation Excess

Unit Area No.	Area	Antecedent Storm Rain, Inches	Antecedent Storm P_e,[a] Inches	Main Storm Rain, Inches	Main Storm P_e,[b] Inches
31	Beaver Creek	6.44	4.52	16.10	14.69
32	Clinch Local (5 areas)	6.44	4.52	15.30	13.89
33	Local above mile 16	6.44	4.52	15.30	13.89
34	Poplar Creek	6.44	4.52	14.90	13.49
35	Emory River	6.44	4.52	13.10	11.69
36	Local Area at Mouth	6.44	4.52	14.90	13.49
37	Watts Bar Local below Clinch River	6.44	4.52	14.40	12.99
38	Chatuge	6.44	2.99	21.40	18.72
39	Nottely	6.44	2.99	19.10	16.42
40	Hiwassee Local	6.44	2.99	18.90	16.22
41	Apalachia	6.44	2.99	17.90	15.22
42	Blue Ridge	6.44	2.99	22.10	19.42
43	Ocoee No. 1, Blue Ridge to Ocoee No. 1	6.44	4.04	18.30	16.31
44	Lower Hiwassee	6.44	4.19	15.20	13.41
45	Chickamauga Local	6.44	4.52	14.50	13.09
46	South Chickamauga Creek	6.44	4.35	12.30	10.89
47	Nickajack Local	6.44	4.52	11.70	10.29
48	Sequatchie	6.44	4.52	9.80	8.39
49	Guntersville N. Local	6.44	4.52	9.80	8.39
50	Guntersville S. Local	6.44	4.52	9.80	8.39
	Average above Guntersville Dam	6.44	4.20	15.56	13.80

a. Adopted API prior to antecedent storm, 1.0 inch.
b. Computed API prior to main storm, 3.65 inches.

4.2 Stream Course Model

4.2.1 Cross-Sectional Data

The cross-sectional data for mainstem and tributary rivers used to develop the SOCH model geometry input were taken from silt range data, pre-reservoir topography, actual field surveys, USGS Topographic Maps, or a combination of these sources. The sections were taken on consistent spacing for a given river reach since the model, at the time, required equal spacing of cross sections. However, the spacing can now be varied depending on available data and that required for model stability. A typical cross section is shown in Figure 20. Table 3 provides a list of sections used for each river reach in the SOCH code. There were several configurations of these cross-sectional data required to model the entire river system which will be covered in the flood routing sequence section of this paper.

Table 3. Cross-Sectional Data by River Reach/Segment

River	River Reach	Number of Sections	Delta X (feet)*
Holston (To Cherokee)	0.00 – 52.31	27	10623.36
French Broad (To Douglas)	0.00 – 32.32	33	5332.85
Tennessee (Fort Loudon)	602.3 – 652.22	25	10982.4
Little Tennessee	0.00 – 32.53	33	5301.83
Tennessee (Watts Bar)	568.23 – 602.3	17	11243.00
Tennessee (Watts Bar)	529.9 – 568.23	19	11243.00
Clinch (To Melton Hill)	0.00 – 23.1	23	5544.00
Clinch (Melton Hill to Norris)	23.1 – 79.8	55	5544.00
Tennessee (Chickamauga)	471.00 – 529.90	29	11107.00
Tennessee (Nickajack)	424.70 – 471.00	23	11112.00
Tennessee (Guntersville)	349.00 – 424.70	37	11102.67

*Distance in feet between cross sections.

A typical set up is shown on Figure 20 for the model of Fort Loudoun Reservoir up to tributary projects Cherokee and Douglas.

Once the cross sections were extracted from the source(s) and plotted they were taken to the field for verification by an experienced engineer. In the field, the engineer would (1) segment the section, (2) estimate the Manning *(n)* values for the channel and overbanks, (3) ensure the section was representative of the flow area for one-half of the reach upstream and downstream of the section, (4) evaluate effective flow areas for passage of flood water (topographic review in the office before hand), (5) review aerial photography where available, and (6) adjust the sections as necessary to account for blockage/obstructions in the overbank areas. The analysis did not include bridges/structures as these were judged to have little, if any, impact on flood flows in the magnitude of interest.

This procedure was followed for development of all cross sections where calibration against historic events was going to be performed. This included the Tennessee River reservoirs from Guntersville through Fort Loudoun to mile 652.22 (confluence of French Broad and Holston Rivers), and the Clinch River to Norris Dam. For the tributary projects like Norris, Cherokee, Douglas, Fontana, and Hiwassee the SOCH model for the reservoir and tributary reaches were set up to allow computation of the outflow hydrograph as a result of the postulated seismically induced dam failure during the ½ PMF or 25-year flood events. Thus these tributary models were not calibrated to the same level of detail as those on the Tennessee River where specific peak discharge and elevation data were going to be required.

After the field review of the cross sections was completed, the data were input to the step backwater/HEC2 program. The step backwater/HEC2 model was then calibrated using all available historic data from reservoir stage records and/or observed high water marks. Final adjustments to the cross sections and Manning *n* values were made during this process to obtain the best possible calibration to the historic flood events.

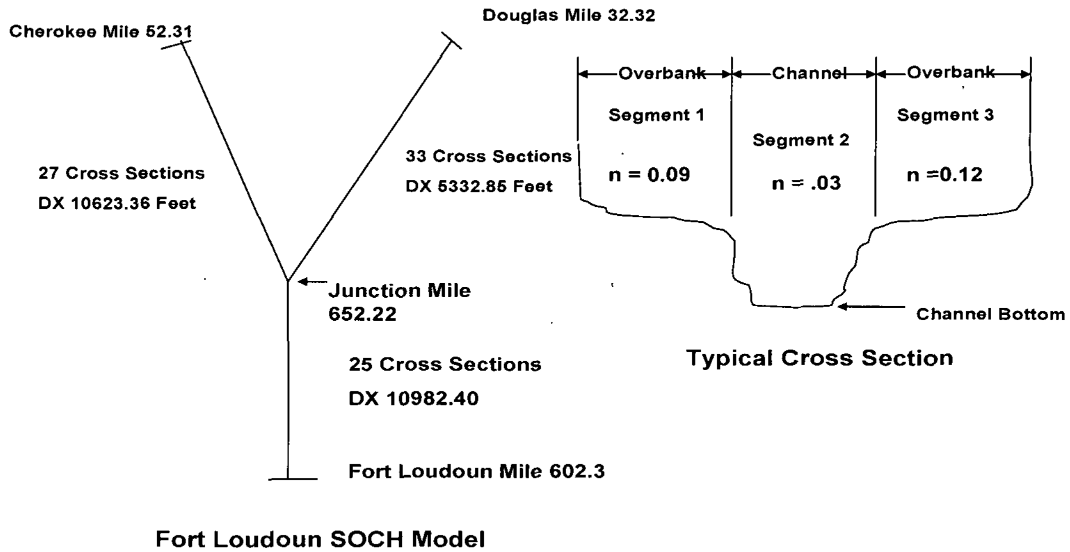


Figure 20. Typical Cross Section and Model Set-Up for Fort Loudoun Reservoir

Using the calibrated backwater/HEC2 model, a series of steady flow profiles were computed for the range of flows from 100,000 to 1,500,000 cfs. The starting levels for these steady flows were based on the headwater rating curve at each dam and the Manning *n* values were held constant. There was no reduction in Manning *n* values for high flows/increased depth. This process resulted in a set of cross sections for the stream reach, adjusted Manning *n* values (channel and overbank), steady flow profiles, and a tailwater rating curve at each project. The cross sections and Manning *n* values became input to the Conveyance Program, which determined parameters used in the SOCH code geometry table. The steady flow profiles and tailwater rating curve were then used to calibrate the SOCH model. The SOCH Code Verification section of this paper provides a discussion of the verification process.

4.2.2 CONVEYANCE Code

The CONVEYANCE code was developed to determine the cross-sectional area and the composite hydraulic radius ($R^{2/3}$) by elevation for a given segmented cross section. A typical cross section with three segments is shown on Figure 21.

The data points to describe the cross section were taken directly from a plot of the section or from the step backwater/HEC2 computation input. The number of points used to describe each section varies and is based on the minimum needed to accurately define the flow area. The number of segments in the cross sections will vary and depend on changes in the Manning *n* values across the section and over one-half reach in either direction as determined by field inspection. The Conveyance Program computes the cross-sectional area and composite $R^{2/3}$ for user-specified elevations (generally from 5- to 20-foot intervals) starting at the channel bottom. The Conveyance is computed for each segment of the cross section by the equation shown on Figure 21. The total cross-sectional area, A, and total conveyance, Ct, is the sum of the segments. A composite $R^{2/3}$ is determined at each elevation step by the following equation with Manning's *n* referenced to the channel *n:*

$$R^{2/3} = Ct\,(n)/1.49(A)$$

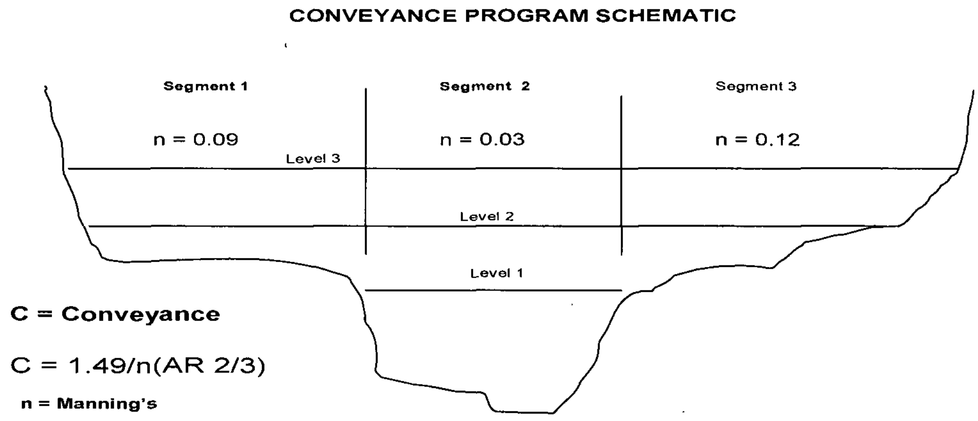


Figure 21. Typical Conveyance Cross Section

The output of the Conveyance Program will be cross-sectional area, A, and composite, $R^{2/3}$ for user-specified elevations. These outputs make up three of the four elements of the geometric table (elevation, cross-sectional area, composite $R^{2/3}$, and B) that the SOCH program uses to define each cross section. The geometric table for each cross section is made up of 21 vertical steps, starting at intervals at or below the channel bottom, which include: elevation, area, $R^{2/3}$, and B. The SOCH program interpolates values for these parameters from the geometry table based on elevation. A spreadsheet verification of this code for a typical cross section is shown in Appendix B.

4.2.3 <u>Weighted Width (B) Code</u>

The fourth component for the SOCH geometry table is B. This is a parameter used to account for reservoir storage. The surface area between cross sections for a series of elevations was determined from topographic maps as shown on Figure 22. Storage in any off-channel areas or tributaries was accounted for by these surface area determinations. The elevations were selected to define the overbank and tributary areas over the range of expected flood depths. Using the surface area by elevation data as input to the Weighted Width program, an equivalent B was determined for each cross section such that the total volume for a reservoir could be determined. Once the B was determined for each cross section, the total volume of the reservoir was computed by the program and a comparison made against the total reservoir volume curve published for the project. If there was a difference, a correction factor was applied to bring the computed volume into agreement with the measured total volume. Once this step was completed, the B in the SOCH geometry table is set for the model.

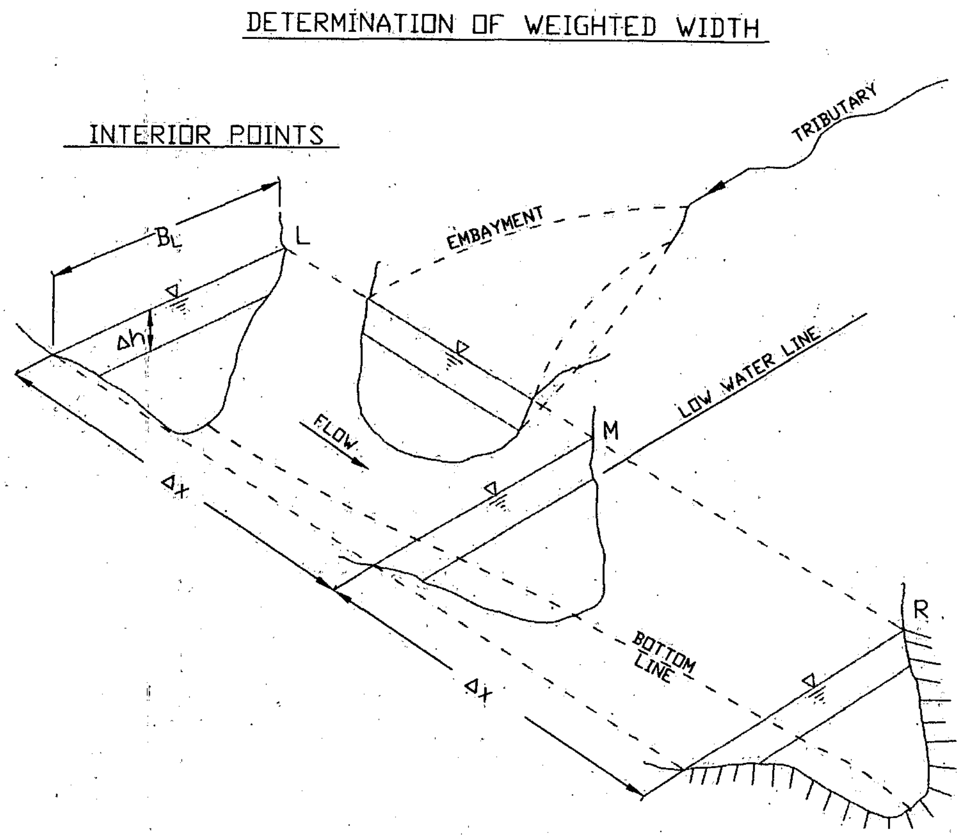


Figure 22. Weighted Width Schematic

A complete narrative for the weighted width code is found in Appendix C.

4.2.4 <u>SOCH Code</u>

The mathematical model for unsteady flow in open channels is assumed to be one-dimensional in the sense that the flow characteristics such as depth and velocity are considered to vary only in the longitudinal (x) direction and with time. The channel geometry is three-dimensional.

The following items are consequences of the one-dimensional assumption.

1. The velocity is uniform across the cross section, so that the water particles in a moving section remain in that section.
2. The transverse water surface is a horizontal line in any cross section.
3. The axis of the river can be considered to be a straight line.

In the development of the mathematical model, the following assumptions are also made.

1. The flow is gradually varied so that the vertical acceleration of the water particles may be neglected, and that the pressure distribution in any cross section is hydrostatic.
2. The bottom slope of the channel is small.
3. The resistance coefficient, as determined for uniform turbulent flow at any given channel cross section, is the same for the given water surface elevation and mean velocity regardless of whether the flow is uniform or nonuniform, steady or unsteady.
4. The mass density ρ is a constant, i.e., no stratification exists.

The two equations of unsteady flow, the continuity equation and the equation of motion, are:

$$\frac{\partial(AV)}{\partial x} + B\frac{\partial h}{\partial t} - q = 0 \tag{1}$$

$$g\frac{\partial h}{\partial x} + V\frac{\partial V}{\partial x} + \frac{\partial v}{\partial t} + gS_f + \frac{q}{A}V = 0 \tag{2}$$

in which A = flow area; V = mean velocity; x = distance; B = surface width; h = water surface elevation; t = time; q = lateral local inflow per unit distance and time; g = the gravitational constant; and S_f = the energy gradient given by:

$$S_f = \frac{n^2 V|V|}{2.21\ R^{4/3}} \tag{3}$$

in which n = Manning's and R = the hydraulic radius. The term ∂A/∂x in the expanded form of Equation (1) can be expressed as a function of ∂h/∂x. Therefore, these equations make up a system of two nonlinear, first order, first degree partial differential equations with two independent variables x and t, and with two unknowns h and V. No analytical solutions to this system of equations exist. However, they may be solved numerically by writing them in finite difference form.

In finite difference methods, the differential equation is replaced by an approximating difference equation, and the continuous region in which the solution is desired is replaced by a set of discrete points called a net as shown on Figure 23. At the time, a variety of net schemes for approximating the differential equations of unsteady flow had been studied by various investigators. A characteristic computation net has several apparent advantages over other schemes, particularly in the stability and convergence of the solution and in optimization of net size. However, this computation scheme has the disadvantage that the net points in the x-t plane are determined as the computation proceeds. It is therefore necessary to compute x and t in addition to h and V, and to use an interpolation procedure if it is desired to obtain results at regular or specific distance and time intervals. The main disadvantage of fixed-net schemes is that the net points in the x-t plane can be selected prior to computation, so that only h and V need to be computed. The major disadvantage of most explicit fixed-net computation schemes is the difficulty in finding a net size that will give a stable and convergent solution. Based on basic studies of many different explicit fixed-net schemes, a centered difference scheme proposed by Stoker (Reference 4) was found sufficiently stable and convergent for the unsteady flow computations if the relation

$$\left(V+\sqrt{g\frac{A}{B}}\right)\frac{\Delta t}{\Delta x}\leq 1-\frac{gn^2|V|}{2.21\,R^{4/3}} \tag{4}$$

is satisfied, in which Δt = the time interval and Δx = the distance interval Equation (4).

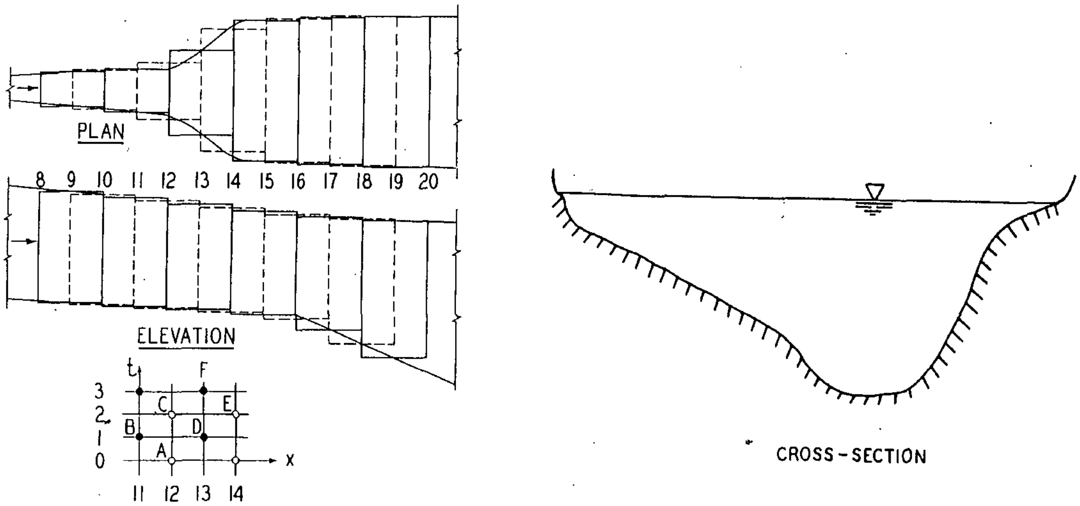


Figure 23. Net Scheme and Channel Geometry

In the solution of the partial differential equations of unsteady flow, it is necessary to specify boundary and initial conditions. Boundary conditions are conditions specified at fixed values of x at various times. Initial conditions are conditions specified at fixed values of time at various spatial locations

The boundary conditions may be given as discharge or water surface elevation versus time, or as a stage-discharge relationship. A steady-flow profile, a flat pool-zero flow profile or a transient flow profile from previous computations may be used as the initial conditions. The initial conditions provide an elevation and discharge for each cross section in the reach.

