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Revision to Flood Hazard Evaluation for the Savannah River Site

R. L. Buckley and D. W. Werth

August 25, 2014

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EXECUTIVE SUMMARY

Requirements for the Natural Phenomena Hazard (NPH) mitigation for new and existing Department of Energy (DOE) facilities are outlined in DOE Order 420.1. This report examines the hazards posed by potential flooding and represents an update to two previous reports. The facility-specific probabilistic flood hazard curve is defined as the water elevation for each annual probability of precipitation occurrence (or inversely, the return period in years). New design hyetographs for both 6-hr and 24-hr precipitation distributions were used in conjunction with hydrological models of various basins within the Savannah River Site (SRS). For numerous locations of interest, peak flow discharge and flood water elevation were determined. In all cases, the probability of flooding of these facilities for a 100,000 year precipitation event is negligible.

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LIST OF ABBREVIATIONS

CAP	Culvert Analysis Program
CLM	Central Climatology Facility
C00	K.-F. Chen report, WSRC-TR-2000-00206
C99	K.-F. Chen report, WSRC-TR-99-00369
DOE	Department of Energy
FDC	Flood Design Category
HEC-HMS	Hydrologic Engineering Center – Hydrologic Modeling System
DOE	Department of Energy
NPH	Natural Phenomena Hazard
SRNL	Savannah River National Laboratory
SRS	Savannah River Site
USGS	United States Geological Survey
WSPRO	Water Surface Profile Computations
W13	D. Werth et al. report, SRNL-STI-2013-00664
W98	A.H. Weber et al. report, WSRC-TR-98-00329

1.0 Background

Flooding at Department of Energy (DOE) sites can cause structural damage and interrupt critical functions, resulting in huge economic losses or possibly an environmental or health hazard. A DOE directive, DOE Order 420.1, Facility Safety, outlines the procedures for Natural Phenomena Hazard (NPH) mitigation for new and existing DOE facilities, and requires a determination of flood elevations as a function of return period up to 100,000 years. Technical standards for various natural phenomena hazards are discussed in DOE-STD-1020-2012 (DOE, 2012). Based on facility-specific probabilistic flood hazard curves and the nature of site operations (e.g., involving hazardous or radioactive materials), managers can design permanent or temporary devices to prevent the propagation of onsite flooding, and develop emergency preparedness plans to mitigate the consequences of floods. The flood hazard curves for various areas within the Savannah River Site (SRS) are presented in this report, covering return periods from 50 years to 100,000 years. The SRS requires a probabilistic flood hazard assessment for return periods out to 100,000 years because it is considered a flood design category (FDC-5) site (as required by Table 5-2A, and discussed in Section 5.5.5 of DOE technical standard 1020 (DOE, 2012)).

Chen (1999, 2000) has twice performed such an analysis, using historic precipitation data to develop a basin model for four basins within the SRS – the Upper Three Runs Creek, the Fourmile Branch, Tims Branch, and the Pen Branch. The former two were first run with 6-hour accumulated precipitation data associated with various return periods (Chen, 1999, henceforth C99), and all four were subsequently rerun with 24-hour accumulated precipitation (Chen 2000, henceforth C00). In addition, L-Lake flooding was considered for the 24-hour accumulation to determine flood elevation levels at L-area. Characterizing river flooding for basins of importance to the SRS addresses Section 5.3.1 of the DOE 1020 technical standard (DOE, 2012). The precipitation data came from a report prepared by Weber et al., (1998, henceforth W98), in which existing precipitation records were used to calculate the probabilities per year of experiencing a range of rainfall amounts. The W98 report has been updated (Werth et al., 2013, henceforth W13), and the C99 and C00 reports must therefore be updated as well, using the newer precipitation values. The basin models, however, are applied as in the previous studies. Therefore, we forego in our report a detailed description of the calibration and validation of those models, which is available in C99/00. Instead, a summary of the current work is described.

2.0 Methodology

A straightforward way to determine probabilistic flood hazard curves is to conduct statistical analyses based on records of gauge-measured stream flow. Gauge records at SRS are short (several decades), however, and for some streams include the effects of significant quantities of cooling water discharged from five SRS production reactors that operated for many years, making it difficult to apply observed flow data to flood hazard analysis. As an alternative, C99/00 employed a basin hydrologic routing method: hyetographs (time series of rainfall depths) for various return periods were synthesized based on existing rainfall data, and the Hydrologic Modeling System computer code (HEC-HMS, 2010) was used to calculate the basin peak flow for each period. Conducting such a site flood and hydrology characterization addresses Section 5.0 of the latest DOE 1020 technical standard (DOE, 2012). The basin peak flow values were in turn used in the Water Surface Profile Computations (WSPRO) (Ameson and Shearman, 1998) computer code to calculate the flood water elevations associated with each return period and location of interest.

Note that in C99 and C00, flood hazard curves for C-, F-, E-, H-, S-, and Z-Areas (Upper Three Runs and Fourmile Branch basins) were estimated for both the 6-hr and 24-hr accumulation periods. For A-Area (Tims Branch basin), K-Area (Pen Branch basin), and L-Area (L-Lake), however, flood hazard curves were estimated for the 24-hr accumulation period only. In the current study, flood hazard curves for both the 6-hr and 24-hr accumulation periods are estimated for all areas of interest.

2.1 Design Hyetographs for SRS

In W98, the extreme rainfall amounts for 6- and 24-hour accumulation periods were calculated as a function of return period based on historical precipitation data at or near SRS. For a six-hour storm, the total accumulated rainfall must be partitioned into an hourly time series, done with a standard fractional allocation. In C99, the hourly rainfall for a given return period storm at SRS is calculated as:

$$I_{ij} = aF_iR_j \qquad (1)$$

where:

I_{ij} = rainfall (inches) in hour "i" (i=1, 6) and for j-year return period,

R_j = total six- hour storm rainfall (inches) for j-year return period, obtained from W98,

F_i = fraction of rainfall in hour "i" for a six-hour storm, obtained from Hunter (1999), and

a = 0.53, conversion factor from point rainfall to regional average rainfall, estimated from Hunter (1999).

For the 24 hour accumulation, C00 expressed this as:

$$I_{ij} = F_iR_j \qquad (2)$$

where:

I_{ij} is the rainfall in hour i (i=1, 24) for return period j (in years),

R_j is the 24-hour accumulated precipitation for the j-year return period (from W98), and

F_i is the fraction of the total precipitation that falls during hour i (from Hunter (1999)).

It is noted that for the longer duration event, a conversion factor was not required.

For the current report, updated return period precipitation data from the W13 report were used as R_j in Equations 1 and 2. As in C99/00, this was used to create 6-hr and 24-hour time series of rainfall for each return period (I_{ij}) (Fig. 1), and this served as input to the HEC-HMS model. The hyetograph for the 6-hr and 24-hr accumulation periods are shown in Tables 1 and 2, respectively. The simulated flow data was then run with WSPRO to obtain flood elevation data for various basins. Use of the C99/00 reports, as well as the update to site characterization of precipitation addresses portions of Sections 5.2.1 and 5.2.2 of the DOE 1020 technical standard (DOE, 2012).

As seen in Fig. 1, precipitation return values from the W13 report are substantially lower than those from the W98 report, particularly for the long return periods. Therefore, it is expected that flow (peak discharge) and flood levels will be correspondingly lower as well.

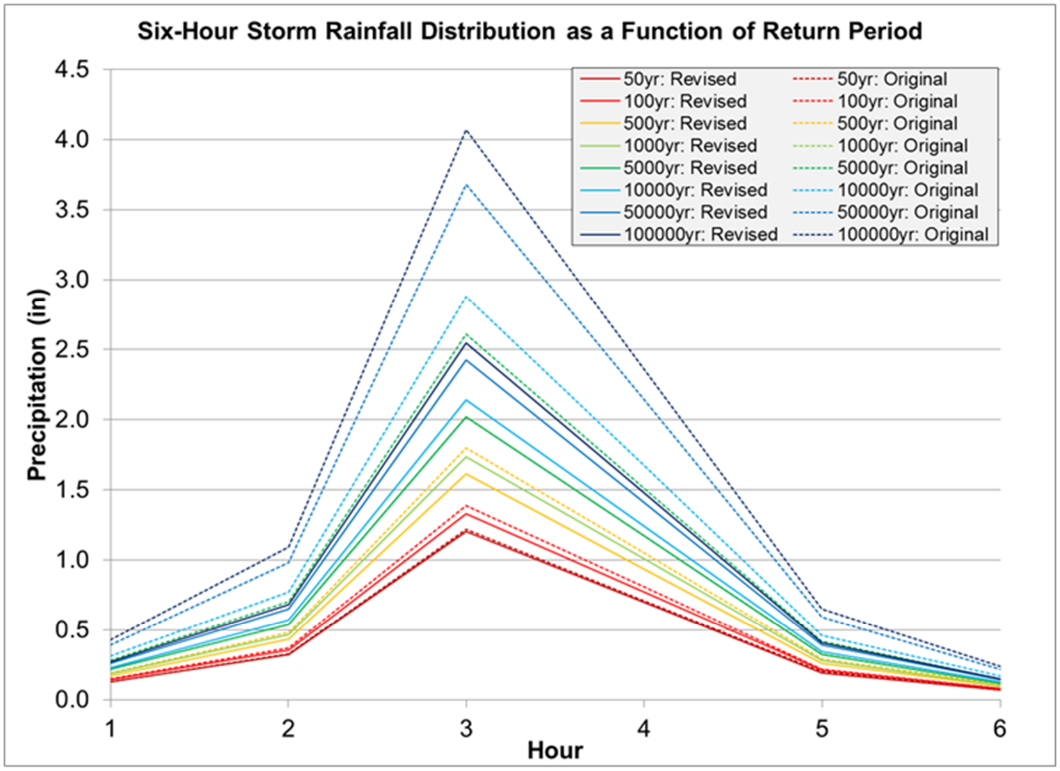
Table 1: 6-hour storm rainfall distribution as a function of return period. Note that the same proportion by hour was used as in C99.

