ASSOCIATED ATTACHMENTS/ENCLOSURES:

Attachment 02.04.03-8AH: Subbasin 35 (Emory River at Mouth) Unit Hydrograph Validation

(69 Pages including Cover Sheet)

NPG CALCULATION COVERSHEET/CCRIS UPDATE

REV 0 EDMS/RIMS NO. L58 090227 002	EDMS TYPE: calculations(nuclear)	EDMS ACCESSION NO (N/A for REV. 0) L58 091230 043

Calc Title: Subbasin 35 (Emory River at Mouth) Unit Hydrograph Validation

CALC ID	TYPE	ORG	PLANT	BRANCH	NUMBER	CUR REV	NEW REV	REVISION APPLICABILITY
CURRENT	CN	NUC	GEN	CEB	CDQ000020080067	1	2	Entire calc ☒ Selected pages ☐

ACTION	NEW ☐ REVISION ☒	DELETE ☐ RENAME ☐	SUPERSEDE ☐ DUPLICATE ☐	CCRIS UPDATE ONLY ☐ (Verifier Approval Signatures Not Required)	No CCRIS Changes ☐ (For calc revision, CCRIS been reviewed and no CCRIS changes required)

UNITS	SYSTEMS	UNIDS
N/A	N/A	N/A

DCN,EDC,N/A	APPLICABLE DESIGN DOCUMENT(S)	CLASSIFICATION
*See Below	N/A	"E"

QUALITY RELATED?	SAFETY RELATED? (If yes, QR = yes)	UNVERIFIED ASSUMPTION	SPECIAL REQUIREMENTS AND/OR LIMITING CONDITIONS?	DESIGN OUTPUT ATTACHMENT?	SAR/TS and/or ISFSI SAR/CoC AFFECTED
Yes ☒ No ☐	Yes ☒ No ☐	Yes ☐ No ☒	Yes ☐ No ☒	Yes ☐ No ☒	Yes ☐ No ☒

PREPARER ID	PREPARER PHONE NO	PREPARING ORG (BRANCH)	VERIFICATION METHOD	NEW METHOD OF ANALYSIS
clstokes	615-252-4343	CEB	See Page 9	☐ Yes ☒ No

PREPARER SIGNATURE	DATE	CHECKER SIGNATURE	DATE
Carrie L. Stokes *Carrie L Stokes*	12/11/09	Karl Stickley *Karl Stickley*	12/11/09
VERIFIER SIGNATURE	**DATE**	**APPROVAL SIGNATURE**	**DATE**
Karl Stickley *Karl Stickley*	12/11/09	K.R. Spates *K.R. Spates*	12/23/09

STATEMENT OF PROBLEM/ABSTRACT

Validate existing unit hydrograph for Emory River at Mouth (Subbasin 35) using recent floods.

*EDCN 22404A (SQN), EDCN 54018A (WBN), Later (BFN)

This calculation contains electronic attachments and must be stored in EDMS as an Adobe .pdf file to maintain the ability to retrieve the electronic attachments.

MICROFICHE/EFICHE Yes ☐ No ☒ FICHE NUMBER(S)

☐ LOAD INTO EDMS AND DESTROY
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L58 090707 003

NPG CALCULATION COVERSHEET/CCRIS UPDATE

REV 0 EDMS/RIMS NO.	EDMS TYPE:	EDMS ACCESSION NO (N/A for REV. 0)
L58 090227 002	calculations(nuclear)	L58 090707 003

Calc Title: Subbasin 35 (Emory River at Mouth) Unit Hydrograph Validation

CALC ID	TYPE	ORG	PLANT	BRANCH	NUMBER	CUR REV	NEW REV	REVISION APPLICABILITY
CURRENT	CN	NUC	GEN	CEB	CDQ000020080067	0	1	Entire calc ☒ Selected pages ☐
NEW	CN	NUC						

ACTION: NEW ☐ REVISION ☒ DELETE ☐ RENAME ☐ SUPERSEDE ☐ DUPLICATE ☐ CCRIS UPDATE ONLY ☐ (Verifier Approval Signatures Not Required) No CCRIS Changes ☐ (For calc revision, CCRIS been reviewed and no CCRIS changes required)

UNITS	SYSTEMS	UNIDS
N/A	N/A	N/A

DCN,EDC,N/A	APPLICABLE DESIGN DOCUMENT(S)	CLASSIFICATION
*See Below	N/A	"E"

QUALITY RELATED?	SAFETY RELATED? (If yes, QR = yes)	UNVERIFIED ASSUMPTION	SPECIAL REQUIREMENTS AND/OR LIMITING CONDITIONS?	DESIGN OUTPUT ATTACHMENT?	SAR/TS and/or ISFSI SAR/CoC AFFECTED
Yes ☒ No ☐	Yes ☒ No ☐	Yes ☐ No ☒	Yes ☐ No ☒	Yes ☐ No ☒	Yes ☐ No ☒

PREPARER ID	PREPARER PHONE NO	PREPARING ORG (BRANCH)	VERIFICATION METHOD	NEW METHOD OF ANALYSIS
PDMADDUX	(423) 751-8015	CEB	Design Review	☐ Yes ☒ No

PREPARER SIGNATURE	DATE	CHECKER SIGNATURE	DATE
Perry D. Maddux	07/01/2009	Dennis L. Lundy	7-1-09
VERIFIER SIGNATURE	**DATE**	**APPROVAL SIGNATURE**	**DATE**
Dennis L. Lundy	7/1/09	K.R. Spates	7-2-09

STATEMENT OF PROBLEM/ABSTRACT

Validate existing unit hydrograph for Emory River at Mouth (Subbasin 35) using recent floods.

Revision 1 Issued to address PER 171268, and lift UVA.

*EDCN Numbers:
WBN: 54018
SQN: 22404
BFN: (Later)

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MICROFICHE/EFICHE Yes ☐ No ☒ FICHE NUMBER(S)

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NPG CALCULATION COVERSHEET/CCRIS UPDATE

CALC ID	TYPE	ORG	PLANT	BRANCH	NUMBER	REV
	CN	NUC	GEN	CEB	CDQ000020080067	2

ALTERNATE CALCULATION IDENTIFICATION

BLDG	ROOM	ELEV	COORD/AZIM	FIRM	Print Report Yes ☐
				BWSC	

CATEGORIES NA

KEY NOUNS (A-add, D-delete)

ACTION (A/D)	KEY NOUN	A/D	KEY NOUN

CROSS-REFERENCES (A-add, C-change, D-delete)

ACTION (A/C/D)	XREF CODE	XREF TYPE	XREF PLANT	XREF BRANCH	XREF NUMBER	XREF REV
A	P	CO	GEN	CEB	FLDHYDRO Version 1.0	
A	P	CO	GEN	CEB	UNITGRPH Version 1.0	

CCRIS ONLY UPDATES:
Following are required only when making keyword/cross reference CCRIS updates and page 1 of form NEDP-2-1 is not included:

PREPARER SIGNATURE	DATE	CHECKER SIGNATURE	DATE
PREPARER PHONE NO.	EDMS ACCESSION NO.		

NPG CALCULATION RECORD OF REVISION	
CALCULATION IDENTIFIER CDQ000020080067	
Title	Subbasin 35 (Emory River at Mouth) Unit Hydrograph Validation
Revision No.	DESCRIPTION OF REVISION
0	Initial issue
1	Revision 1 lifts the UVA under Section 3.2, and addresses PER 171268, which identified an incorrect version of FLDHYDRO that was used in development of Revision 0 of this calculation. The UVA was lifted by issuance of the SPP 2.6 software documentation (Users Manual - Ref. 7) for FLDHYDRO, and Reference 2.7 was revised to reflect the EDMS number of the issued document. Four electronically attached FLDHYDRO output files were replaced with equivalent files generated using current QA Version 1.0 of FLDHYDRO, dated 11/04/2008. Results of FLDHYDRO and this calculation were unaffected by the version change. Revised page 14 to add the word "data" following the word "precipitation" in the ANSI/ANS-2.8-1992 quotation. Corrected FILEKEEPER Number for Attachment 3-1. Significant changes in Revision 1 are noted with a right margin revision bar. Pages replaced: 1, 2, 3, 8, 11, 12, 14 Pages Added / Deleted: None Attachments 2-5, 2-7, 2-9, and 2-11 (electronically attached FLDHYDRO output files) were replaced. See page 8 for the listing of file names and their associated attachment numbers. Total pages of calculation hardcopy for Revision 1 = 66 pages
2	This calculation was revised to address the following: • PER 203951- The verification of the original calculation was completed by personnel who had not completed the required NEDP-7 Job Performance Record (JPR). A verification JPR is now in place for all personnel engaged in verification tasks. Checking includes only changes made in this revision as the checking of the calculation was not impacted by PER 203951. The verification is inclusive of work completed prior to this revision. • PER 204081- The verification of Rev 1 of the calculation was completed by a TVA Project Engineer with expired qualifications. • PER 203872- replace NEDP-2 forms on Pages 2 through 7 with the forms from the NEDP-2 Revision in effect at the time of calculations issuance. Significant changes in Revision 2 are noted with a right margin revision bar. Administrative changes and typos are excluded. Pages Added: 1a & 9a Pages Replaced: 1-7, 9, 11, 16 & 56 Total pages of calculation hard copy for Revision 2= 68

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CALCULATION IDENTIFIER : CDQ000020080067	Revision: 2

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NPG COMPUTER INPUT FILE STORAGE INFORMATION SHEET			
Document CDQ000020080067	Rev. 2	Plant: GEN	
Subject: Subbasin 35 (Emory River at Mouth) Unit Hydrograph Validation			
☐ Electronic storage of the input files for this calculation is not required. Comments:			
☒ Input files for this calculation have been stored electronically and sufficient identifying information is provided below for each input file. (Any retrieved file requires re-verification of its contents before use.)			
See listing of electronically attached Input & Output files on the following page			
☐ Microfiche/eFiche			

ELECTRONIC FILE ATTACHMENTS			
Document: CDQ000020080067	Rev. 1	Plant: Gen	
Subject: Subbasin 35 (Emory River at Mouth) Unit Hydrograph Validation			

The files listed below, which contain both input and output data, are electronically attached to the parent Adobe .pdf calculation file. All files are therefore stored in an unalterable medium and are retrievable through the EDMS number for this calculation. Click on the "Attachments" Tab within Adobe to view the attachment listing, to access and view the files as needed.

Electronic Attachment: Name of File or Folder	File Location
Supporting Spreadsheets	
Attachment 1- 1: EmoryAtOakdale_Rev0.xls (provided by the TVA)	Attached to PDF
Attachment 1- 2: Basin_35-UH_Derivation.xls	Attached to PDF
Attachment 1- 3: UH_S-graph_Translation.xls	Attached to PDF
Attachment 1- 4: Emory_at_Mouth_Calculations.xls	Attached to PDF
Attachment 1- 5: May1973_Flood.xls	Attached to PDF
Attachment 1- 6: Largest_Events.xls	Attached to PDF
Attachment 1- 7: Gage-data_prob-reccurrence.xls	Attached to PDF
Attachment 1- 8: Basin_35-Base_Flow_Separation.xls	Attached to PDF
Attachment 1- 9: NWS_Precip_Basins-35_CT.xls	Attached to PDF
Attachment 1- 10: rain1990.xls (provided by the TVA)	Attached to PDF
Attachment 1- 11: rain2003.xls (provided by the TVA)	Attached to PDF
Attachment 1- 12: rain2004.xls (provided by the TVA)	Attached to PDF
Attachment 1- 13: Rainfall_1973.xls	Attached to PDF
Attachment 1- 14: Rainfall_1990.xls	Attached to PDF
Attachment 1- 15: NWS_6hr-Comp.xls	Attached to PDF
Attachment 1- 16: ThiessenWts_andGages_forDelivery_Rev0.xls (provided by the TVA)	Attached to PDF
Attachment 1- 17: Basin_35-USGS_Scale.xls	Attached to PDF
FLDHYDRO and UNITGRPH Files	
Attachment 2- 1: UGin_35.dat	Attached to PDF
Attachment 2- 2: B35_L1.prn	Attached to PDF
Attachment 2- 3: B35_L1.plt	Attached to PDF
Attachment 2- 4: Bas35_03FS.dat	Attached to PDF
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Attachment 2- 6: Bas35_04.dat	Attached to PDF
Attachment 2- 7: Bas35_04.out (Revised by Revision 1)	Attached to PDF
Attachment 2- 8: Bas35_90BA.dat	Attached to PDF
Attachment 2- 9: Bas35_90BA.out (Revised by Revision 1)	Attached to PDF
Attachment 2- 10: Bas35_73-AVE.dat	Attached to PDF
Attachment 2- 11: Bas35_73-AVE.out (Revised by Revision 1)	Attached to PDF
HEC-HMS File Folders	
Attachment 3- 1: Basin-35_20080067.zip	Filekeeper No. 311415
Attachment 3- 2: Basin-35-4hr_20080067.zip	Filekeeper No. 311414

NPG CALCULATION VERIFICATION FORM

Calculation Identifier CDQ000020080067 Revision 2

Method of verification used:		
1. Design Review	☒	Verifier [signature] Date 12/11/09
2. Alternate Calculation	☐	Karl E. Stickley
3. Qualification Test	☐	

Comments:

This calculation entitled, "Subbasin 35 (Emory River at Mouth) Unit Hydrograph Validation" was verified by independent design review. The process involved a critical review of the calculation to ensure that it is correct and complete, uses appropriate methodologies, and achieves its intended purpose. The inputs were reviewed and determined to be appropriate inputs for this calculation. The results of the calculation were reviewed and were found to be reasonable and consistent with the inputs provided. Backup files and documents were consulted as necessary to verify data and analysis details found in the calculation.

Detailed comments and editorial suggestions for the changes made in this revision were transmitted to the author and reviewer by email along with a marked up copy of the calculation.

FLDHYDRO input files for this calculation were developed with a check volume used to calibrate the modeled. This is an acceptable practice, but the FLDHYDRO output calibrated with a check volume should have been compared to FLDHYDRO output for the same storm that was not calibrated with a check volume. This comparison allows better selection of storms that have runoff and environmental characteristics most compatible with the FLDHYDRO program. This verification process included a comparison of FLDHYDRO runs calibrated with check volumes and FLDHYDRO runs that were not calibrated with check volumes, no notable discrepancies were found.

The calculations tried to use a straight-line interpolation between data points for the S-Graph Method. This resulted in "stair-step" hydrographs with a steep rise in one hour followed by a horizontal line for the subsequent time periods (Figures 2 and 3). A curvilinear approximation of the S-Graph should have been used to determine the hourly data points, which would have resulted in a smooth hydrograph. As the "stair-step" hydrograph was not adopted for use, the methodology does not materially affect the results of the calculations.

For transferring the gaged flows downstream to the mouth of the river, regression coefficients based upon equations developed by USGS for ungaged sites were used. These coefficients (used with an area ratio) vary from 0.694 to 0.753 (Table 2). In the past, TVA calculations have used a coefficient of 0.5 when transferring gaged flows downstream. Use of the USGS coefficients in these calculations is appropriate.

The recession arm of the earlier storm in Figure 10 appears to begin too late, resulting in a steep ascension arm for the main storm. However, this is a judgment call and does not materially affect the results of the calculations.

(Note: The design verification of this calculation revision is for the total calculation, not just the changes made in the revision. This complete re-verification is performed to disposition PER 203951 as described in the Calculation Revision Log on Page 3)

NPG CALCULATION VERIFICATION FORM

Calculation Identifier CDQ000020080067 Revision 0

Method of verification used:		
1. Design Review	☒	Verifier Bob Swain Date 2/4/2009
2. Alternate Calculation	☐	
3. Qualification Test	☐	

Comments:

The calculation entitled, "Subbasin 35 (Emory River at Mouth) Unit Hydrograph Validation" was verified by an independent design review. The process involved a critical review of the calculation to ensure that it is correct and complete, uses appropriate methodologies, and achieves its intended purpose. Backup files and documents were consulted as necessary to verify data and analysis details found in the calculation. Detailed comments and editorial suggestions were transmitted to the author and reviewer by email along with a marked up copy of the calculation. All of the comments were minor in nature. Most of the editorial suggestions were adopted in the final document.

Two primary issues were discussed and resolved during the verification process. The issues involved the choice of floods selected for verification and the method for a really distributing storm rainfall amounts. The 1973 and 1990 floods were more than double the size of the 2003 and 2004 floods, so they were added to the analysis. Gaged precipitation data were available for use in all of the flood simulations; however, radar-based precipitation data were only available for the 2003 and 2004 flood simulations. The 2003 and 2004 floods were successfully reproduced using radar-based precipitation data. For the 1973 and 1990 flood simulations, rainfall was distributed using an area-weighted approach (Theissen polygons) and equal-weighted approach (arithmetic average). The method for distributing the rainfall over the watershed did not make much difference. The critical parameter in simulating the floods was the temporal distribution of rainfall, which was determined by using a temporal distribution at a rain gage. The model did not reproduce the 1990 flood very well, but successfully reproduced the 1973 flood.

Based on the successful simulations of the 1973, 2003, and 2004 floods, the calculation supports the conclusion that the unit hydrograph developed for Subbasin 35 (Emory River at Mouth) has been validated against recent floods.

1 Purpose

The TVA's Water Management Group has adapted computer codes and data sets developed from flood studies carried out over the past 40 years to develop a dynamic hydrologic model (Reference 1) of the Tennessee River upstream of the Guntersville Dam for use in the Probable Maximum Flood (PMF) and dam break analysis for the Sequoyah, Watts Bar, and planned Bellefonte Nuclear plant sites (Note that this calculation will also be used in similar future PMF and dam break analyses for the Browns Ferry Nuclear plant).

Inputs to the dynamic model include hydrographs for 47 subbasins developed from design rainfall inputs convoluted with unit hydrographs developed specifically for each subbasin. These unit hydrographs were developed by the TVA in previous studies, mostly in the 1970s and early 1980s, utilizing observed rainfall and streamflow and reservoir headwater elevation and discharge data, and are being validated by checking their performance in reproducing recent floods.

This calculation presents the validation of the unit hydrograph for the Emory River at Mouth local area, Subbasin 35. This subbasin is located within the Tennessee River watershed as shown in Figure 1.

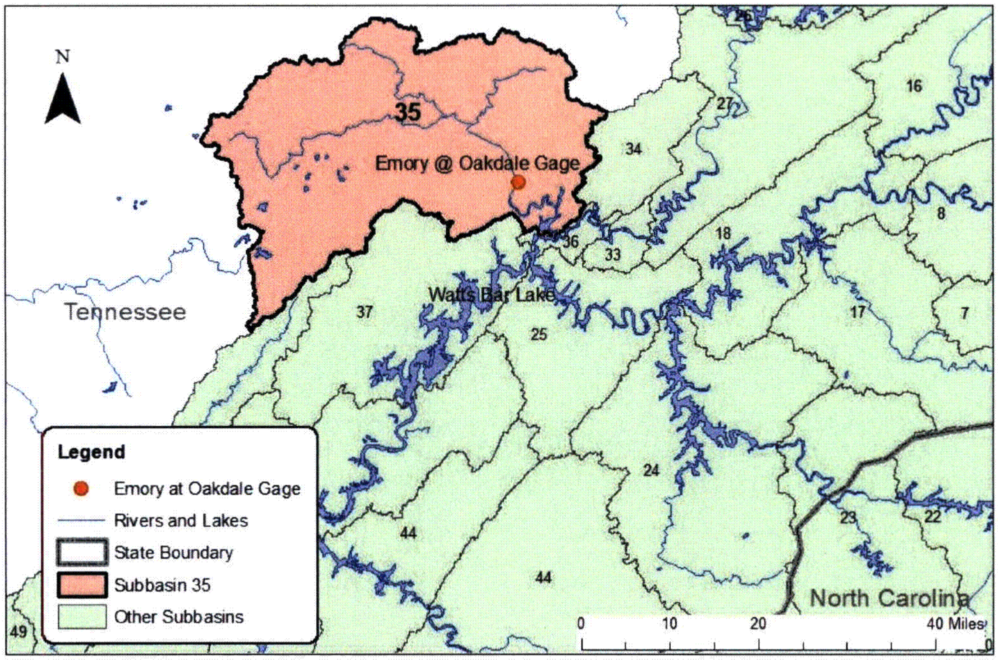

Figure 1: Emory River at Mouth, Subbasin 35, showing streamflow gage and nearby subbasins

2 References

Reference 1: Tennessee Valley Authority, Bellefonte Nuclear Plant - White Paper, Hydrologic Analysis, Revision 1, July 25, 2008, (EDMS No. L58 081219 800). FOR INFORMATION ONLY

Reference 2: Viessman, W., J.W. Knapp, G.L. Lewis, and T.E. Harbaugh, Introduction to Hydrology, Second Edition, Harper & Row Publishers, 1977.

Reference 3: Chow, V.T., D.R. Maidment, and L.W. Mays, *Applied Hydrology*, McGraw-Hill Book Company, 1988.

Reference 4: Watts Bar Reservoir. Available online from the Tennessee Valley Authority (TVA) at http://www.tva.com/sites/wattsbarres.htm; Accessed 5 December 2008.

Reference 5: American Nuclear Society, American National Standard for Determining Design Basis Flooding at Power Reactor Sites, ANSI/ANS-2.8-1992, 1992.

Reference 6: U.S. Nuclear Regulatory Commission, Standard Review Plan 2.4.3, Probable Maximum Flood (PMF) on Streams and Rivers, NUREG-0800, Revision 4, March 2007.

Reference 7: Tennessee Valley Authority, *UNITGRPH-FLDHYDRO-TRBROUTE-CHANROUT User's Manual,* Version 1.0, November 2008 (EDMS No. L58 090325 001).

Reference 8: U.S. Army Corps of Engineers, *Hydrologic Modeling System HEC-HMS User's Manual*, Version 3.2, April 2008.

Reference 9: U.S. Army Corps of Engineers, *Hydrologic Modeling System HEC-HMS Technical Reference Manual*, March 2000.

Reference 10: Tennessee Valley Authority, Unit Area 35, Emory River at Mouth, File Book Reference. (EDMS No. L58 090 123 802)

Reference 11: Emory River at Oakdale gage data. Available from the U.S. Geological Survey (USGS) at http://waterdata.usgs.gov/nwis/inventory/?site_no=03540500& Accessed 9 October 2008.