In addition to boundary and initial conditions, input data on local inflows, channel geometry, and Manning's *n* must be prescribed. From these input data the SOCH code determines flows, mean velocities, and water surface elevations at any number of desired locations and times for the channel reach under study.

A typical reservoir or river link geometric representation is shown schematically on Figure 23. As this figure illustrates, the mathematical model approximates the actual channel geometry by a series of adjacent prisms. Channel geometry data are determined by interpolation from a 21-step geometry table for each cross section that is in the form of elevation, area, $R^{2/3}$, and B. Off-channel storage in a reach, i.e., embayments or tributaries, is accounted for in the mathematical model by adjusting the B width in the equation of continuity to give the correct volume in the reach. B is generally different from the cross section top width upon which the cross-sectional area and hydraulic radius are based.

Inflows to the model generated by the FLDHYDRO code can be entered as point source or distributed locals over a reach length Δx depending on how they were developed. A more detailed description of the inflows to the SOCH model is covered in the Flood Hydrograph section.

Typically, TVA has used a computational time interval Δt ranging from 10 seconds to 2.5 minutes, with a longitudinal spacing of net points Δx ranging from 0.1 to 2.5 miles. The initial model set-up procedure included testing the cross section spacing and time step to obtain a stable/convergent model result. Once that point was established, further testing with additional cross sections and/or reduced time step did not result in any change in the computed values.

The SOCH code has evolved since it was first developed in the late 1960s. There is very little documentation on code changes that were made between its initial application and the SOCH90 code (PC Version). However, most of the code changes related to how input and/or output to the code would be handled to streamline its use. The basic computation scheme was not modified as a result of these changes. For any code change made there were test runs to ensure duplication of previous results.

A comparison of the headwater, tailwater and discharge from Fort Loudoun Dam as a result of the postulated seismic failure of Cherokee and Douglas is shown on Figure 24. This clearly shows for this case there is no difference in the computed results between the SOCH88 code and the SOCH90 PC version. Further comparative analyses between the two versions of the codes will be completed during the update and verification process.

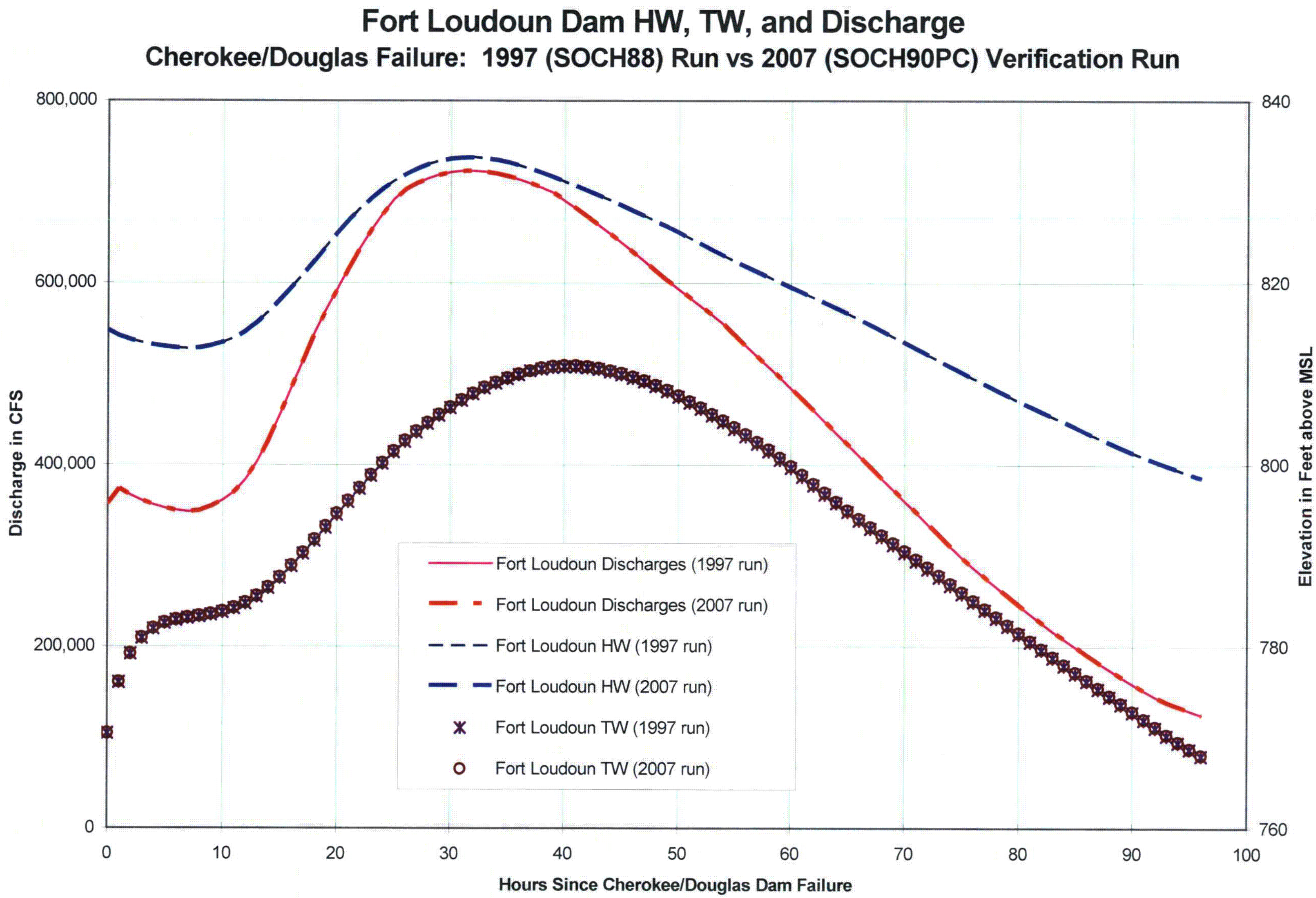


Figure 24. Comparison of SOCH88 and SOCH90 PC Version

4.2.5 SOCH Code Verification (Steady-State HEC2 Profiles/Historic Floods)

The calibration of the SOCH model for the mainstem reservoir was an iterative process completed in two steps: (1) comparison of SOCH vs. steady-state elevation profiles along the length of the reservoir for flows ranging from 100,000 to 1,500,000 cfs and the tailwater rating curve at the upstream dam and (2) duplication of historic floods. The inputs to the SOCH model at this point include: (1) conveyance and B code outputs which make-up the four parameters (elevation, cross sectional area, $R^{2/3}$, B in the 21-step geometry table for each cross section, and (2) standard step/HEC2 backwater profiles for flow ranges from 100,000 to 1,500,000 cfs.

The initial set of Manning *n* values for use in the SOCH code are the channel *n*'s taken from the calibrated HEC2 computations. These Manning *n* values were then adjusted in the SOCH model as appropriate to produce the best agreement with the elevation profiles for all flow ranges. The final SOCH-adjusted Manning *n* values from this process were then used in step two to calibrate the largest historic floods in that reservoir (typically two historic floods in each reservoir). On Figure 25 a comparison of the steady flow step backwater profiles and the profiles computed by the SOCH model are shown.

To verify the SOCH model against historic events such as the 1967 and 1973 flood events, several pieces of actual observed data had to be collected. For the flood period of study, typically about two weeks, data included (1) observed discharge and tailwater elevation data at the upstream project, (2) observed elevations for all gage recorders along the length of the reservoir, and (3) observed discharge and headwater elevation data at the downstream project. The inflows along the reservoir were determined from the calibrated runoff model (FLDHYDRO code) and provided as local inflows to the SOCH model.

A simulation run for the flood event (1967, 1973, etc.) using the SOCH model was then made using the observed discharge at the upstream boundary, local inflows from the runoff model, and observed discharge at the downstream boundary. A comparison of the observed data (elevation and discharge) and that computed by the SOCH code for five stage locations is shown on Figure 26 for Guntersville Reservoir for the March 1973 flood. The verification of the 1967 flood in Watts Bar Reservoir is shown on Figure 27.

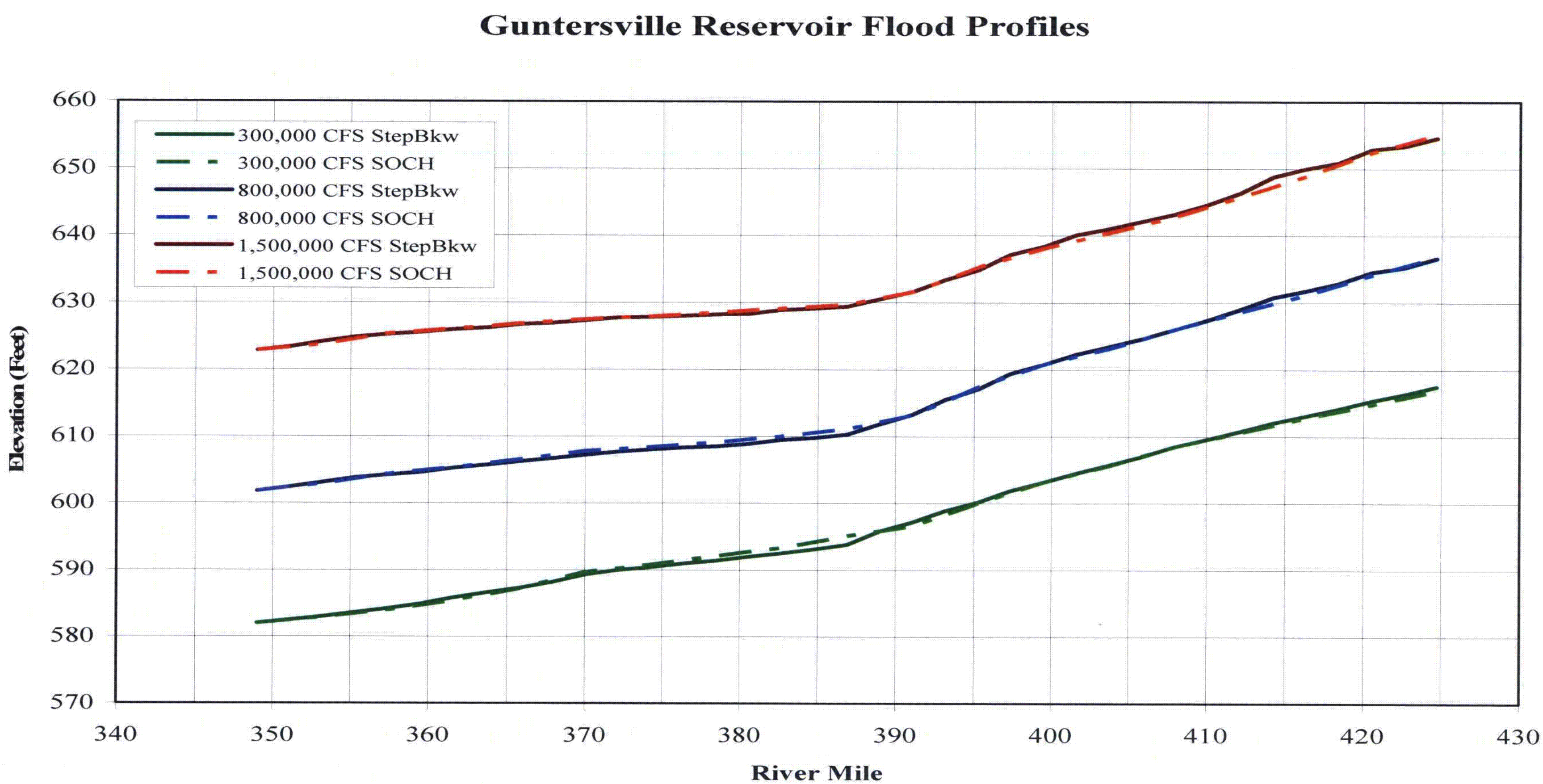


Figure 25. Comparison Profile Plot

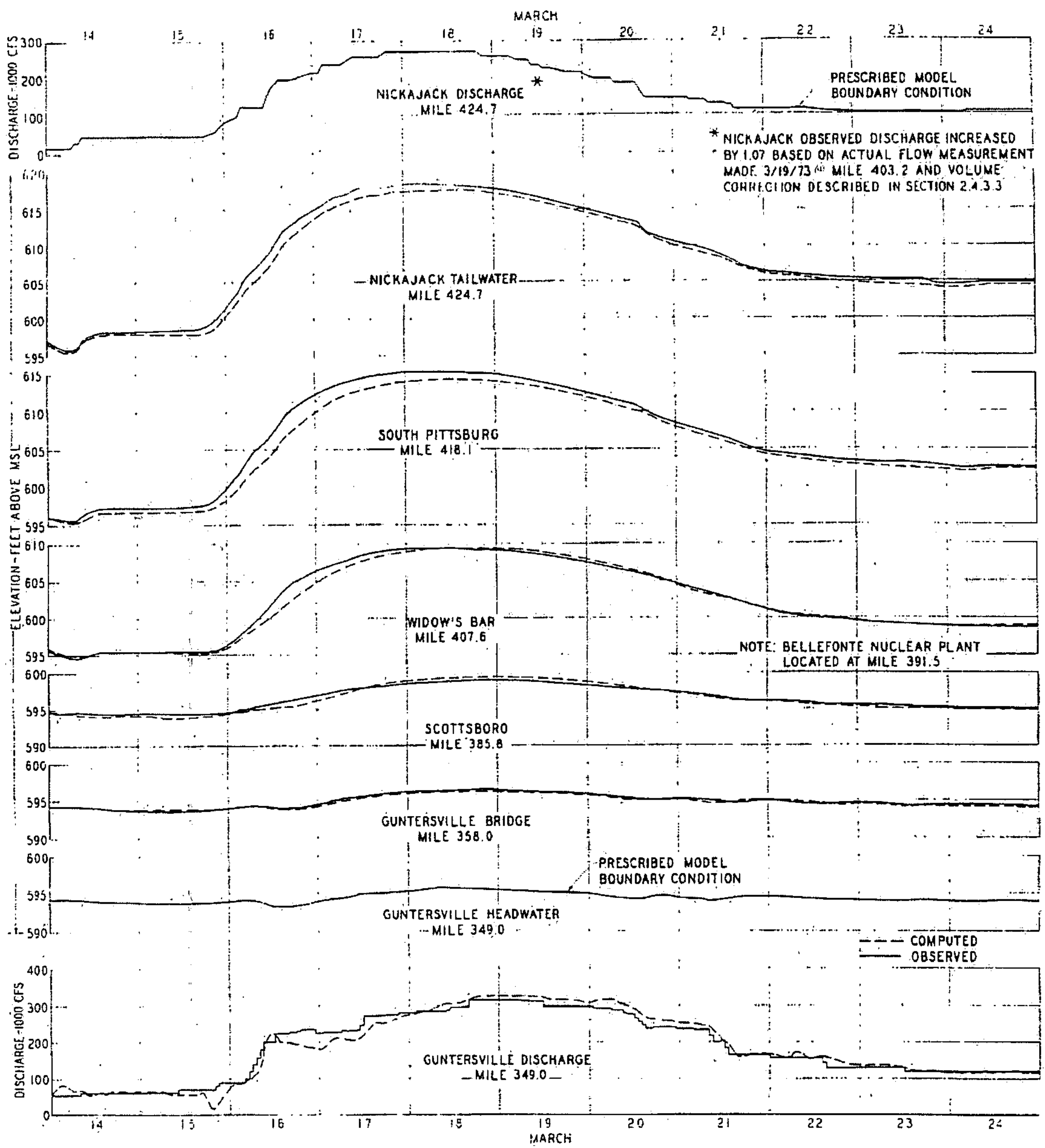


Figure 26. March 1973 Flood Calibration - Guntersville Reservoir

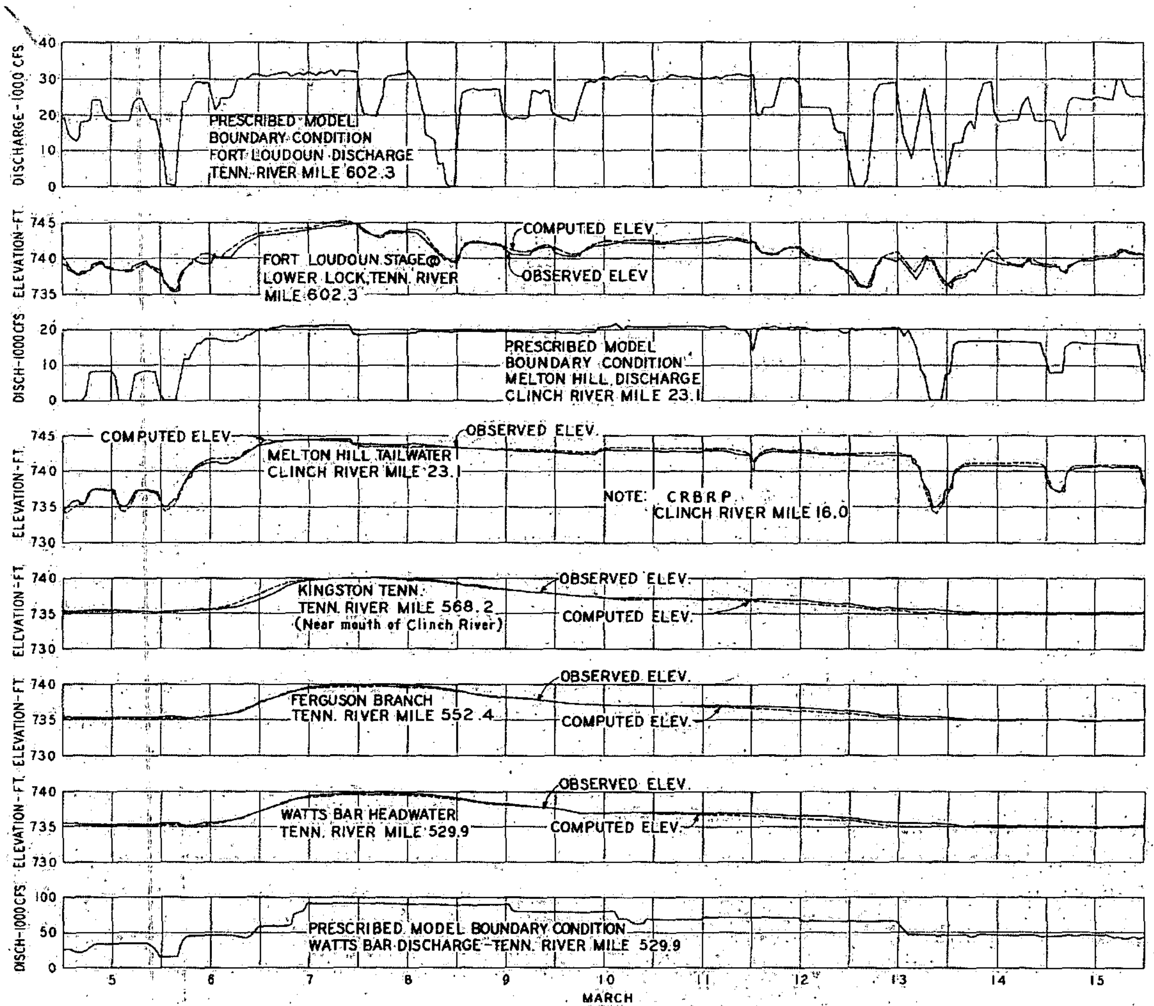


Figure 27. March 1967 Flood Calibration - Watts Bar Reservoir

If the computed elevations for the SOCH code did not show good agreement with the observed historic data all inputs to SOCH were checked for accuracy. If no input errors were found then the Manning *n* values in the SOCH model would be adjusted to obtain the best match possible to the historic data. At this point the steady-state step backwater profiles would be rerun using the SOCH model with the Manning *n* values as modified by the historic event calibration. Depending on the magnitude of any changes at a given location, a final set of Manning *n* values would be determined for that reservoir that was based on an iterative process between the best fit of the historic flood events and the steady flow step backwater profile calibration.