	Hourly Precipitation						
Return Period	1	2	3	4	5	6	Total
Years	in	in	in	in	in	in	in
50	0.128	0.322	1.205	0.699	0.194	0.071	2.619
100	0.141	0.355	1.328	0.771	0.214	0.078	2.886
500	0.172	0.431	1.612	0.936	0.259	0.095	3.505
1,000	0.185	0.464	1.734	1.007	0.279	0.102	3.770
5,000	0.215	0.540	2.018	1.171	0.325	0.118	4.387
10,000	0.228	0.572	2.140	1.242	0.344	0.126	4.653
50,000	0.258	0.648	2.424	1.407	0.390	0.142	5.269
100,000	0.271	0.681	2.546	1.478	0.410	0.149	5.534

Table 2: 24-hour storm rainfall distribution as a function of return period. Note that the same proportion by hour was used as in C00

	Return Period (years)							
	50	100	500	1000	5000	10000	50000	100000
HOUR	in	in	in	in	in	in	in	in
1	0.030	0.033	0.039	0.042	0.049	0.052	0.058	0.061
2	0.054	0.059	0.071	0.076	0.088	0.093	0.105	0.110
3	0.072	0.079	0.095	0.102	0.117	0.124	0.140	0.147
4	0.210	0.230	0.276	0.296	0.343	0.363	0.409	0.429
5	0.341	0.374	0.450	0.482	0.558	0.590	0.666	0.698
6	0.455	0.499	0.600	0.643	0.744	0.787	0.888	0.931
7	0.629	0.689	0.829	0.889	1.028	1.088	1.227	1.286
8	1.617	1.772	2.131	2.285	2.643	2.797	3.154	3.308
9	0.988	1.083	1.302	1.396	1.615	1.709	1.927	2.022
10	0.545	0.597	0.718	0.770	0.891	0.943	1.063	1.115
11	0.359	0.394	0.473	0.508	0.587	0.621	0.701	0.735
12	0.293	0.322	0.387	0.415	0.480	0.508	0.572	0.600
13	0.102	0.112	0.134	0.144	0.166	0.176	0.199	0.208
14	0.066	0.072	0.087	0.093	0.108	0.114	0.128	0.135
15	0.042	0.046	0.055	0.059	0.069	0.073	0.082	0.086
16	0.030	0.033	0.039	0.042	0.049	0.052	0.058	0.061
17	0.030	0.033	0.039	0.042	0.049	0.052	0.058	0.061
18	0.024	0.026	0.032	0.034	0.039	0.041	0.047	0.049
19	0.024	0.026	0.032	0.034	0.039	0.041	0.047	0.049
20	0.018	0.020	0.024	0.025	0.029	0.031	0.035	0.037
21	0.018	0.020	0.024	0.025	0.029	0.031	0.035	0.037
22	0.018	0.020	0.024	0.025	0.029	0.031	0.035	0.037
23	0.012	0.013	0.016	0.017	0.020	0.021	0.023	0.025
24	0.012	0.013	0.016	0.017	0.020	0.021	0.023	0.025
Total	5.988	6.563	7.891	8.462	9.787	10.358	11.682	12.252

a)



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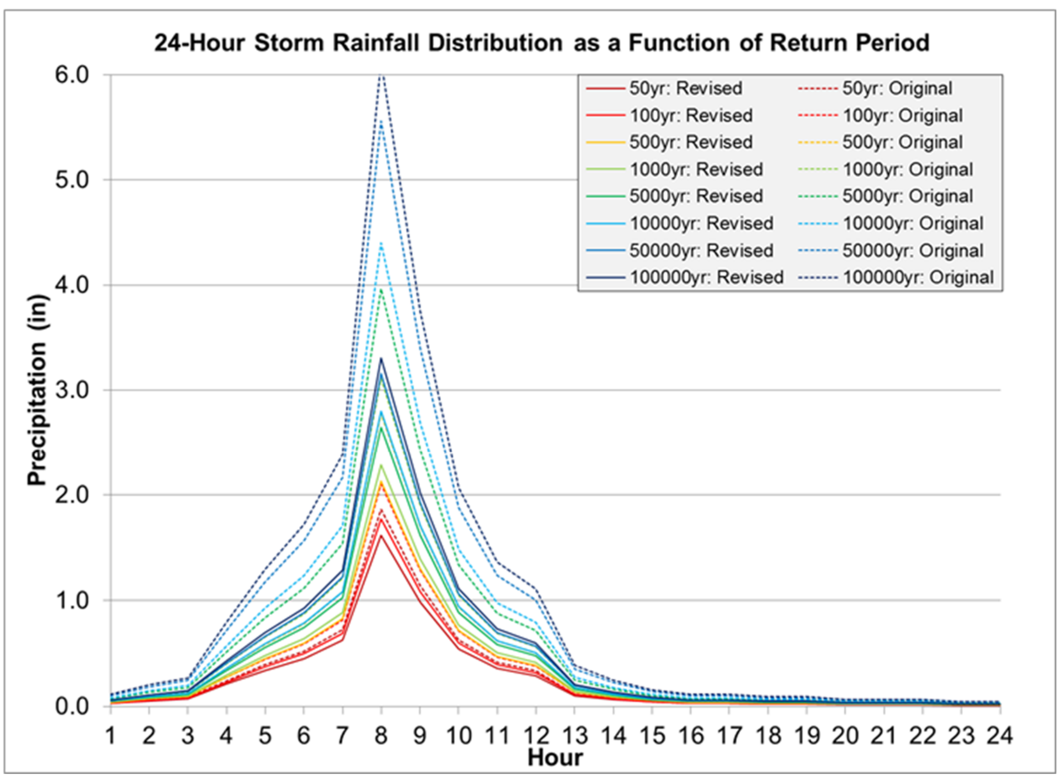


Figure 1: Hyetographs calculated from the W13 data for (a) 6-hr distribution, (b) 24-hr distribution. Note that dashed lines represent distributions used in C99 and C00, respectively.

2.2 The HEC-HMS Model

HEC-HMS is a hydrologic modeling system developed by the US Army Corps of Engineers, Hydrologic Engineering Center, to model flood hydrology through precipitation and runoff simulations. The HEC-HMS requires both precipitation data and prescribed model parameters (i.e., losses, runoff transformation and base flow) characterizing the properties of the basin, and the output is the basin runoff discharge. (In this report, "basin runoff discharge" means the total volumetric flow rate in the creek, stream, or river.)

The optimum parameter values of importance are determined by simulating past flows and setting model values so that the HEC-HMS output runoff discharge matches the measured runoff discharge for selected historical storm events. This was done in C99/00 by selecting a few representative storm events and obtaining the associated precipitation and flow data. Rainfall data is recorded at 13 rain gauge stations distributed inside the SRS (Figure 2a). Measurements are taken once a day (usually at 6 AM), except for the rain gauge at the Central Climatology Facility (CLM), where it is recorded automatically every 15 minutes. C99/00 then used this 15 minute data to partition the daily precipitation data into hourly increments to obtain a time series of precipitation for each event. For each storm, the measured hourly flows were provided by the United States Geological Survey (USGS), Columbia, SC District. The USGS previously utilized a network of monitoring stations at strategic locations on the Savannah River and SRS streams (Figure 2a) to measure the flows and stage highs. The various basins of interest and associated topography are illustrated in Figure 2b.

HEC-HMS requires basin drainage area, loss rate, transformation, and base flow for the basin in question. C99/00 used established sources to get the basin drainage area (Cooney et al., 1995) and the area within each basin impervious to rain infiltration (using the ArcView GIS system from Environmental Systems Research Institute, (ESRI, 2003)). C99/00 then set the basin-specific parameters so as to match the simulated flow (run with the historic storm data) to the observed flow - the parameters for loss were adjusted to match the measured peak flow, those for the runoff transformation model were adjusted