Reference 12: Tennessee Valley Authority, Calculation No. CDQ000020080055, Processing and Validation of National Weather Service's NEXRAD Stage III Hourly Precipitation Data for Hydrologic Analysis of TVA Subbasins, Revision 3

Reference 13: Newton, D.R., and J.W. Vinyard, Computer-Determined Unit Hydrograph From Floods, *Journal of the Hydraulics Division*, ASCE, Vol. 93, No. HY5, September, 1967.

Reference 14: Singh, K.P. and D.R. Dawdy, Computer-Determined Unit Hydrograph From Floods, *Journal of Hydraulics Division*, ASCE, Vol. 93, No. HY6, November 1968.

Reference 15: Singh, V. P. *Elementary Hydrology*, Prentice-Hall, 1992.

Reference 16: Weaver, J.D. and Gamble, C.R., Flood Frequency of Streams in Rural Basins of Tennessee, *U.S. Geological Survey Water-Resources Investigations Report 92-4165*, 1993.

Reference 17: Stedinger, J.R., Vogel, R.M., and Foufoula-Georgiou, E. "Chapter 18: Frequency Analysis of Extreme Events" in *Handbook of Hydrology*, D.R. Maidment ed. McGraw-Hill, 1993.

Reference 18: Bechtel, Request for Information RFI 25447-000-GRI-GEX-00062, January 5, 2009. (EDMS No. L58 090113 800)

Reference 19: Linsley, R.K., J.B. Franzini, D.L. Freyberg, and G. Tchobanogolous. *Water Resources Engineering*, Fourth Edition, Irwin McGraw-Hill, 1992.

Reference 20: Linsley, R.K., Kohler, M.A., and Paulhus, J.H., *Hydrology for Engineers*, McGraw-Hill Book Company, 1982.

Reference 21: Kohler, M.A., and R.K. Linsley, Predicting the Runoff from Storm Rainfall, Research Paper No. 34, U.S. Department of Commerce, September 1951. (EDMS No. L58 080910 001).

3 Assumptions

3.1 *General Assumptions*

None

3.2 *Unverified Assumptions*

None

4 Background

4.1 *Unit Hydrograph Theory*

The unit hydrograph (UH) is used to predict the runoff response at the outlet of a watershed, or subbasin, to the input of one unit of excess rainfall applied uniformly over a given duration of time. Runoff from other depths of excess rainfall can be obtained by scaling (Reference 2 and Reference 3).

The unit hydrograph is used to obtain the streamflow hydrograph resulting from a series of excess rainfall inputs of any depth using the process of "convolution." The discrete convolution equation, states that the streamflow, Q, is obtained by summing the products of the excess rainfall depths (direct runoff depths), P, and the unit hydrograph ordinates, U (Reference 2 and Reference 3). The reverse process, called deconvolution, is used to derive the ordinates of the unit hydrograph by reconstituting floods from precipitation and streamflow data. The unit hydrograph is derived from the unit duration of uniform excess precipitation applied evenly across the watershed.

Unit hydrograph theory is applicable under the following conditions (Reference 3):

1. Excess rainfall has a constant intensity within the effective duration.
2. Excess rainfall is uniformly distributed over the entire subbasin.
3. The duration of direct runoff resulting from a unit of excess rainfall is constant.
4. The ordinates of the unit hydrograph are directly proportional to the total amount of direct runoff (linear response).
5. The surface runoff hydrograph reflects all the unique physical characteristics and runoff processes in the drainage basin in a given "epoch."

4.2 *Subbasin Location and Layout*

Subbasin 35 is located to the north of Watts Bar Lake. The drainage area of the subbasin was calculated in GIS as 868.8 mi^2. The subbasin is a headwater watershed, and no runoff enters this watershed from areas outside of, or upstream of, the basin. The Emory River flows from the northwest to the southeast corners of the subbasin where it joins the Clinch River. The intersection of Emory River with the subbasin boundary in the southeast denotes the subbasin outlet; see Figure 1. A gage, Emory River at Oakdale, Tennessee, is located within the subbasin boundary at river mile 18.3. The distance along the Emory River from the subbasin outlet to the gage location was measured in GIS as approximately 16 miles. Watts Bar Dam was constructed between 1939 and 1942; the dam creates a slack-water arm that extends 12 miles up the Emory River from its historical confluence with the Clinch River (Reference 4).

5 Methodology

The methodology used for unit hydrograph validation follows that described in ANSI/ANS-2.8-1992 (Reference 5). This document is included as a reference in the Nuclear Regulatory Commission's (NRC's) Standard Review Plan for Section 2.4.3, Probable Maximum Flood on Streams and Rivers (Reference 6). With regard to verifying runoff models, ANSI/ANS-2.8-1992 indicates the following:

> "Deterministic simulation models including unit hydrographs should be verified or calibrated by comparing results of the simulation with the highest two or more floods for which suitable precipitation data are available."

For the purpose of validating the UH for Subbasin 35, the period of record from which the highest two or more floods are selected extends from 1997 through 2007. This period was targeted because of the availability of high resolution, radar-based, hourly precipitation, as described in Section 5.3. Furthermore, since the original UH for Subbasin 35 was developed from floods that occurred between 1939 and 1963 (Section 5.1), it was necessary to use recent rainfall and streamflow data to evaluate the possibility that changes in watershed characteristics over the intervening years might have altered the rainfall-runoff response of the watershed to such an extent as to invalidate the original UH.

In general, the methodology used for UH validation includes the following steps:

1. Screen historical streamflow data from 1997-2007 to identify the two highest floods. These floods are used for unit hydrograph validation.
2. Obtain the observed hydrograph data for the two floods and transfer the flow series to the subbasin outlet using established hydrologic procedures, as necessary, to develop the local basin hydrograph.
3. Separate baseflow from the local basin hydrograph to obtain the "observed" direct runoff hydrograph for the basin, and calculate the volume of the direct runoff based on the hydrograph ordinates.
4. Obtain observed rainfall data for the selected floods and calculate the basin average precipitation for the adopted time step.
5. Convert the observed rainfall series to an effective rainfall series using the TVA's API-RI method as implemented in FLDHYDRO (Reference 7). This includes inputting the observed runoff volume obtained in Step 3 to ensure that the effective rainfall volume calculated by FLDHYDRO equals the observed runoff volume.

6. Run HEC-HMS (Reference 8 and Reference 9) utilizing the TVA unit hydrograph and the effective rainfall series as input and compare the resulting simulated hydrograph with the observed direct runoff hydrograph in terms of total volume, and the timing and magnitude of peak discharge.

6 Design Input Data

The input data necessary for validating the UH for the Emory River at Mouth unit area, Subbasin 35, are summarized below.

- Unit hydrograph ordinates and duration
- Observed streamflow from the gage on the Emory River at Oakdale, TN.
- Observed rainfall data associated with the selected floods

Each of these inputs is described in more detail in the following subsections.

6.1 *Original Unit Hydrograph Ordinates*

The UH for Subbasin 35 is described in the corresponding TVA File Book Reference (Reference 10). The data used (by the TVA) to develop the subbasin UH include streamflow records at the Oakdale gage (Section 5.2) from the following historical floods:

- February 3, 1939 – peak discharge 97,390 cfs after removal of baseflow
- February 13, 1948 – peak discharge 99,250 cfs after removal of baseflow
- May 23, 1957 – peak discharge 5,740 cfs after removal of baseflow
- November 19, 1957 – peak discharge 74,876 cfs after removal of baseflow
- February 28, 1962 – peak discharge 48,524 cfs after removal of baseflow
- March 12, 1963 – peak discharge 86,200 cfs after removal of baseflow

The drainage area of the Emory River at Mouth subbasin is given in the TVA File Book Reference as 865 mi^2 (Reference 10) and was calculated in GIS as 868.8 mi^2. According to Reference 10, the February 1939, November 1957, and March 1963 floods were caused by rainstorms centered downstream. The February 1948 and February 1963 floods occurred from rainstorms centered upstream. “Upstream” and “downstream” are not defined in Reference 10.

For each of the six floods listed above, a UH was developed for the gage location (note that the gage location is upstream of the subbasin outlet) using the TVA’s UNITGRPH program (Reference 7). Then, two composite UHs were derived using the UNITGRPH program: 1) from the three floods with storms centered downstream and 2) from the two floods with storms centered upstream. The with storms centered downstream and 2) from the two floods with storms centered upstream. The three-flood, "downstream" composite was adopted. The adopted hydrograph at the Oakdale gage had a four-hour period; this hydrograph was converted (by the TVA) to a six-hour unit hydrograph using the S-graph method (Section 7.1.3).

The Subbasin 35 UH was then derived from the adopted hydrograph at the Oakdale, TN gage on the Emory River by multiplying the peak discharge from the gage hydrograph by 1.064 (the square root of the ratio of the two watershed areas). The remaining ordinates, for the Subbasin 35 UH, were obtained by adjusting the gage UH ordinates to obtain a unit volume. The drainage area in the watershed above the gage is 764 mi^2 (Reference 11). No rationale is provided in Reference 10 for the area scaling relationship applied to the peak discharge.

6.2 *Streamflow Data from the Oakdale Gage*

Streamflow data have been collected at U.S. Geological Survey (USGS) gage 03540500 on the Emory River near Oakdale, TN since 1927 (Reference 11). Data from this gage were used to develop the UH for Subbasin 35 (Section 6.1). The TVA provided bihourly discharge measurements from this gage in spreadsheet format for 1985 through 2007; these data, as provided by the TVA, are enclosed as Attachment 1- 1. The drainage area above this gage is 764 mi^2 (Reference 11). Discharge data from this gage are used as streamflow data for analysis of the Subbasin 35 unit hydrograph. These data were adjusted to represent discharge at the subbasin outlet as discussed in Section 7.2.

6.3 *Observed Rainfall*

Radar-based, geospatially referenced precipitation data is extremely useful for hydrologic analysis because of its comprehensive spatial and temporal detail. Gridded daily precipitation data are available at http://water.weather.gov/ for 2005 to present. Hourly precipitation data are not generally available without special arrangements with the United States National Weather Service (NWS).

NWS NEXRAD Stage III hourly precipitation data were obtained from the Lower Mississippi River Forecast Center (LMRFC) from January 1997 to April 2008 for unit hydrograph validation. A Microsoft.Net utility was developed to generate radar-based Mean Areal Precipitation (MAPX) time series for each of the subbasins (Reference 12). The utility reads the raw hourly precipitation depth data for each 4-km square grid cell, performs necessary coordinate system and projection calculations, and then calculates the average precipitation depth within each subbasin, grouping output into a matrix of MAPX elements arrayed by subbasin and time (Greenwich Mean Time, GMT). Each column of this matrix is equivalent to an annual hyetograph for each subbasin in the TVA model. The results are stored in an Excel spreadsheet for each year of record. Reference 12 describes the methodology used to process the precipitation data and includes resulting subbasin averaged hourly values for the January 1997 to April 2008 period of record.

7 Computations and Analyses

7.1 *Regenerated Unit Hydrograph*

7.1.1 UNITGRPH Program

The TVA developed the UH for Subbasin 35 in 1966 using the computer program UNITGRPH (Reference 7). This program employs the methodology proposed by Newton and Vinyard (Reference 13) for estimating a UH from complex floods using matrix algebra and statistical curve fitting techniques. In the method, the UH ordinates are determined from estimates of observed direct runoff and excess precipitation. The method determines the best fit unit hydrograph from a single or a series of floods. The Newton and Vinyard method (Reference 13) also provides a means to adjust, if necessary, runoff, or excess precipitation, based on the excess precipitation required to generate the observed direct runoff. Implicit in the adjustment is the requirement that the estimated time series of direct runoff (e.g. streamflow with baseflow removed) is more accurate than the estimated time series of excess precipitation.

To develop a UH using the methods of Newton and Vinyard contained within the UNITGRPH program, the flood or floods of interest are identified. Baseflow is removed from the flood(s) to obtain observed direct runoff. Excess precipitation is estimated from observed rainfall for each flood. Direct runoff and excess precipitation are then determined for time intervals that match the desired UH period. These values are provided to the program along with the "list" of ordinates to be computed directly. The remaining ordinates are linearly interpolated from the "listed" ordinates. Suggestions for deriving the list values are provided in Reference 14.

The UNITGRPH program first estimates UH ordinates using matrix inversion. The first iteration UH is then employed to estimate "adjusted" runoff which is simply an estimate of the excess precipitation that would provide a better match to the observed direct runoff when convolved with the first iteration UH. In the second iteration, the program computes a new UH using the adjusted excess precipitation and the observed direct runoff. The updated UH is used to estimate a new series of adjusted runoff, and the process is repeated for the specified number of iterations or until a specified average error criterion is met. Newton and Vinyard (Reference 13) suggest that five iterations or an average error of five percent be adopted as limits.

7.1.2 Regenerated Unit Hydrograph Ordinates

The TVA's UNITGRPH program was revised to correct a code error in 2008. Consequently, a UH for the Emory River at Oakdale gage location was regenerated from estimated February 1939, November 1957, and March 1963 direct runoff and excess precipitation provided in Reference 10 employing the revised UNITGRPH program (Reference 7). List values were obtained from Reference 10.

The Subbasin 35 UH is a composite (i.e. generated from multiple floods). The composite UH was derived in 1966 by running the UNITGRPH program for each of the three floods and then using the adjusted excess precipitation from each single flood run (along with the observed direct runoff) in a three flood, or composite, UNITGRPH run. The regenerated UH was derived using estimated excess precipitation for the floods (i.e., from FLDHYDRO). UNITGRPH input and output files for each of the single flood runs and for the composite run are enclosed as Attachment 2- 1 through Attachment 2- 3. As mentioned, the subbasin boundaries do not coincide with the gage location. The UH for the gage location was adjusted to the subbasin outlet as described in Section 5.1, and enclosed as Attachment 1- 2, to obtain the regenerated UH for Subbasin 35.

The regenerated UH for Subbasin 35 is plotted in Figure 2 as "Regenerated 4-hr UH." The time base and ordinates are listed in Table 1 along with a volume check demonstrating that volume of runoff is equivalent to one inch of excess rainfall over the entire basin. One-hour and two-hour period UHs were derived from the four-hour UH using the S-graph method (Section 6.1.3) to facilitate convolution with one-hour and two-hour (Section 6.5) excess precipitation values derived from rainfall data recorded at one- and two-hour intervals. The HEC-HMS software was used to calculate the one- and two-hour UHs with the S-graph method. One- and two-hour UHs were also calculated in a spreadsheet, Attachment 1- 3, as a check on the HEC-HMS calculations. The derived one-hour UHs are plotted in Figure 2 along with the regenerated four-hour UH. Figure 3 provides an equivalent plot for the two-hour UHs. The HEC-HMS software applies some form of smoothing/shaping as part of the S-graph transformation. Consequently, the two one-hour UHs and two-hour UHs are slightly different.

A six-hour period UH was also derived from the four-hour UH using the S-graph method; these calculations are enclosed as Attachment 1- 3. This six-hour UH is plotted in Figure 4 as "Regenerated 6-hr UH." The time base and ordinates for the regenerated six-hour UH are listed in Table 1. Plots of output for HEC-HMS simulations employing the regenerated composite UH and the excess precipitation estimated by the TVA for the February 1939, November 1957, and March 1963 floods are contained in the Appendix, Section 8.

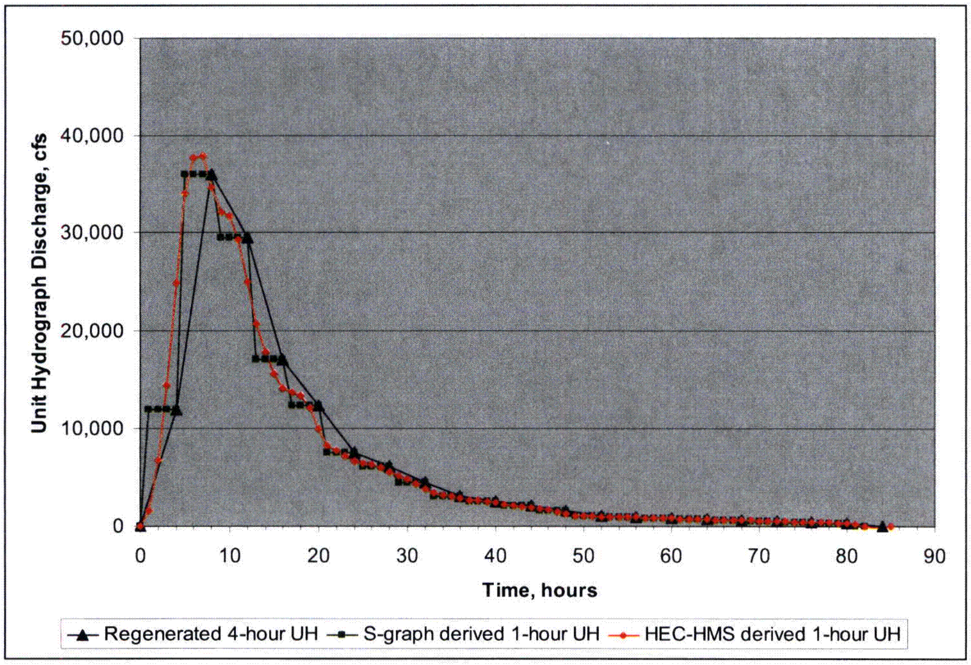

Figure 2: Regenerated four-hour unit hydrograph (UH) and one-hour UHs derived using the S-graph method for Subbasin 35 (Emory River at Mouth)

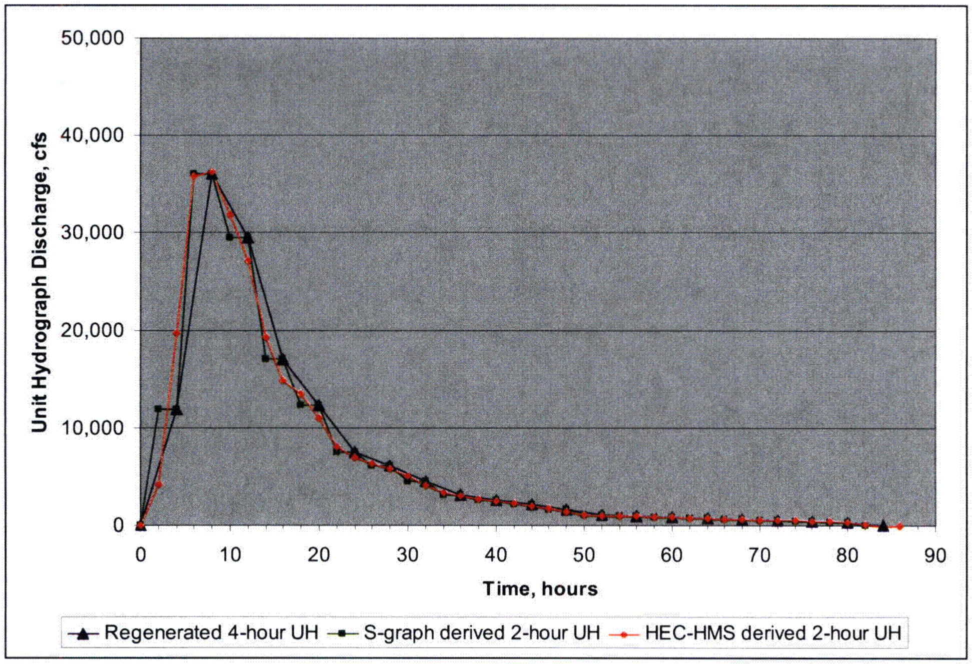

Figure 3: Regenerated four-hour unit hydrograph (UH) and two-hour UHs derived using the S-graph method for Subbasin 35 (Emory River at Mouth)

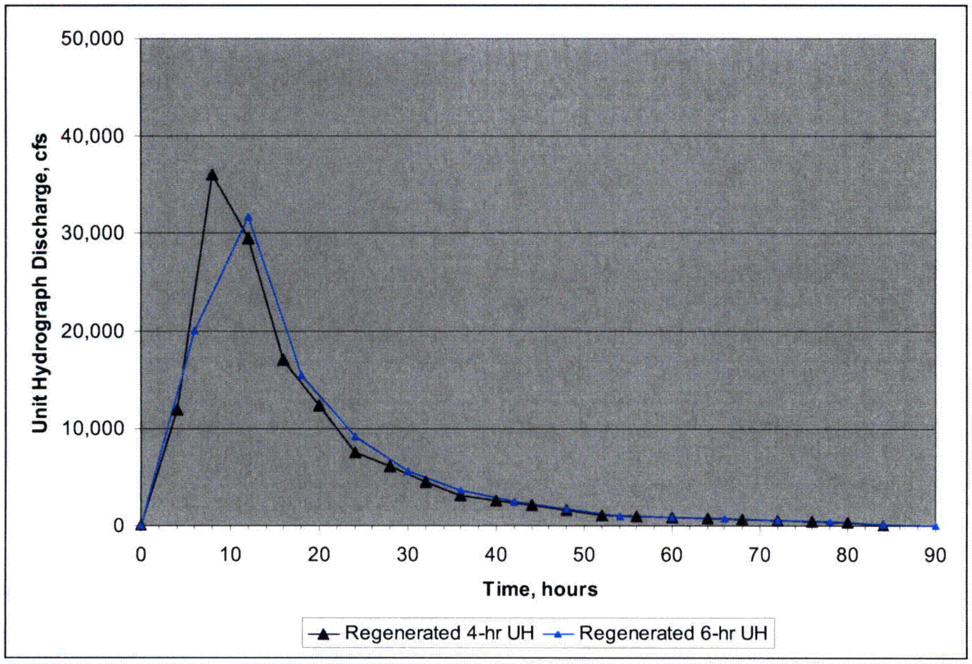

Figure 4: Regenerated four-hour unit hydrograph (UH) and six-hour UH derived using the S-graph method for Subbasin 35 (Emory River at Mouth)

Table 1: Time base and ordinates for regenerated four-hour UH and transformed six-hour UH

4-hr UH		6-hr UH	
Hour	Discharge, cfs	Hour	Discharge, cfs
0	0	0	0
4	11,950	6	19,997
8	36,090	12	31,697
12	29,500	18	15,500
16	17,100	24	9,100
20	12,300	30	5,583
24	7,500	36	3,617
28	6,100	42	2,433
32	4,550	48	1,747
36	3,150	54	1,017
40	2,600	60	883
44	2,100	66	717
48	1,570	72	590
52	1,050	78	437
56	950	84	123
60	850	90	0
64	750		
68	650		
72	560		
76	470		
80	370		
84	0		

	4-hr UH	6-hr UH	
Total Volume (1)	46,333.8	46,333.8	acre-ft
Basin Area	868.8	868.8	mi^2
Runoff Depth (2)	0.99995	0.99995	in

Notes:

1) $$Volume = \sum Q \frac{ft^3}{\sec} \times 3600 \frac{\sec}{hr} \times Period\ in\ hrs \times \frac{1acre - ft}{43560 ft^3}$$

2) $$Depth = \frac{Volume.acft}{Area.mi^2} \frac{mi^2}{640.acre} \frac{12.inch}{ft}$$

7.1.3 S-Graph Method

A UH is derived for a specific effective duration. Often, the UH is applied to rainfall data that may be better represented with a different effective duration than that used to derive the UH. A UH for any effective duration can be derived from an existing UH using the summation hydrograph, or S-graph, method (Reference 15).