Once the geometric tables are fixed, the Manning *n* values calibrated and the runoff model verification complete, the runoff and stream course model was then considered to be verified and ready for use in simulating any combination of design storms, seismic failure events, or normal operation studies.

The SOCH model has also been applied to several unsteady flow problems with very good agreement with observed data. Figure 28 shows application of the SOCH model in Nickajack Reservoir to a special turbine operation where the velocity at Moccasin Bend (TRM 458.4) was computed by the model and measured in the field for verification. During this operation, only observed data at the boundaries was used with minimum local inflows along the reservoir. Figure 29 shows application of the SOCH model below Barkley Dam on the Cumberland River to define the impact on elevations at selected locations that would result from turbine operations.

To calibrate the runoff and stream course model a more recent event (the 2003 flood) will be used to compare predicted values against observed data in Watts Bar, Chickamauga, Nickajack and Guntersville reservoirs.

4.2.6 SOCH Code Computer Run Sequence – 21,400 Mi^2 PMF

To better understand the SOCH code computer runs required to compute the elevation and discharge at BLN the following Figures 30 to 39 defines the run sequence by reservoir set-up. At this point the inputs to the SOCH code include the following data and information:

- Geometric table for each cross section and stream reach (input from Conveyance and Weighted Width codes)
- Final calibrated Manning's *n* values
- Inflow hydrographs at the boundaries and local inflows generated by the FLDHYDRO code (inputs from UNITGRPH, TRBROUTE, CHANROUT)
- Fixed rule curves and headwater rating curves at the downstream boundary
- All data input to the SOCH code with initial conditions defined for the river reach (elevation and discharge specified for each cross section)
- If the earth embankments are over topped and judged to fail, the time of failure will be determined by DBREACH code
- Outputs from SOCH code include elevation and discharge data at each dam and at any major point of interest i.e., nuclear plant sites

The computer runs will follow the fixed rule operation during the antecedent storm and the 3-day dry period, and then during the main storm follow the appropriate before-failure or after failure rating curves at each dam.

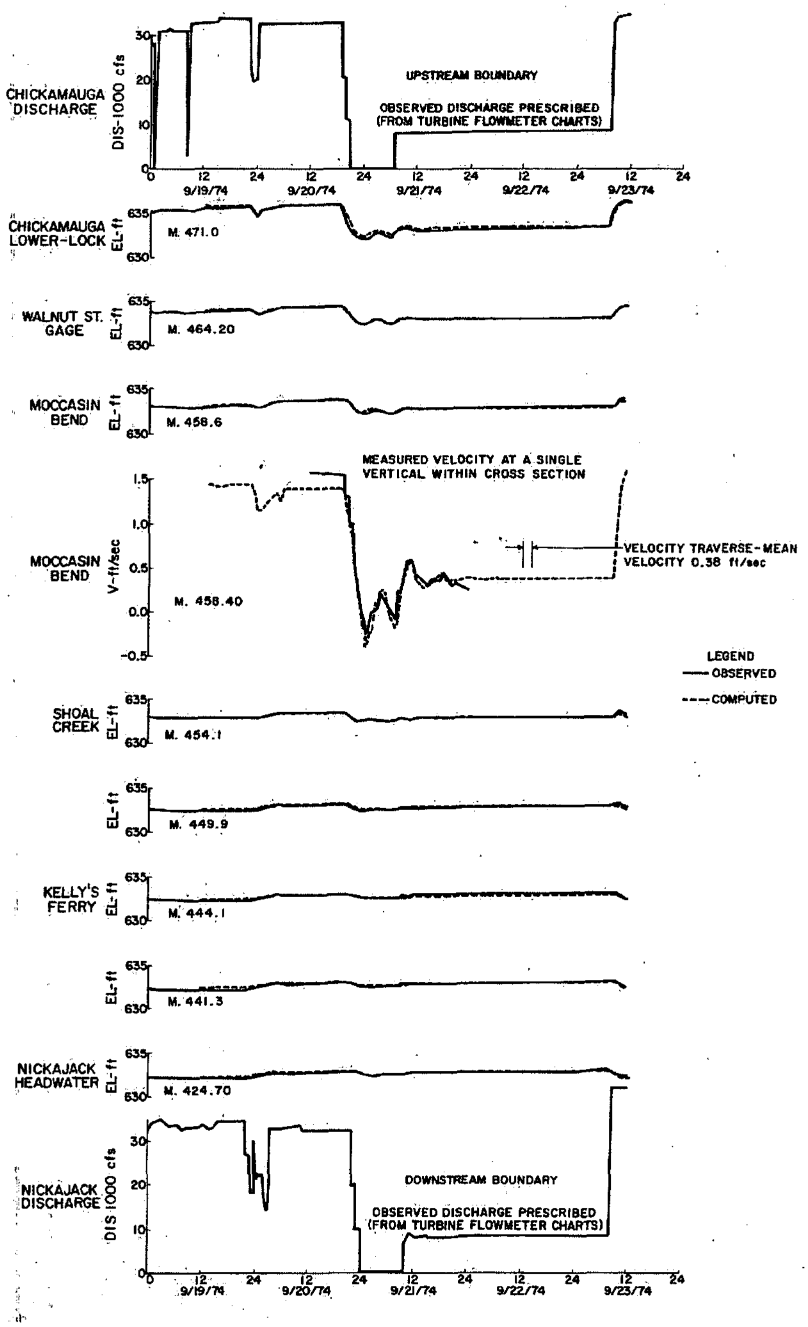


Figure 28. Nickajack Reservoir Velocity Survey

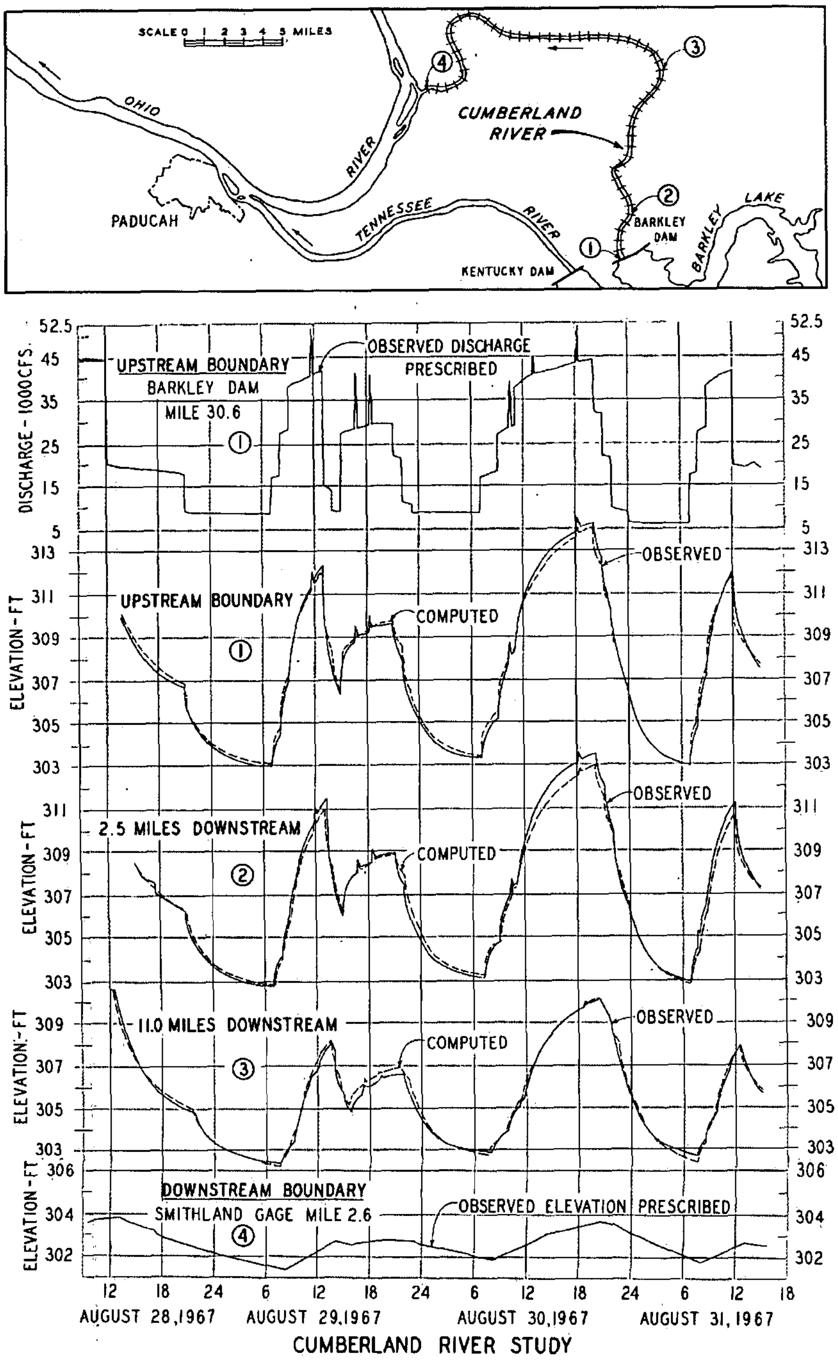


Figure 29. Cumberland River Turbine Operation Study

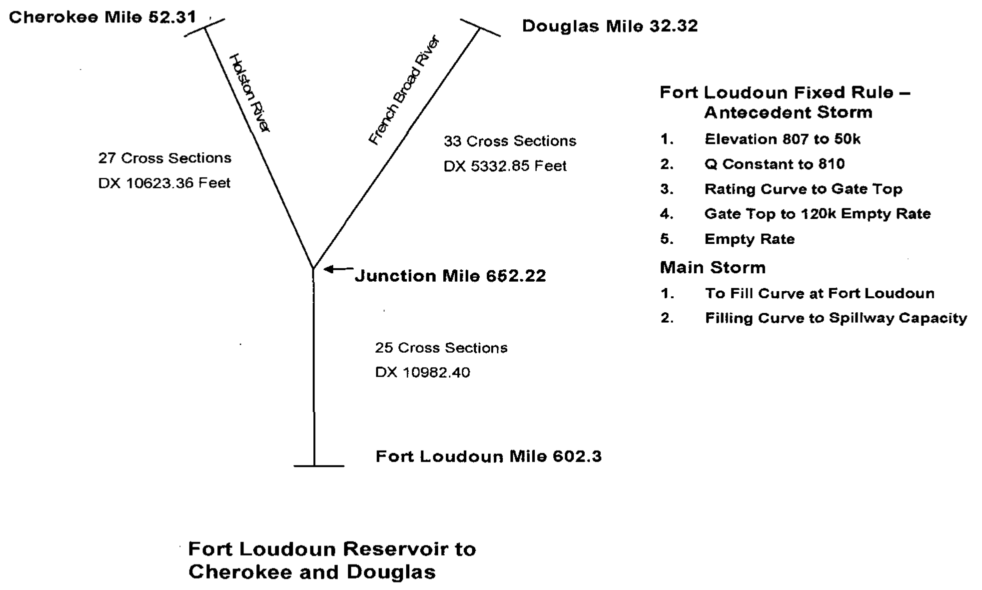


Figure 30. Run Sequence, Fort Loudon

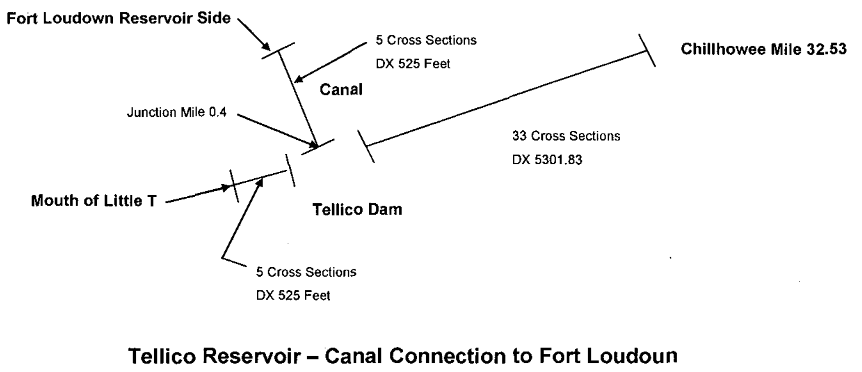


Figure 31. Run Sequence, Tellico Reservoir/Fort Loudoun Canal

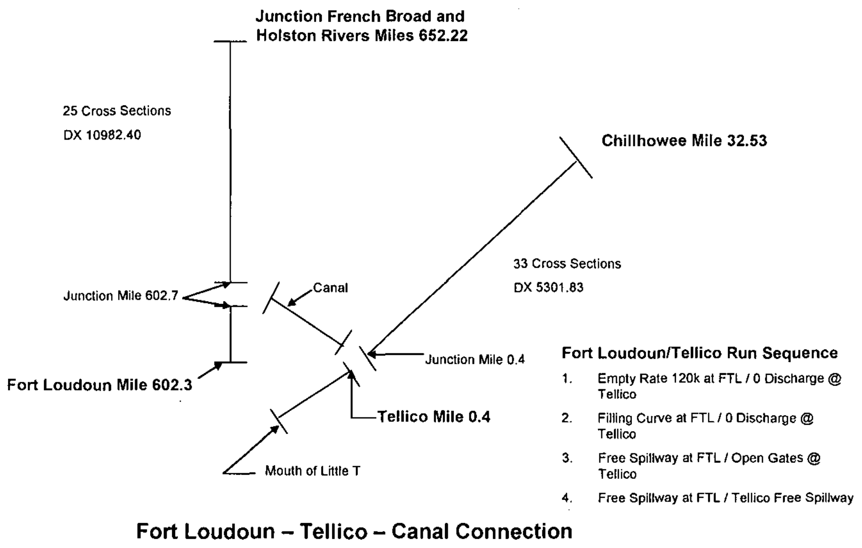


Figure 32. Run Sequence, Fort Loudoun/Tellico Reservoirs

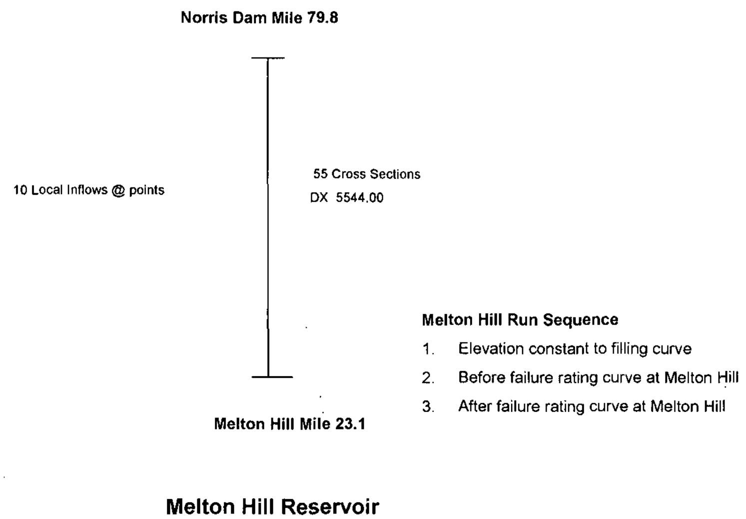


Figure 33. Run Sequence, Melton Hill Reservoir

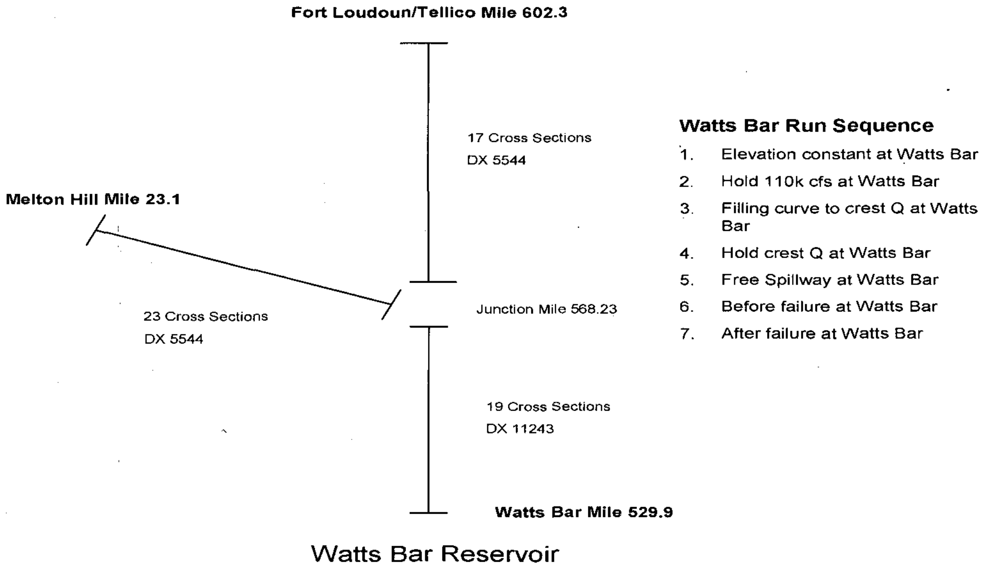


Figure 34. Run Sequence, Watts Bar Reservoir/Clinch River Arm

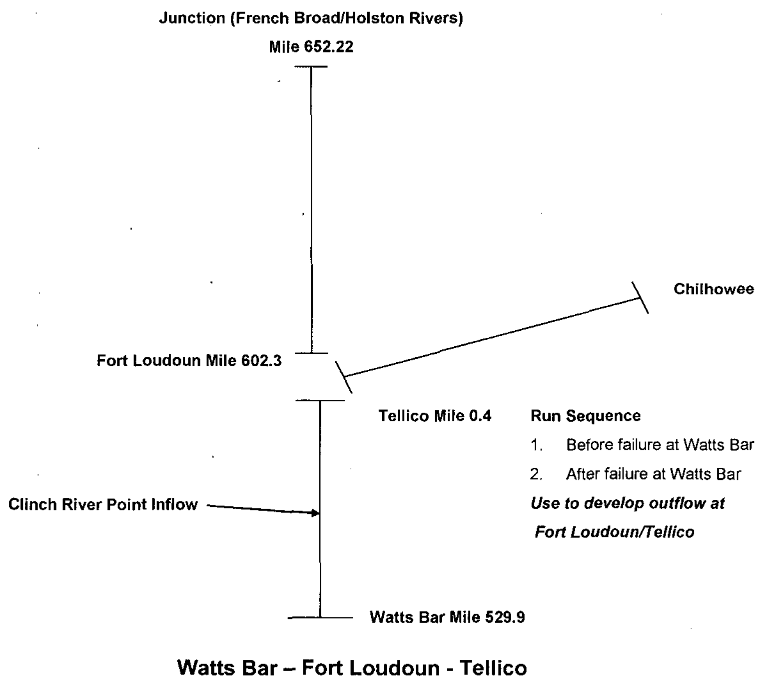


Figure 35. Run Sequence, Watts Bar/Fort Loudoun Reservoirs (Clinch River as Point Inflow to Obtain Combined Inflow at Fort Loudoun/Tellico)