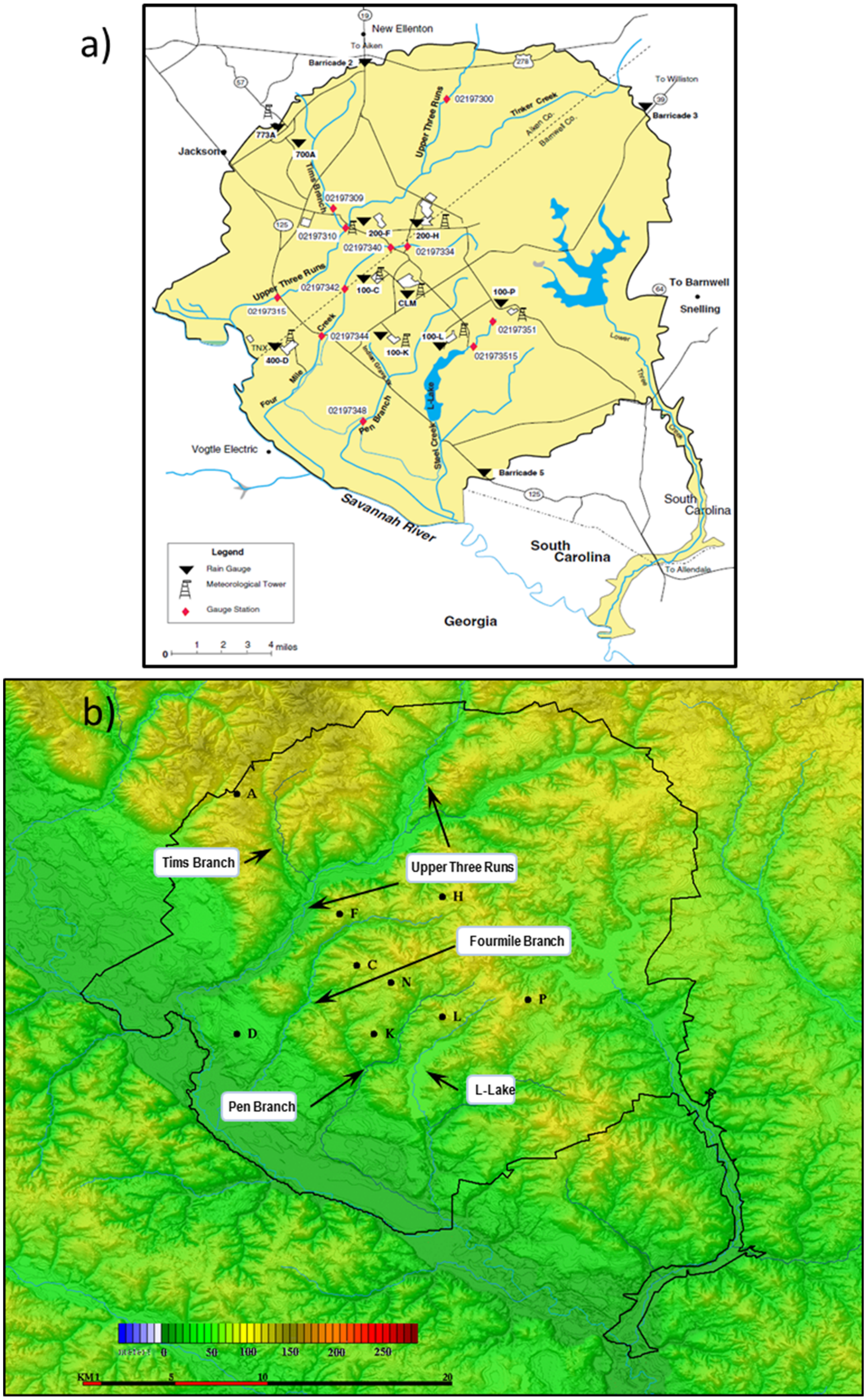


Figure 2: (a) Savannah River Site (SRS) map showing location of gauge stations and meteorological towers. (b) SRS map showing topographic elevation relative to various basins of interest.

to match the measured shape of the hydrograph, and those for the base flow model were adjusted to match the measured base flow. The resulting parameters were then used by HEC-HMS to calculate basin peak flow using the precipitation hyetographs from the W98 report. Example input parameters for Upper Three Runs Basin and Fourmile Branch Basin are given in Tables 2 and 3 of C99.

2.3 WSPRO

These peak flows were then used in WSPRO (Ameson and Shearman, 1998) to calculate the flood water elevations associated with each return period. WSPRO was developed by the United States Geological Survey (USGS) for the Federal Highway Administration, and uses a step-backwater analysis method to calculate water surface elevations for one-dimensional, gradually-varied, steady flow under bridges, and overtopping of embankments. The Culvert Analysis Program (CAP, Fulford 1998), developed by the United States Geological Survey (USGS) was also used to account for changes in elevation due to the presence of bridges and culverts (see Section 5.3.2 of the DOE 1020 technical standard (DOE, 2012)). The data required for WSPRO are flow, boundary condition, channel geometry and losses, and hydraulic characteristics of the bridges and road crossings. WSPRO calculates the elevation at numerous cross-sections along the basin, using detailed channel geometry available from prior studies (Lanier 1996). The use of detailed basin geometry satisfies Section 5.2.4 of the DOE 1020 technical standard (DOE, 2012). This two-step process – using HEC-HMS to generate peak discharge rates, and then using WSPRO (and CAP) to obtain the elevation – was applied to various sub-basins within the SRS in C99 and C00, and is repeated here using the updated precipitation data.

3.0 Modeling Results

3.1 Upper Three Runs Basin

Upper Three Runs is the longest and northernmost system in SRS and has a drainage area of over 195 square miles. The main channel flows in a southwesterly direction until it empties into the Savannah River (Fig. 2a). Three main tributaries are the Tinker Creek, McQueen Branch, and Tims Branch. SRS facilities within the Upper Three Runs basin include B -, M-, A-, F-, H-, S-, and Z-Areas. Upper Three Runs is gauged near Highway 278 (station 02197300), at SRS road C (station 02197310), and at SRS Road A (station

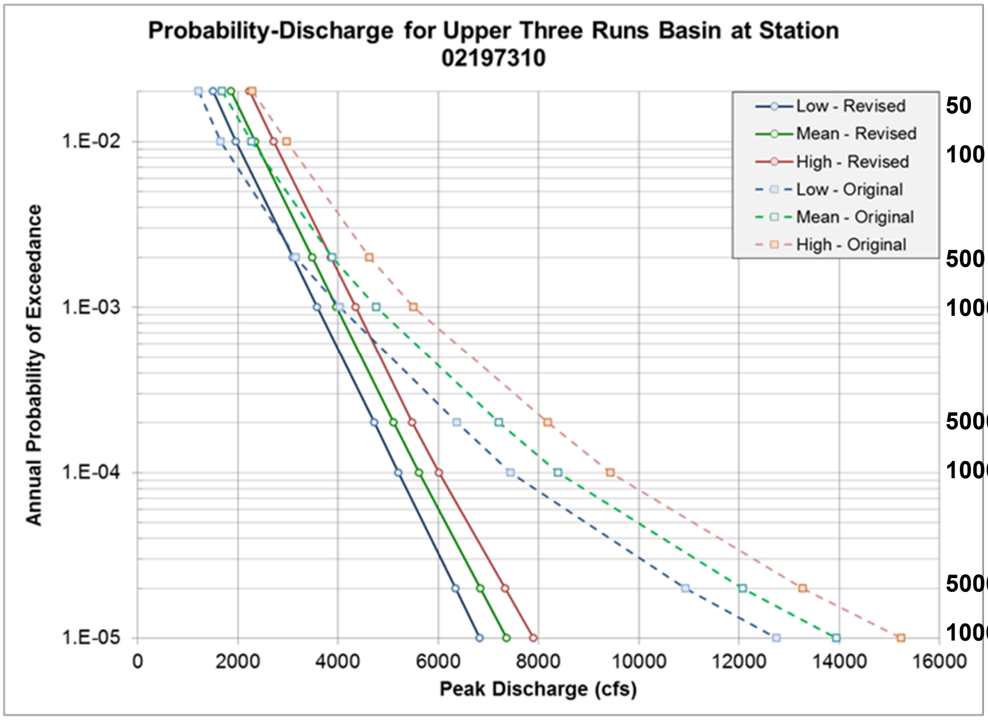
02197315) (Figure 2a). There are six highway bridges and two railway bridges that cross Upper Three Runs. In addition, there are six power-line roads near the channel, but these do not cross it. Upper Three Runs differs from the other five onsite streams in two respects: it is the only stream with headwaters arising outside the SRS, and it is the only stream that has never received heated discharges of cooling water from the production reactors.

Three storm events were used to determine the HEC-HMS input parameters that characterize the Upper Three Runs basin - March 29, 1991; January 6, 1995; and May 3, 1997 (C99). These selected storms were isolated events, and there was no rainfall for several days before or after each event. The average rainfall for each storm for Upper Three Runs basin was determined from the average measured rainfall from the six rain gauges that cover the basin (773A, Barricade 2, 700A, 200-F, 200-H, and Barricade 3). These storms represent typical winter, spring and summer storm events that occur at the SRS.

C99 selected the HEC-HMS input parameters for the Upper Three Runs basin to match the simulated to the measured flows at gauge station 02197310 for the three selected storm events. The parameters for loss rate and base flow varied for the different storm events because of the differences in ground soil and ground water conditions at the time of the events. These same three parameter sets are used with the current simulations.

Three sets of peak flows (1 per parameter set) at station 02197310 for each return period were calculated by HEC-HMS using the 6-hour hyetographs derived from Eq. 1, which were based on the W13 data in Table 1. The highest, the lowest, and the average of the three sets of the calculated peak flows are presented in Figure 3a. Note that annual probability of exceedance is the inverse of the return period (e.g. a probability of exceedance of 2×10^{-2} equals a 50 year return period, and a probability of exceedance of 1.0×10^{-5} equals a 100,000 year return period). The calculated 100-year return flood at station 02197310 varies from 1964 to 2723 cfs, while the 100,000-year return flood at that station varies between 6833 cfs and 7900 cfs. These can be compared to the C99

a)



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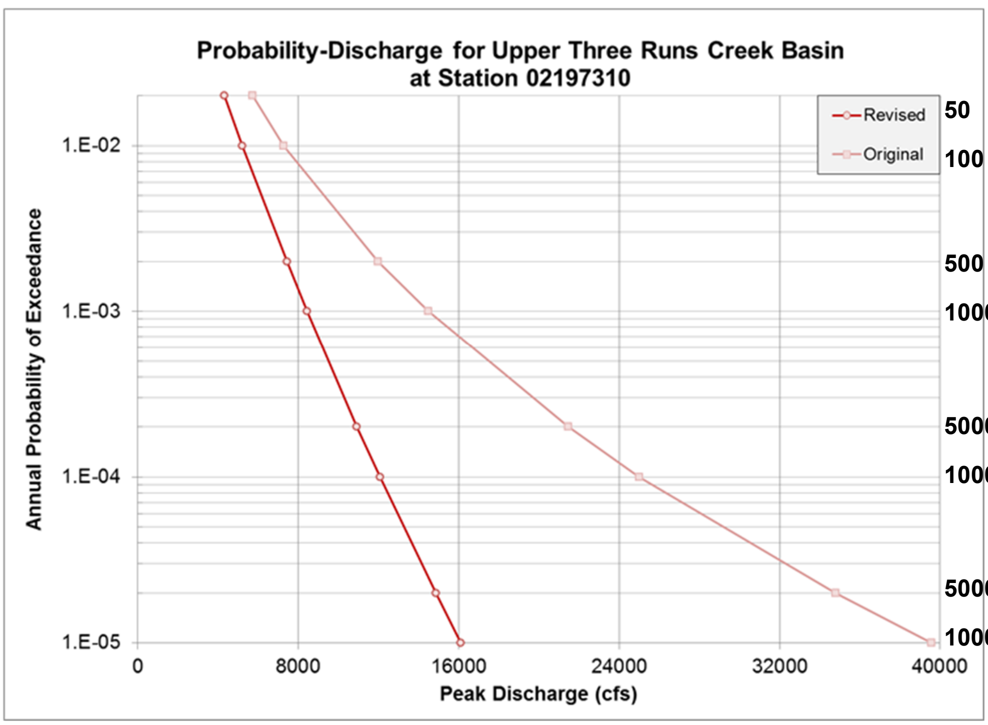


Figure 3: Probability of exceedance versus peak discharge for a) 6-hour accumulation, and b) 24 hour accumulation for the Upper Three Runs basin. Data for the prior C99 and C00 studies are shown in addition to the current estimates. Note that annual probability of exceedance is the inverse of the annual return period in years (i.e. the range presented here is 50 to 100,000 years).