In this method, a summation hydrograph is constructed from a series of unit hydrographs (all of the same effective duration) using the principle of superposition. This involves successively displacing the original UH by the effective duration and summing the ordinates of the original and displaced graphs. The S-graph represents the runoff that would result from a continuous, constant excess rainfall rate per specified period that produces a unit depth runoff volume. The UH with the desired effective duration is derived from the S-graph by offsetting the S-graph an amount equal to the desired effective duration and subtracting the offset S-graph from the original S-graph. The pertinent calculations following the methods in Reference 15 for this subbasin are provided in Attachment 1- 3.

Derivation of a short-period hydrograph from one of longer duration does not work as well as derivation of long-period hydrograph from one of shorter duration (Reference 15). The S-graph process involves averaging of ordinates; consequently, small errors in the ordinates of a shorter duration hydrograph are smoothed as part of the calculation. However, small errors in a longer duration unit hydrograph may lead to larger errors in the derived, shorter-period UH (Reference 15). Also, errors in the original UH may result in oscillations in the S-graph (Reference 15). These errors come about if the original UH is not the "true" UH in the sense that the watershed response may be nonlinear (Reference 15).

The derivation of a one-hour period UH from the four-hour UH, as discussed above for Subbasin 35, involves derivation of a short-period UH from one of longer duration. The rainfall data used here suggests that constant intensity rainfall and thus constant intensity excess precipitation can be more closely approximated by using periods shorter than the effective duration of the TVA UH. Consequently, a one-hour period UH was derived for use with one-hour precipitation data in order to minimize potential errors associated with the constant rainfall intensity condition underlying the UH method (Section 3.3).

7.2 *"Observed" Subbasin Discharge Calculation Methods*

The available Emory River streamflow data for Subbasin 35 are collected at Oakdale, TN. The outlet for the subbasin is located approximately 16 miles downstream from this gage. Observed streamflow at the basin outlet location is needed for comparison with that estimated with the unit hydrograph for the subbasin. Discharge at the subbasin outlet was estimated using a method for calculating peak discharges for 2-, 5-, 10-, 25-, 50-, 100-, and 500-year recurrence intervals at ungaged sites in rural Tennessee (Reference 16). The Emory River at Oakdale gage was among those used in developing the methods provided by this reference.

Reference 16 provides a methodology for estimating peak discharges of various recurrence intervals at an ungaged site from a relatively near gage site (on the same stream) when the ungaged site drainage area is within 50 percent of the drainage area of the gaged site. This methodology includes a means to estimate the discharge magnitude, transferred downstream to the ungaged site, from the known discharge at the gaged site as shown in Equation (1):

$$Q_w^{'} = \left(\frac{A_u}{A_g} \right)^b Q_w \qquad (1)$$

where Q'_w is the discharge for the ungaged site; A_u is the watershed area of the ungaged site; A_g is the watershed area of the gaged site; b is the regression coefficient of drainage area of the gaged site; and, Q_w is the discharge for the gaged site. In Reference 16, the Q'_w value for the selected recurrence interval is then further adjusted based on the regression analysis underlying the method.

For this calculation, the bihourly gage measurements need to be transferred downstream to the subbasin outlet. A scaling factor was used to estimate discharge at the subbasin outlet from each bihourly measured discharge value at the Oakdale gage. The selected scaling factor comes from the area ratio raised to the regression coefficient, as shown in Equation (1). Table 2 provides the calculated scaling factor for each recurrence interval. When the calculated scaling factors are rounded to two significant digits, a single scaling factor of 1.1 is obtained.

Table 2: Scaling Factors for each recurrence interval calculated from Equation (1)

Recurrence Interval of Discharge (years)	Area Ratio (A_u/A_g)	Regression Coefficient b	Scaling Factor $(A_u/A_g)^b$	Scaling Factor rounded to 2 significant digits
2	1.137	0.753	1.102	1.1
5	1.137	0.736	1.099	1.1
10	1.137	0.727	1.098	1.1
25	1.137	0.717	1.097	1.1
50	1.137	0.711	1.096	1.1
100	1.137	0.703	1.095	1.1
500	1.137	0.694	1.093	1.1

To estimate Emory River discharge at the subbasin outlet, the measured discharge values at the Oakdale gage were multiplied by the scaling factor of 1.1. Complete calculations are enclosed as Attachment 1- 4 and Attachment 1- 5. This calculation simply increases discharge by a weighted ratio of the watershed area of the ungaged outlet to the watershed area of the gage location. This calculation method does not account for possible changes in channel storage from the additional reach of the Emory River or changes in unit hydrograph timing and shape as a result of increased drainage area.

7.3 *Floods for Unit Hydrograph Validation*

Two recent storms/floods were selected for the validation process for the period from 1997 to 2007. This period is of interest because of the availability of hourly precipitation data from the U.S. National Weather Service (NWS) Lower Mississippi River Forecast Center (LMRFC). Streamflow data from the Oakdale gage were provided by the TVA for 1985 through 2007; these data are enclosed as Attachment 1- 1. Consequently, the interval 1997 to 2007 provides the period for identifying two recent floods.

For the Emory River at Mouth watershed, it was necessary to develop streamflow time series at the watershed outlet by transferring the Oakdale gage time series downstream to the outlet (Section 6.2). The two largest floods within the period of interest were identified from Oakdale gage annual peak discharge data (Reference 11).

Table 3 provides the peak discharge for the Oakdale gage for each water year from 1997-2007. These discharges and the corresponding dates were obtained from Reference 11. In Table 3 the Weibull Plotting Position is used as an estimate of the exceedance probability for each annual peak discharge (Reference 17). It provides the exceedance probability of the *i*th-largest flood from the total number, *n*, of measured floods as shown in Equation (2). The probability plotting position, q_i, is based on the 81 years of annual peak discharge data obtained from the USGS. Complete calculations are enclosed in Attachment 1- 6 and Attachment 1- 7.

$$q_i = \frac{i}{n+1} \tag{2}$$

The dates obtained from the USGS and presented in Table 3 agree with the corresponding peak discharge dates provided by the TVA. The peak discharge values obtained from the two sources are within 3%; the small discrepancy is attributed to different data durations (i.e. USGS data are instantaneous annual peaks, and the TVA data are two-hour discharge values).

Reference 16 provides estimates of peak discharge values for the Oakdale gage for recurrence intervals between two and 500 years. Regional regression equations are used in Reference 16 to estimate peak discharges, and this reference provides a 100-yr peak discharge magnitude of 166,000 cfs, a 50-yr peak of 143,000, a 10-yr peak of 93,800 cfs, and a 5-yr peak discharge magnitude of 73,500 cfs for the Oakdale gage. Average recurrence intervals provided in Table 3 were calculated from the probability plotting position, Equation (2). Probability plotting position analysis only accounts for difference in rank and ordered position. It does not account for relative discharge magnitude. As a result, the second highest discharge will have an estimated average recurrence interval from Equation 2 of about 40.5 years and the highest will have an estimated average recurrence interval of 81 years from Equation 2 even if the highest and second highest discharges are only separated by 1 cfs.

Given the accuracy limitations of the average recurrence intervals estimated from Equation 2, the recurrence interval and peak discharge combinations provided by Reference 16 should be considered more accurate than the average recurrence intervals provided in Table 3. However, the recurrence intervals for floods in Table 3 approximately agree with the recurrence intervals provided in Reference 16 for the Oakdale gage for average recurrence intervals less than ten years (e.g. the January 2002 flood has an average recurrence interval of 5 years and peak discharge of 75,000 cfs and the February 1939 flood has an average recurrence interval of 10 years and a peak discharge of 100,000 cfs). The recurrence intervals estimated with Equation (2) become less accurate than those in Reference 16 as the rank of the corresponding discharge approaches one. Reasonable average recurrence intervals are provided in Table 3 for the floods during the period from 1997 to 2007 because these floods have relatively high ranks (i.e. closer to 81 than to one) for the period from 1927 to 2007.

In Table 3, the floods in 1990 and 1973 are significantly larger than those occurring during 1997-2007. These two floods were not used in the derivation of the UH (Section 5.1). Although these floods occurred outside of the 1997-2007 analysis interval, they are included in the UH validation process because of their relatively large peak discharges. The recurrence interval for these floods based on plotting position in Table 3 is between 27 and 41 years; however, the 170,000 cfs peak discharge magnitude for these floods exceeds the 166,000 cfs peak discharge magnitude identified for the 100-yr flood at the Oakdale gage in Reference 16. Because the 1990 and 1973 floods are ranked two and three, the approximately 100-year recurrence interval generated from comparison with the values in Reference 16 provides the better estimate of recurrence interval for these two floods. Data for December 1990 and May 1973 floods were obtained from the TVA (Reference 18).

Two floods were also selected from 1997-2007 for use in unit hydrograph validation. Rainfall data are missing from the NWS gridded precipitation data set for January 6, 2002 through January 31, 2002 and for January 6, 1998 through January 31, 1998. These periods of missing data encompass the second and third largest floods during 1997-2007 as shown in Table 3. As a result, the following two floods were selected for unit hydrograph validation from 1997-2007:

- February 14, 2003, 00:00 hrs to February 21, 2003, 00:00 hrs, the "February 2003" flood
- September 16, 2004, 00:00 hrs to September 22, 2004, 00:00 hrs, the "September 2004" flood

Table 3: Rank of annual peak discharges from 1927-2007 and for floods used in UH validation and derivation

1997 to 2007							
Rank from 1997 - 2007*	Rank from 1927 - 2007	Date	Water Year	Peak discharge (cfs)	Stage (ft)	Weibull Plotting Position+	Average Recurrence Interval Based on Plotting Position (yrs)
1	12	9/17/2004	2004	83,800	30.03	0.148	6.8
2	18	1/23/2002	2002	75,200	28.85	0.222	4.5
3	23	1/7/1998	1998	65,900	27.15	0.284	3.5
4	29	2/16/2003	2003	62,800	26.55	0.358	2.8
5	35	12/1/2004	2005	53,700	24.88	0.432	2.3
6	42	4/8/2006	2006	49,600	24.11	0.519	1.9
7	48	1/23/1999	1999	42,200	22.63	0.593	1.7
8	50	4/4/2000	2000	41,600	22.51	0.617	1.6
9	53	2/17/2001	2001	38,300	21.92	0.654	1.5
10	75	5/5/2007	2007	22,500	17.64	0.926	1.1
Three Floods Used to Derive TVA Unit Hydrograph							
Rank from 1997 - 2007*	Rank from 1927 - 2007	Date	Water Year	Peak discharge (cfs)	Stage (ft)	Weibull Plotting Position+	Average Recurrence Interval Based on Plotting Position (yrs)
N/A	8	2/3/1939	1939	100,000	N/A	0.099	10.1
N/A	9	3/12/1963	1963	89,400	N/A	0.111	9.0
N/A	17	11/19/1957	1958	76,700	N/A	0.210	4.8
Largest Floods with Data Available from the TVA							
Rank from 1997 - 2007*	Rank from 1927 - 2007	Date	Water Year	Peak discharge (cfs)	Stage (ft)	Weibull Plotting Position+	Average Recurrence Interval Based on Plotting Position (yrs)
N/A	2	12/23/1990	1990	170,000	38.71	0.025	40.5
N/A	3	5/28/1973	1973	170,000	38.68	0.037	27.0

* Floods in the USGS dataset are provided by water year. The largest flood in water year 1996-97 occurred at the end of 1996. Consequently, only 10 floods are provided in this table.

+ Plotting position based on 81 (1927 - 2007) years of USGS annual peak discharge data.

Plots of discharge at the Oakdale gage and that calculated for the subbasin outlet (i.e. Emory River at Mouth) are shown in Figure 5 and Figure 6 along with NWS basin average precipitation data for the February 2003 and September 2004 floods. Figure 7 and Figure 8 display plots of discharge at the Oakdale gage, and that calculated for the Emory River at Mouth, for the December 1990 and May 1973 floods along with the average rainfall among TVA rain gages in the vicinity of the subbasin. NWS basin average precipitation data are not available for the December 1990 and May 1973 floods.

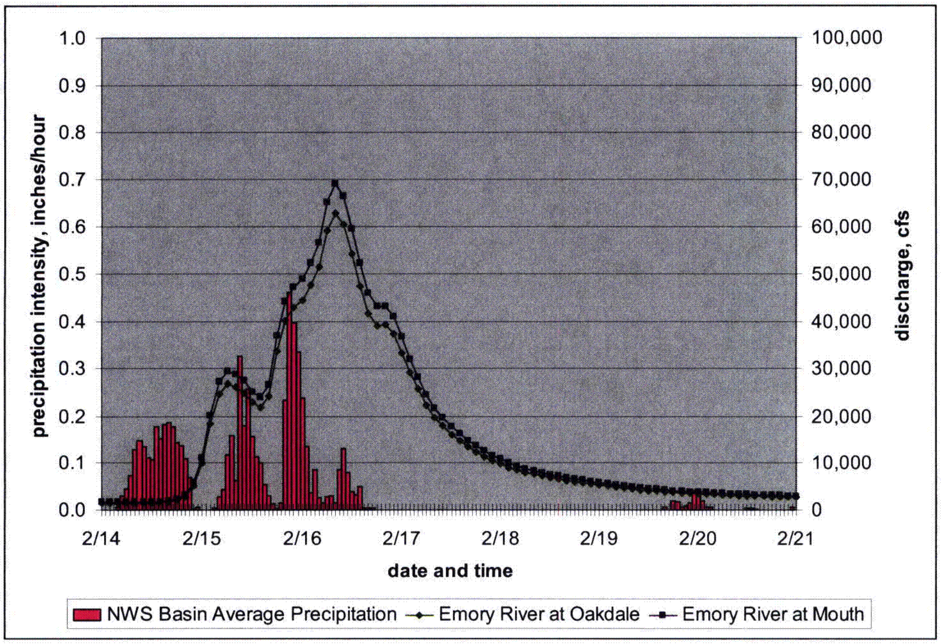

Figure 5: Emory River at Mouth "observed" hydrograph and precipitation for February 2003 flood

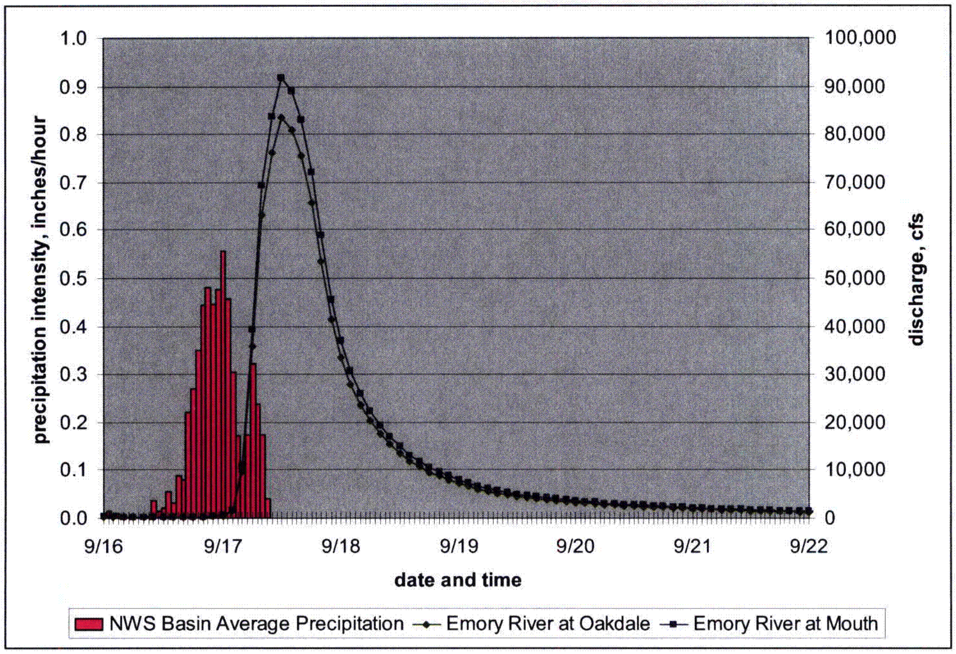

Figure 6: Emory River at Mouth "observed" hydrograph and precipitation for September 2004 flood

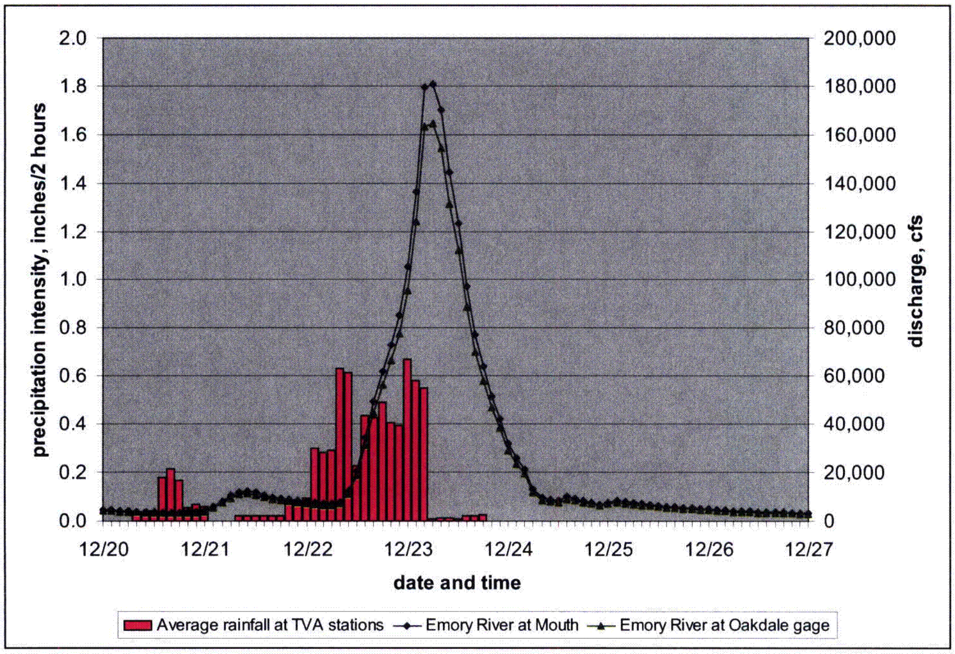

Figure 7: Emory River at Mouth "observed" hydrograph and precipitation for December 1990 flood

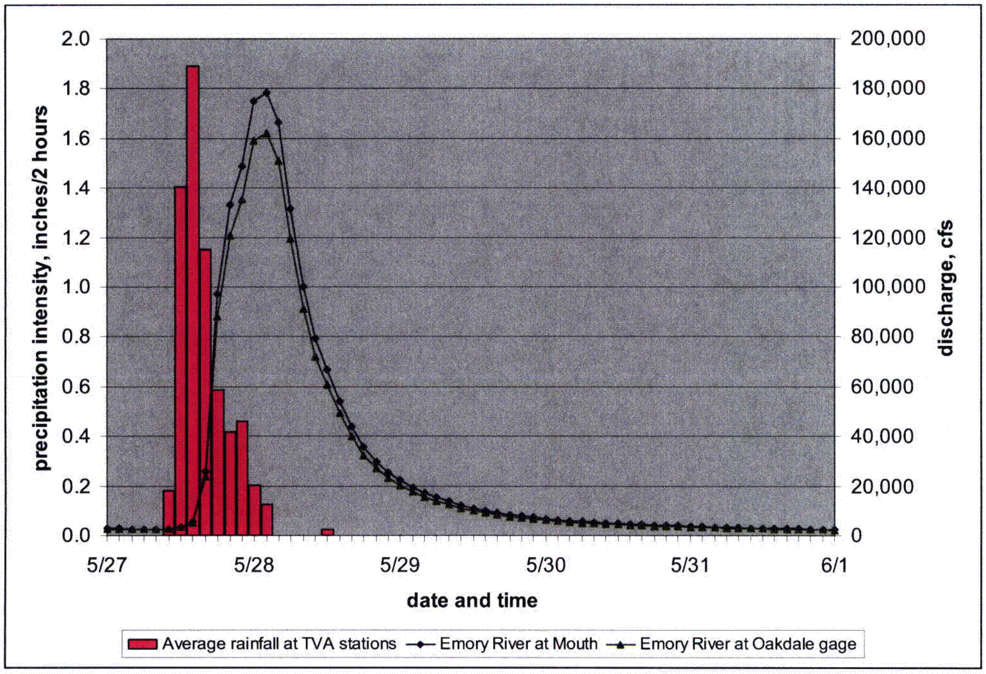

Figure 8: Emory River at Mouth "observed" hydrograph and precipitation for May 1973 flood

7.4 *Baseflow Separation*

Baseflow separation is required to provide an estimate of direct runoff associated with the rain storm. For this calculation, the three-point (ABC) method was employed, as illustrated in Figure 9 (Reference 19, page 45). The flow recession existing prior to the storm was extended from the starting point of runoff (point A) to a point immediately beneath the peak (point B). The starting point, point A, was selected via visual examination of the calculated hydrograph. Recession, in this calculation, was estimated by fitting a line to the observed hydrograph across one to three days prior to the flood; calculated hydrograph points were omitted from the line fitting process as necessary to obtain a trend line with a negative slope (i.e. recession) and to provide the best "visual" fit. Point B was then connected to the point on the receding limb of the hydrograph when storm runoff ends, point C (Reference 19). The approximate location of the point on the hydrograph when storm runoff ends (point C) was estimated using Equation (3) (Reference 19; Reference 20), where N is the length between point B and C in days, and A is the basin area in square miles.

$$N = A^{0.2} \tag{3}$$

The observed hydrograph for the 2003 flood has one smaller peak followed by the main, larger peak. The main peak for this flood was isolated by removing the smaller peak using a recession curve for the initial peak. The direct runoff volume for the 2003 flood was obtained by removing baseflow and the initial, smaller peak. Direct runoff volume for the 1973, 1990, and 2004 floods was estimated by removing baseflow from the observed flood hydrograph. These direct runoff volumes are used in adjusting the excess rainfall volumes, as noted in Section 4. Direct runoff volume, V, is calculated from period average flow rate, Q_i, where there are a number of periods, P, with a period duration of Δt as:

$$V(ac-ft) = \sum_{i=1}^{P} Qi(cfs)\ x \left(\Delta t(hr) * \frac{3{,}600(s/hr)}{43{,}560(ft^2/ac)} \right) \tag{4}$$

Table 4 provides a summary of the direct runoff obtained from baseflow separation for each flood. Local hydrographs for each flood along with estimated baseflow and direct runoff are provided in Figure 10, Figure 11, Figure 12, and Figure 13. Baseflow separation calculations are enclosed as Attachment 1- 8.