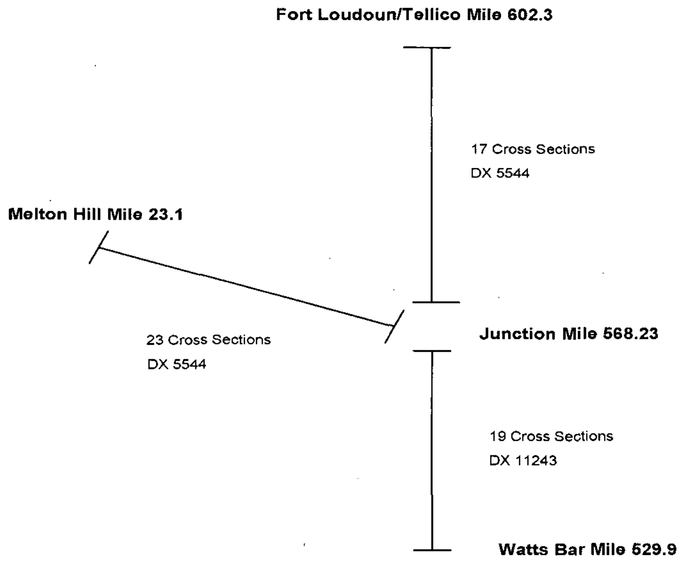


Figure 36. Run Sequence, Watts Bar Reservoir/Cinch River Arm (Use Fort Loudoun/Tellico Combined Inflow with Clinch River)

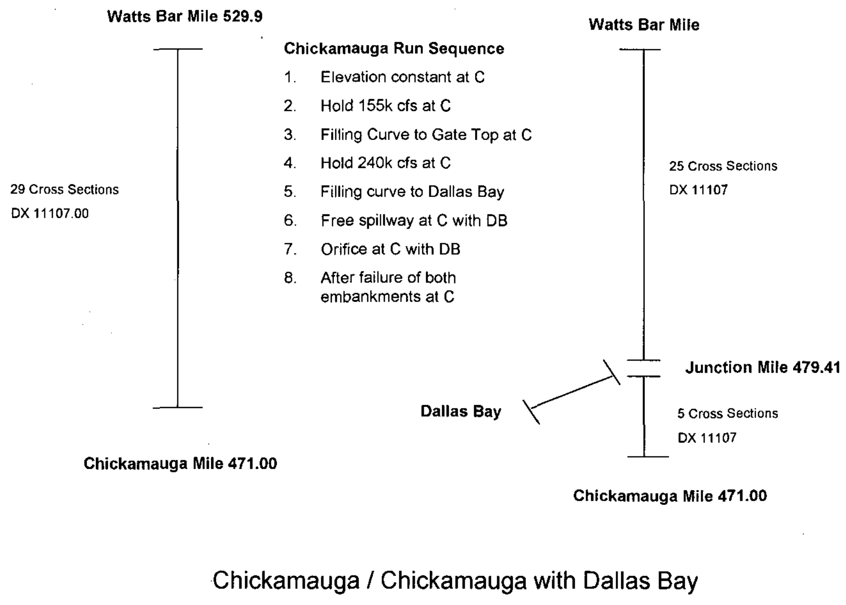


Figure 37. Run Sequence, Chickamauga Reservoir

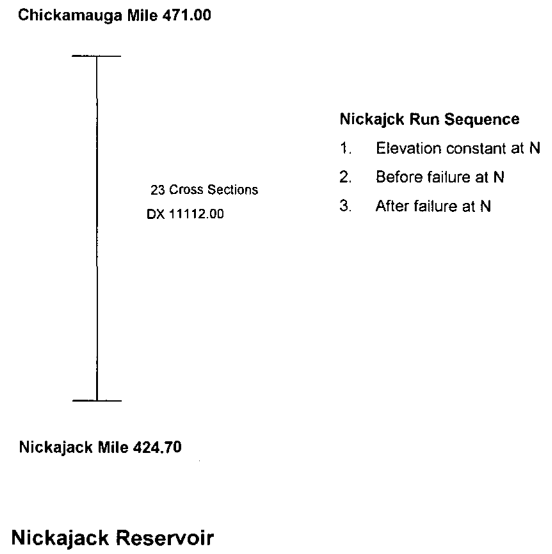


Figure 38. Run Sequence, Nickajack Reservoir

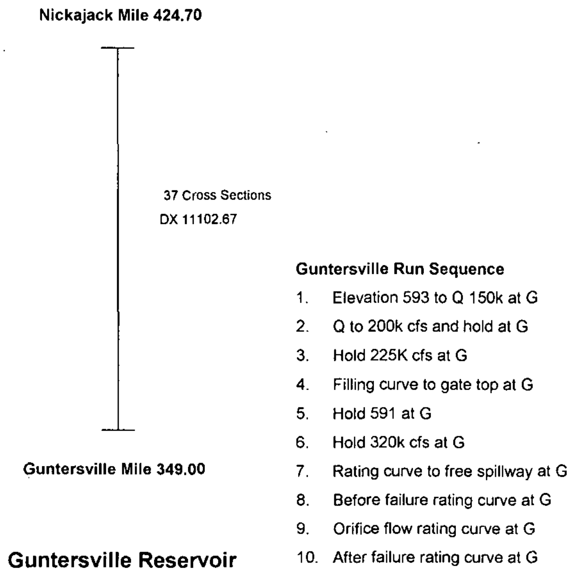


Figure 39. Run Sequence, Guntersville Reservoir

5. Reservoir Routing

5.1 Reservoir Operating Rule (Improved Fixed Rules)

Reservoirs were operated according to guides for flood operation, known as improved fixed rules, in routing the candidate storms to determine the controlling events. Figure 40 shows the operation guide for Chickamauga on the mainstem and Figure 41 shows the operation guide for Douglas Dam, a tributary dam. There is a fixed rule guide for each project that is followed for the flood routing sequence.

5.2 Project Rating Curves

The spillway discharge ratings for TVA's dams, which are based on test results from scale models, are published. These ratings cover the complete range of expected operating conditions. However, for use in determining the PMF, the discharge ratings had to be extended to account for flow over the top of the dam. The discharge for these higher elevations was computed using standard hydraulic equations (weir, orifice, etc.).

The reservoir routings through Chickamauga Dam were made before the recent modifications to construct a new lock. The new lock will eliminate at least four spillway bays. The impact to the spillway discharge rating with four bays removed is shown on Figure 42. The single line rating curve (existing project before any modifications) used in the original routings is also shown. Site flood analyses will be re-performed once the new lock design details are finalized.

5.3 Reservoir Operation

Median initial reservoir elevations were used at the start of the storm sequence used to define the PMF to be consistent with statistical experience and to avoid unreasonable combinations of extreme events. As a result, 53 percent of the total reserved system flood detention capacity was occupied at the start of the main flood based on SOCH code output. Studies made by TVA for the Watts Bar and Sequoyah Nuclear Plants have shown that the initial reservoir levels would not have a significant effect on maximum flood discharges and elevations at the plant site because spillway capacities, and hence uncontrolled conditions, are reached early in the flood.

Normal reservoir operating procedures were used in the antecedent storm. This included use of turbine and sluice discharge in the tributary reservoirs. Turbine discharges are not used in the main river reservoirs after large flood flows develop because head differentials are too small. Flood operating procedures were used in the main storm. Turbine discharge was not used in either the tributary or main river dams. All spillway gates were determined to be operable without failures during the flood. Gate crews would be called to respective dams during or before the first hours of the main storm when access would not be a problem. Normal practice of having gate crews remain at the dams during major floods would be followed. Gates on main river dams would be fully raised, thus requiring no additional operations, by the last day of the main storm which is before the structures and access roads would be inundated.

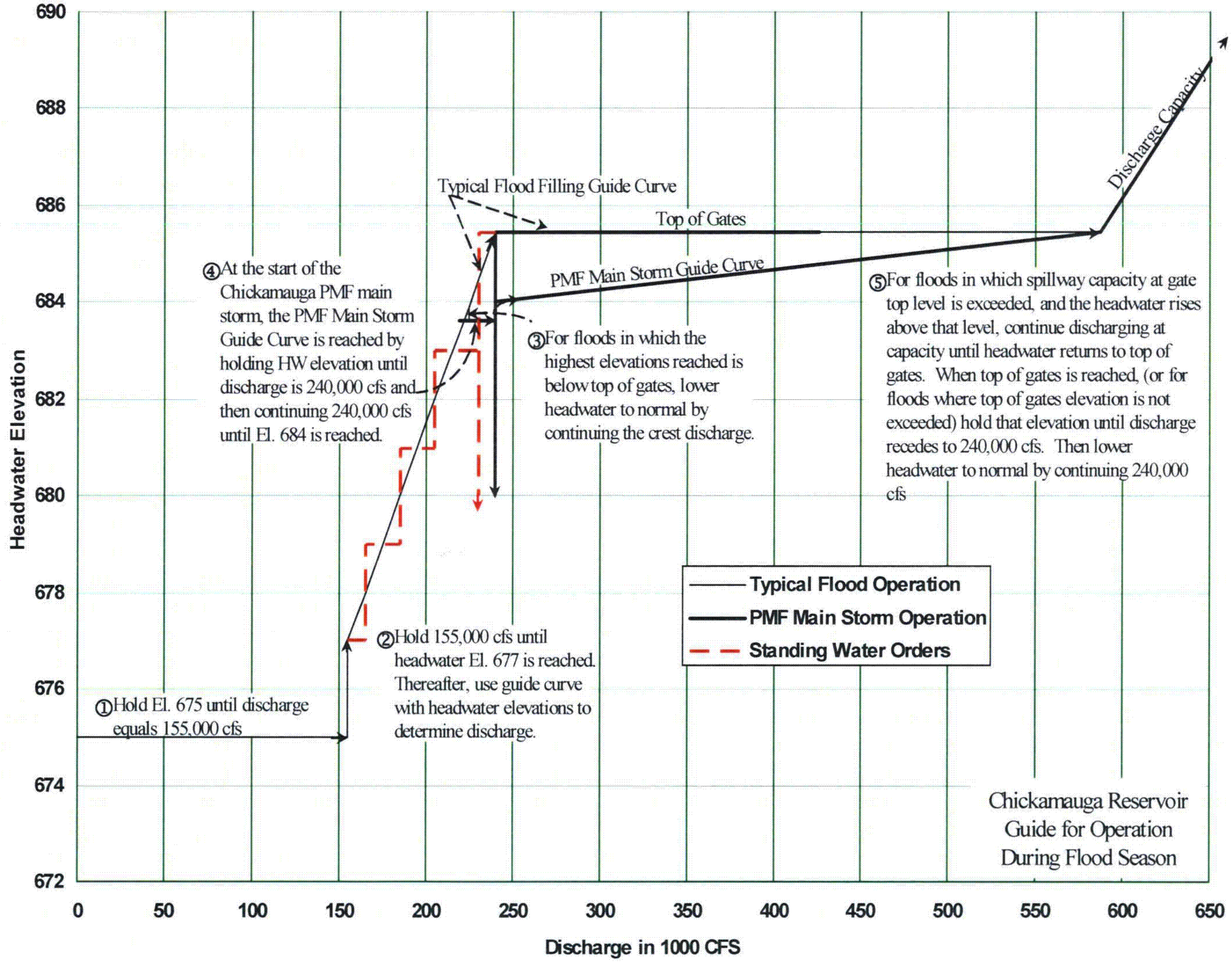


Figure 40. Chickamauga Reservoir Guide for Operation

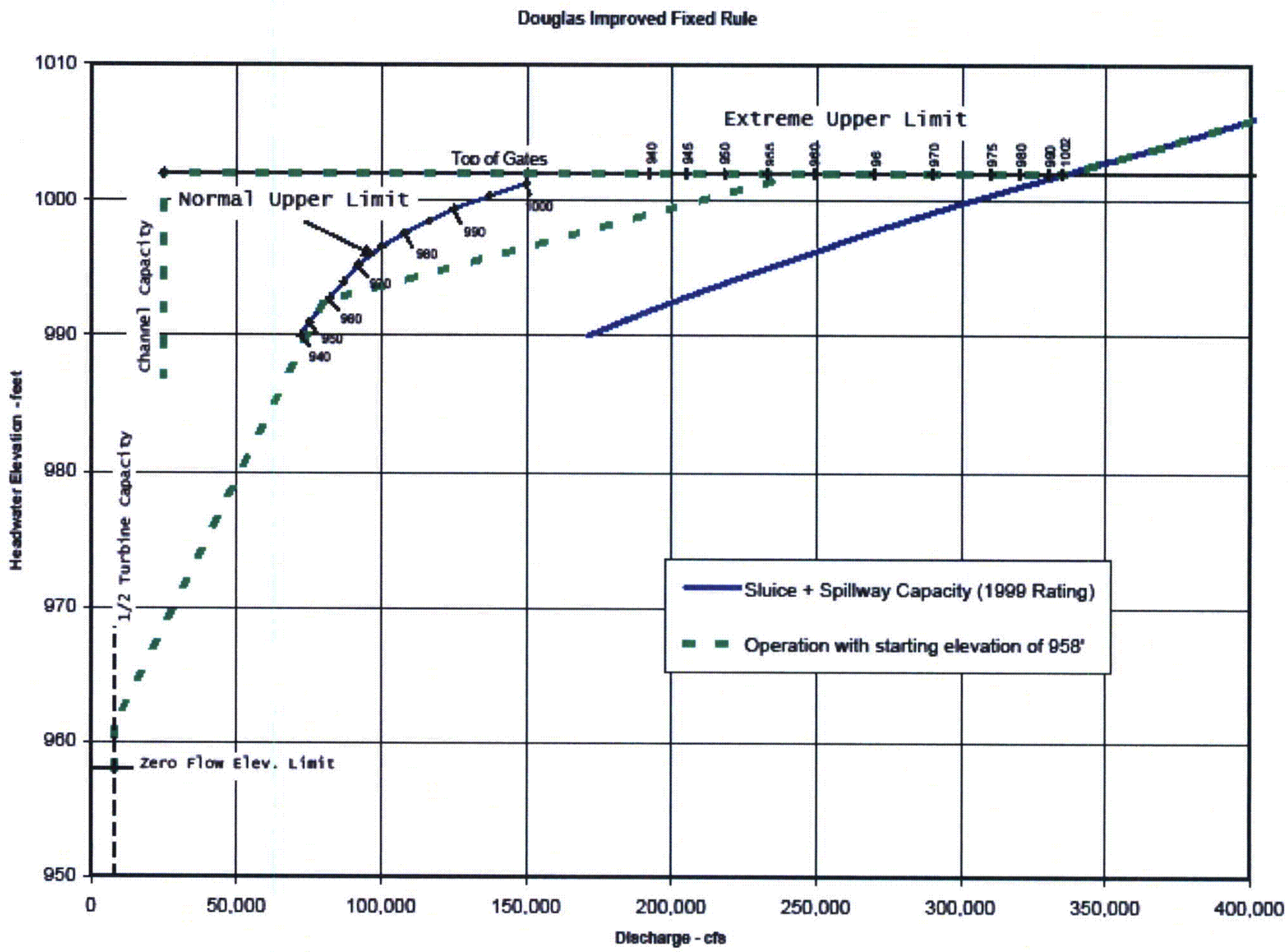


Figure 41. Douglas Reservoir Guide for Operation

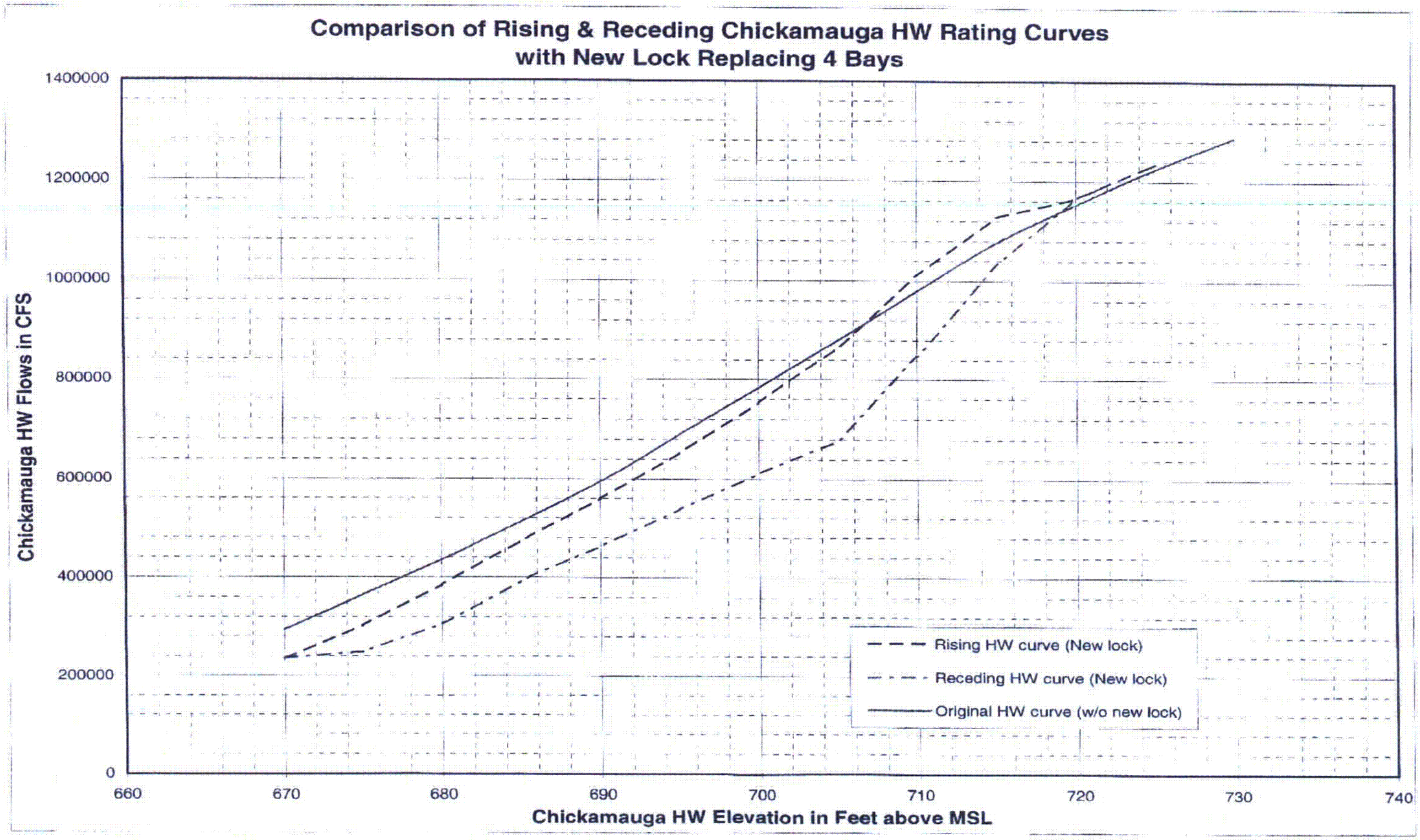


Figure 42. Chickamauga Headwater Rating Curves

5.3.1 Embankment Breaching (DBREACH Code)

In the 1998 reanalysis which addressed dam safety modifications, only the west saddle dike at Watts Bar Dam and the north embankment at Nickajack Dam would be overtopped and breached. At Nickajack Dam, the north embankment would fail down to the roller-compacted concrete overflow dam with top at elevation 634. These are the only failures. Chickamauga Dam 79.5 miles upstream from the plant, would be overtopped but was assumed not to fail. This assumption was based upon the pending dam safety modification planned for Chickamauga Dam to prevent failure from overtopping. Failure of Chickamauga Dam would increase flood levels at Bellefonte, but the increase would be small (dam safety studies showed that with both Watts Bar and Chickamauga failures, the flood level at BLN would be increased only 0.4 foot). As part of the update and verification process Chickamauga discharges with and without dam failure (those that would overtop the dam) will be computed past BLN site.

The adopted relationship to compute the rate of erosion in an earth dam failure is that developed and used by the Bureau of Reclamation in connection with its safety of dams program (Reference 5). The expression relates the volume of eroded fill material to the volume of water flowing through the breach. The equation is:

$$\frac{Q_{soil}}{Q_{water}} = Ke^{-X}$$

Where

Q_{soil} = Volume of soil eroded in each time period

Q_{water} = Volume of water discharged each time period

K = Constant of proportionality, 1 for the soil and discharge relationships in this study

e = Base of natural logarithm system

$$X = \frac{b}{H}\tan\phi_d$$

Where

b = Base length of overflow channel at any given time

H = Hydraulic head at any given time

ϕ_d = Developed angle of friction of soil material.