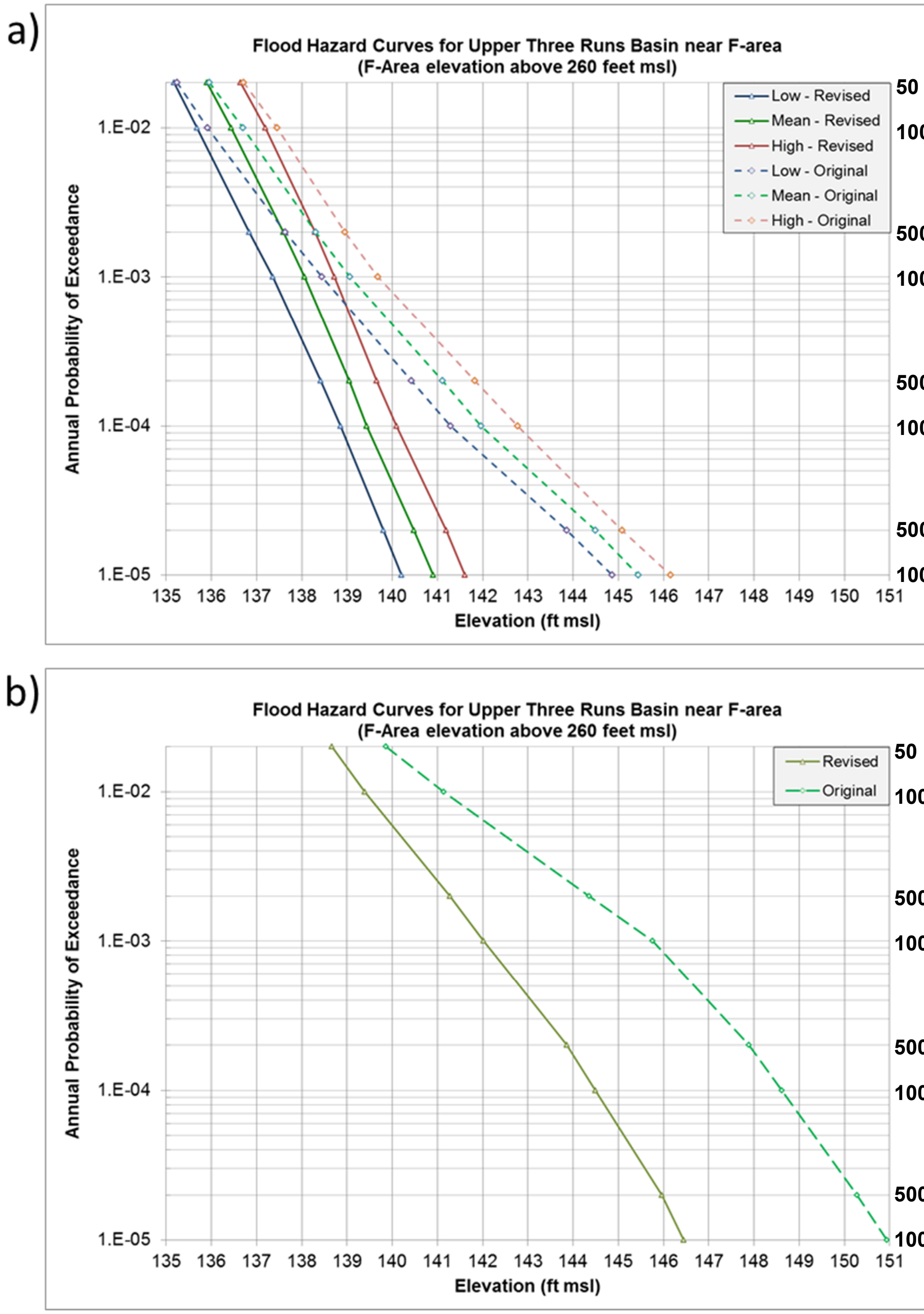
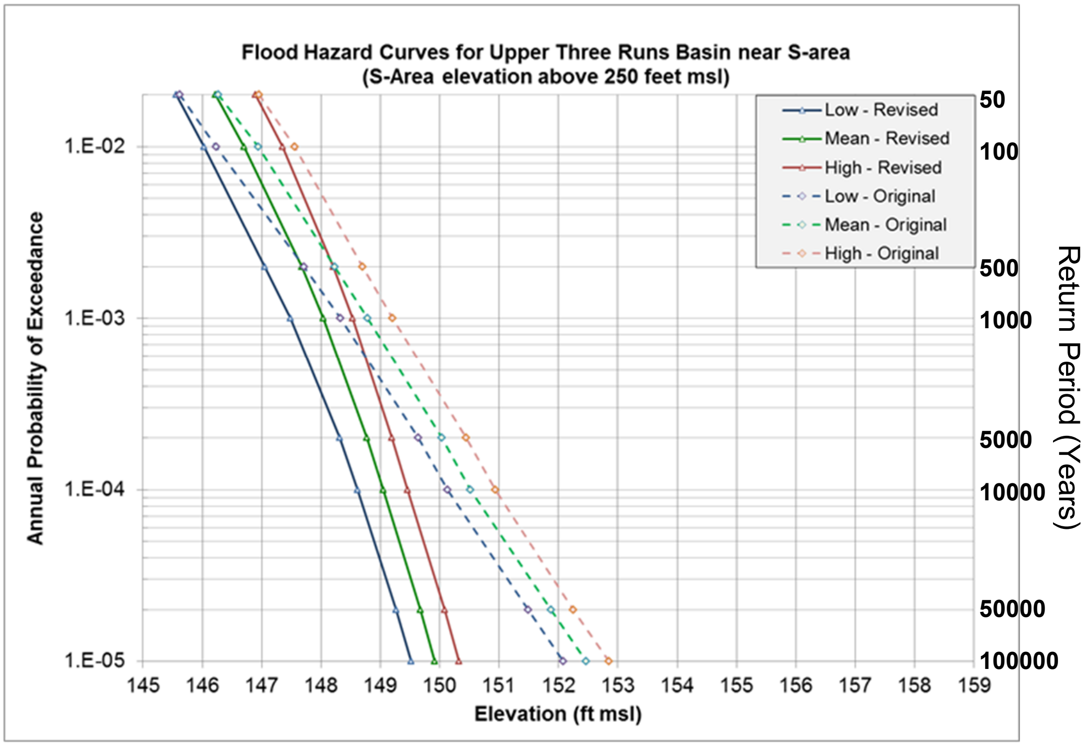


Figure 4: Flood hazard curves for Upper Three Runs basin near F-area for a) 6-hour accumulation, and b) 24-hour accumulation. Data for the prior C99 and C00 studies are shown in addition to the current estimates. The F-area elevation is above 260 feet msl.

a)



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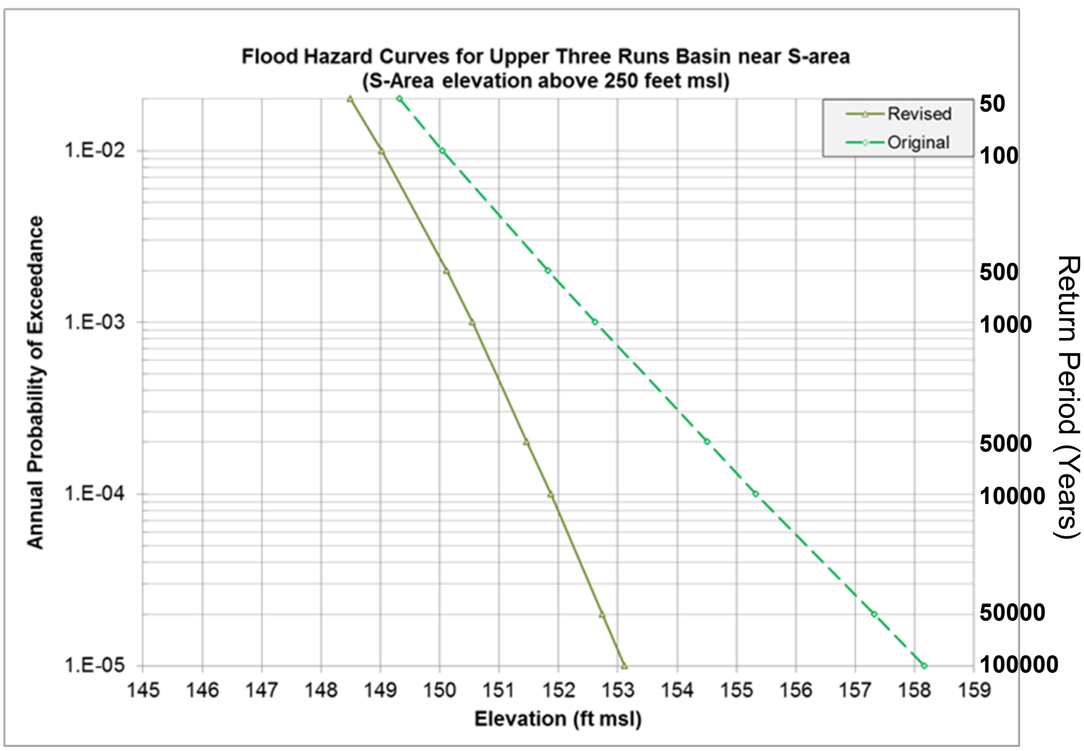


Figure 5: As in Fig. 4, but for S-area. The S-area elevation is above 250 feet msl.