Table 4: Direct runoff (RO) volume obtained from baseflow separation for each flood

Subbasin	Drainage Area (sq. mi.)	Flood	Total Runoff Volume (acre-ft)	Runoff Depth (in)
Emory River at Mouth, Sub-basin 35	868.8	February 2003	173,667	3.75
		September 2004	153,449	3.31
		May 1973	269,716	5.82
		December 1990	327,051	7.06

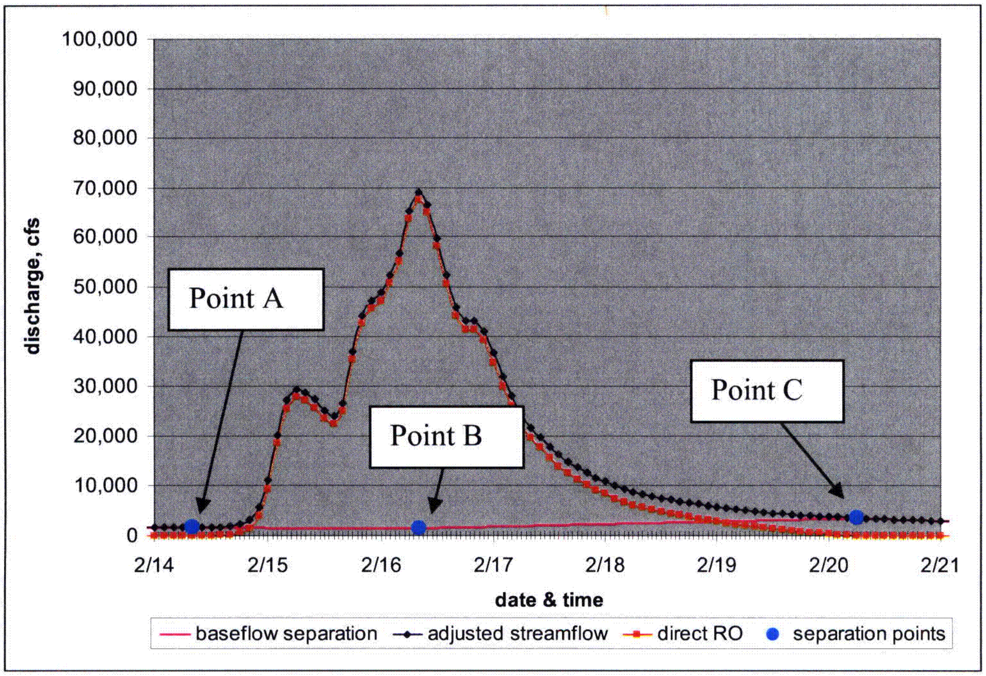

Figure 9: Emory River at Mouth baseflow separation for the February 2003 flood

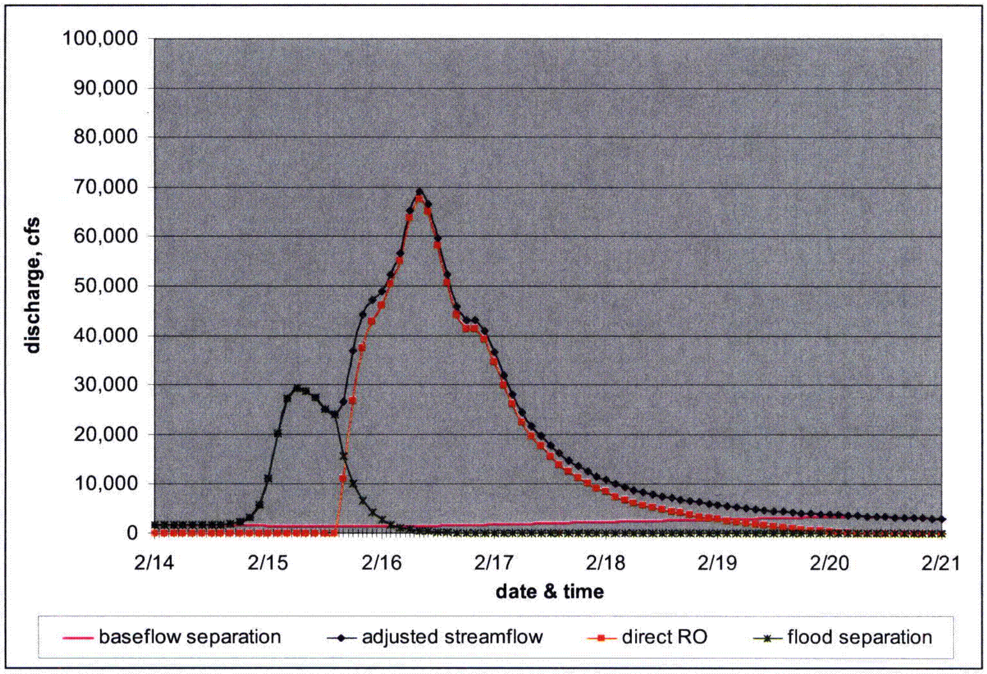

Figure 10: Emory River at Mouth direct runoff (RO) for the main peak of the February 2003 flood

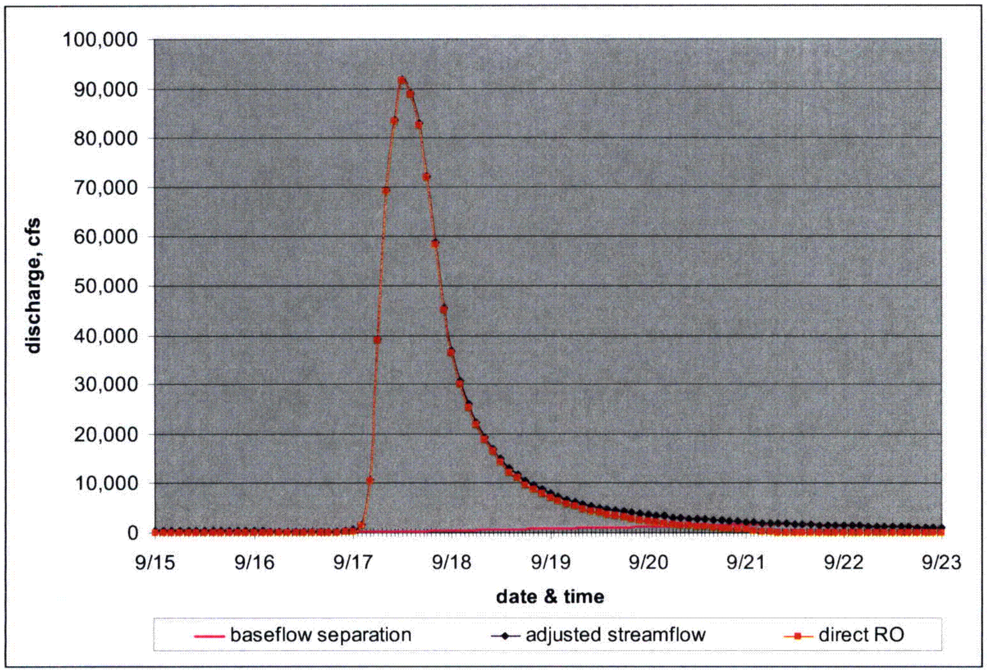

Figure 11: Emory River at Mouth baseflow separation for September 2004 flood

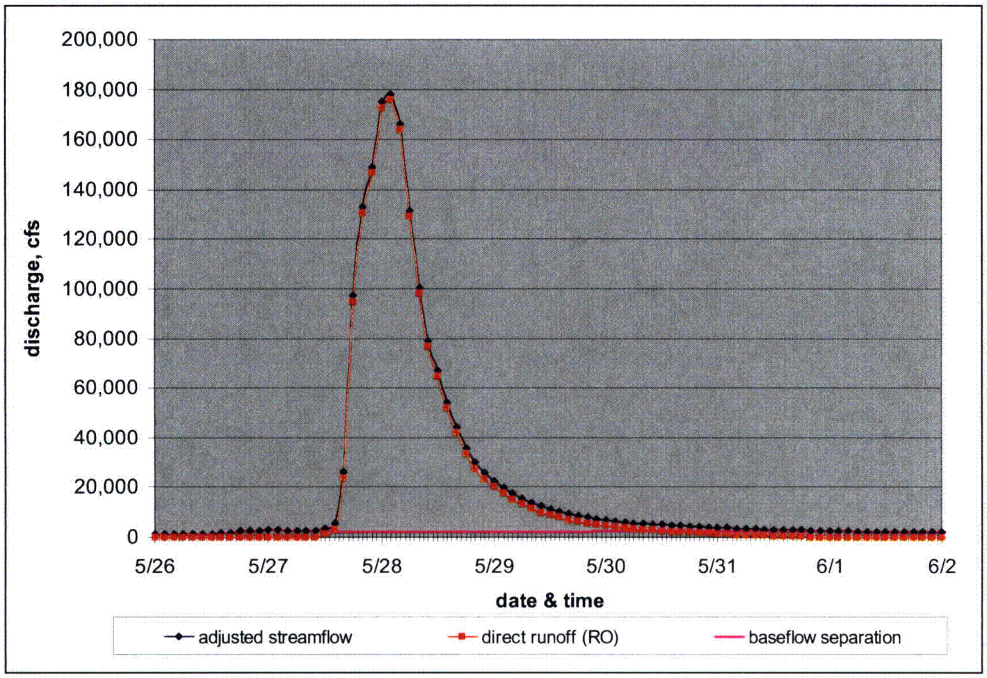

Figure 12: Emory River at Mouth baseflow separation for May 1973 flood

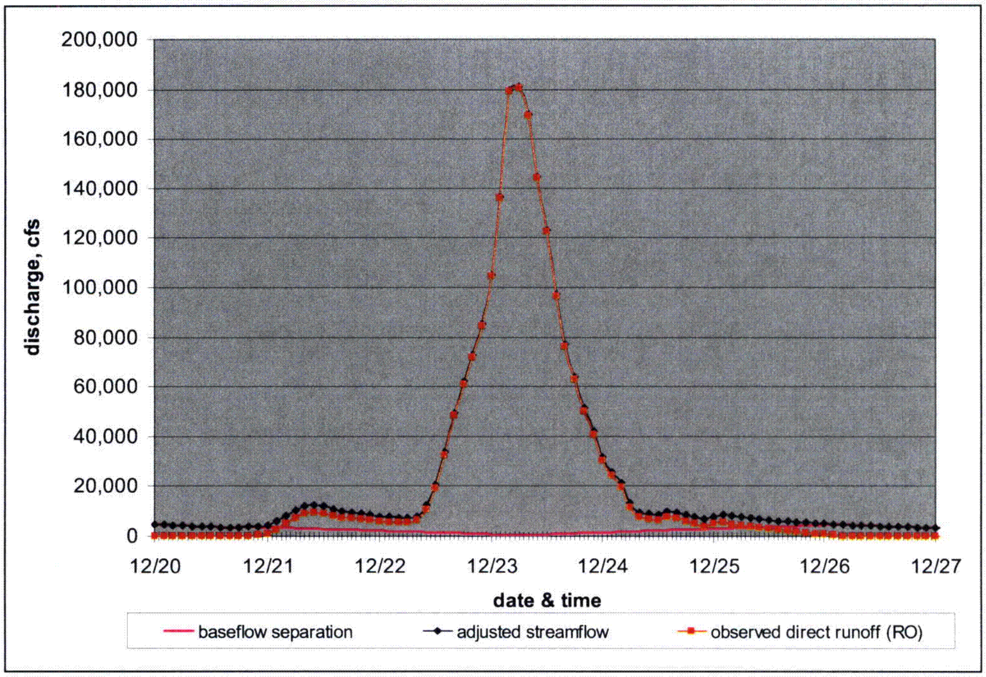

Figure 13: Emory River at Mouth baseflow separation for December 1990 flood

7.5 *Observed Basin Average Rainfall*

Observed basin average rainfall data for the storms during 1997-2007 were obtained from the National Weather Service (NWS) (Reference 12). The NWS basin average precipitation data are considered the best available for this calculation. The hourly precipitation series developed from NWS gridded data for these storms are provided in Attachment 1- 9 along with adjustments to Central time and unit conversion.

7.5.1 Rainfall Data Available for the May 1973 and December 1990 Storms

Hourly data from the NWS are not available for the 1990 and 1973 storms. Rainfall data corresponding to the 1973 and 1990 floods were requested and obtained from the TVA (Reference 18). The rainfall data for these storms were collected at two-hour intervals. Rainfall data were also available at six-hour intervals in the TVA six-hour rainfall database for the 1990, 2003, and 2004 storms; these data are enclosed as Attachment 1- 10, Attachment 1- 11, and Attachment 1- 12, respectively. Daily rainfall data for the 1973 storm were obtained from the TVA daily rainfall database (Reference 18). TVA rainfall data are reported in Central Time.

These rainfall data from the various TVA databases (two-hour, six-hour, and daily) are compared in Table 5, Table 6, and Table 7. The calculations underlying these tables are enclosed as Attachment 1- 13, Attachment 1- 14, and Attachment 1- 15. The locations of gages available in the TVA's rainfall databases are shown in Figure 14. Of note, the gages for which data are available vary by storm (e.g. year) and by measurement interval (i.e. two-hour, six-hour, or daily); consequently, data are not available from the same gages for all of the storms. The three tables provide summary data for the closest gages to the subbasin provided in Reference 18. For gages listed as "N/A" in these three tables, data are available at one measurement interval (e.g. two-hour or six-hour) but not at the comparison measurement interval. Additional gages, which are not shown in the tables, are included in the various databases; however, the gages provided in the tables are those closest to the subbasin.

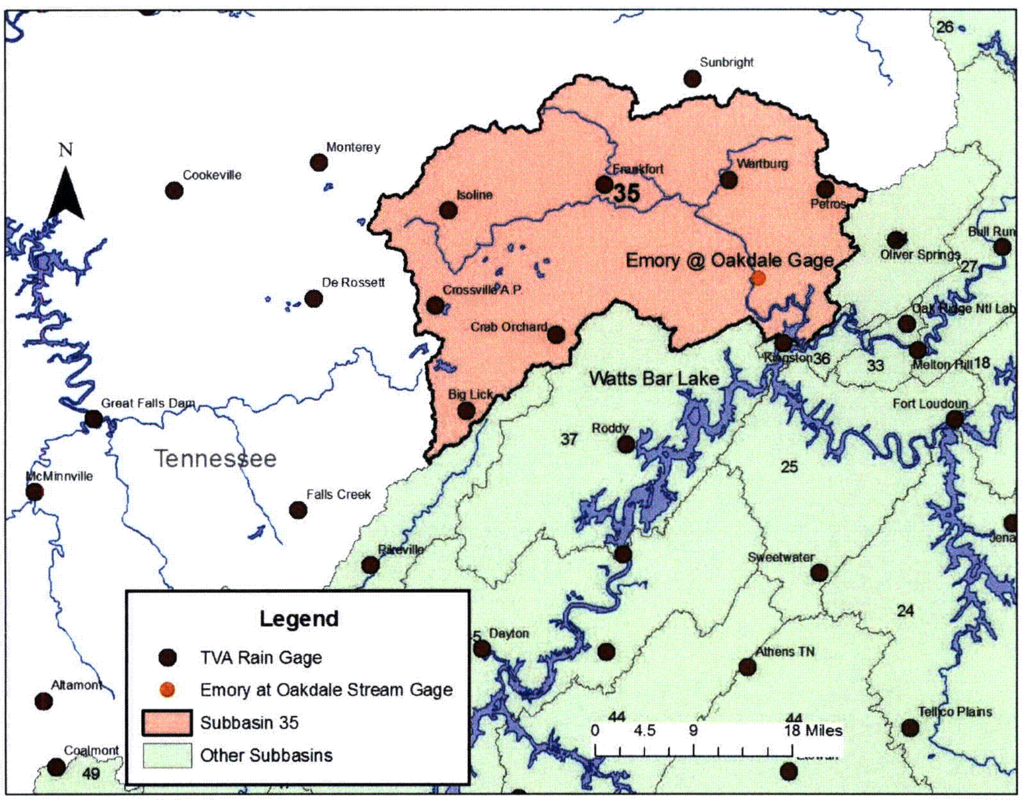

Figure 14: Locations of rain gages in the active TVA rainfall databases relative to Subbasin 35

Table 5: Summary of rainfall data for the May 1973 storm (5/27/1973 08:00 through 5/28/1973 08:00)

Gage	Thiessen Weights provided by the TVA	Rainfall Data (in)	
		Summation of Daily Data	Summation of 2-hour Data
Kingston	0.023	4.84	N/A
Petros	0.107	5.96	N/A
Isoline	0.102	6.50	N/A
Near Crossville (Crab Orchard)	0.043	7.07	N/A
Big Lick	0.026	7.17	N/A
Frankfort	0.165	5.40	5.10
Hebbertsburg*	0.152	8.29	N/A
Pilot Mountain*	0.137	5.83	N/A
Clarkrange*	0.056	5.85	N/A
Harriman*	0.092	5.85	N/A
Lantana*	0.082	6.04	N/A
Jewett*	0.015	5.84	N/A
Roddy	N/A	6.33	6.31
Falls Creek	N/A	N/A	7.40
Altamont	N/A	N/A	7.00
Arithmetic Average		6.228	6.428
Basin Average - Thiessen Polygon Weighting		6.301	N/A

*Gage is not part of the TVA active rain gage database

In Table 5, Table 6, and Table 7 three different types of average rainfall values are listed. Each type of average is calculated in a different manner. "Basin Average - Thiessen Polygon Weighting," is calculated using the Thiessen polygon weights. For six-hour data, the Thiessen weights were provided by the TVA in Attachment 1- 16. Thiessen polygon weights were derived for the two-hour rainfall data for the 1990 storm because the same gages are not available in the two-hour and six-hour data sets; these Thiessen polygon areas and weights are shown on Figure 32 and tabulated in Table 10 in the Appendix, Section 8. Thiessen polygon weights were also provided by the TVA for the daily data for the 1973 storm (Reference 18). Data are not available from the same gages for the 1973 flood as for either the two-hour or six-hour data for the 1990 flood. "Arithmetic Average," is the average of the listed values. "NWS Basin Average Rainfall" is the basin average calculated from hourly NWS data discussed in Section 5.3.

Because the NWS basin average precipitation data are not available for the December 1990 and May 1973 storms, a different source of rainfall data needs to be selected for these two storms. Two-hour data are available for both December 1990 and May 1973; two-hour data provide better temporal resolution, relative to the six-hour and daily data, of the distribution of rainfall. Two-hour rainfall data are used for both storms.

Table 6: Summary of rainfall data for the December 1990 storm (12/20/1990 00:00 through 12/25/1990 00:00)

Gage	Rainfall Data (in)	
	Summation of 6-hr Data	Summation of 2-hr Data
Wartburg	6.57	6.57
Frankfort	6.05	6.05
Petros	8.42	8.41
Isoline	6.37	6.84
Oliver Springs	6.43	6.42
Crossville A.P.	5.89	N/A
Crab Orchard	11.42	11.42
Kingston Fossil	7.32	N/A
Big Lick	6.31	6.25
Roddy	9.36	N/A
Sunbright	6.98	6.97
DeRossett	N/A	9.84
Arithmetic Average	7.375	7.641
Basin Average - Thiessen Polygon Weighting	7.196	7.468

Table 7: Summary of TVA six-hour and NWS rainfall data for the 2003 and 2004 storms

Gage		Rainfall Totals (in)	
		Summation of Rainfall between 2/14/2003 00:00 and 2/17/2003 00:00	Summation of Rainfall between 9/16/2004 00:00 and 9/17/2004 12:00
TVA 6-hr Rainfall Data (in)	Wartburg	5.14	4.44
	Frankfort	4.61	5.15
	Petros	6.97	5.39
	Isoline	5.39	4.99
	Oliver Springs	8.53	4.65
	Crossville A.P.	5.54	4.81
	Crab Orchard	5.28	5.61
	Kingston (fossil)	7.81	5.44
	Big Lick	5.34	6.10
	Roddy	8.24	5.76
	Sunbright	4.61	4.28
	Basin Average - Thiessen Polygon Weighting	5.542	5.109
	Arithmetic Average	6.133	5.147
NWS Basin Average Rainfall (in)		6.260	5.626

7.5.1.1 Two-Hour Rainfall Data for the May 1973 Storm

The four two-hour gages available for May 1973 are shown in Figure 15. Only one of these gages, Frankfort, is situated within Subbasin 35. The other three gages are located to the southwest of the subbasin. The total measured rainfall at the Frankfort gage of 5.1 inches is less than the 5.82 inches of observed direct runoff (Table 4) in the May 1973 flood. The observed rainfall at the other three gages was larger than the depth of direct runoff.