A conservative value of 13 degrees was adopted for materials in the dams investigated.

Solving the equation, which is computerized (DBREACH code), involves a trial and error procedure over short depth and time increments. In the program, depth changes of 0.1 foot or less are used to keep time increments to less than one second during rapid failure and up to about 350 seconds prior to breaching.

The solution of an earth embankment breach begins by solving the erosion equation using a headwater elevation hydrograph assuming no failure, as provided by SOCH model outputs. Erosion is postulated to occur across the entire earth section and to start at the downstream edge when headwater elevations reached a selected depth above the dam top elevation. Subsequently, when erosion reaches the upstream edge of the embankment, breaching and rapid lowering of the embankment begins. Thereafter, computations include headwater adjustments for increased reservoir outflow resulting from the breach.

Figure 43 is a general plan and section of the west saddle dike at Watts Bar Dam. Erosion calculations to determine time of failure were made for the dike. The computed erosion rate and estimated time of failure are shown on Figure 44. The failure was assumed to be a complete washout of the 1300-foot-long dike, down to about elevation 750. An instantaneous disappearance was postulated to occur at the calculated failure time. Similar erosion calculations were made to determine the failure time of the north embankment at Nickajack Dam.

The time of failure as determined by DBREACH was used as an input to the SOCH reservoir routing. At the time of failure the SOCH routing would shift to an after failure rating curve or a total failure of the embankment would be assumed to occur instantaneously.

6. Maximum Water Level Determination

6.1 Probable Maximum Flood

The flood event producing the maximum plant site level was determined to be the 21,400 mi^2 storm. The maximum flood elevation was computed to be 622.1 (Chickamauga Dam safety modifications not complete would add +0.4 feet, elevation 622.5) at the BLN site. Elevations were computed concurrently with discharges for the site using the unsteady flow reservoir model (SOCH code). This is 2.7 feet lower than elevation 624.8 documented in the Final Safety Analysis Report (FSAR – Bellefonte U1/U2). The lower elevation is a result of the dam safety modifications TVA has made beginning in 1982. Table 4 provides information regarding the dam safety modifications that have been made starting at Guntersville dam and for those projects upstream.

A schematic of the runoff and stream course model for the Fort Loudoun Reservoir is shown on Figure 45.

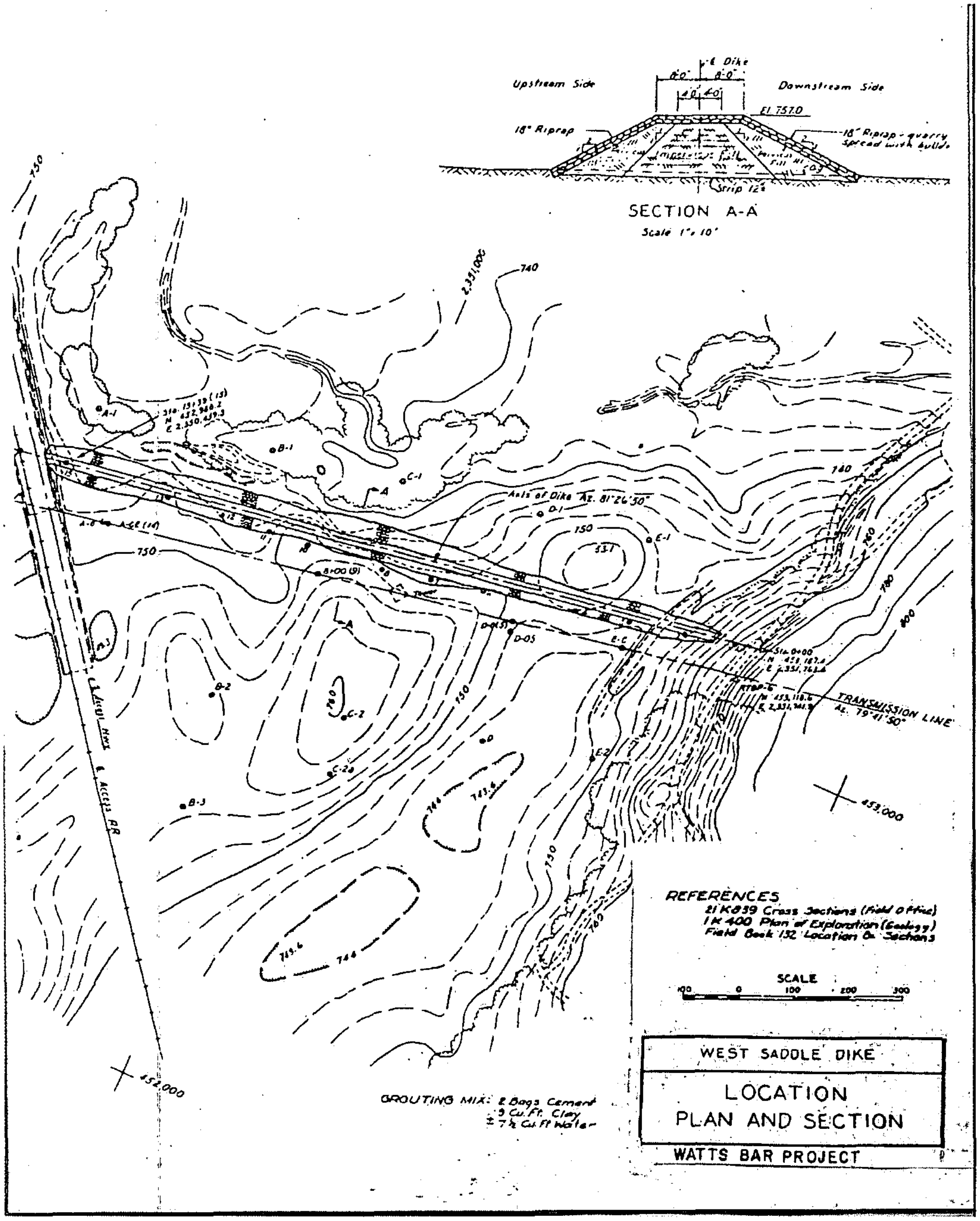


Figure 43. West Saddle at Watts Bar Dam

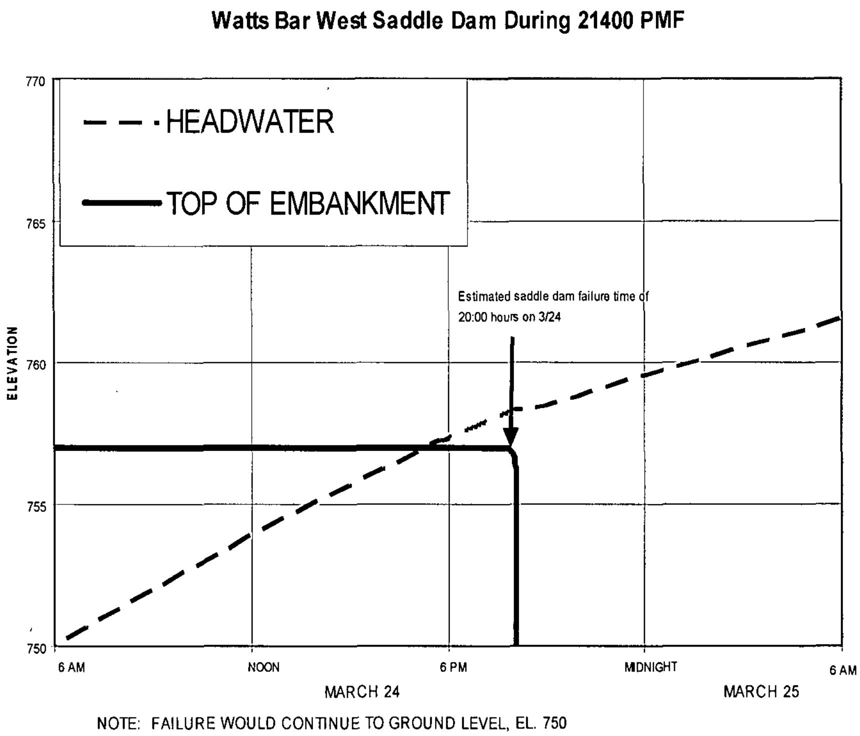


Figure 44. DBREACH Time of Failure – Watts Bar Saddle Dam – 21400 Mi^2 PMF

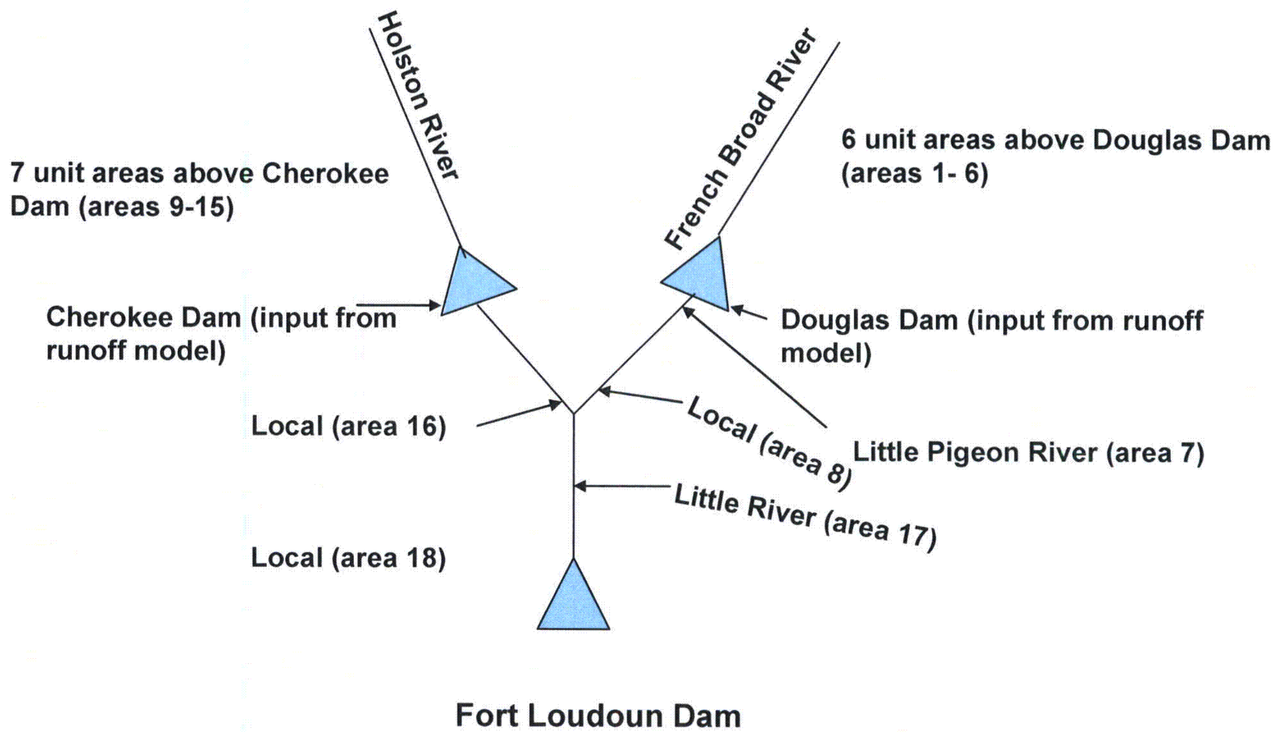


Figure 45. Schematic of Inputs to Runoff and Stream Course Model – Fort Loudoun Reservoir

Table 4. Dam Safety Modifications (Hydrologic)
Above Guntersville Dam

Dam	*Dam Modification	Year Modifications Completed
Main River Dams		
Fort Loudoun - Tellico	Fort Loudoun Dam was raised 3.25 feet with a concrete wall to elevation 833.25. A 2000-foot uncontrolled spillway with crest at elevation 817 was added at Tellico Dam.	1989
Watts Bar	Embankment was raised 10 feet with earthfill/concrete wall to elevation 767.	1997
Nickajack	South embankment was raised 5 feet with earthfill/concrete wall to elevation 657. A 1900-foot roller-compacted concrete overflow dam with top at elevation 634 was added below the north embankment.	1992
Guntersville	Embankments were raised 7.5 feet with earthfill and concrete walls to elevation 617.5.	1996
Tributary Dams		
Blue Ridge	Three (3) additional spillway bays were added in 1982. Embankment was raised 7 feet with earthfill/concrete wall to elevation 1713, and a 295-foot uncontrolled spillway with crest at elevation 1691 was added in 1995.	1995
Boone	Embankment was raised 8.5 feet with earthfill to elevation 1408.5	1984
Chatuge	Embankment was raised 6.5 feet with earthfill to elevation 1946.5.	1986
Cherokee	A portion (600 feet) of the nonoverflow dam was raised 7.75 feet to elevation 1089.75.	1982
Douglas	A portion of the nonoverflow dam was raised 13.5 feet to elevation 1022.5, and eight saddle dams were raised 6.5 feet with earthfill to elevation 1023.5	1988
Nottely	Embankment was raised 13.5 feet with rockfill to elevation 1807.5.	1988
Watauga	Embankment was raised 10 feet with rockfill to elevation 2012.	1983
Fontana	Dam post-tensioned.	1988
Melton Hill	Dam post-tensioned.	1988

* These dam safety modifications enable these projects to safely pass the PMF.
Note: Plans are to armor the embankment at Chickamauga Dam to permit overtopping.

6.2 Potential Dam Failures (Seismically Induced)

The procedures described in ANSI/ANS-2.8-1992 were followed when evaluating potential flood levels from seismically induced dam failures.

There are 21 major dams above BLN. These were examined individually and in groups as documented in the Sequoyah Nuclear Plant FSAR (Reference 6) to determine if dam failure might result from a seismic event and if so, if such a failure concurrent with storm runoff create maximum flood levels at the plant. Dam locations with respect to the plant site are shown in Figure 1.

Two situations were examined for the seismic failure analysis, consistent with ANSI/ANS-2.8-1992:

1. Determination of the water level at the plant during one-half the PMF with full reservoirs if its crest were augmented by flood waves from the postulated failure of upstream dams during an Operating Basis Earthquake (OBE).
2. Determination of the water level at the plant during a 25-year flood with full reservoirs if its crest were augmented by flood waves from the postulated failure of upstream dams during a Safe Shutdown Earthquake (SSE).

The SQN FSAR fully describes the investigation of potential single and multiple failures of all dams upstream of Chickamauga Dam during the two postulated seismic and flood conditions. The postulated failure conditions and assumptions for BLN are identical to those described for SQN. All events referred to in the SQN FSAR were examined using Bellefonte flood conditions. As an example, results of the structural analysis for the OBE plus ½ PMF for Norris Dam is shown on Figure 46. The postulated failure condition at Norris for this event which shows the location of the debris pile is shown on Figure 47.

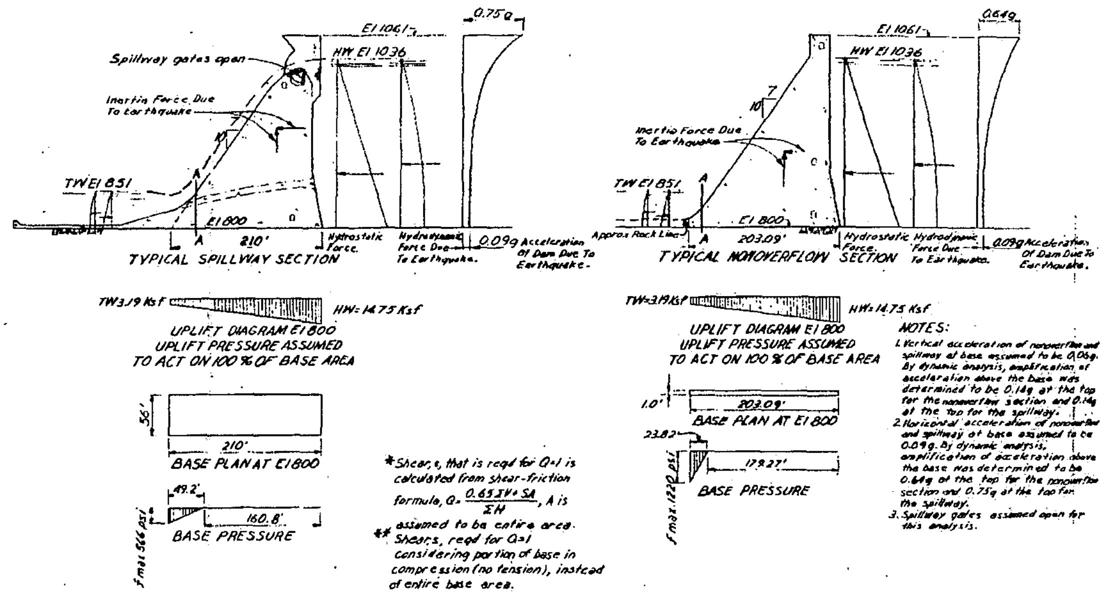
All potentially critical seismic events involving dam failure upstream of the plant were evaluated for the BLN Unit 1 and Unit 2 FSAR. This was done prior to the dam safety modifications. The five postulated events included:

<u>OBE Failures with ½ PMF</u>

1. Norris
2. Cherokee – Douglas
3. Fontana – Hiwassee – Apalachia – Blue Ridge

<u>SSE Failures with 25-year Flood</u>

4. Norris – Cherokee – Douglas
5. Norris – Douglas – Fort Loudoun – Tellico



ΣV	ΣH	ΣH/ΣV	Avg Shear,s	S Reqd For Q=1	f max	FS=ΣMR/ΣMo	Vertical Shear on plane A-A
112,616 K	143,587 K	1.28	85 psi (entire base)	42 psi* (171 psi)**	566 psi	1.25	247 psi

ΣV	ΣH	ΣH/ΣV	Avg Shear,s	S Reqd For Q=1	f max	FS=ΣMR/ΣMo	Vertical Shear on plane A-A
2101 K	2786 K	1.33	95 psi (entire base)	49 psi* (415 psi)**	1220 psi	1.03	535 psi

Figure 2.4.4-8
Spillway & Nonoverflow
Norris Dam, Results
of Analysis for
1/2 SSE

Figure 46. Norris Seismic Analysis SQN FSAR – For OBE and ½ PMF

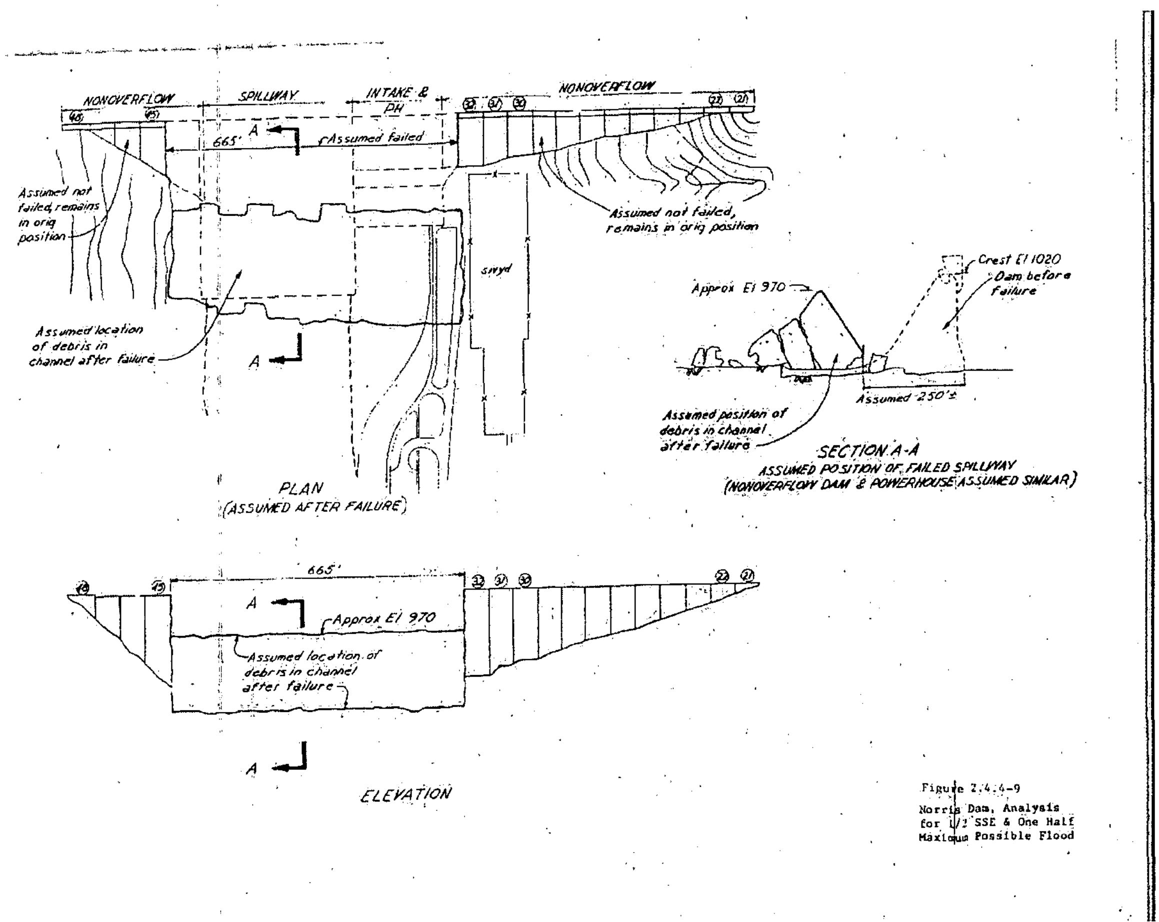


Figure 47. Norris Postulated Seismic Failure Mode – Model Test Results SQN FSAR

In addition to the mainstem SOCH models used for routing floods from postulated seismically induced dam failures of tributary dams, unsteady flow using SOCH were used as adjuncts to the unsteady flow model previously used for the PMF determination. These included unsteady flow SOCH models for Norris, Cherokee, Douglas and Fontana Reservoirs and tributary's and an unsteady flow model developed during TVA's dam safety studies to route the Hiwassee, Apalachia, and Blue Ridge failures.