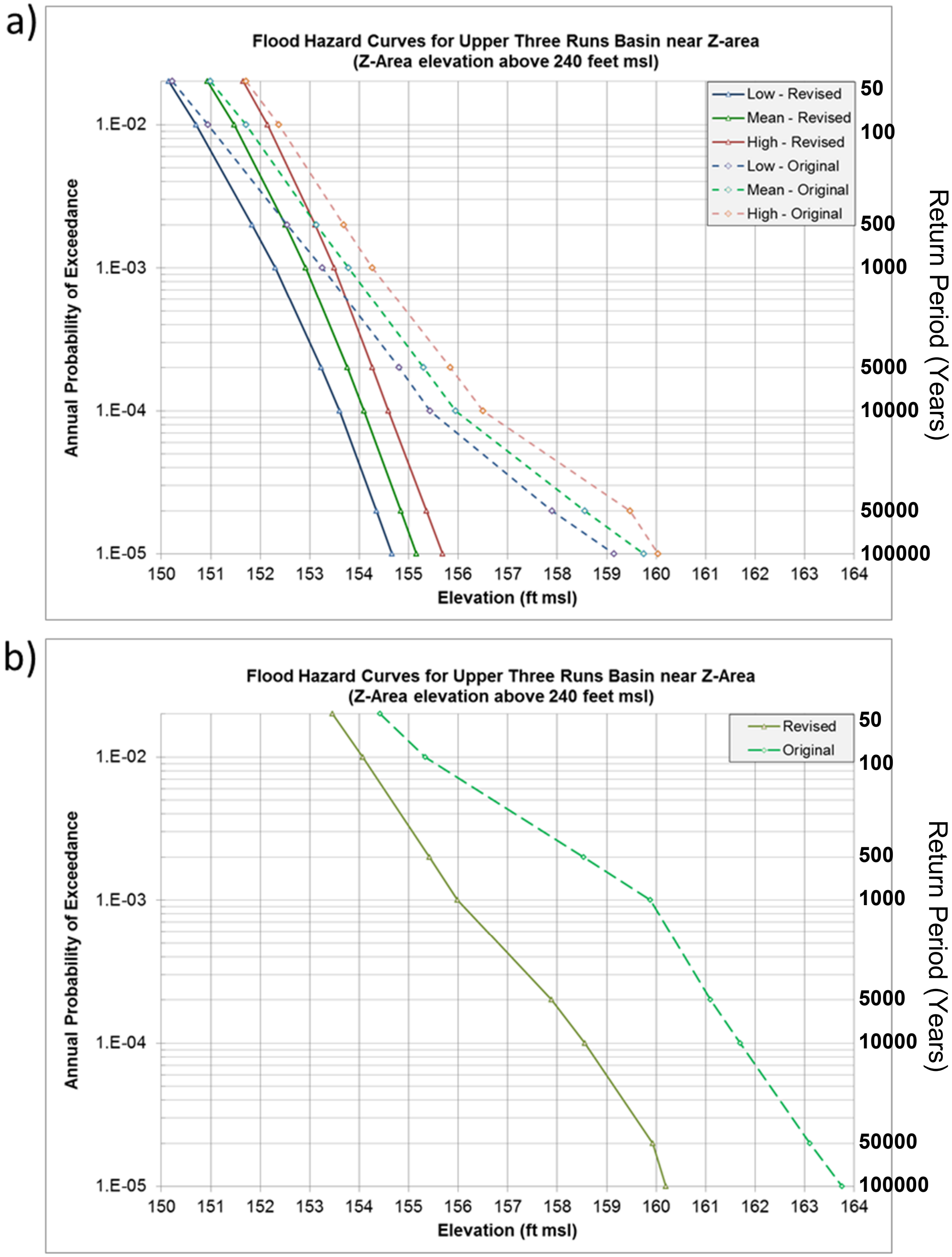


Figure 6: As in Fig. 4, but for Z-area. The Z-area elevation is above 240 feet msl.

values of 1660 to 2972 cfs for the 100-year return period, and 12756 cfs and 15241 cfs for the 100,000-year period. For the 24-hr accumulation, only the highest calculated peak flows were used in the resulting calculations (Fig. 3b). The calculated 100- year return flood at station 02197310 is at 5197 cfs, while the 100,000-year return flood at that station is 16120 cfs. This is much lower than the corresponding C00 values of 7269 and 39576 cfs, respectively.

The flood elevations of the Upper Three Runs basin for various flows were calculated by the WSPRO computer code. For a 100,000-year return 6-hour flood, the maximum calculated flood elevations at F-, S-, and Z- Areas are approximately 142 (Fig. 4a), 150 (Fig. 5a) and 156 feet (Fig. 6a) above mean sea level (msl), respectively. The elevations of F-, S-, and Z- Areas are above 260, 250 and 240 feet msl, respectively. Therefore, the chances of flooding for those facilities would be very small, similar to the conclusion from C99. If we instead do the calculations for a 24-hour accumulation, the calculated flood elevations at F-, S-, and Y- and Z- Areas are 146 (Fig. 4b), 153 (Fig. 5b) and 160 feet (Fig. 6b) msl, respectively, all well below flood level at those locations.

3.2 Fourmile Branch Basin

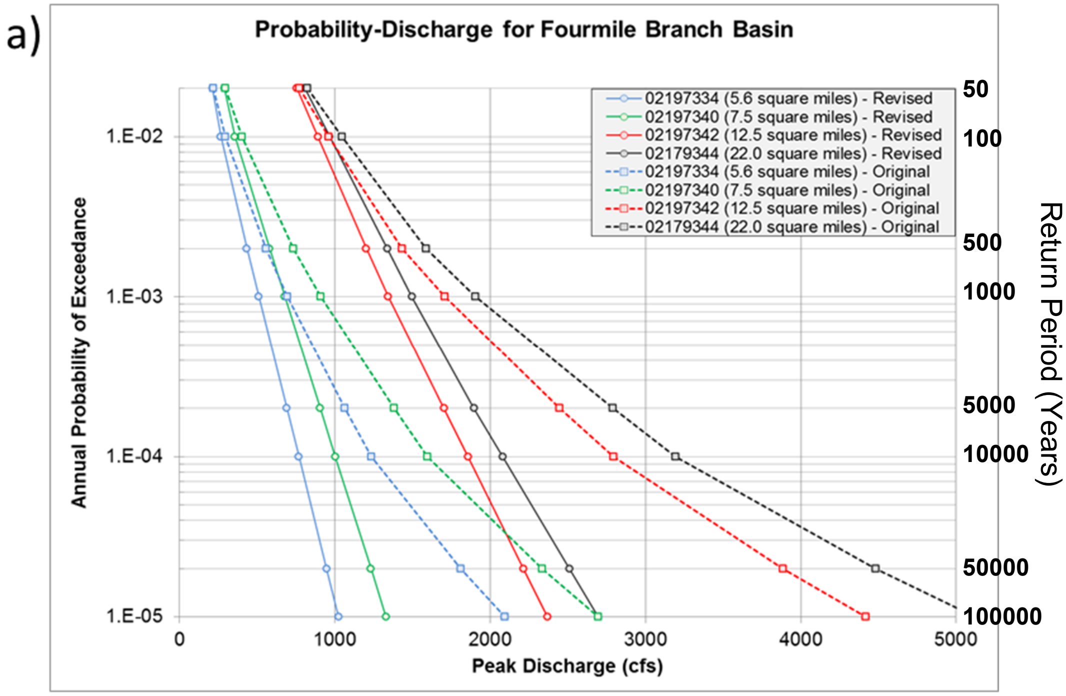
The Fourmile Branch basin has about 23 square miles of drainage area, including much of F-, H-, and C-Areas (Fig. 2a). The stream flows to the southwest into the Savannah River swamp and then into the Savannah River, and the banks vary from gently sloping to fairly steep. The floodplain is up to 1,000 feet wide. Fourmile Branch receives effluents from F-, H-, and C-Areas, from a groundwater plume from the Burial Ground and F and H seepage basins, and, until June 1985, received large volumes of cooling water from the production reactor in C Area. Figure 2a shows the gauge stations 02197334, 02197340, 02197342, and 02197344 on the river. There are four highway bridges, one railway bridge, five culvert crossings, and ten breached dams or road beds that cross Fourmile Branch.

Because C99 found that the effect of variations in basin infiltration on flood elevation is very small in comparison to the flood margin, only one storm event (January 6, 1995 storm) was used to develop the Fourmile Branch basin runoff characteristics (obviating the need for high/low/average curves). The January 6, 1995 storm was chosen because it had the highest rainfall intensity and the largest accumulated rainfall. C99 used the averaged measured rainfall from the six rain gauges (200-H, 200-F, 100-C, CLM, 100-K and 400-D) that cover the Fourmile Branch basin. The basin was divided into four sub-basins at stream gauge stations 02197334, 02197340, 02197342 and 02197344. As for the Upper Three Runs basin, the sub-basin properties (HEC- HMS input parameters) were determined by matching the model hydrographs with the measured hydrographs at the four gauge stations.