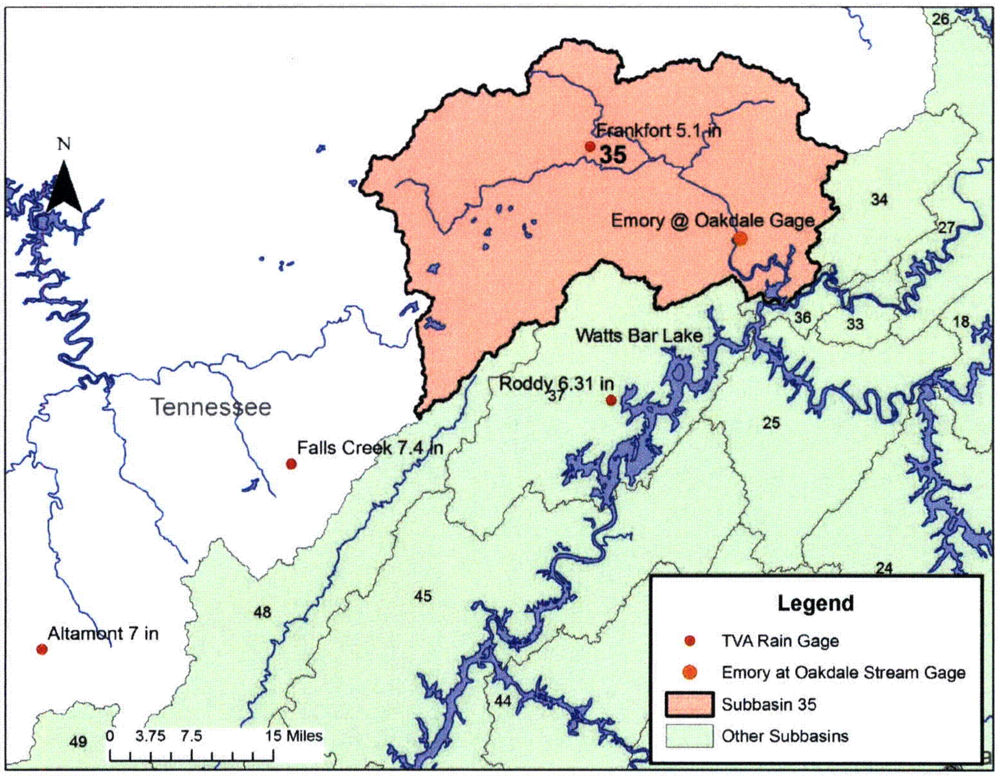

Figure 15: Locations and total rainfall depth for TVA rain gages with two-hour rainfall data available for the May 1973 storm

Estimates of average rainfall across the subbasin were derived in two different ways for the May 1973 storm. The area-weighted average depth from the daily gages, calculated using Thiessen polygons/weights obtained from the TVA (Reference 18), provides one estimate of basin average rainfall. This depth was distributed to two-hour intervals using the FLDHYDRO program and the time distribution of rainfall measured at the Frankfort gage. The FLDHYDRO program estimates the distribution of weighted runoff during a storm from the distribution of rainfall measured at hourly intervals for one or more stations/gages; the bihourly data from the Frankfort gage interpolated to one-hour intervals provided the time distribution to FLDHYDRO for the May 1973 storm. The total depth of runoff is determined by FLDHYDRO from both the hourly rainfall data and the daily rainfall total depths measured at one or more recorders/gages according to the Thiessen weight allocated to each recorder. As shown in Table 5, the area-weighted total depth is approximately 6.3 inches.

Figure 16 shows the distribution over time of the bihourly rainfall at the Frankfort gage, which was used to distribute the daily Thiessen-weighted precipitation runoff values during the May 1973 storm. In this figure, the measured rainfall at the Frankfort gage is compared to the observed rainfall from the two-hour gage with the largest measured rainfall depth for the storm, Falls Creek, and the average rainfall from the four two-hour gages shown in Table 5 for each bihourly measurement interval. The peak of the measured rainfall at the Frankfort gage is relatively subdued and broad. Preliminary HEC-HMS results, obtained using excess precipitation derived from the area-weighted basin average rainfall and shown in Figure 31 in the Appendix (Section 9), provide a significant under-prediction of the observed direct runoff. The significant under-prediction of the peak discharge for this flood is attributed to limiting the rainfall distribution to the Frankfort gage which is not adequate to define the basin rainfall.

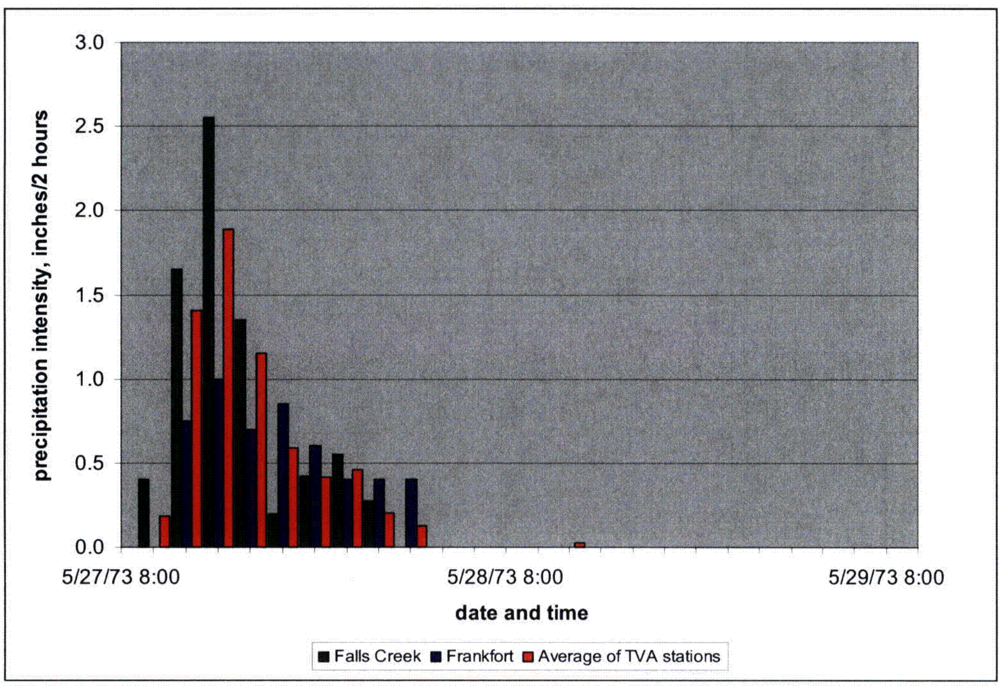

Figure 16: Comparison of average rainfall to the gage with the largest observed rainfall depth, Falls Creek, and the smallest, Frankfort, for the May 1973 storm

The arithmetic average of the data from the two-hour gages in the vicinity of the subbasin (Frankfort, Roddy, Falls Creek, and Altamont) provides another means to estimate the basin average rainfall for the May 1973 storm. This arithmetic average is only about 0.1 inches larger than the basin average calculated using Thiessen polygons. However, the arithmetic average of the four, two-hour stations provides a relatively defined peak of rainfall (and thus excess precipitation) as shown in Figure 16. The average of the four gages in Figure 15 for each two-hour measurement interval was used as rainfall data for the May 1973 storm. Results obtained using the arithmetic average for basin average rainfall data are presented in Section 6.7.

7.5.1.2 Two-Hour Rainfall Data for the December 1990 Storm

The nine two-hour gages available in the vicinity of Subbasin 35 for the December 1990 storm are shown in Figure 17. The Kingston gage, which is shown at the southeastern corner of the subbasin in Figure 14, was not available in the two-hour dataset (Reference 18). Consequently, Figure 17 displays a gap in gage coverage in the southeastern corner of the subbasin. The gages surrounding this gap (Crab Orchard, Frankfort, Wartburg, Petros, and Oliver Springs) provide a wide range of measured rainfall depths (6.05 to 11.42 inches) for the December 1990 storm. Six of the nine gages (Big Lick, Isoline, Frankfort, Sunbright, Wartburg, and Oliver Springs) have a total depth of measured rainfall for the December 1990 storm that is less than the observed direct runoff of 7.06 inches (Table 4) for the December 1990 flood.

In a similar fashion to the May 1973 storm, basin average rainfall values were estimated in two ways. In one method, area-weighted basin average rainfall depths for each two-hour measurement interval were calculated for the 1990 storm by applying the Thiessen weights to the data from the nine rain gages shown in Table 6. The area-weighted total depth for the storm of 7.47 inches, shown in Table 6, is only six percent larger than the observed direct runoff volume of 7.06 inches for the December 1990 flood. This relatively low total depth is obtained because the measured depths at the Frankfort, Wartburg, Isoline, Oliver Springs, and Big Lick gages, which are all smaller than the observed runoff, account for 70 percent of the area-weighted average.

This small difference between the area-weighted total rainfall depth and the depth of runoff means that essentially all precipitation is converted to runoff and that none of the rainfall infiltrates or evaporates. While it is possible that all rainfall could be converted to runoff, it is more likely that some of the rainfall (i.e. more than six percent) either infiltrates or evaporates, especially as rainfall depths that exceed the observed runoff by more than one inch were measured at several of the gages shown in Table 6. Preliminary HEC-HMS results, obtained using excess precipitation derived from the area-weighted basin average rainfall and shown in Figure 33 in the Appendix (Section 9), demonstrate a significant under-prediction of the observed direct runoff.

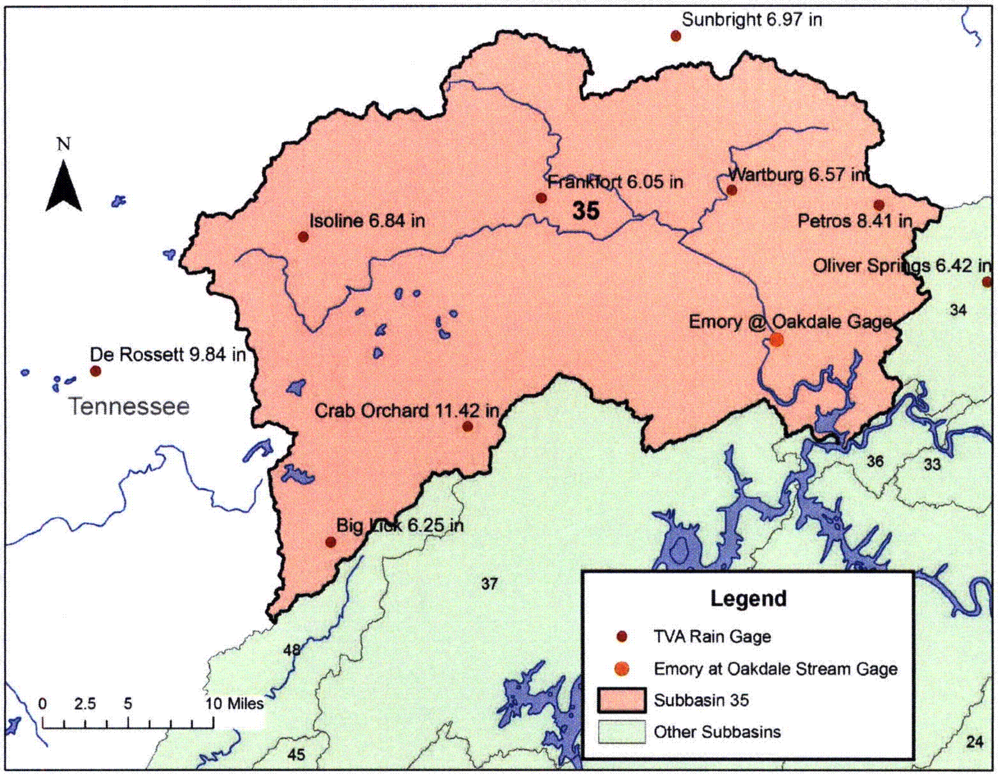

Figure 17: Locations and total rainfall depth for TVA rain gages with two-hour rainfall data available for the December 1990 storm

The arithmetic average of the data from the nine, two-hour gages in the vicinity of the subbasin provides another way to estimate the basin average rainfall for the December 1990 storm. The total depth provided by the arithmetic average is only about 0.2 inches larger than the total depth calculated using area-weighting. Figure 18 provides a comparison of the measured rainfall at the Crab Orchard gage which has the largest total depth during the December 1990 storm, to the Frankfort gage, which has the smallest total depth, and to the average of the nine gages. The average of the nine gages in Table 6 for each two-hour measurement interval was used as rainfall data for the December 1990 storm. Estimates of direct runoff obtained using the arithmetic average for basin average rainfall data are presented in Section 6.7.

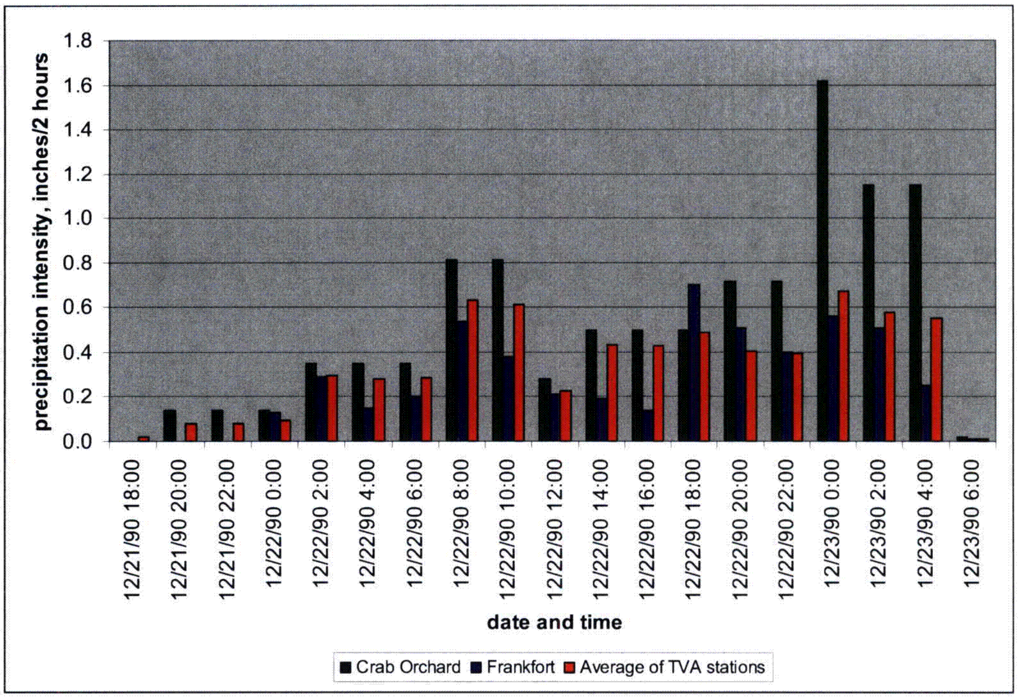

Figure 18: Comparison of average rainfall to the gage with the largest observed rainfall depth, Crab Orchard, and the smallest, Frankfort, for the December 1990 storm

7.6 *Basin Average Effective Rainfall*

Effective rainfall, or excess precipitation, is the input to the linear basin model that is converted into direct runoff at the basin outlet via convolution with the UH. The amount of excess can be developed from observed rainfall by the application of a loss function which incorporates the hydrologic abstractions of evaporation and transpiration, interception, depression storage, and infiltration (Reference 2). The amount of excess precipitation, or runoff, produced by a given storm is dependent on the soil and land use characteristics, state of the basin at the beginning of the storm, and the characteristics of the storm (Reference 20 and Reference 21). Storm characteristics related to excess rainfall generation include precipitation intensity, total rainfall amount, and spatial and temporal distribution of rainfall across the watershed (although use of the unit hydrograph method precludes incorporating the spatial distribution of rainfall into the analysis of storm runoff). The state of the basin encompasses antecedent soil moisture conditions, the amount of depression storage remaining in the watershed after recent rains, and vegetation-related concerns like evapotranspiration and interception.

The TVA utilizes the FLDHYDRO computer program (Reference 7) to estimate excess precipitation from a given rain storm for use with the UH for runoff prediction. The TVA created this program to implement the Antecedent Precipitation Index (API)/Rainfall Index (RI) methodology developed by the U.S. Weather Bureau (USWB) and described in Reference 20 and Reference 21. In this method, antecedent precipitation data are used to define the basin state at the beginning of the storm through the API. Seasonal, empirical relationships (the RI component) are employed to account for expected seasonal variation in runoff resulting from observed seasonal variations in evapotranspiration.

7.6.1 FLDHYDRO Operation

FLDHYDRO can be employed in two different ways to generate excess precipitation. One way, referred to here as the "forward excess precipitation estimation mode" uses the Antecedent Precipitation Index (API) for a given day, which is calculated on the basis of a recession constant normally reported to range from 0.85 to 0.98 (Reference 2, page 101). A recession constant of 0.9 is used in FLDHYDRO for this calculation. The API is used to obtain a Rainfall Index (RI) that has been determined for the Tennessee River Valley region as a function of precipitation, location, and season. The RI is then used to obtain precipitation losses for each increment of rainfall. The use of the loss function is discussed in the TVA White Paper (Reference 1) and the methodology is described in detail in the USWB publication (Reference 21).

The other FLDHYDRO excess precipitation estimation method, referred to here as the "CHKVOL mode," distributes and scales excess precipitation, independently of antecedent precipitation, so that the total volume of excess precipitation approximately matches the calculated direct runoff volume. The direct runoff volume comes from the baseflow separation calculations and is provided to the program with the CHKVOL variable. The time distribution of rainfall excess within the storm occurs according to the region provided to the FLDHYDRO model. Excess precipitation, as a percentage of observed rainfall, is larger at later times in the storm. The CHKVOL mode was used to estimate excess precipitation for use in HEC-HMS simulations of floods for UH validation.

FLDHYDRO, regardless of operation mode, requires a region specification in order to provide excess precipitation for a storm. Reference 7 provides information concerning the methods of specifying the region within the model. Subbasin 35 is in the North (N) region.

7.6.2 FLDHYDRO Input and Output

Table 8 provides a summary of the FLDHYDRO input and output for each storm and the resulting volume of excess precipitation obtained from the model. The input files and corresponding outputs files for FLDHYDRO are enclosed as Attachment 2- 4 through Attachment 2- 11. The time series of NWS basin average precipitation provides the main FLDHYDRO input for the 2003 and 2004 storms. The time series of average rainfall at the TVA's two-hour rain gages provides the primary input for the 1990 and 1973 storms.

FLDHYDRO derives the time distribution of excess precipitation from the precipitation input. Comparisons of cumulative precipitation and excess precipitation and of the distribution over time of precipitation and excess precipitation are provided in Figure 19 through Figure 26. The FLDHYDRO output obtained using the CHKVOL mode was adjusted using the ratio of the FLDHYDRO output total excess precipitation volume to the observed direct runoff volume so that estimated excess precipitation volume matches observed direct runoff volume. These estimated excess precipitation values were then used simulate direct runoff hydrographs in HEC-HMS.

Table 8: Selected FLDHYDRO inputs and resulting excess precipitation volumes

Subbasin	Storm	Volume of Rainfall	FLDHYDRO Input File	Region	Excess Precipitation Derived from FLDHYDRO Mode	
					CHKVOL	Forward Estimation
		(in)			(in)	(in)
Emory River at Mouth, Subbasin 35	February 2003	4.13*	B35_03FS.dat	N	3.63	2.19
	September 2004	5.63	Bas35_04.dat	N	3.32	1.71
	May 1973	6.46**	Bas35_73-AVE.dat	N	5.78	4.21
	December 1990	7.64	Bas35_90BA.dat	N	7.14	5.86

* The volume of rainfall shown in Table 8 differs from that shown in Table 7 due to separation of rainfall data to correspond with the hydrograph separation shown in Figure 10.

** The volume of rainfall shown in Table 8 differs from that shown in Table 5 due to a small amount of rainfall after 08:00 on 5/28/1973.