In addition rating curves for failed conditions had to be determined. Figure 48 shows rating curves computed manually for OBE and SSE failures of Norris Dam. On this figure are also shown the lab model ratings which agree well with the manual computations.

The event producing the maximum flood level from seismically induced dam failures at Bellefonte was originally determined to be the postulated simultaneous failure of Fontana, Hiwassee, Apalachia, and Blue Ridge dams in the OBE coincident with ½ the PMF. The resulting flood level at BLN was determined to be 615.1 and is documented in the Bellefonte U1 and U2 FSAR. In the 1998 reanalysis for dam safety modifications the seismically induced dam failure flood routings were only computed down to Chickamauga Dam. During the update and verification process all flood routings will be computed down to Guntersville Dam which will allow determination of flood levels at BLN.

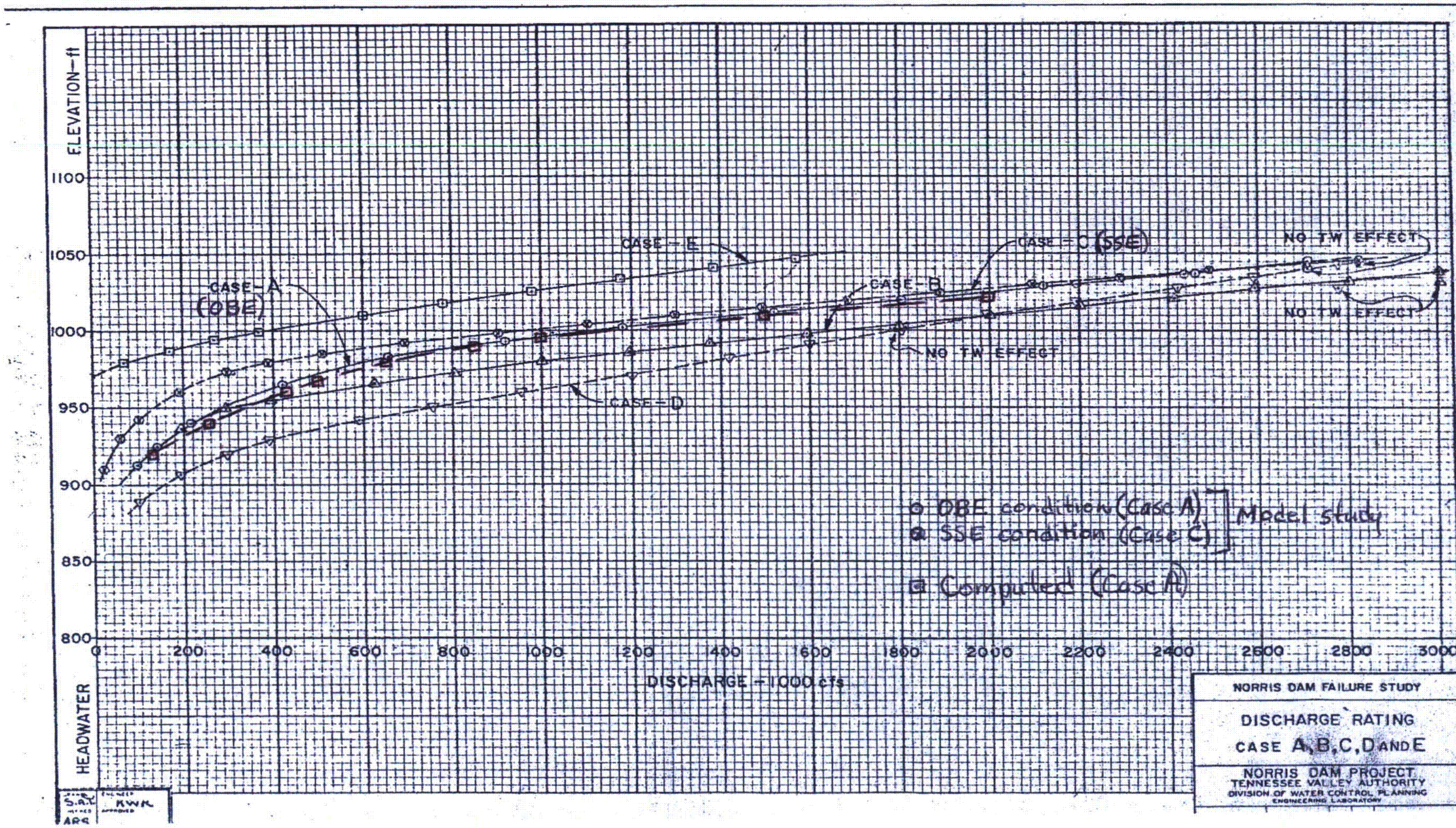


Figure 48. Discharge Rating Curves – Norris Failure

7. Update and Verification of Runoff and Stream Course Models

The following steps are being taken to update and verify the inputs to the computer codes used by TVA to determine maximum elevations at the BLN site.

- River cross-sectional geometry data used in the SOCH code are being checked for accuracy and being updated where appropriate by current bathymetry data. The source of this information is the U.S. Army Corps of Engineers (USACE) Inland Electronic Navigation Charts program. New bathymetry data is available for Guntersville, Nickajack, and Chickamauga Reservoirs from this source. Additional bathymetry information will soon be available from the same source for Watts Bar and Fort Loudoun Reservoirs.

- If significant changes are noted as a result of the updated bathymetry data the SOCH geometric tables will be updated. The SOCH code will be recalibrated and verified to updated steady-state step backwater profiles for the affected reservoirs. This will include calibration of the 2003 flood against observed data in Watts Bar, Chickamauga, Nickajack, and Guntersville Reservoirs.

- Unit hydrographs developed by TVA for the 45 sub-basins above BLN are being independently verified by using more recent storm data, where appropriate, and by use of the HEC1 computer code. Where the unit hydrographs differ, the basis for change will be documented.

- The updated unit hydrographs will be used to generate new design storm inflows, which will be verified by use of the HEC1 code.

- The updated runoff and stream course models will be used to compute PMF and seismic event maximum flood levels at BLN. This will ensure that the data and computer codes used to establish licensing basis flood values have been documented and validated.

- Reservoir operating guides and spillway rating curves are being reviewed, checked and updated as appropriate.

8. Summary and Conclusions

The runoff and stream course model codes developed and used in the original studies by TVA to determine maximum elevations and discharges at BLN were run on a main frame computer. Some of the computer codes used in the runoff model was single purpose. The SOCH code used for the stream course model has been updated to a PC version. The single-purpose codes used in the runoff model have been modified to handle multiple operations as well as additional codes added to facilitate some calculations that originally were done manually. These are also PC versions. These nine computer codes (UNITGRPH, FLDHYDRO, TRBROUTE, CHANROUT, DBREACH, CONVEYANCE, WEIGHTED WIDTH, THIESSEN, and SOCH) will be documented. TVA is confident that there will be good agreement with previous work, which will validate that, given a consistent set of inputs, the PC codes will duplicate results obtained from previous versions of the codes.

In addition all input data (cross sectional data, unit hydrographs, flood hydrographs, etc) will be independently reviewed and updated where necessary and flood levels computed using the updated data. The runoff and stream course model, with updates and verifications, developed by TVA to determine maximum elevations and discharges at BLN are still valid and meet criteria set forth in ANSI/ANS-2.8-1992.

9. References

1. Newton, Donald W., and J.W. Vinyard, "Computer-Determined Unit Hydrographs from Floods." Journal of the Hydraulics Divisions, ASCE, Volume 93, No. HY5, September 1967.

2. U. S. Weather Bureau, "Probable Maximum and TVA Precipitation Over the Tennessee River Basin above Chickamauga," Hydrometeorological Report No. 41, 1965.

3. Kohler, M.A., and R.K. Linsley, Research Paper No. 34, "Predicting the Runoff form Storm Rainfall," U.S. Department of Commerce, Weather Bureau, Washington, September 1951.

4. Stoker, J.J., "Numerical Solution of Flood Prediction and River Regulation Problems," New York University, Institute of Mathematical Sciences, Reports I and II, New York, 1953-54.

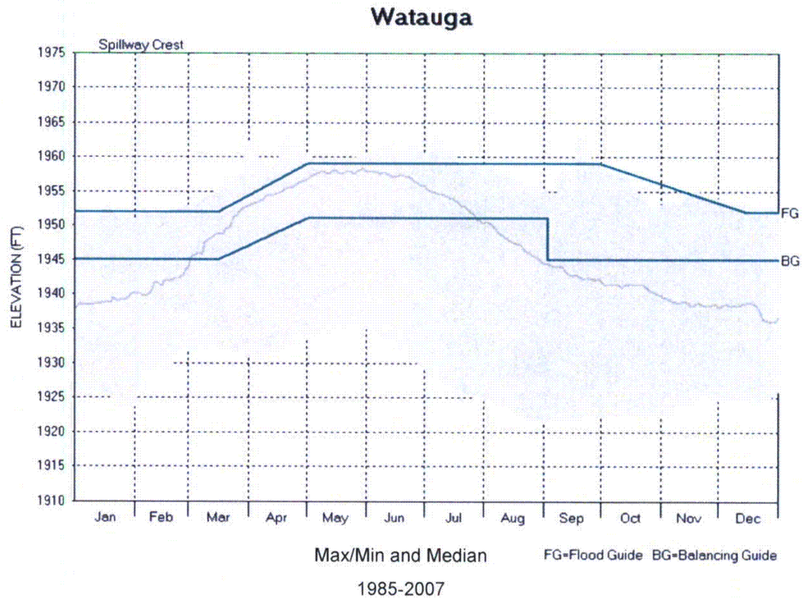
5. Cristofano, E.A., "Method of Computing Erosion Rate for Failure of Earthfill Dams," Engineering and Research Center, Bureau of Reclamation, Denver 1966.

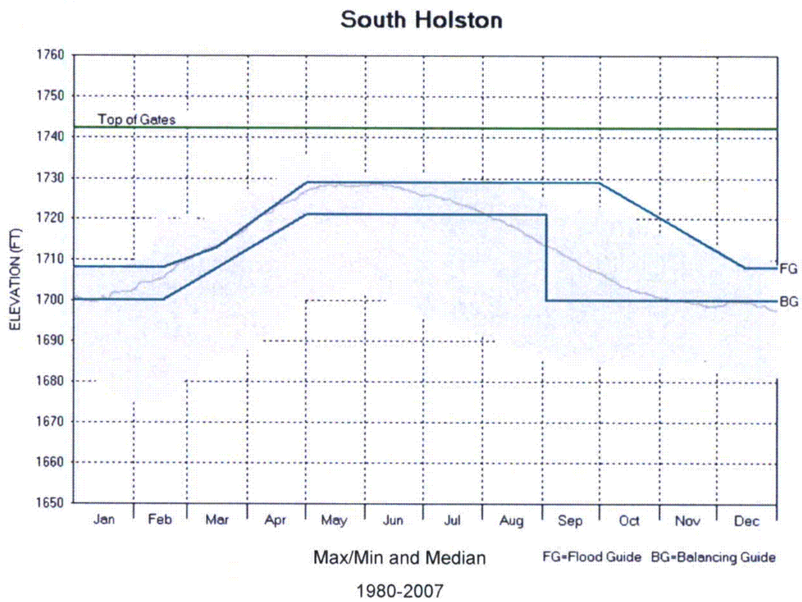
6. FSAR, Sequoyah Nuclear Plant, Docket Nos. 50-327 and 50-328.

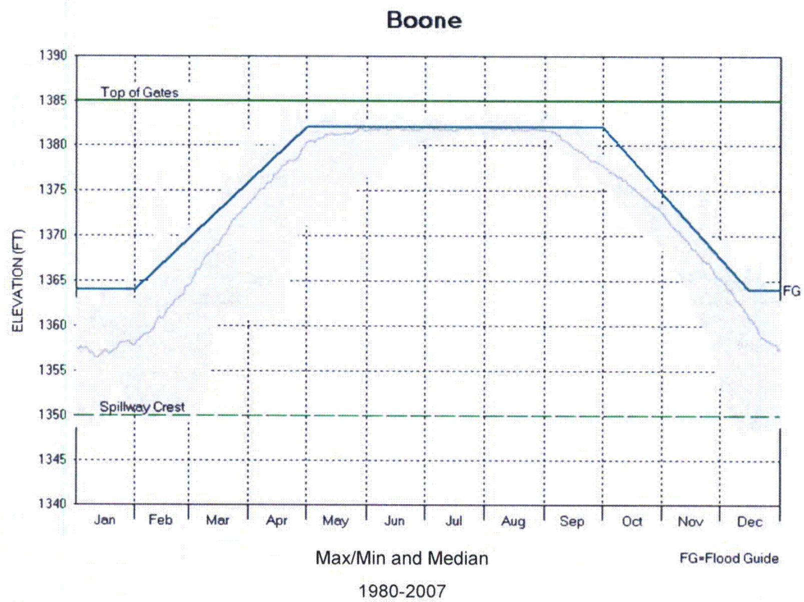
Appendices

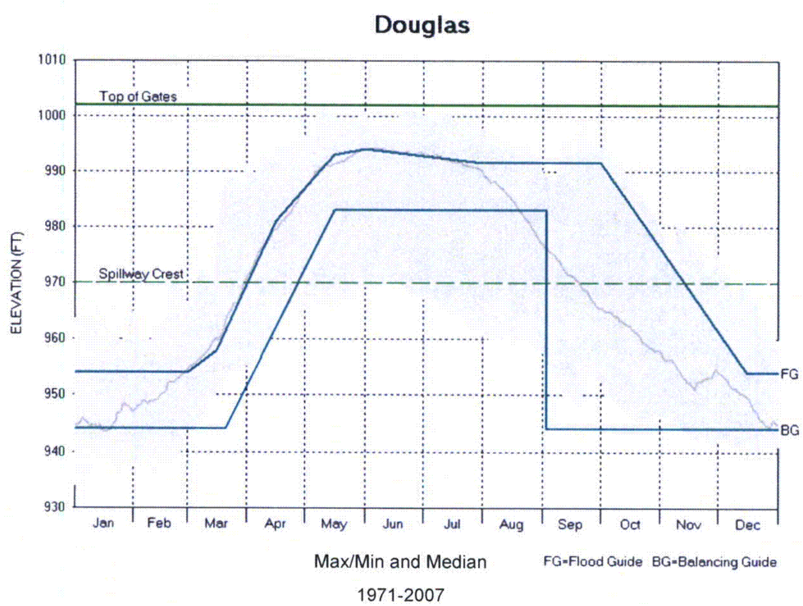
Appendix A – Guide Curves for Tributary and Mainstem Reservoir above BLN

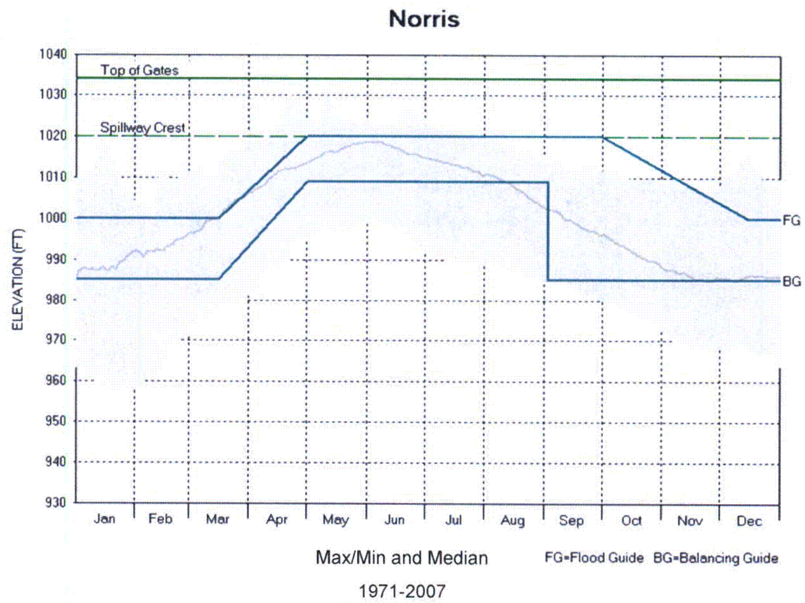
Tributary Reservoirs

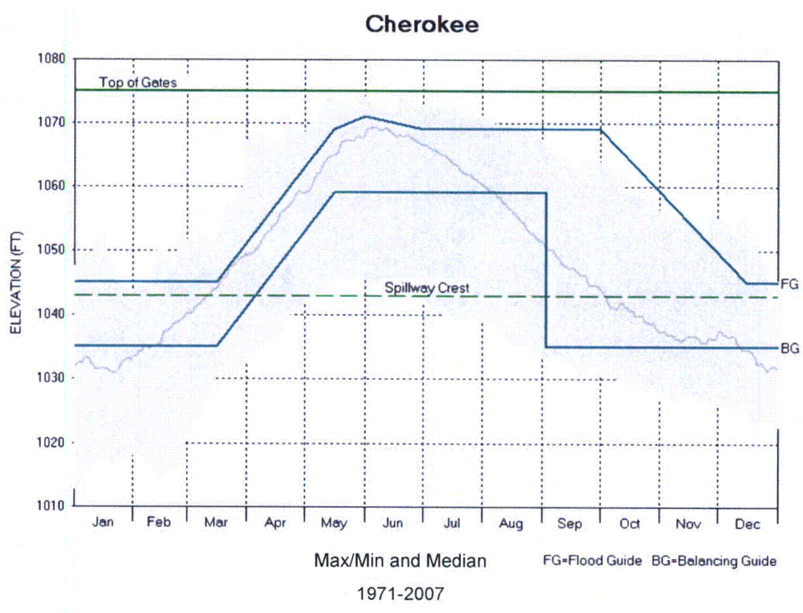




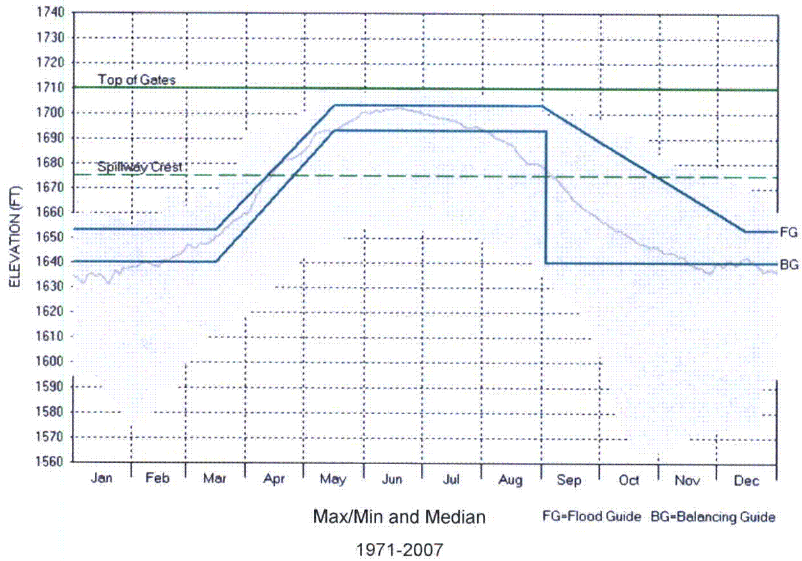

Boone
Top of Gates
Spillway Crest
FG
ELEVATION (FT)
1390
1385
1380
1375
1370
1365
1360
1355
1350
1345
1340
Jan
Feb
Mar
Apr
May
Jun
Jul
Aug
Sep
Oct
Nov
Dec
Max/Min and Median
1980-2007
FG=Flood Guide