With the W13 data, the peak flow for various return periods of 6-hour accumulation was calculated for the four stream gauges (Fig. 7a). The peak discharge at the 100-year return period varies from 265 cfs for gauge 02197334 to 967 cfs for gauge 02179344, while the 100,000-year return period discharge varies from 1025 cfs for gauge 02197334 to 2695 cfs for gauge 02179344. The corresponding values from C99 are much higher: 2096 cfs for gauge 02197334 to 5121 cfs for gauge 02179344 (assuming a 100,000-year return period). When we assume a 24-hour discharge (Fig. 7b), the 100-year return period varies from 587 cfs for gauge 02197334 to 1864 cfs for gauge 02179344, while the 100,000-year return period discharge varies from 1929 cfs for gauge 02197334 to 5515 cfs for gauge 02179344.

The WSPRO basin models of the Fourmile basin (C99) were then rerun using these updated peak discharge values. Similar to Upper Three Runs basin, prior channel geometry data for Fourmile Branch were used as input to WSPRO (Lanier 1997). To accommodate the presence of culverts and bridges, the Culvert Analysis Program (CAP) was used as well. Fourmile Branch was subdivided into 5 separate components, and least square curve fits were used to find the values of elevation most closely matching the peak discharge values for a particular cross-section. For a six-hour accumulation period, the calculated annual probability of 1×10^{-5} (100,000-year return) flood elevation at C-Area is 189 feet msl (Fig. 8a), F-Area is 193 feet msl (Fig. 9a), E-Area is 201 feet msl (Fig. 10a), and H-Area is 236 feet msl (Fig. 11a). For the 24-hr accumulation period, the calculated annual probability of 1×10^{-5} flood elevation at C-Area is 190 feet msl (Fig. 8b), F-Area is 194 feet msl (Fig. 9b), E-Area is 203 feet msl (Fig. 10b), and H-Area is 237 feet msl (Fig. 11b). The elevations of C-, F-, E-, and H-Areas are 280, 260, 280, and 270 feet above msl, respectively. Therefore, the chances of flooding the facilities at C-, F-, E-, and H-

Areas would be significantly less than 1×10^{-5} per year, in keeping with the conclusion of C99 and C00.



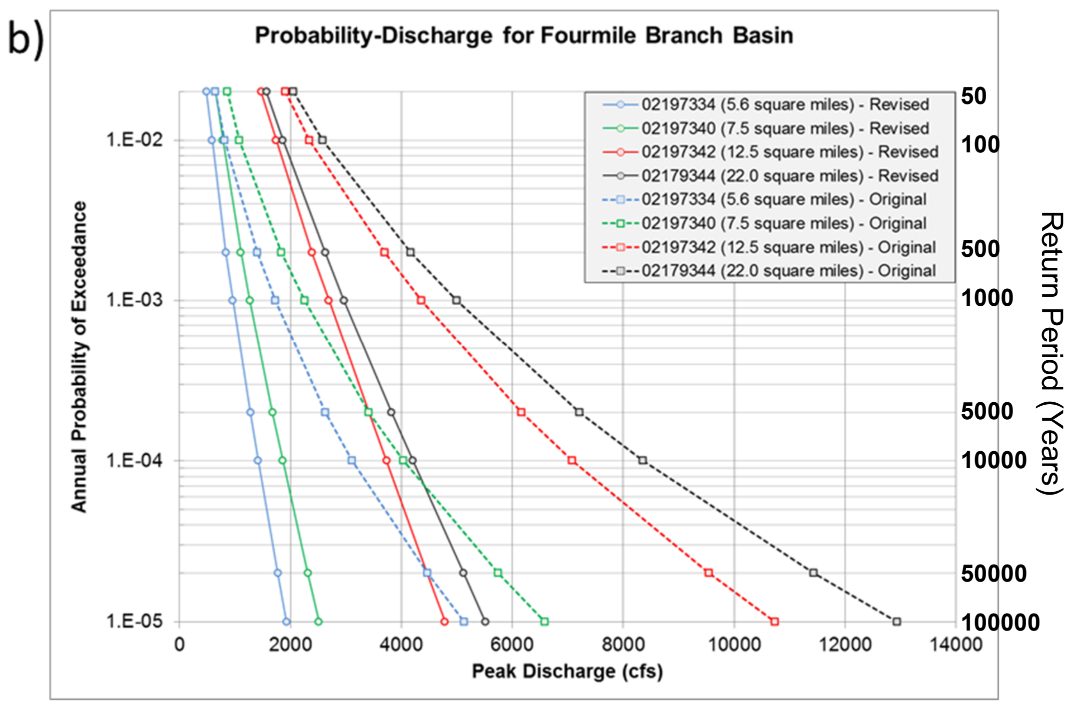


Figure 7: Probability of exceedance versus peak discharge for a) 6-hour accumulation, and b) 24 hour accumulation for the Fourmile basin at different gauge stations. Data for the prior C99 and C00 studies are shown in addition to the current estimates.

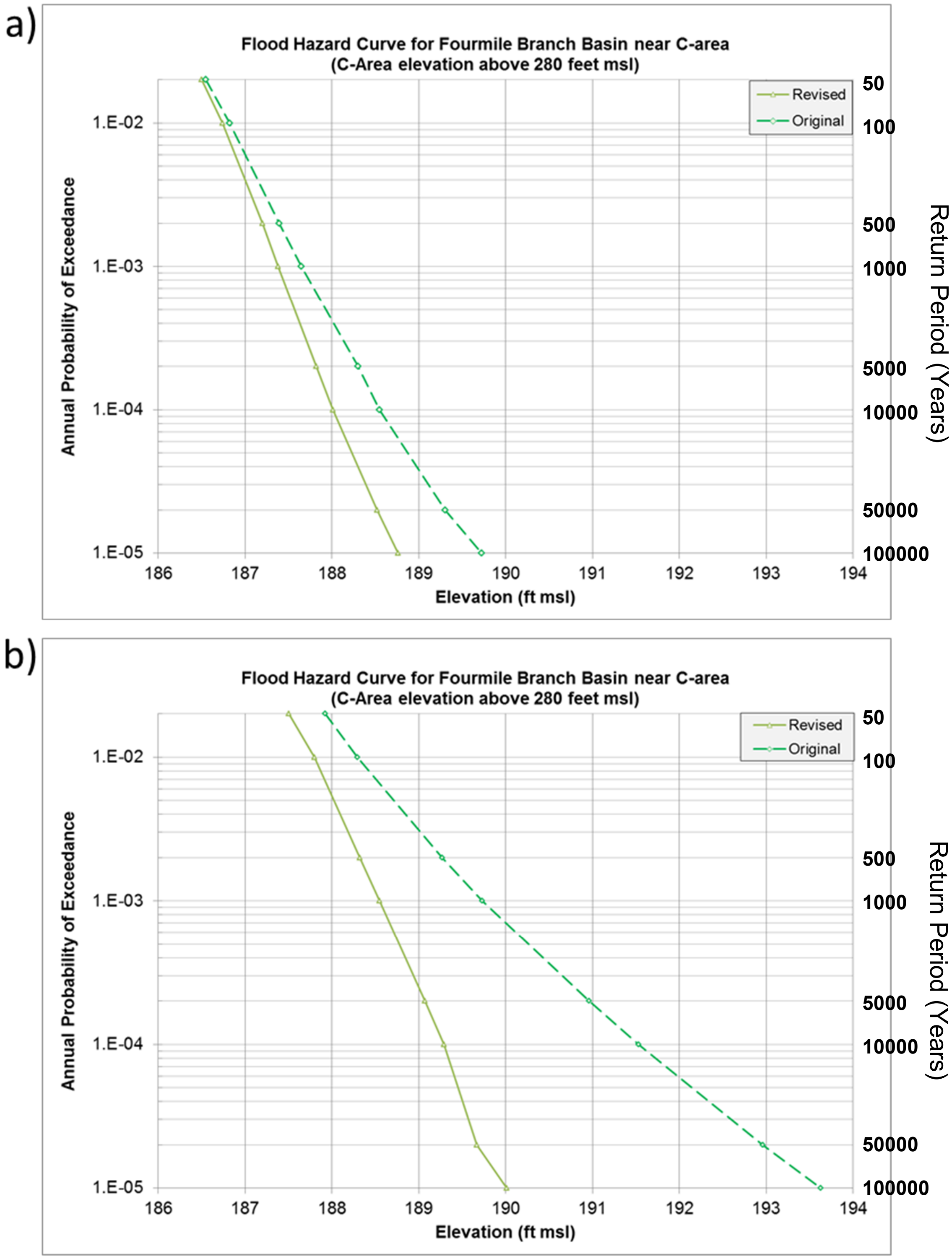


Figure 8: Flood hazard curves for Fourmile basin near C-area for a) 6-hour accumulation, and b) 24-hour accumulation. Data for the prior C99 and C00 studies are shown in addition to the current estimates. The C-area elevation is above 280 feet msl.