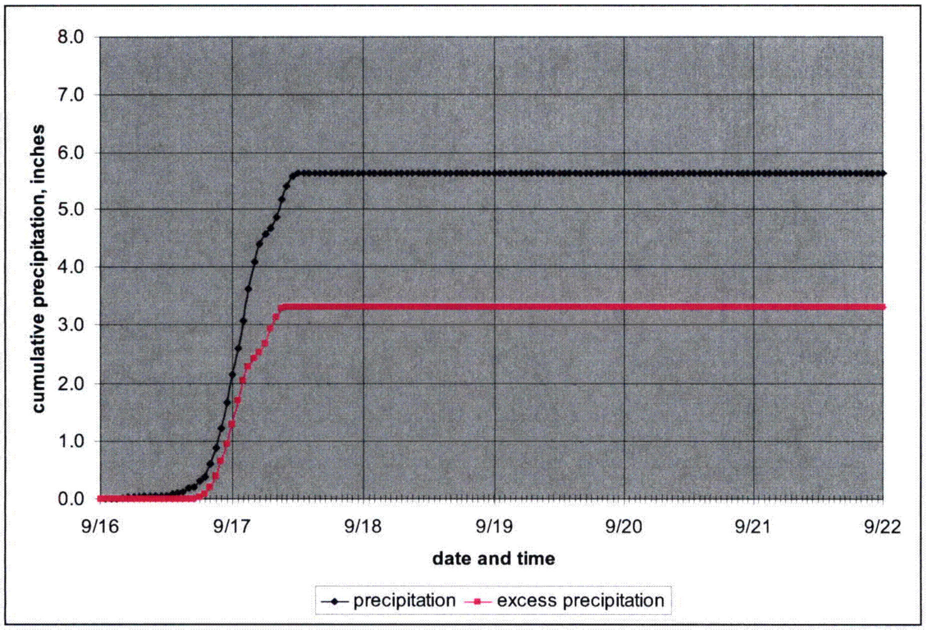

Figure 19: Emory River at Mouth cumulative precipitation and excess precipitation for the September 2004 storm

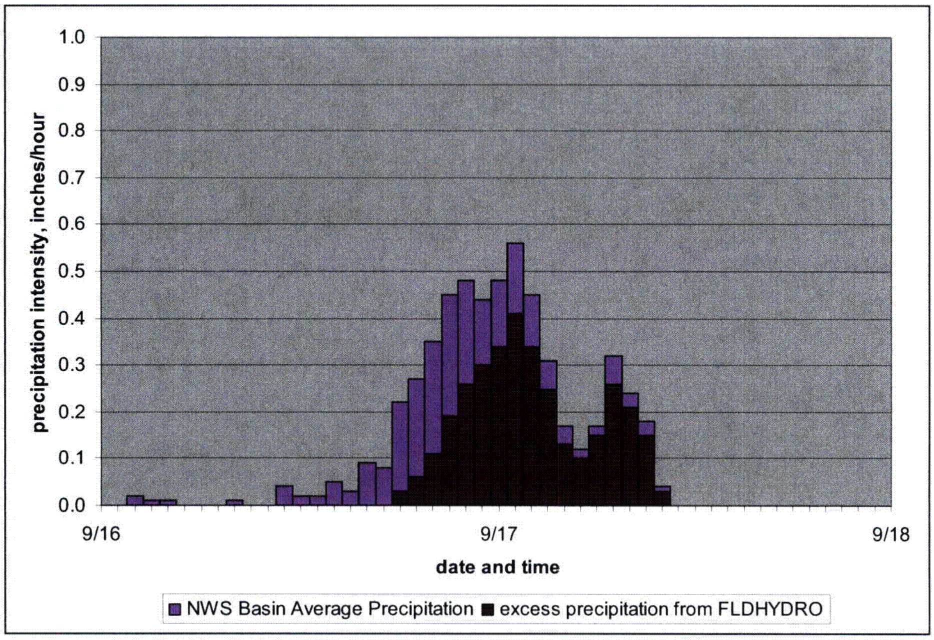

Figure 20: Emory River at Mouth precipitation and excess precipitation time series for the September 2004 storm

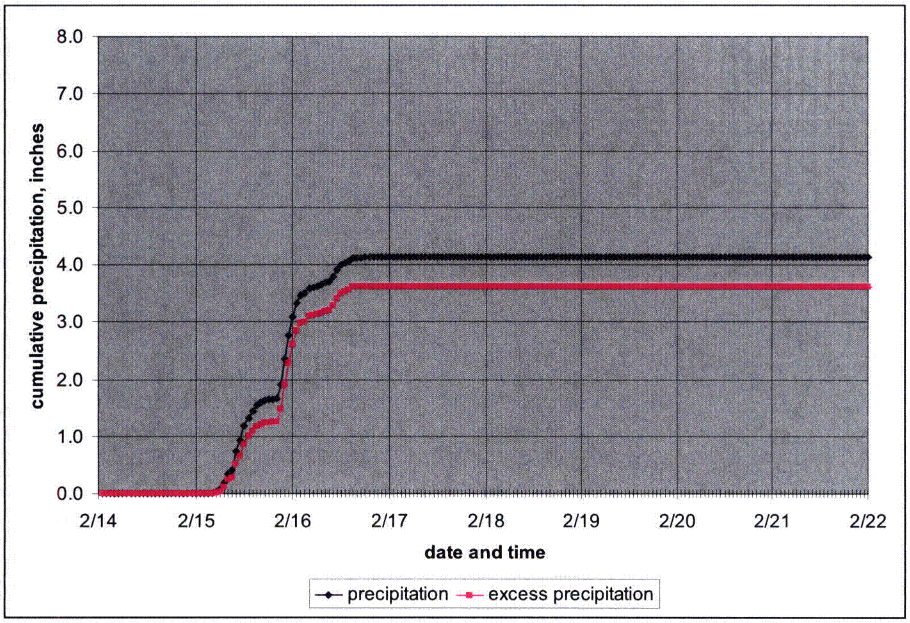

Figure 21: Emory River at Mouth cumulative precipitation and excess precipitation for the February 2003 storm

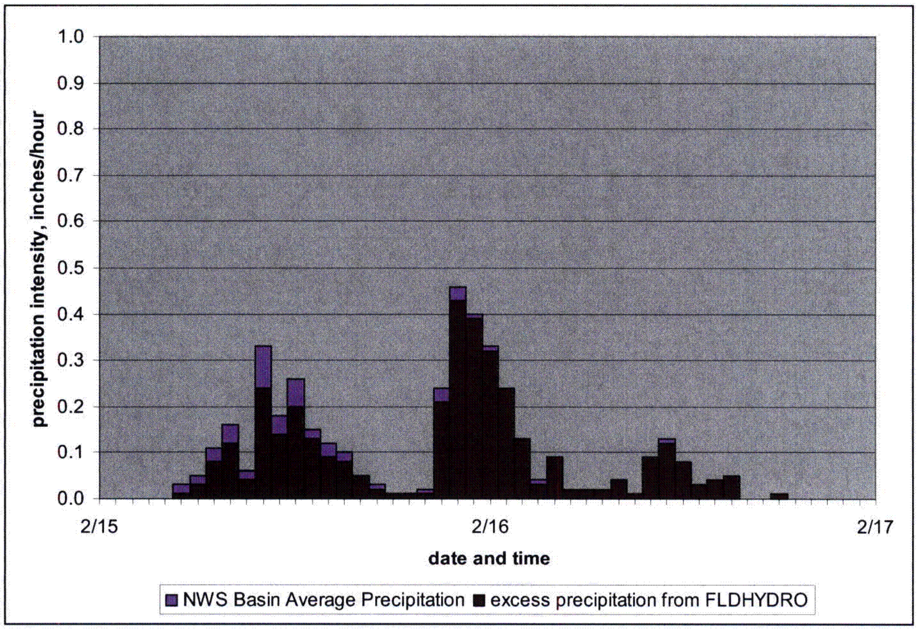

Figure 22: Emory River at Mouth precipitation and excess precipitation time series for the February 2003 storm

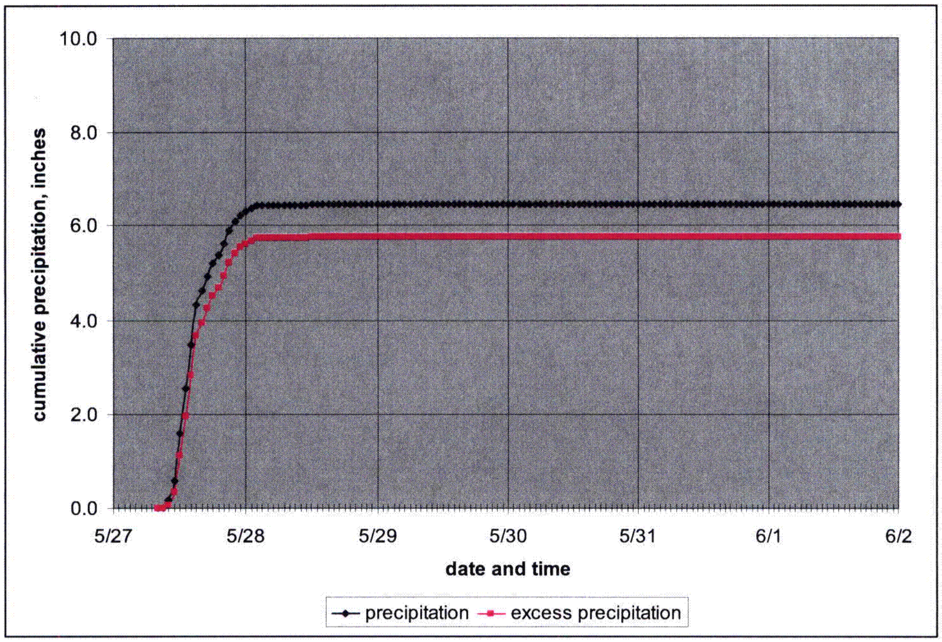

Figure 23: Emory River at Mouth cumulative precipitation and excess precipitation for the May 1973 storm

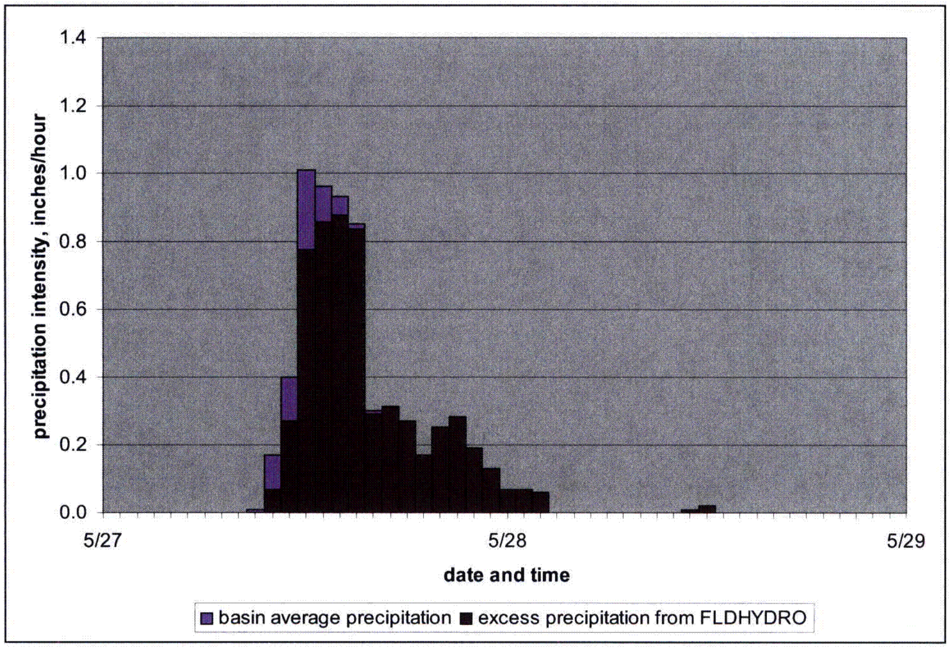

Figure 24: Emory River at Mouth precipitation and excess precipitation time series for the May 1973 storm

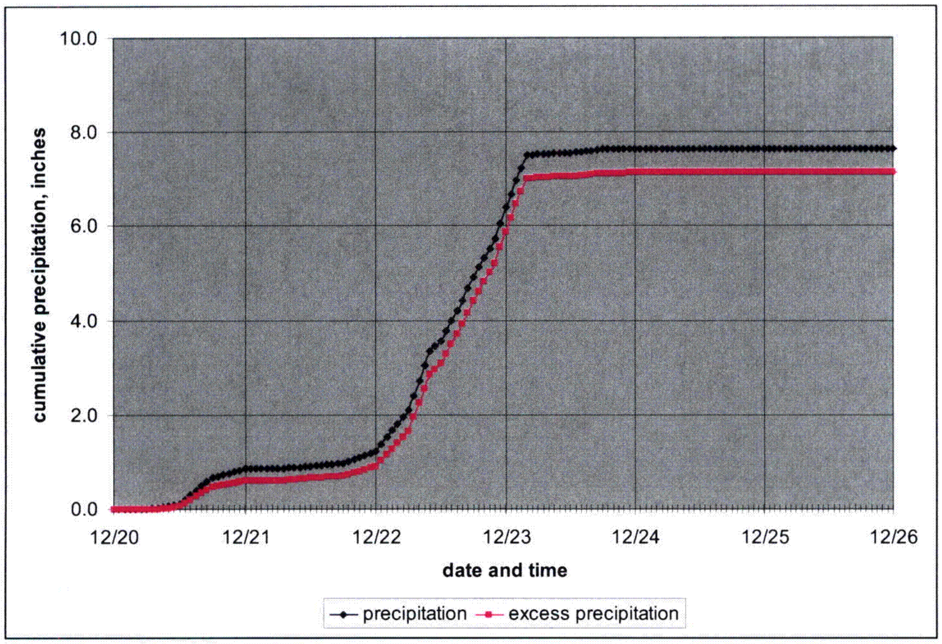

Figure 25: Emory River at Mouth cumulative precipitation and excess precipitation for the December 1990 storm

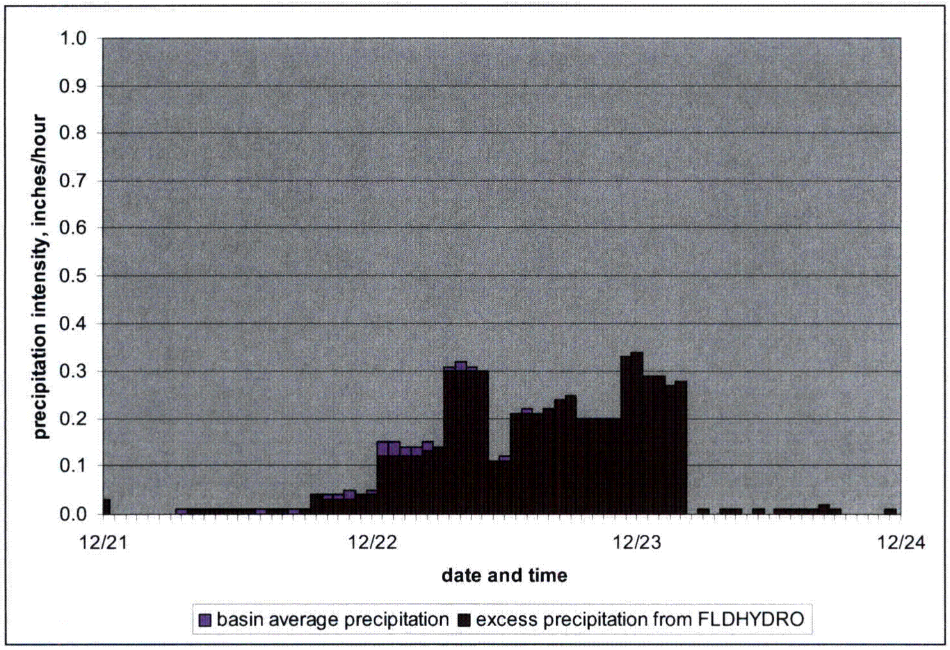

Figure 26: Emory River at Mouth precipitation and excess precipitation time series for the December 1990 storm

7.7 *HEC-HMS Simulations of Floods*

Two HEC-HMS project files were developed for testing the unit hydrograph developed for the Emory River at Mouth subbasin. One project file (Attachment 3- 1) provides validation of the regenerated UH. The following basin models were developed within this project:

- Basin_35-1973
- Basin_35-1990
- Basin_35-2003
- Basin_35-2004

One-hour excess precipitation values were employed with the one-hour regenerated UH to simulate the February 2003 and September 2004 floods. NWS basin average rainfall data are available for these two storms and have one-hour measurement intervals. Because two-hour rainfall data are the finest resolution data available for the May 1973 and December 1990 storms, two-hour excess precipitation values were used with the two-hour regenerated UH to simulate the 1973 and 1990 floods. Simulated hydrographs are compared to observed direct runoff for each flood.

The following input files were developed for the project and input to HEC-HMS (Reference 8) via the Time Series Data Manager (all time series are adjusted to Central Time for this calculation):

- Precipitation Gage "Effect_May1973-Ave" with two-hour incremental depths of excess precipitation derived from the arithmetic average of two-hour TVA rainfall data
- Precipitation Gage "Effect_Dec1990-Ave" with two-hour incremental depths of excess precipitation derived from the arithmetic average of two-hour TVA rainfall data
- Precipitation Gage "Effect_Feb2003" with hourly incremental depths of excess rainfall derived from NWS basin average rainfall
- Precipitation Gage "Effect_Sep2004" with hourly incremental depths of excess rainfall derived from NWS basin average rainfall
- Discharge Gage "ObsRO_May1973" with two-hour local direct runoff discharge in cfs
- Discharge Gage "ObsRO_Dec1990" with two-hour local direct runoff discharge in cfs
- Discharge Gage "ObsRO_Feb2003" with hourly local direct runoff discharge in cfs
- Discharge Gage "ObsRO_Sep2004" with hourly local direct runoff discharge in cfs

Note that instead of inputting observed basin average precipitation and utilizing a loss function for the subbasin, the excess basin average rainfall (or runoff) output from FLDHYDRO was utilized as "precipitation data" for all simulations. The simulated hydrograph is compared to the observed hydrograph for the February 2003, September 2004, May 1973, and December 1990 floods in Figure 27, Figure 28, Figure 29, and Figure 30 obtained from the HEC-HMS GUI. An assessment of the results of the validation simulations is presented in Table 9.

Table 9: Summary of HEC-HMS validation simulations

Flood		February 2003	September 2004	December 1990	May 1973
Volume (in)	HMS	3.63	3.32	7.14	5.78
	Observed RO	3.75	3.31	7.06	5.82
	Residual	-0.12	0.01	0.08	-0.04
	% Error*	-3.2	0.3	1.1	-0.7
Peak Discharge (cfs)	HMS	76,348	86,303	118,299	156,081
	Observed RO	67,556	91,686	180,742	176,058
	Residual	8,792.1	-5,383.0	-62,443.2	-19,977.5
	% Error*	13.0	-5.9	-34.5	-11.3
Time of Peak (hrs)	HMS	2/16/2003 5:00	9/17/2004 11:00	12/23/1990 6:00	5/27/1973 22:00
	Observed RO	2/16/2003 8:00	9/17/2004 12:00	12/23/1990 6:00	5/28/1973 2:00
	Residual	-3.00	-1.00	0.00	-4.00
	% Error**	10.7	5.3	0.0	25.0

* % Error is the Residual divided by Observed RO value as a percentage.

** % Error is the observed time to peak less the simulated time to peak divided by the observed time to peak. The time to peak is measured from the onset of excess precipitation in the FLDHYDRO output.

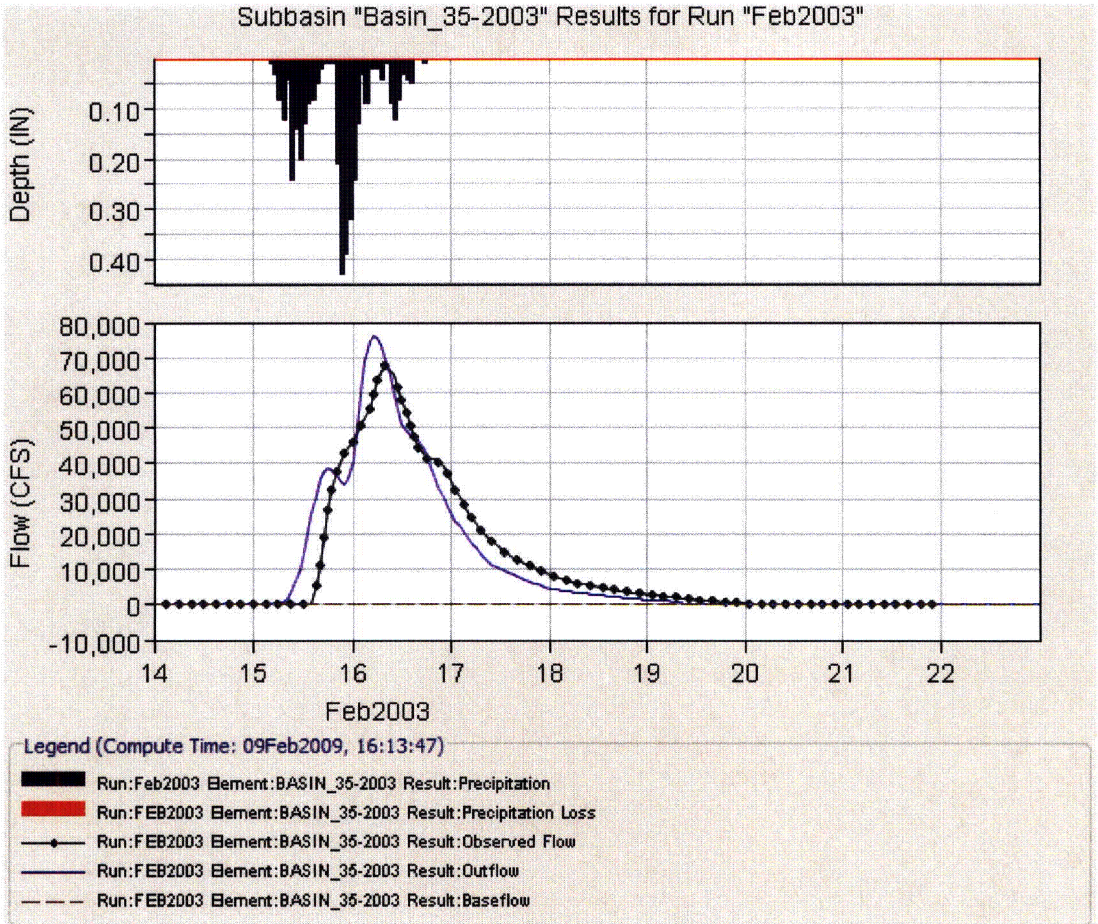

Figure 27: Emory River at Mouth HEC-HMS output (1-hr UH) for February 2003 flood

For the February 2003 simulation:

1. The simulated discharge occurred three hours prior to the observed discharge.
2. The magnitude of the peak was 13 percent higher in the simulation than in the observed hydrograph.

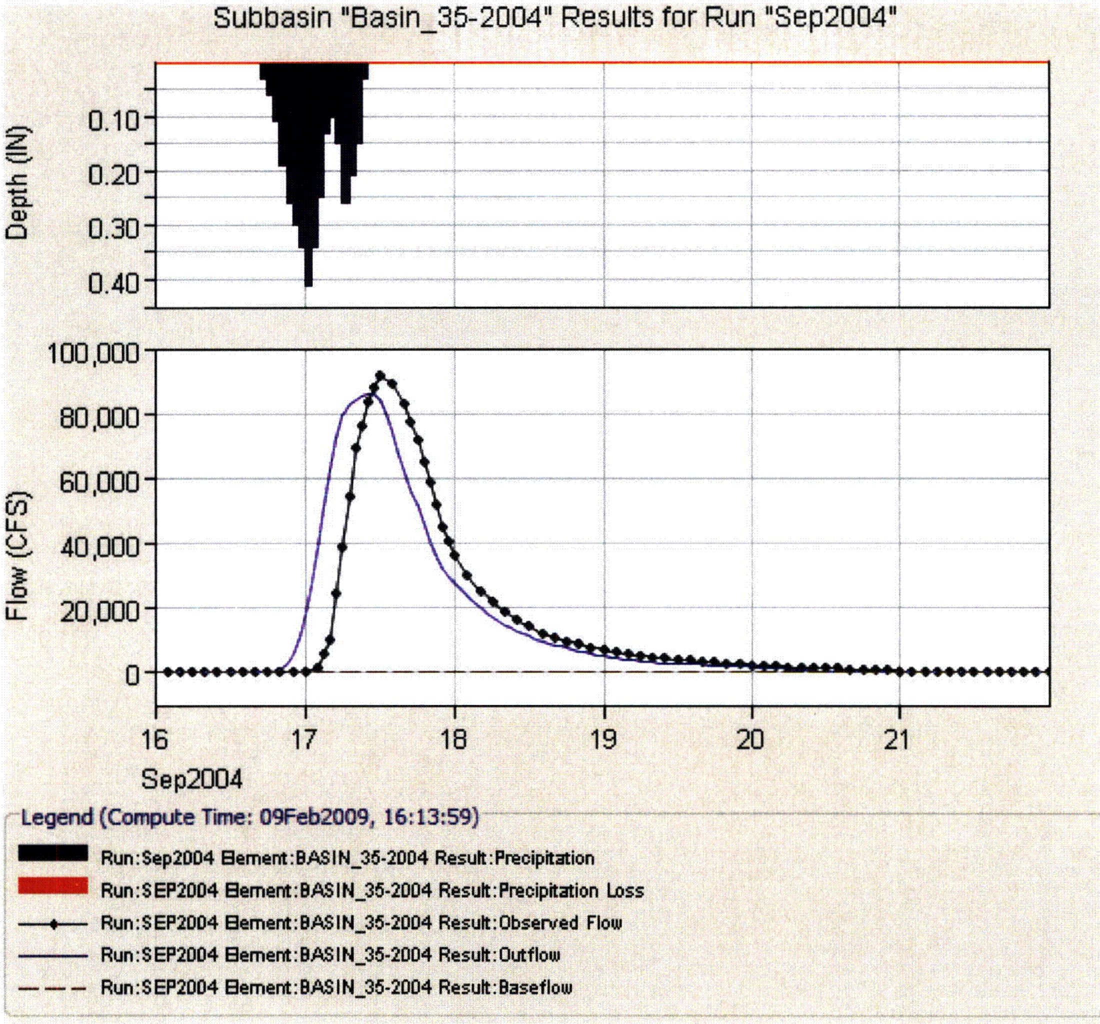

Figure 28: Emory River at Mouth HEC-HMS output (1-hr UH) for September 2004 flood

For the September 2004 simulation:

1. The simulated discharge occurred one hour prior to the observed discharge.
2. The magnitude of the peak was 6 percent lower in the simulation than in the observed hydrograph.