Douglas
Top of Gates
Spillway Crest
FG
BG
ELEVATION (FT)
1010
1000
990
980
970
960
950
940
930
Jan
Feb
Mar
Apr
May
Jun
Jul
Aug
Sep
Oct
Nov
Dec
Max/Min and Median
1971-2007
FG=Flood Guide BG=Balancing Guide

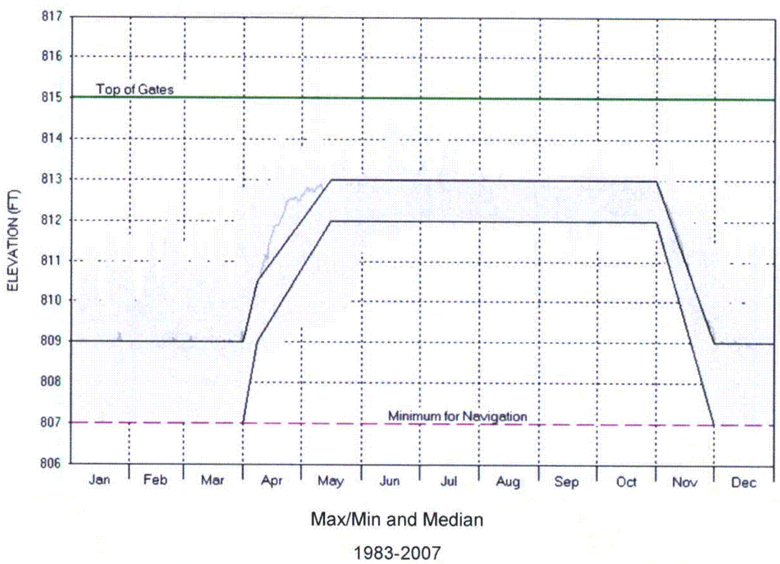

Norris
Top of Gates
Spillway Crest
ELEVATION (FT)
1040
1030
1020
1010
1000
990
980
970
960
950
940
930
FG
BG
Jan Feb Mar Apr May Jun Jul Aug Sep Oct Nov Dec
Max/Min and Median
FG=Flood Guide BG=Balancing Guide
1971-2007

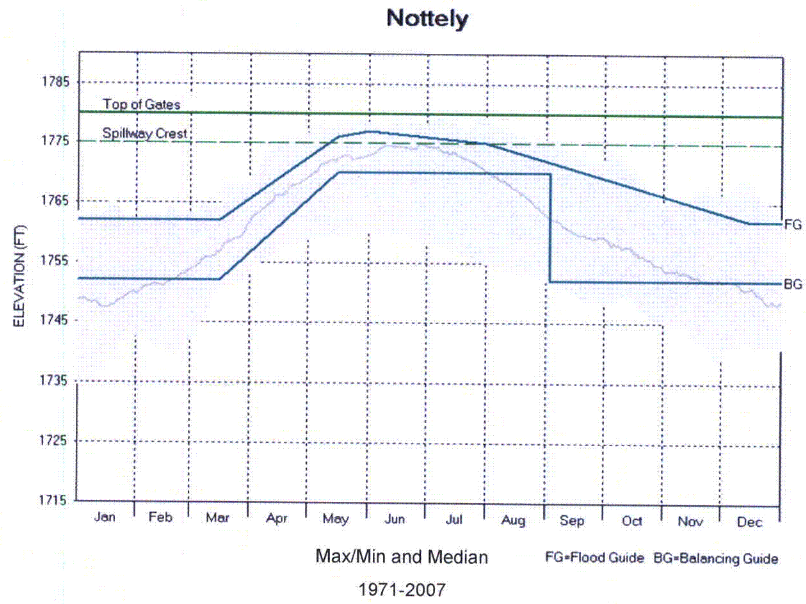

Cherokee
Top of Gates
Spillway Crest
ELEVATION (FT)
1080
1070
1060
1050
1040
1030
1020
1010
FG
BG
Jan Feb Mar Apr May Jun Jul Aug Sep Oct Nov Dec
Max/Min and Median
FG=Flood Guide BG=Balancing Guide
1971-2007

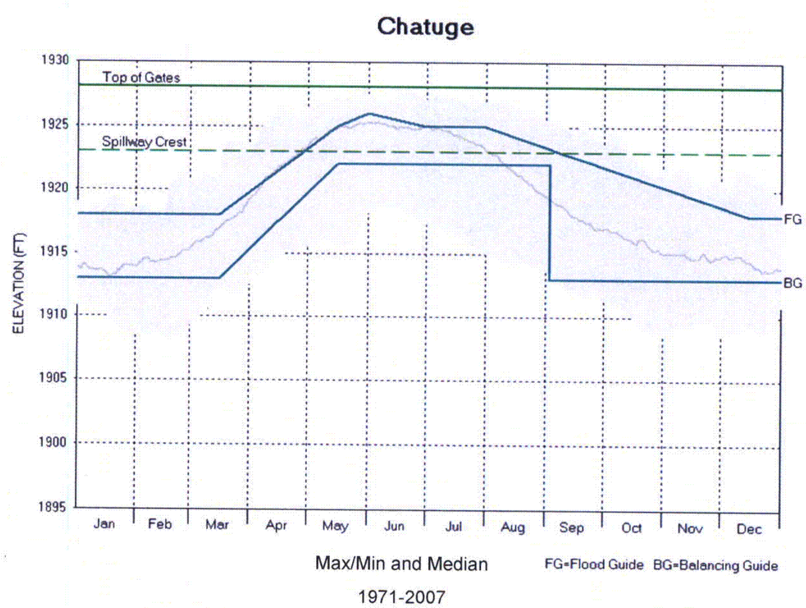
Fontana

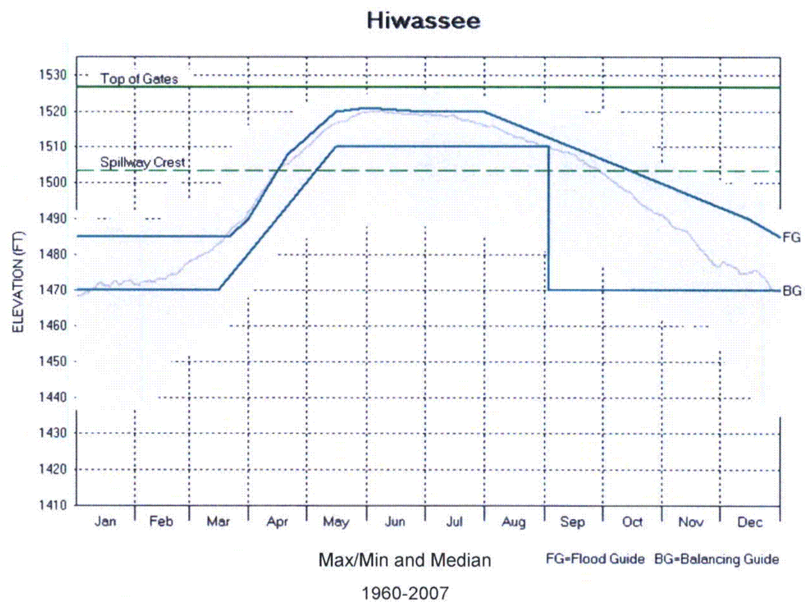


Tellico

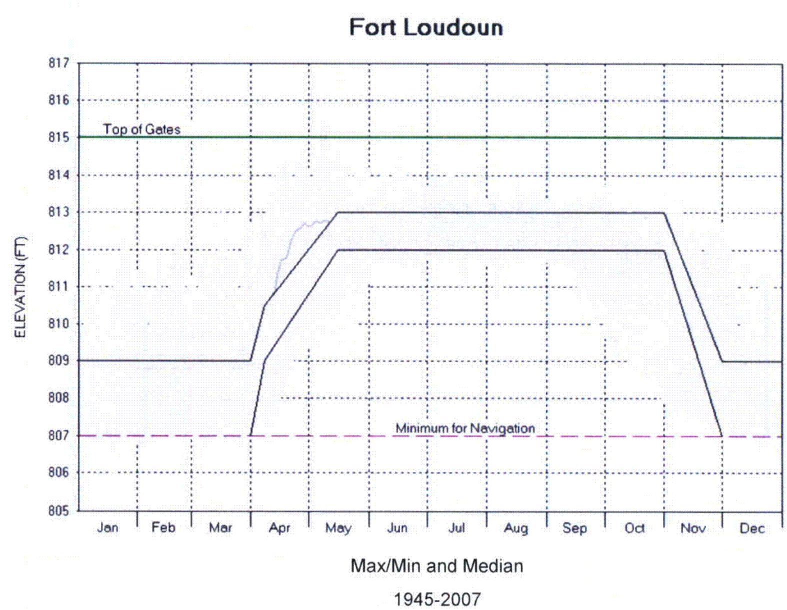


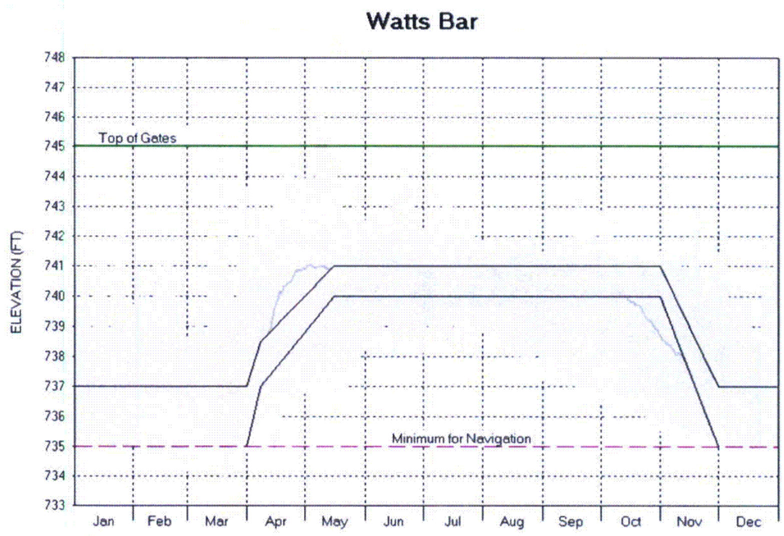

Nottely
Top of Gates
Spillway Crest
ELEVATION (FT)
1785
1775
1765
1755
1745
1735
1725
1715
Jan
Feb
Mar
Apr
May
Jun
Jul
Aug
Sep
Oct
Nov
Dec
FG
BG
Max/Min and Median
1971-2007
FG=Flood Guide BG=Balancing Guide


Chatuge
Top of Gates
Spillway Crest
ELEVATION (FT)
1930
1925
1920
1915
1910
1905
1900
1895
Jan
Feb
Mar
Apr
May
Jun
Jul
Aug
Sep
Oct
Nov
Dec
FG
BG
Max/Min and Median
1971-2007
FG=Flood Guide BG=Balancing Guide



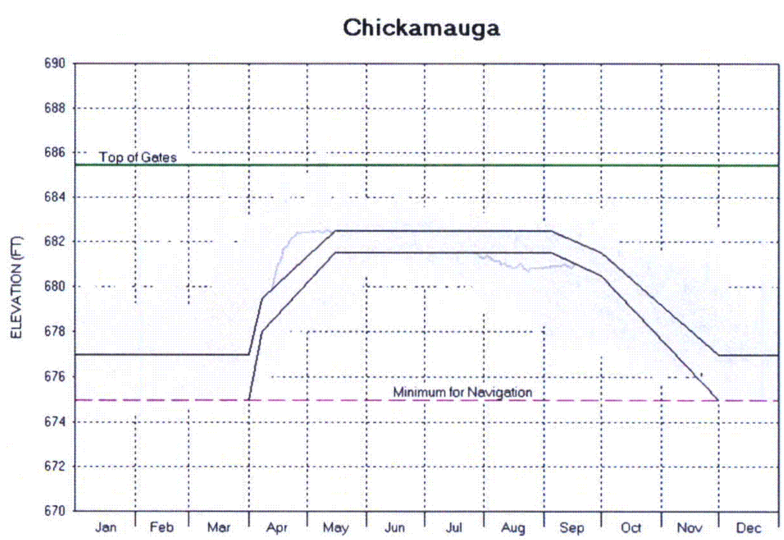
Mainstem Reservoirs





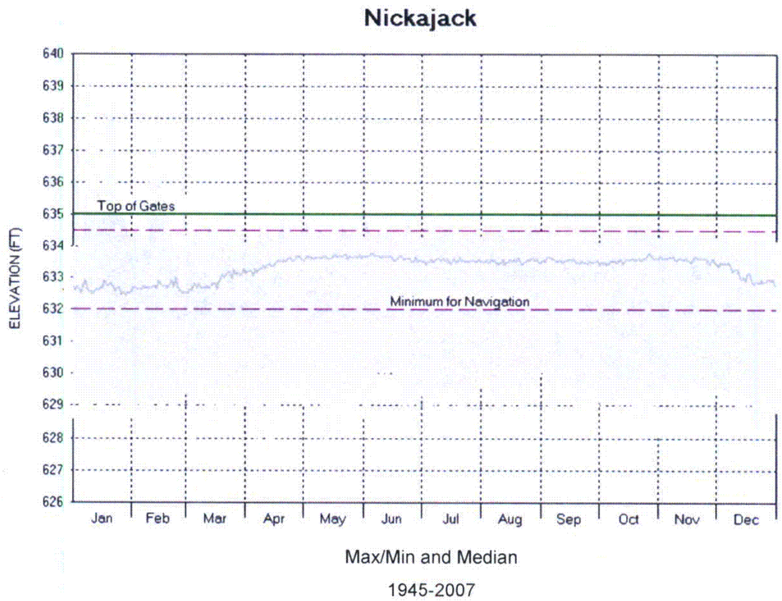
Max/Min and Median

1945-2007

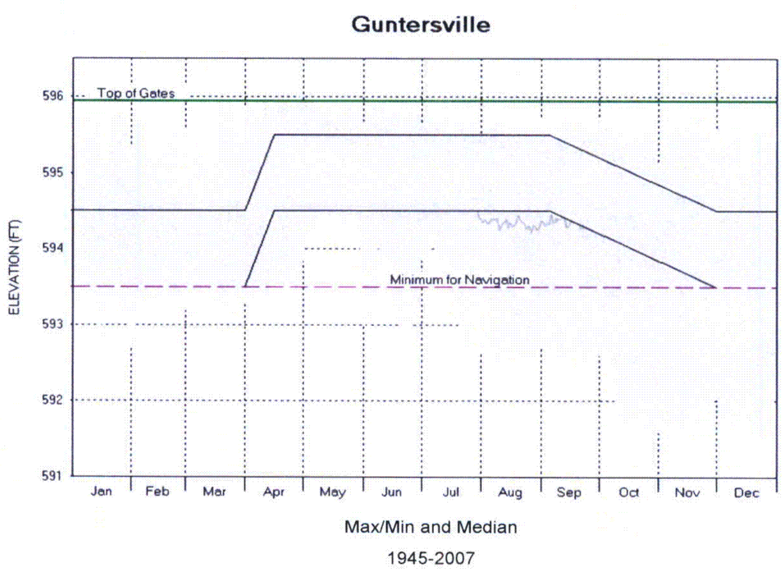


Max/Min and Median

1945-2007



Note – Nickajack is run-of-river project with no flood storage.



Note – Guntersville Located Downstream of BLN

Appendix B – Conveyance Code Details and Excel Verification

CONVEYANCE INPUT DATA ORDER

1	1	0	RIVER MI	52.31										
1	40	3	919	39	5	0.033								
RIVER MI	52.31	1	1	1										
-2501	1	1110	-2501	1	1060	-2500	1	1060	-2000	1	1040	-1845	1	1020
-1045	1	1000	-981	1	980	-840	1	960	-781	1	948.9	-775	1	942.5
-740	1	925	-730	1	921.8	-720	1	922	-715	1	921.5	-700	1	921.5
-620	1	921.6	-595	1	920.9	-540	2	920.4	-480	2	919	-440	2	920.6
-375	2	920.4	-330	2	920.8	-275	2	921.4	-215	2	919.1	-170	2	919.5
-135	2	920.4	-134	2	921.4	-95	2	921.8	-60	2	921.9	-30	2	921.1
0	2	921.6	11	3	924.6	39	3	941.5	45	3	945.5	285	3	980
405	3	1000	500	3	1020	1000	3	1040	1700	3	1060	1700	3	1110
0.1	0.033	0.1												

INPUT DATA (MODIFIED)

TITLE
RIVER MI 52.31

General title up to 40 characters

If value is 1 then geometry with conveyance is output to a separate file

If value is 1 then width value is added to output geometry file

If value is 1 then each segment has a variable N

IP	IB	IN
1	1	0

Section name up to 5 characters long

No. of natural ground points

No. of segments for current cross section

No. of computational elevations

Composite cross section Manning's N value

SEC	NG	NSA	WO	NSTEP	INTV	ENN
1	40	3	919	39	5	0.033

Cross section description up to 20 Characters

Lowest elevation of cross section

Computational elevation interval

TOT
RIVER MI 52.31

If value is 1 then include segment 1's contribution to conveyance

If value is 1 then include segment 2's contribution to conveyance

If value is 1 then include segment 3's contribution to conveyance

LS(1)	LS(2)	LS(3)
1	1	1

Natural ground station

Segment no.