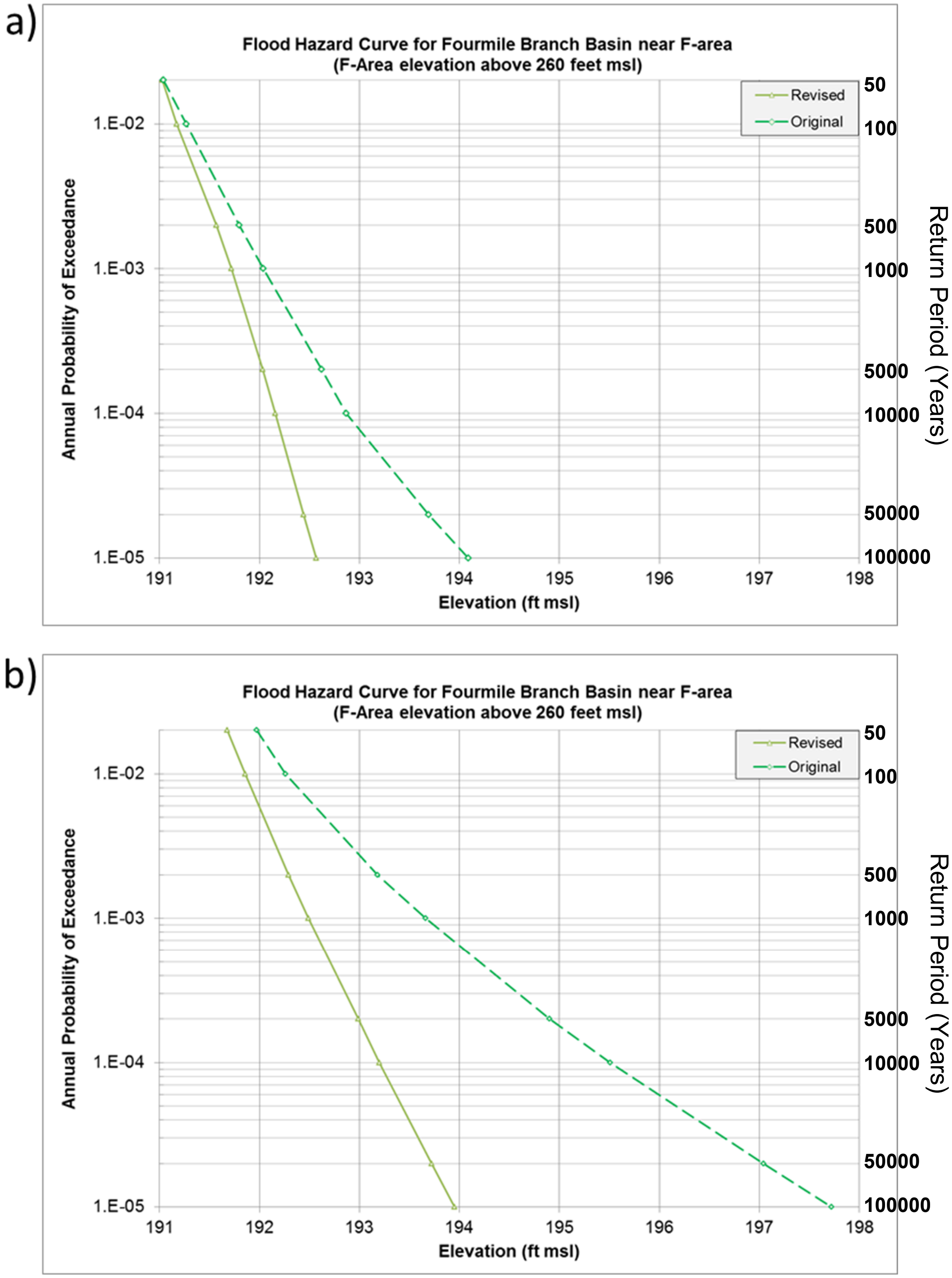


Figure 9: As in Fig. 8, but for F-area. The F-area elevation is above 260 feet msl.

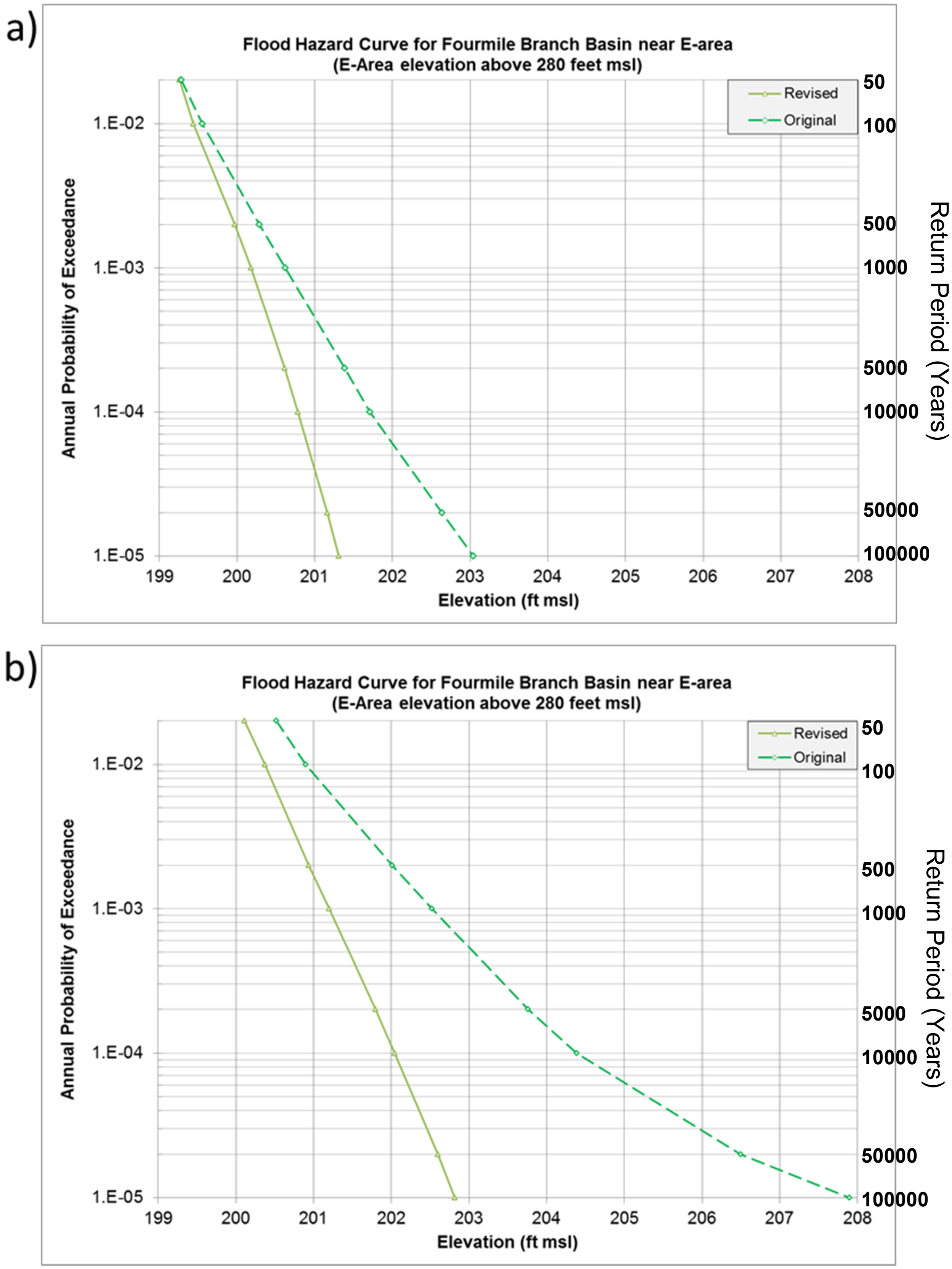


Figure 10: As in Fig. 8, but for E-area. The E-area elevation is above 280 feet msl.

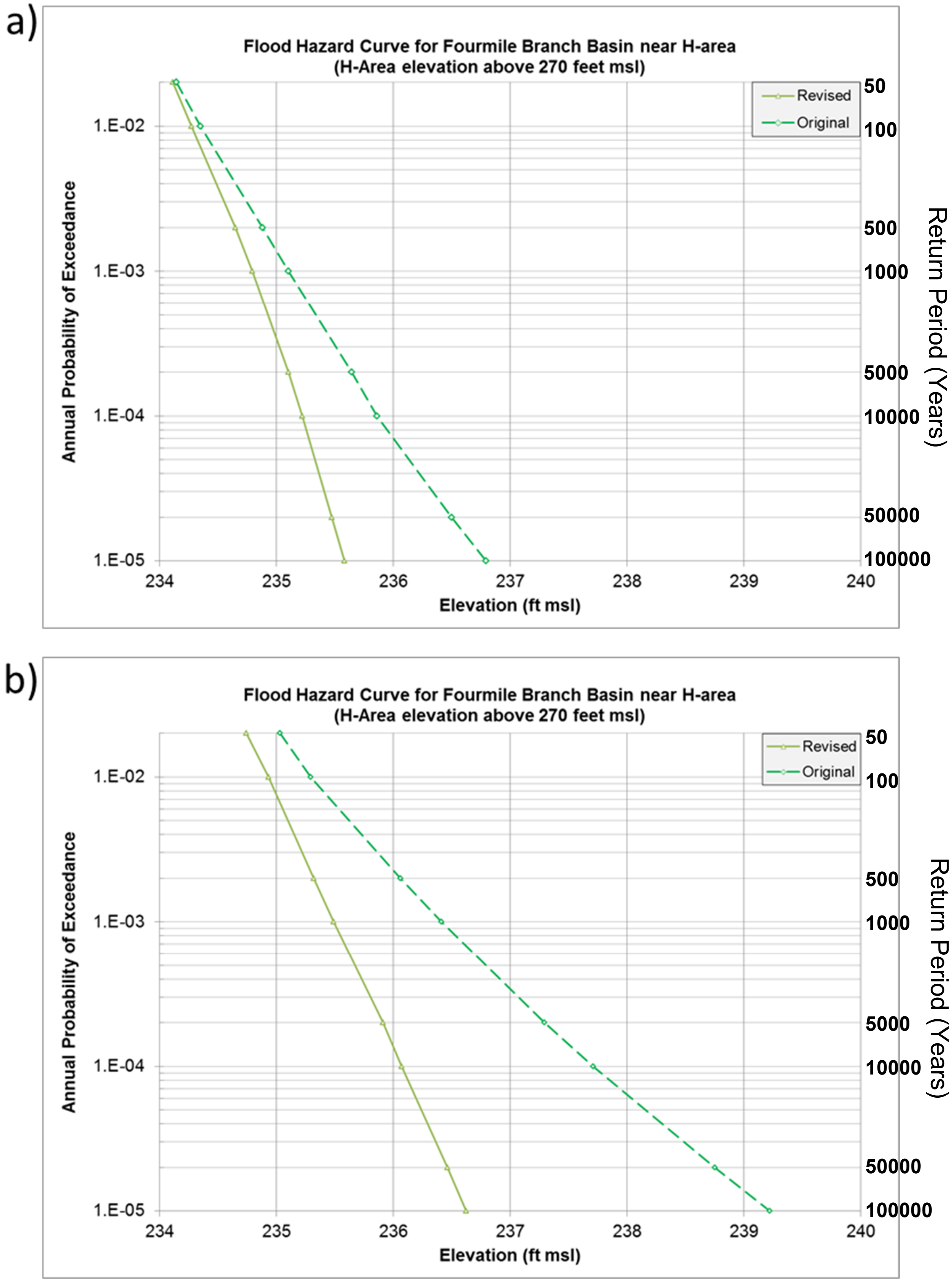


Figure 11: As in Fig. 8, but for H-area. The H-area elevation is above 270 feet msl.