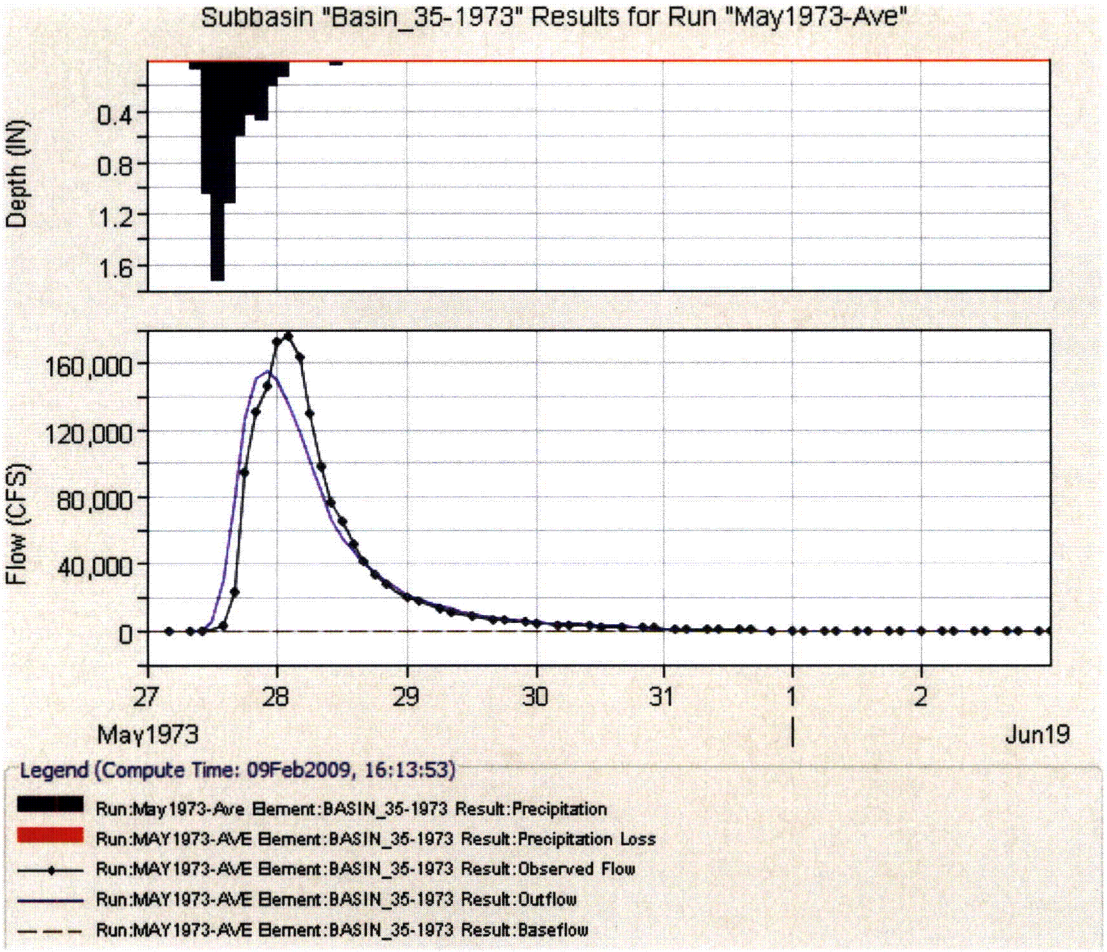

Figure 29: Emory River at Mouth HEC-HMS output (2-hr UH) for May 1973 flood

For the May 1973 simulation:

1. The simulated discharge occurred four hours prior to the observed discharge.
2. The magnitude of the peak was 11 percent lower in the simulation than in the observed hydrograph.

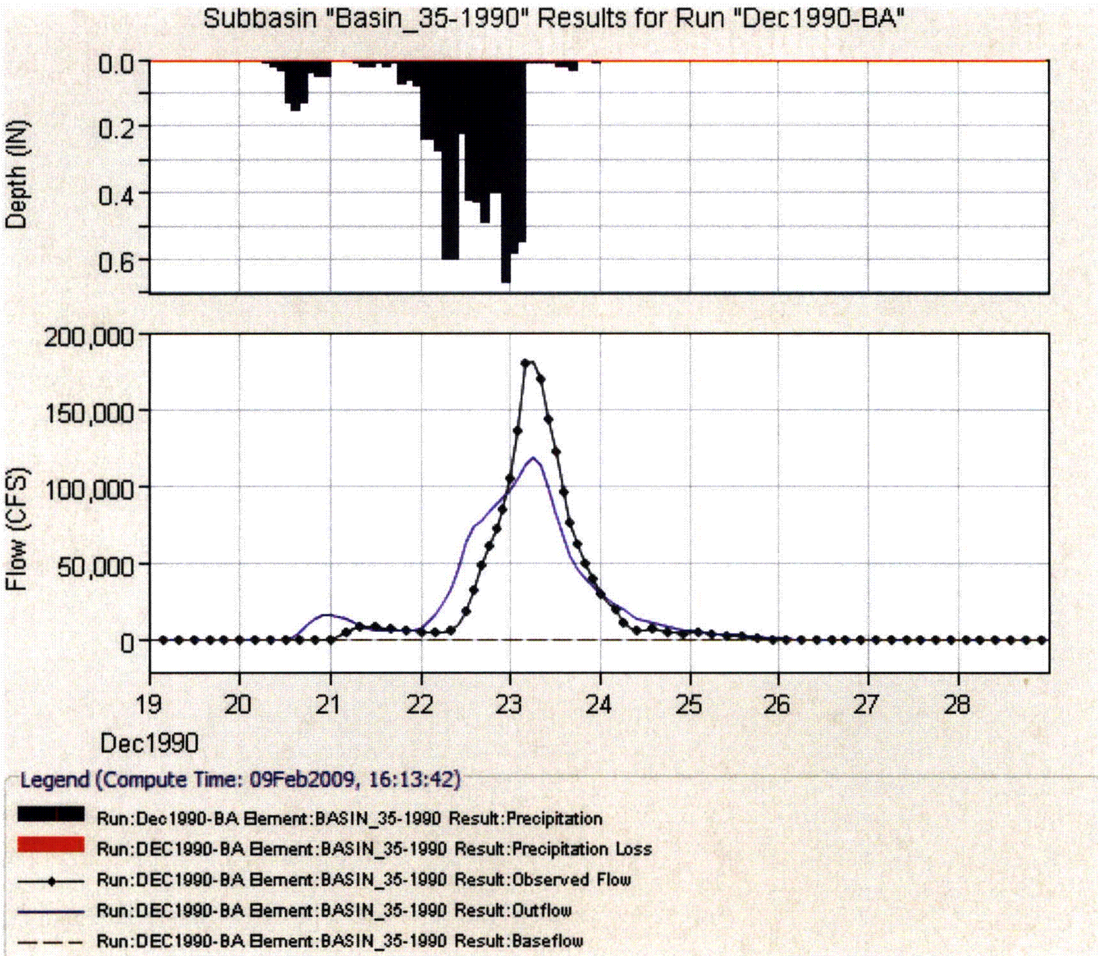

Figure 30: Emory River at Mouth HEC-HMS output (2-hr UH) for December 1990 flood

For the December 1990 simulation:

1. The simulated peak discharge occurred at approximately the same time as the observed peak.
2. The magnitude of the peak was 35 percent lower in the simulation than in the observed hydrograph.

The other HEC-HMS project file (Attachment 3- 2) employs the regenerated 4-hr UH with precipitation data aggregated to four-hour intervals to confirm the performance of the UH on the floods used in its derivation. This confirmation is necessary since the UNITGRPH program was used to calculate a composite unit hydrograph for this subbasin. As a result, the performance of the regenerated unit graph should be checked against each flood. The following basin models were developed within this project:

- Basin_35-1939
- Basin_35-1957
- Basin_35-1963
- Basin_35-2003
- Basin_35-2004

The following input files were developed for the project and input to HEC-HMS (Reference 8) via the Time Series Data Manager (all time series are adjusted to Central Time for this calculation):

- Precipitation Gage "Effect_Feb1939" with incremental depths of excess rainfall
- Precipitation Gage "Effect_Nov1957" with incremental depths of excess rainfall
- Precipitation Gage "Effect_Mar1963" with incremental depths of excess rainfall
- Precipitation Gage "Effect_Feb2003" with incremental depths of excess rainfall
- Precipitation Gage "Effect_Sep2004" with incremental depths of excess rainfall
- Discharge Gage "ObsRO_Feb1939" with local direct runoff discharge in cfs
- Discharge Gage "ObsRO_Nov1957" with local direct runoff discharge in cfs
- Discharge Gage "ObsRO_Mar1963" with local direct runoff discharge in cfs
- Discharge Gage "ObsRO_Feb2003" with local direct runoff discharge in cfs
- Discharge Gage "ObsRO_Sep2004" with local direct runoff discharge in cfs

Data for the 1939, 1957, and 1963 floods were obtained from Reference 10. The discharge data for these early floods were scaled as discussed in Section 6.2 for comparison to the flood runoff estimated with the four-hour UH for Subbasin 35. These calculations are enclosed as Attachment 1-17. For comparison, figures and a table summarizing the results of simulations using the four-hour UH are provided in the Appendix, Section 9 (Figure 34 through Figure 38 and Table 11).

8 Discussion and Conclusions

One- and two-hour UHs were derived using the S-graph method from the four-hour UH regenerated for Subbasin 35. The TVA provided bihourly discharge data for the gage on the Emory River at Oakdale, TN. Four floods were selected from these gage data for UH validation. Two of the selected floods represent the largest annual discharge values, which have corresponding hourly precipitation data available from the NWS, for the 11-year period spanning 1997-2007. The

December 1990 and May 1973 floods were also used for unit hydrograph validation; these floods are the second and third largest floods on record.

Hydrographs were calculated for the four selected floods from the streamflow gage data using a scaling factor (Section 6.2). Baseflow was then estimated and removed from flood hydrographs to obtain "observed" direct runoff hydrographs. Hourly, basin-average precipitation data were obtained from the NWS for the two storms selected during 1997- 2007. Two-hour precipitation data were obtained from the TVA for the December 1990 and May 1973 storms. FLDHYDRO was used to estimate excess precipitation from the rainfall data for each validation storm. The UH for the subbasin and the estimated excess precipitation values were then used in HEC-HMS to simulate flood runoff.

A subjective, visual comparison of the HEC-HMS simulated hydrograph to the corresponding time series of "observed" direct runoff was used to determine UH validity. This comparison involved examination of: 1) overall flood hydrograph shape; 2) timing of flood hydrograph peak, and 3) magnitude of flood hydrograph peak. Subjectivity enters the validation process because the conditions underlying the unit hydrograph method (Section 3.3), the determination of excess precipitation (Section 6.6), and the calculation of the "observed" direct runoff hydrograph (Section 6.2) preclude an exact match between a discharge series calculated with a UH for a particular rain storm and the observed discharge series at the basin outlet.

Floods in Subbasin 35 during February 2003 and September 2004 were simulated in HEC-HMS using excess precipitation derived from NWS basin averaged rainfall data. The simulated flood hydrograph for the 2003 flood had a peak that exceeded the observed peak by 13 percent. The simulated peak for the 2004 flood was lower than the observed peak but only by about six percent (Table 9).

HEC-HMS was used to simulate floods in Subbasin 35 during December 1990 and May 1973 using excess precipitation derived from rainfall data in the TVA's two-hour database. Arithmetic averages of rainfall amounts measured at the TVA two-hour gages in the vicinity of the subbasin were used in FLDHYDRO to derive excess precipitation for each storm. The simulated flood hydrograph for the 1990 flood had a peak that was 34 percent lower than the observed peak. The simulated peak for the 1973 flood was also lower than the observed peak by about 12 percent (Table 9).

The timing of the four simulated hydrographs matched the timing of the observed hydrographs moderately well. The peak discharge in the 2003, 2004, and 1973 simulations occurred prior to the observed peak discharge by one to four hours. The simulated peak discharge for the 1990 flood occurred at the same time as the observed peak discharge. The difference in timing between simulated and observed peak discharge was less than or equal to the period (i.e. four hours) of the calculated UH for all four simulations. In reproduction of the three floods used to derive the UH for this subbasin, the simulated and observed peaks occurred simultaneously for the 1939 flood (Figure 34); the simulated peak preceded the observed for the 1963 flood (Figure 36); and, the simulated peak lagged the observed peak for the 1957 flood (Figure 35). In general, the simulated hydrographs led the observed hydrographs slightly; however, this bias is not consistent across the floods used in the validation analysis.

Arithmetic averages of the available rainfall data in the TVA's two-hour rainfall database were employed as rainfall data in the analysis of the 1973 and 1990 floods. Average values, either arithmetic or area-weighted, of the TVA rain gages in the vicinity of the subbasin for a particular storm are not necessarily equivalent to the NWS basin average rainfall values. For both the December 1990 and May 1973 storms, significant spatial gaps exist among rain gages. Additionally, the total measured depth of rainfall at the TVA two-hour rain gages varies by more than one inch, or by more than 15 percent, for both the 1973 and 1990 storms. Given concerns with spatial coverage combined with variation in measured rainfall depths among the gages, the TVA rainfall data are not considered as accurate as the NWS basin average data for use in unit hydrograph validation.

It is important to note that different results are obtained from simulations of the May 1973 and December 1990 floods when different rainfall data are used with FLDHYDRO to derive excess precipitation. When excess precipitation for the 1990 flood is estimated from rainfall measured at the Crab Orchard gage (which recorded 11.42 inches of rainfall, much more than at other gages), the flood peak discharge is under-predicted by less than 7 percent as shown in Figure 39 in the Appendix, Section 9. Visual comparison of the precipitation obtained from the average rainfall with the precipitation measured at the Crab Orchard data (Figure 18) suggests that the lack of a defined basin average precipitation series is responsible for the poor representation of the peak discharge (Figure 30). The total volume of excess precipitation is approximately the same for the simulation results for the December 1990 flood shown in Figure 30 and Figure 39 because the CHKVOL mode was used in FLDHYDRO to derive the excess precipitation from both sets of rainfall data.

To provide another example of results obtained using different data, rainfall data from the Roddy rain gage were used to simulate the May 1973 flood. These rainfall data were employed without abstractions (i.e. FLDHYDRO was not used). The May 1973 flood can be categorized as a 100-year flood based on the magnitude of peak discharge. In estimation of floods of this size, rainfall data are sometimes used without abstractions to provide a conservative estimation of flooding for scenarios with already saturated soils and without available storage in the watershed. The Roddy rain gage was chosen since the total depth measured at the this gage (6.31 in) is closest to the volume of observed direct runoff for the May 1973 flood (5.82 in) among the gages available for this storm. The simulated peak discharge in this case is nearly equal to the observed peak discharge as shown in Figure 40 in Section 9, although the simulated runoff volume was greater than the observed runoff volume.

Different rainfall data series provide different simulation results. Because data are not available from the same rain gages for the May 1973 and December 1990 floods and because NWS basin average data are not available for these floods, it is unclear how well the average rainfall values employed for the 1973 and 1990 storms represent the "actual" rainfall across the subbasin. The results for the May 1973 and December 1990 floods obtained using excess precipitation derived from the average of the available rain gages significantly under-predict the observed peak discharges for these two floods. However, results obtained from different rainfall data (i.e. from the Crab Orchard gage in 1990 and from the Roddy gage in 1973) suggest that the Subbasin 35 unit hydrograph could adequately reproduce the observed direct runoff if an "optimal" series of excess precipitation is derived. As a result, the simulation results presented here for the May 1973 and December 1990 floods neither validate nor invalidate the unit hydrograph for Subbasin 35 because of uncertainty related to the available rainfall data for the May 1973 and December 1990 storms.

Given the uncertainty in results obtained for the May 1973 and December 1990 floods, the February 2003 and September 2004 floods provide unit hydrograph validation. The unit hydrograph developed for the Emory River at Mouth watershed (Subbasin 35) has been validated against more recent floods that occurred in February 2003 and September 2004. Although simulated hydrographs for both floods led the observed hydrographs slightly, this bias is counterbalanced by the predictive results obtained for the three floods used to develop the unit hydrograph (i.e. February 1939 and November 1957). The validated unit hydrograph is listed in tabular form in Table 1 and provided in graphical form in Figure 2 as the line labeled "Regenerated 4-hr UH". The regenerated UH with a six-hour period is provided in the same figure and table.

9 Appendix

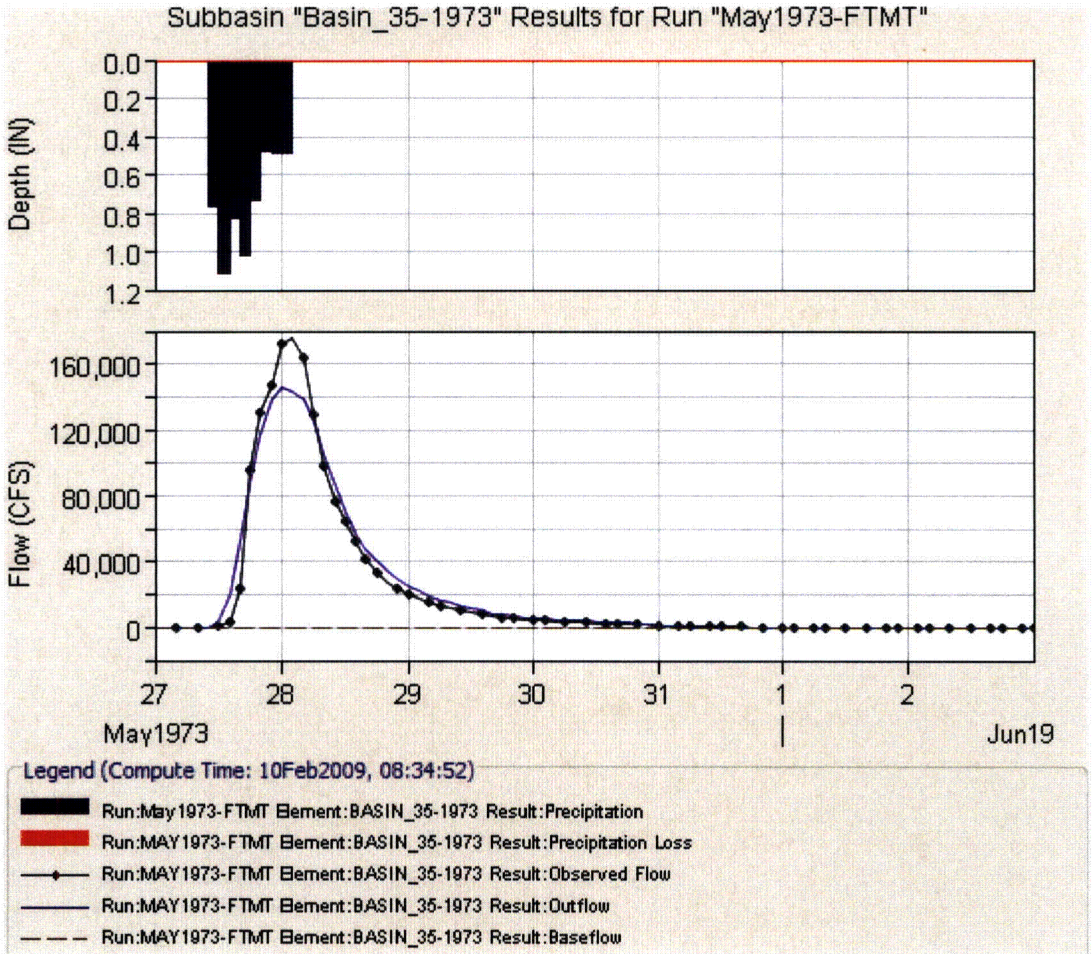

Figure 31: Emory River at Mouth HEC-HMS output (2-hr UH) for May 1973 flood using excess precipitation derived from area-weighted (Thiessen polygons), basin average rainfall