Natural ground elevation

S(i)	SA(i)	G(i)
-2501	1	1110
-2501	1	1060
-2500	1	1060
.	.	.
.	.	.
.	.	.
1000	3	1040
1700	3	1060
1700	3	1110

Segment 1's Manning's N value

Segment 2's Manning's N value

Segment 3's Manning's N value

UN(1)	UN(2)	UN(3)
0.1	0.033	0.1

COMPUTATIONS (DESCRIPTIONS)

				Sums	
H =	1	2	3		Segment nos.
N(H) =	0.1	0.033	3		Segment N's
A(H) =	201965.8	103815.6	145504.6	451286.0	Total cross sectional area
B(H) =	1961.0	551.0	1689.0	4201.0	Total surface width
WP(H) =	2024.9	551.9	1750.8	4327.6	Total wetted perimeter
RS(H) =	99.74	188.09	83.11		Segment hydraulic radii
RS23(H) =	21.51	32.83	19.04		Segment hydraulic radii to the 2/3 power
K(H) =	64721742	153879218	1376261	219977221	Segment conveyance: (1.49/N)*$R^{2/3}$*A Total conveyance
FN(H) =	0.1	0.033	3		
PERC(H) =	29.4%	70.0%	0.6%		Segment's % of conveyance
			RH23 =	10.80	Composite hydraulic radius to the 2/3 power: N*SUMK/(SUMA*1.49)
			RH =	35.47	Composite hydraulic radius
			RP23 =	22.158	
			PN =	0.068	
Current Elevation =	1109				

Change this elevation to make recalculations

Natural ground station	Segment no.	Natural ground elevation	Original	Original	Original	Original	Original	Original	Original	Original	Original	Original	Original	Original	Original	Original	Original
			1109	1109	1109	1109	1109	1109	1109	1109	1109	1109	1109	1109	1109	1109	1109
S(i)	SA(i)	G(i)	Height	Base	A(1)	B(1)	P(1)	Height	Base	A(2)	B(2)	P(2)	Height	Base	A(3)	B(3)	P(3)
			1	1	1	1	1	2	2	2	2	2	3	3	3	3	3
							0.0					0.0					0.0
-2501	1	1110	49	0	0.0	0.0	49.0	0	0	0.0	0.0	0.0	0	0	0.0	0.0	0.0
-2501	1	1060	0	1	49.0	1.0	50.0	0	0	0.0	0.0	0.0	0	0	0.0	0.0	0.0
-2500	1	1060	-20	500	29549.0	501.0	550.4	0	0	0.0	0.0	0.0	0	0	0.0	0.0	0.0
-2000	1	1040	-20	155	41794.0	656.0	706.7	0	0	0.0	0.0	0.0	0	0	0.0	0.0	0.0
-1845	1	1020	-20	800	120994.0	1456.0	1506.9	0	0	0.0	0.0	0.0	0	0	0.0	0.0	0.0
-1045	1	1000	-20	64	128610.0	1520.0	1574.0	0	0	0.0	0.0	0.0	0	0	0.0	0.0	0.0
-981	1	980	-20	141	148209.0	1661.0	1716.4	0	0	0.0	0.0	0.0	0	0	0.0	0.0	0.0
-840	1	960	-11.1	59	157327.5	1720.0	1776.4	0	0	0.0	0.0	0.0	0	0	0.0	0.0	0.0
-781	1	948.9	-6.4	6	158307.3	1726.0	1785.2	0	0	0.0	0.0	0.0	0	0	0.0	0.0	0.0
-775	1	942.5	-17.5	35	164441.0	1761.0	1824.3	0	0	0.0	0.0	0.0	0	0	0.0	0.0	0.0
-740	1	925	-3.2	10	166297.0	1771.0	1834.8	0	0	0.0	0.0	0.0	0	0	0.0	0.0	0.0
-730	1	921.8	0.2	10	168168.0	1781.0	1844.8	0	0	0.0	0.0	0.0	0	0	0.0	0.0	0.0
-720	1	922	-0.5	5	169104.3	1786.0	1849.9	0	0	0.0	0.0	0.0	0	0	0.0	0.0	0.0
-715	1	921.5	0	15	171916.8	1801.0	1864.9	0	0	0.0	0.0	0.0	0	0	0.0	0.0	0.0
-700	1	921.5	0.1	80	186912.8	1881.0	1944.9	0	0	0.0	0.0	0.0	0	0	0.0	0.0	0.0
-620	1	921.6	-0.7	25	191606.5	1906.0	1969.9	0	0	0.0	0.0	0.0	0	0	0.0	0.0	0.0
-595	1	920.9	-0.5	55	201965.8	1961.0	2024.9	0	0	0.0	0.0	0.0	0	0	0.0	0.0	0.0
-540	2	920.4	0	0	201965.8	1961.0	2024.9	-1.4	60	11358.0	60.0	60.0	0	0	0.0	0.0	0.0
-480	2	919	0	0	201965.8	1961.0	2024.9	1.6	40	18926.0	100.0	100.0	0	0	0.0	0.0	0.0
-440	2	920.6	0	0	201965.8	1961.0	2024.9	-0.2	65	31178.5	165.0	165.0	0	0	0.0	0.0	0.0
-375	2	920.4	0	0	201965.8	1961.0	2024.9	0.4	45	39656.5	210.0	210.1	0	0	0.0	0.0	0.0
-330	2	920.8	0	0	201965.8	1961.0	2024.9	0.6	55	49991.0	265.0	265.1	0	0	0.0	0.0	0.0
-275	2	921.4	0	0	201965.8	1961.0	2024.9	-2.3	60	61316.0	325.0	325.1	0	0	0.0	0.0	0.0
-215	2	919.1	0	0	201965.8	1961.0	2024.9	0.4	45	69852.5	370.0	370.1	0	0	0.0	0.0	0.0
-170	2	919.5	0	0	201965.8	1961.0	2024.9	0.9	35	76469.3	405.0	405.1	0	0	0.0	0.0	0.0
-135	2	920.4	0	0	201965.8	1961.0	2024.9	1	1	76657.4	406.0	406.5	0	0	0.0	0.0	0.0
-134	2	921.4	0	0	201965.8	1961.0	2024.9	0.4	39	83966.0	445.0	445.5	0	0	0.0	0.0	0.0
-95	2	921.8	0	0	201965.8	1961.0	2024.9	0.1	35	90516.2	480.0	480.5	0	0	0.0	0.0	0.0
-60	2	921.9	0	0	201965.8	1961.0	2024.9	-0.8	30	96141.2	510.0	510.5	0	0	0.0	0.0	0.0
-30	2	921.1	0	0	201965.8	1961.0	2024.9	0.5	30	101770.7	540.0	540.5	0	0	0.0	0.0	0.0
0	2	921.6	0	0	201965.8	1961.0	2024.9	3	11	103815.6	551.0	551.9	0	0	0.0	0.0	0.0
11	3	924.6	0	0	201965.8	1961.0	2024.9	0	0	103815.6	551.0	551.9	16.9	28	4926.6	28.0	32.7
39	3	941.5	0	0	201965.8	1961.0	2024.9	0	0	103815.6	551.0	551.9	4	6	5919.6	34.0	39.9
45	3	945.5	0	0	201965.8	1961.0	2024.9	0	0	103815.6	551.0	551.9	34.5	240	41019.6	274.0	282.4
285	3	980	0	0	201965.8	1961.0	2024.9	0	0	103815.6	551.0	551.9	20	120	55299.6	394.0	404.0
405	3	1000	0	0	201965.8	1961.0	2024.9	0	0	103815.6	551.0	551.9	20	95	64704.6	489.0	501.1
500	3	1020	0	0	201965.8	1961.0	2024.9	0	0	103815.6	551.0	551.9	20	500	104204.6	989.0	1001.5
1000	3	1040	0	0	201965.8	1961.0	2024.9	0	0	103815.6	551.0	551.9	20	700	145504.6	1689.0	1701.8
1700	3	1060	0	0	201965.8	1961.0	2024.9	0	0	103815.6	551.0	551.9	49	0	145504.6	1689.0	1750.8
1700	3	1110					2024.9					551.9					1750.8

Appendix C – Weighted Width Details

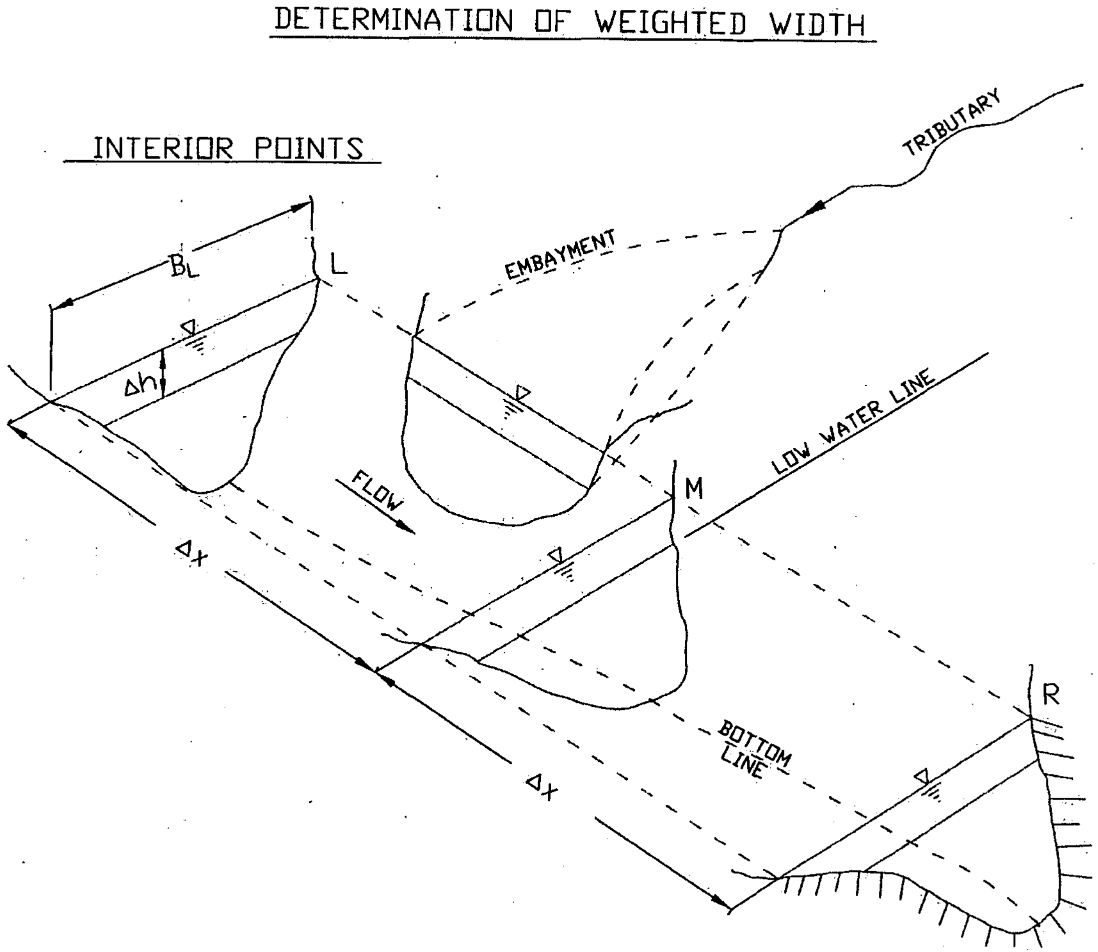


Figure A1. Schematic Representation of Open Channel

Referring to Figure A1, above, the volume of water stored between cross sections L and R located 2ΔX apart, $\cancel{V}_{2\Delta X}$, at any instant of time and for any given water surface elevation may be expressed by:

$$\cancel{V}_{2\Delta X} = \cancel{V}_o + \Delta \cancel{V} \tag{1}$$

in which $\cancel{V}_o$ is a reference volume equal to the volume in the reach 2ΔX corresponding to the minimum water surface elevation expected, and $\Delta\cancel{V}$ is the volume increment due to water levels located Δh above the minimum water level. Equation 1 may be written as:

$$\cancel{V}_{2\Delta X} - \cancel{V}_o = \Delta \cancel{V} = B_W \Delta h \cdot 2\Delta X \tag{2}$$

where B_W is the weighted width and Δh is the water depth above minimum pool. Solving Equation 2 for B_W gives:

$$B_W = \frac{\Delta V}{2\Delta h \Delta X} = \frac{\bar{A}_S \Delta h}{2\Delta h \Delta X} = \frac{\bar{A}_S}{2\Delta X} \qquad (3)$$

in which $\bar{A}_S$ is the mean water surface area in the reach $2\Delta X$ and in the elevation interval Δh. As may be computed from:

$$A_S \approx \frac{A_{S1} + A_{S2}}{Z} \qquad (4)$$

where $\bar{A}_{S1}$ and $\bar{A}_{S2}$ are water surface areas corresponding to the bottom and top of the elevation interval Δh, respectively. For fairly small elevation intervals, $\bar{A}_{S1}$ and $\bar{A}_{S2}$ are practically equal.

The weighted width at interior points (Section M in Figure 1) may be expressed by:

$$B_W = \frac{\overbrace{B_L + 2B_M + B_R}^{\textit{BASED ON FLOW SECTIONS}}}{4} + \overbrace{B_S}^{\textit{STORAGE}} \qquad (5)$$

in which L, M, and R refer to sections at the upstream end, middle, and downstream end of the reach $2\Delta X$, respectively, and subscript S refers to off-channel storage. Equation 3 may be written as:

$$B_W = \frac{\bar{A}_M}{2\Delta X} + \frac{\bar{A}_T}{2\Delta X} \qquad (6)$$

where subscripts M and T refer to main channel and tributary, respectively. Comparing Equations 5 and 6, one can write:

$$B_W = \frac{B_L + 2B_M + B_R}{4} + \frac{\bar{A}_T}{2\Delta X} \qquad (7)$$

Equation 7 is the relation used in practice to get B_W at interior points.

Boundary Points

In this case, only two cross sections located ΔX apart are used. For left, or upstream, boundaries:

$$B_W = \frac{B_L + B_M}{2} + \frac{\bar{A}T}{\Delta X} \qquad (8)$$

and for downstream boundaries:

$$B_W = \frac{B_M + B_R}{2} + \frac{\bar{A}T}{\Delta X} \qquad (9)$$

B_W in Equation 7 is assumed to apply at the center of the reach $2\Delta X$, while B_W in Equations 8 and 9 is assumed to apply at the left or right end of the reach, respectively. After B_W is computed for each desired water surface elevation at each net point, the total volume at any elevation is computed from:

$$\forall_{TOTAL} = \forall_O + \sum_{1}^{N-1} B_{W_{NBL}} \Delta h \cdot 2\Delta X \tag{10}$$

where subscript NBL refers to net points on lines not having points at the boundaries, and N is the number of net points on a line containing boundary points. The total volume is also computed from:

$$\forall_{TOTAL} = \forall_O + \sum_{1}^{N-1} B_{W_{NBL}} \Delta h \cdot 2\Delta X + (B_{W_{BL}} + B_{W_{RB}}) \cdot \Delta h \cdot \Delta X \tag{11}$$

in which subscript BL refers to net points on lines with points at the boundaries, and subscripts LB and RB refer to the left and right boundaries, respectively. Results from equations 10 and 11 are compared to an elevation-volume relation, if available. If the computed total volume is different from that given by the elevation-volume relation, a constant correction factor is applied to all values of B_W to give the same volume as the relation. The constant correction factor is, in general, different for each elevation involved. As an example, if the total volumes, at a certain elevation, computed from Equations 10 and 11 were 10% larger and 5% smaller, respectively, than the volumes from the volume-elevation curve, all values of B_W on non-boundary lines would be multiplied by 0.9 and all values of B_W on boundary lines would be multiplied by 1.05, thereby bringing the computed volumes into agreement with the measured total volume. It is noted that the weighted width, B_W at the boundaries is not used in the unsteady flow calculations, but is used when volumes are balanced.

XC OBTA

20555
DC-US
IAD

Emp# 545362 17APR08 CHAA

482
800

FedEx Express US Airbill

FedEx Tracking Number 8616 8118 7844

Recipient's Copy

RECIPIENT: PEEL HERE

fedex.com 1.800.GoFedEx 1.800.463.3339

1 From This portion can be removed for Recipient's records.

Date 4/17/08 FedEx Tracking Number 861681187844

Sender's Name P. Ray Phone 423 751-2450

Company TVA/CENTRAL MAILROOM

Address 1101 MARKET ST

Dept./Floor/Suite/Room

City CHATTANOOGA State TN ZIP 37402-2881

2 Your Internal Billing Reference

3 To Recipient's Name Document Control Desk Phone 301 415-1132

Company U.S. NRC

Recipient's Address

We cannot deliver to P.O. boxes or P.O. ZIP codes.

Dept./Floor/Suite/Room

Address

To request a package be held at a specific FedEx location, print FedEx address here.

City Washington State DC ZIP 20555

0357156838

8616 8118 7844

4a Express Package Service — Packages up to 150 lbs.

- [x] FedEx Priority Overnight — Next business morning.* Friday shipments will be delivered on Monday unless SATURDAY Delivery is selected.
- [ ] FedEx Standard Overnight — Next business afternoon.* Saturday Delivery NOT available.
- [ ] FedEx First Overnight — Earliest next business morning delivery to select locations.* Saturday Delivery NOT available.
- [ ] FedEx 2Day — Second business day.* Thursday shipments will be delivered on Monday unless SATURDAY Delivery is selected.
- [ ] FedEx Express Saver — Third business day.* Saturday Delivery NOT available.

FedEx Envelope rate not available. Minimum charge: One-pound rate.

* To most locations.

4b Express Freight Service — Packages over 150 lbs.

- [ ] FedEx 1Day Freight* — Next business day.** Friday shipments will be delivered on Monday unless SATURDAY Delivery is selected.
- [ ] FedEx 2Day Freight — Second business day.** Thursday shipments will be delivered on Monday unless SATURDAY Delivery is selected.
- [ ] FedEx 3Day Freight — Third business day.** Saturday Delivery NOT available.

* Call for Confirmation: ** To most locations.

5 Packaging

- [ ] FedEx Envelope*
- [x] FedEx Pak* — Includes FedEx Small Pak, FedEx Large Pak, and FedEx Sturdy Pak.
- [ ] FedEx Box
- [ ] FedEx Tube
- [ ] Other

* Declared value limit $500.

6 Special Handling — Include FedEx address in Section 3.

- [ ] SATURDAY Delivery — Not available for FedEx Standard Overnight, FedEx First Overnight, FedEx Express Saver, or FedEx 3Day Freight.
- [ ] HOLD Weekday at FedEx Location — Not available for FedEx First Overnight.
- [ ] HOLD Saturday at FedEx Location — Available ONLY for FedEx Priority Overnight and FedEx 2Day to select locations.

Does this shipment contain dangerous goods? One box must be checked.

- [ ] No
- [ ] Yes — As per attached Shipper's Declaration.
- [ ] Yes — Shipper's Declaration not required.
- [ ] Dry Ice — Dry Ice, 9, UN 1845 ______ x ______ kg
- [ ] Cargo Aircraft Only

Dangerous goods (including dry ice) cannot be shipped in FedEx packaging.

7 Payment Bill to: Enter FedEx Acct. No. or Credit Card No. below. Obtain Recip. Acct. No.

- [ ] Sender — Acct. No. in Section 1 will be billed.
- [ ] Recipient
- [ ] Third Party
- [ ] Credit Card
- [ ] Cash/Check

Total Packages Total Weight Credit Card Auth.

†Our liability is limited to $100 unless you declare a higher value. See the current FedEx Service Guide for details.

8 Residential Delivery Signature Options If you require a signature, check Direct or Indirect.

- [ ] No Signature Required — Package may be left without obtaining a signature for delivery.
- [x] Direct Signature — Someone at recipient's address may sign for delivery. Fee applies.
- [ ] Indirect Signature — If no one is available at recipient's address, someone at a neighboring address may sign for delivery. Fee applies.

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This pouch is resealable.

Align bottom of Airbill Pouch here.