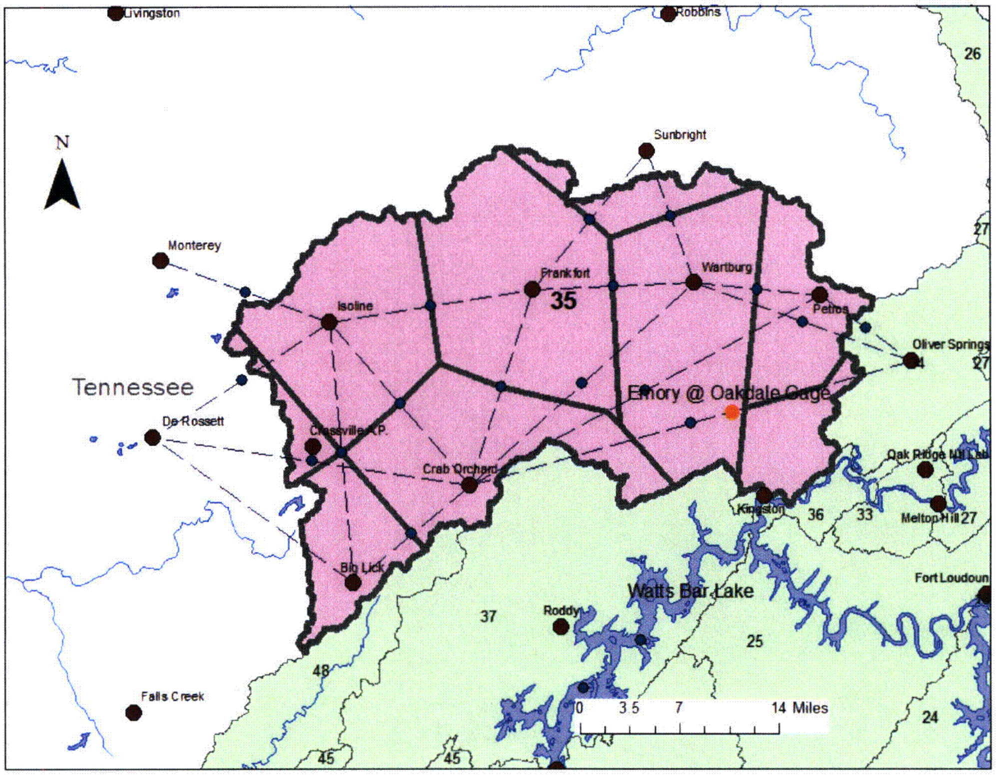

Figure 32: Thiessen areas for two-hour rainfall data for the December 1990 storm

Table 10: Area weights for two-hour rainfall data for the December 1990 storm

Gage	Area (mi^2)	Weight
Big Lick	61.8	0.071
DeRossett	18.4	0.021
Crab Orchard	135.0	0.155
Isoline	143.7	0.165
Frankfort	185.0	0.213
Wartburg	171.1	0.197
Petros	74.9	0.086
Oliver Springs	44.9	0.052
Sunbright	34.4	0.040
sum	869.0	1.0

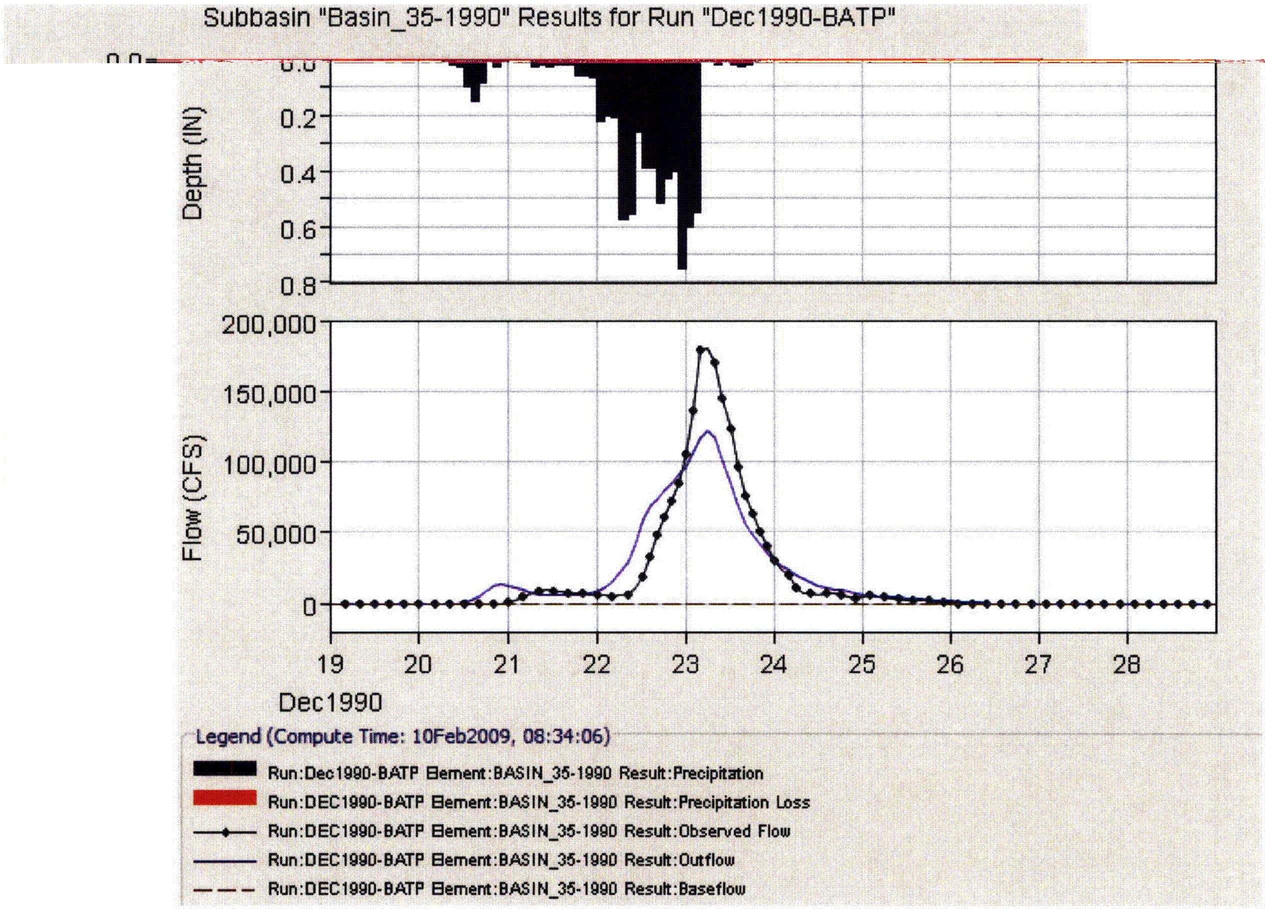

Figure 33: Emory River at Mouth HEC-HMS output (2-hr UH) for December 1990 flood using excess precipitation derived from area-weighted (Thiessen polygons), basin average rainfall

Table 11: Summary of HEC-HMS simulations using 4-hr regenerated UH

Flood		February 2003	September 2004		February 1939	November 1957	March 1963
Volume (in)	HMS	3.63	3.32		3.30	5.70	3.30
	Observed RO	3.75	3.32		3.19	5.76	3.19
	Residual	-0.12	0.00		0.11	-0.06	0.11
	% Error*	-3.2	0.0		3.4	-1.0	3.4
Peak Discharge (cfs)	HMS	71,463	84,961		85,542	72,546	86,805
	Observed RO	67,556	91,686		107,129	82,364	94,820
	Residual	3,907.0	-6,725.3		-21,587.0	-9,818.0	-8,015.0
	% Error*	5.8	-7.3		-20.2	-11.9	-8.5
Time of Peak (hrs)	HMS	2/16/03 4:00	9/17/04 8:00		2/2/39 22:00	11/19/57 1:00	3/12/63 3:00
	Observed RO	2/16/03 8:00	9/17/04 12:00		2/2/39 22:00	11/18/57 17:00	3/12/63 7:00
	Residual	-4.00	-4.00		0.00	8.00	-4.00
	% Error**	14.3	21.1		0.0	114.3	57.1

* % Error is the Residual divided by Observed RO value as a percentage.

** % Error is the observed time to peak less the simulated time to peak divided by the observed time to peak. The time to peak is measured from the onset of excess precipitation in the FLDHYDRO output.

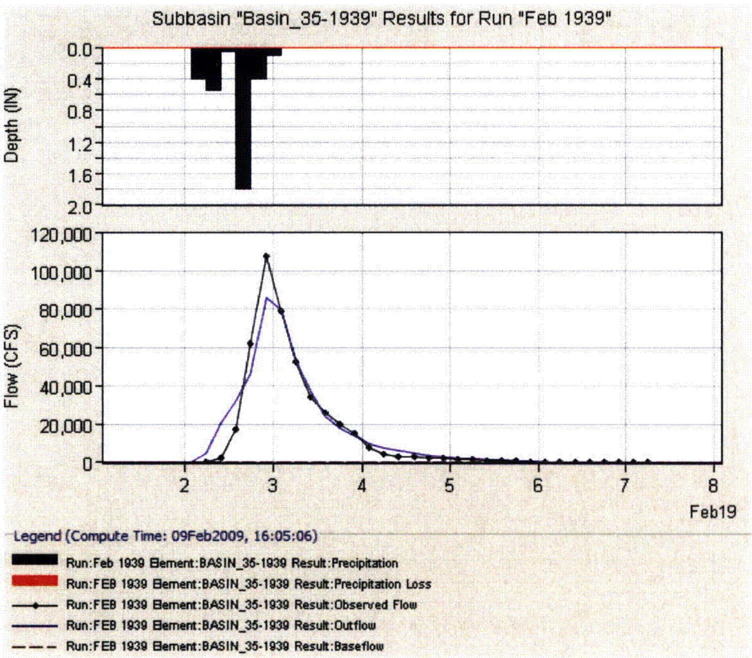

Figure 34: Emory River at Mouth HEC-HMS output (4-hr UH) for February 1939 flood

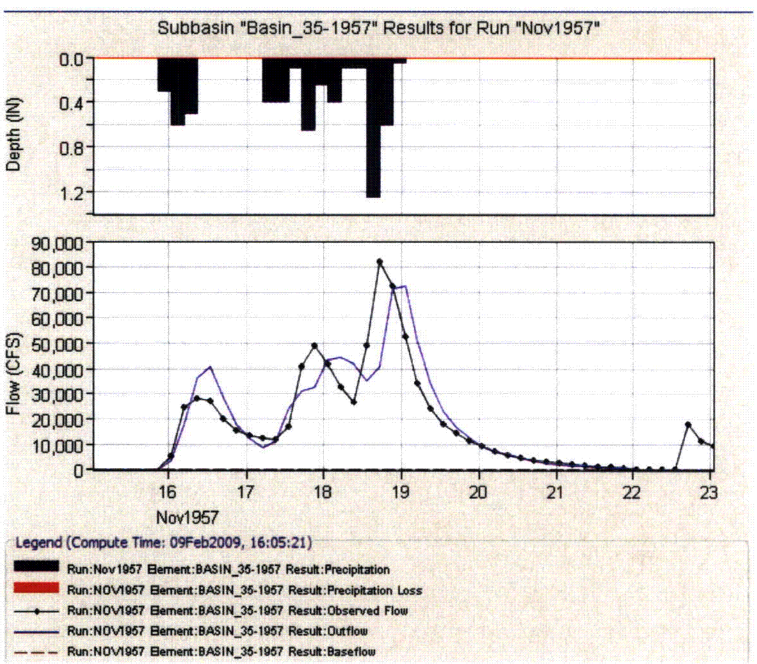

Figure 35: Emory River at Mouth HEC-HMS output (4-hr UH) for November 1957 flood

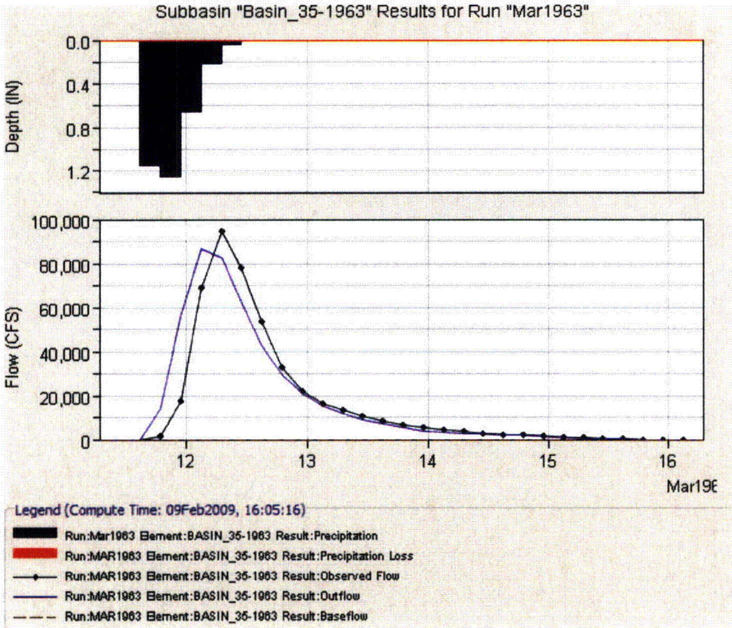

Figure 36: Emory River at Mouth HEC-HMS output (4-hr UH) for March 1963 flood

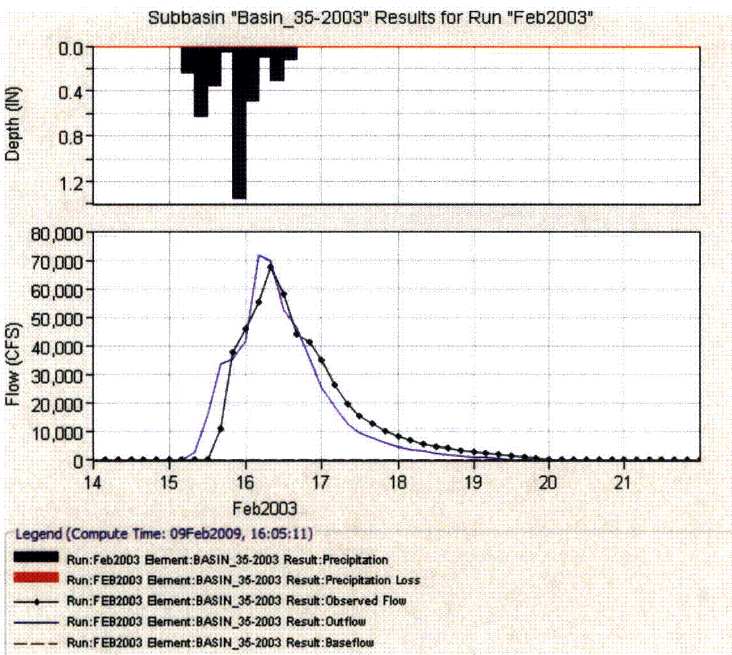

Figure 37: Emory River at Mouth HEC-HMS output (4-hr UH) for February 2003 flood

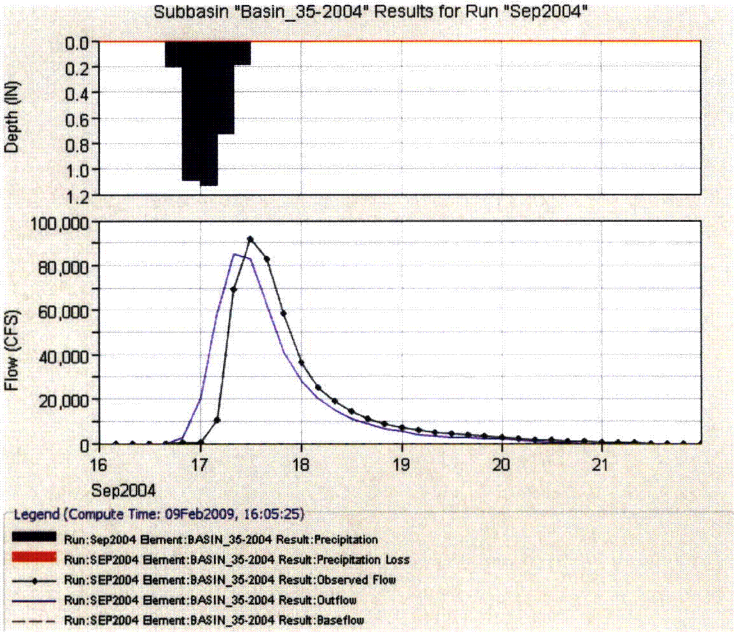

Figure 38: Emory River at Mouth HEC-HMS output (4-hr UH) for September 2004 flood

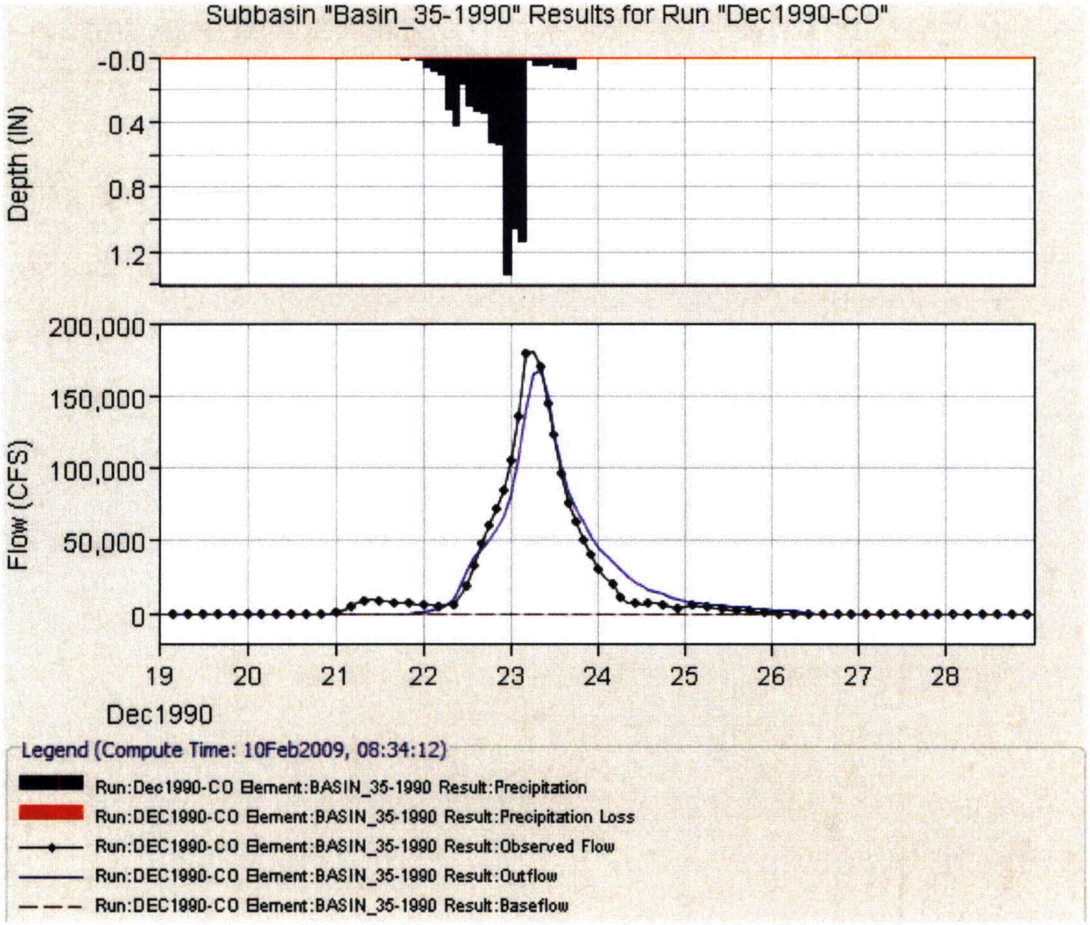

Figure 39: Emory River at Mouth HEC-HMS output (2-hr UH) for December 1990 flood using excess precipitation derived from 2-hr rainfall series for Crab Orchard gage

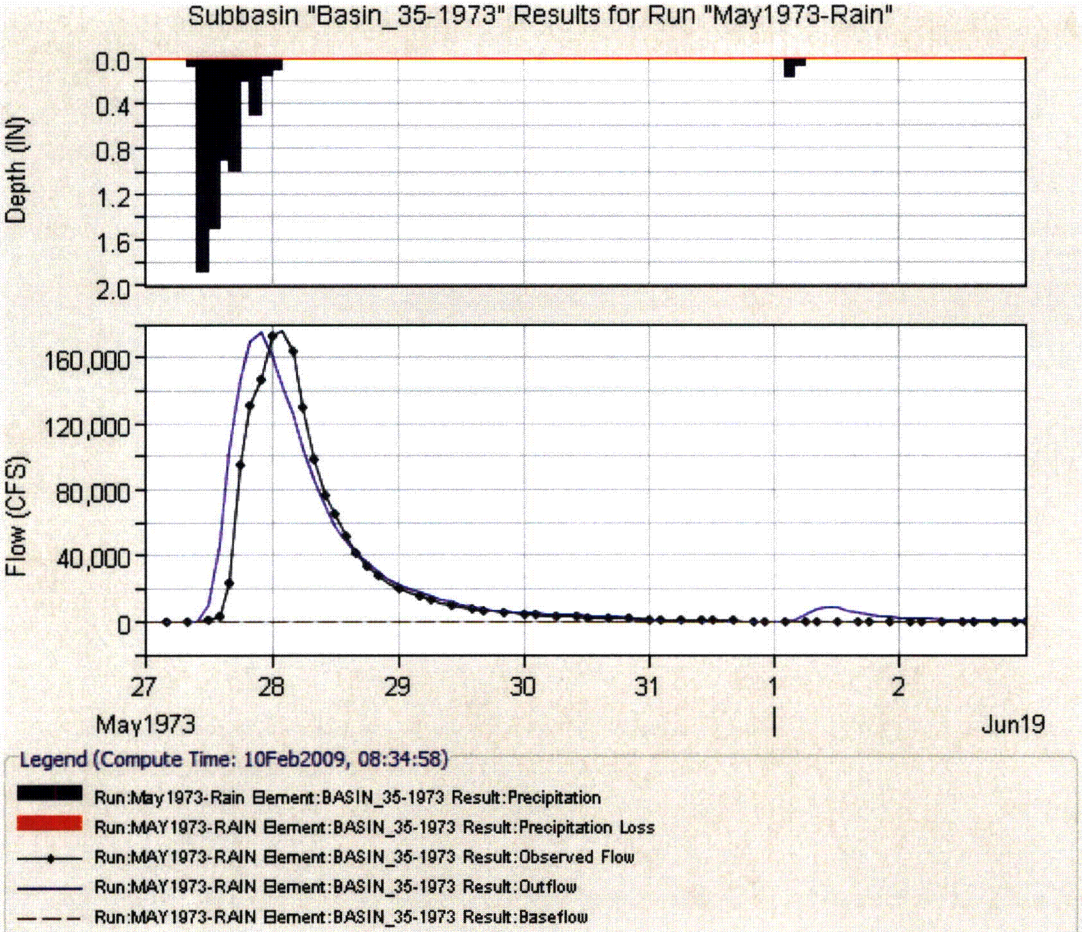

Figure 40: Emory River at Mouth HEC-HMS output (2-hr UH) for May 1973 flood using 2-hr rainfall data (no loss subtracted) from the Roddy gage