3.3 Tims Branch Basin

The Tims Branch drainage basin is about 18.86 square miles, most of which lies within SRS (Fig. 2). Tims Branch drains much of the M- and A- areas, flows south-southeast into Upper Three Runs Creek, and has an elevation gradient ranging from 10 to 30 ft/mile. The valley is V- shaped with the sides varying from fairly steep to gently sloping and the floodplain is up to 1,000 feet wide. Water flow measurements were recorded on Tims Branch near the confluence of Tims Branch with Upper Three Runs Creek (station 02197309) between March 1974 and November 1982 and from May 1984 through 1999.

Based on historical available storm and flow records, a storm event that occurred on March 29, 1991 was used to determine the HEC-HMS input parameters that best characterize the Tims Branch basin (C99). The average of the measured rainfall for a given storm event from the four rain gauges (773A, Barricade 2, 700A, and 200-F) that cover the Tims Branch basin was taken to be the average rainfall of that storm for the basin. (Note that calculations for the 6-hr accumulation were not generated in C99).

With the W13 data, the peak flow for various return periods of 6- and 24-hour accumulations was calculated (Fig. 12). The peak discharge for a 6-hour accumulation and 100-year return period is 515 cfs for gauge 02197309, while the 100,000-year return period discharge is 1615 cfs (Fig. 12a). Similarly, for the 24-hr accumulation (Fig. 12b), the peak discharge for the 100 and 100,000-year return periods is 995 and 3268 cfs, respectively. Again, these are substantially lower than the C00 values. For a 100,000-year return flood, the calculated flood elevation at A-Area is 245 and 246 feet above mean sea level (msl) (Fig. 13) for the 6-hr and 24-hr accumulation periods, respectively. The elevation of A-Area is above 350 feet msl. Therefore, the probability of flooding for A-Area is significantly less than 1×10-5 per year.

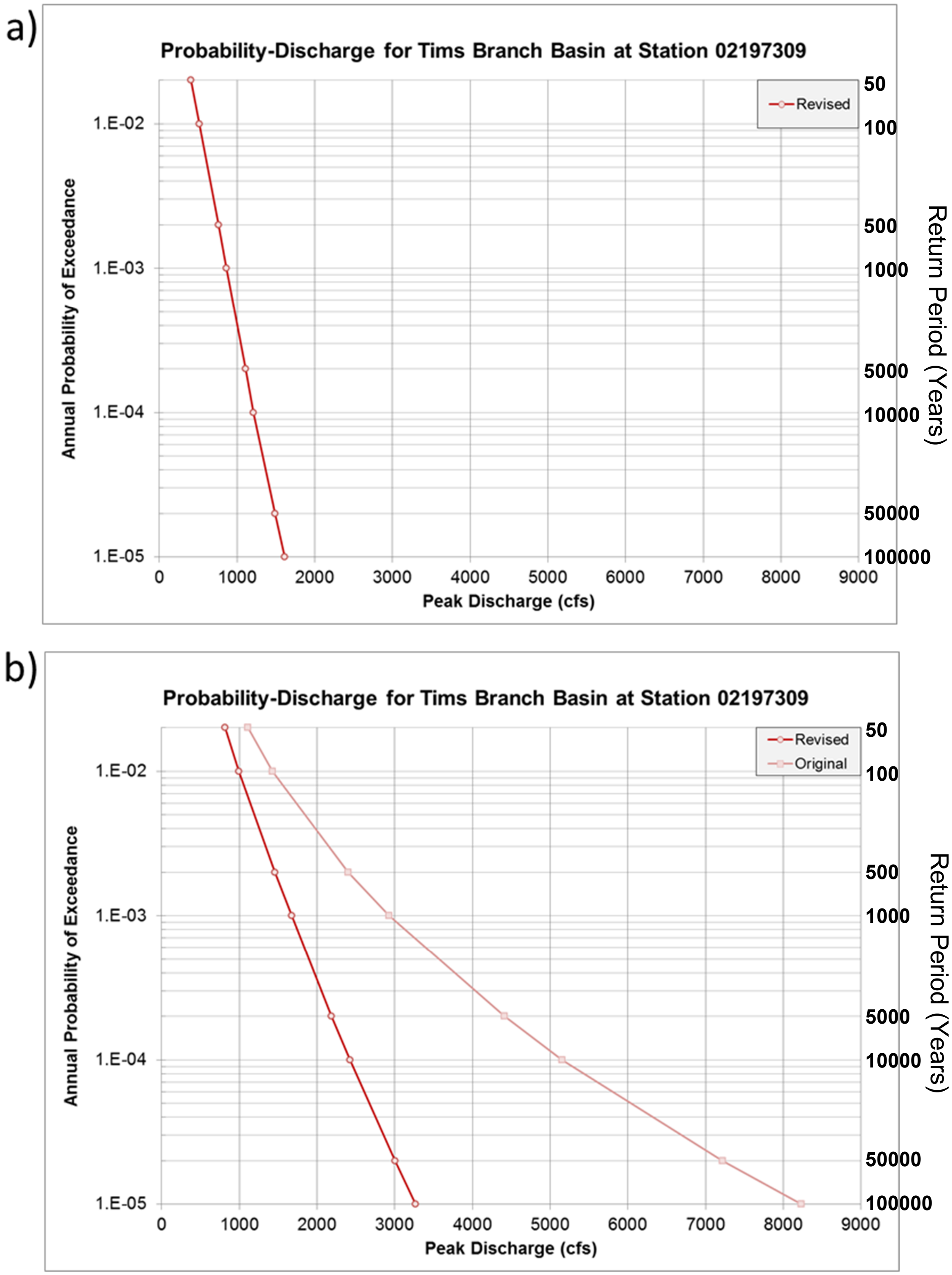
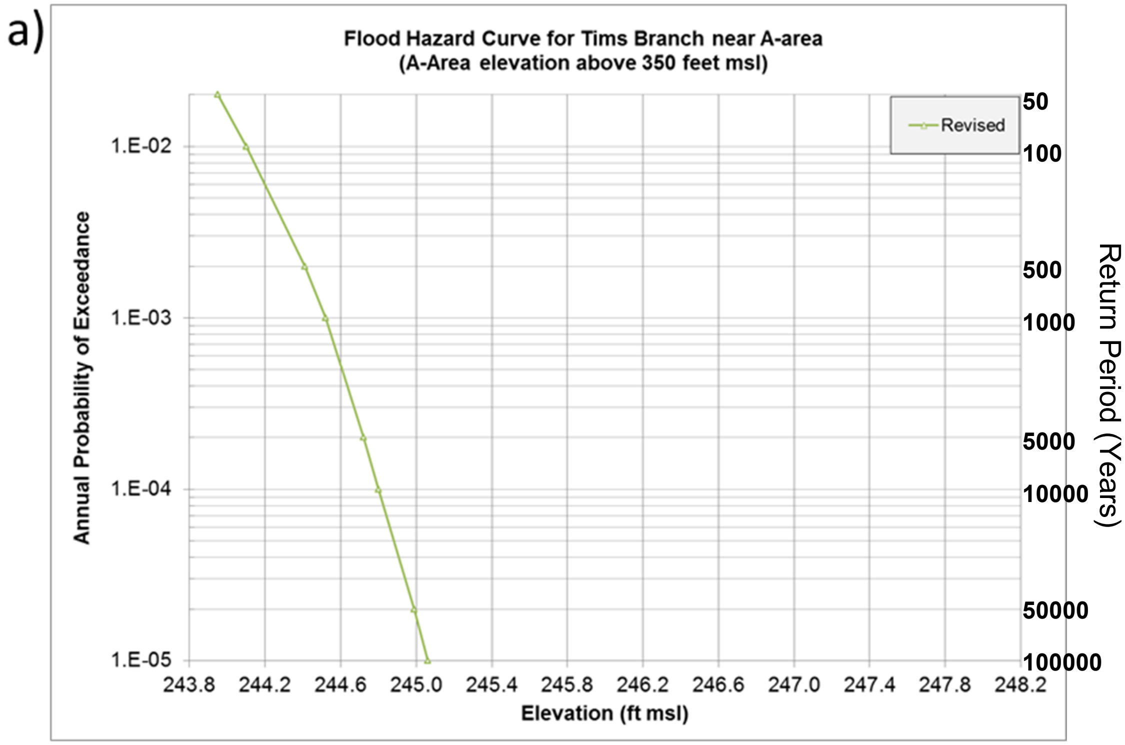


Figure 12: Probability of exceedance versus peak discharge for a) 6-hour accumulation, and b) 24 hour accumulation for Tims Branch basin. Data for the prior C00 study is shown in b). Note that no data were generated in the C99 study for Tims Branch basin.



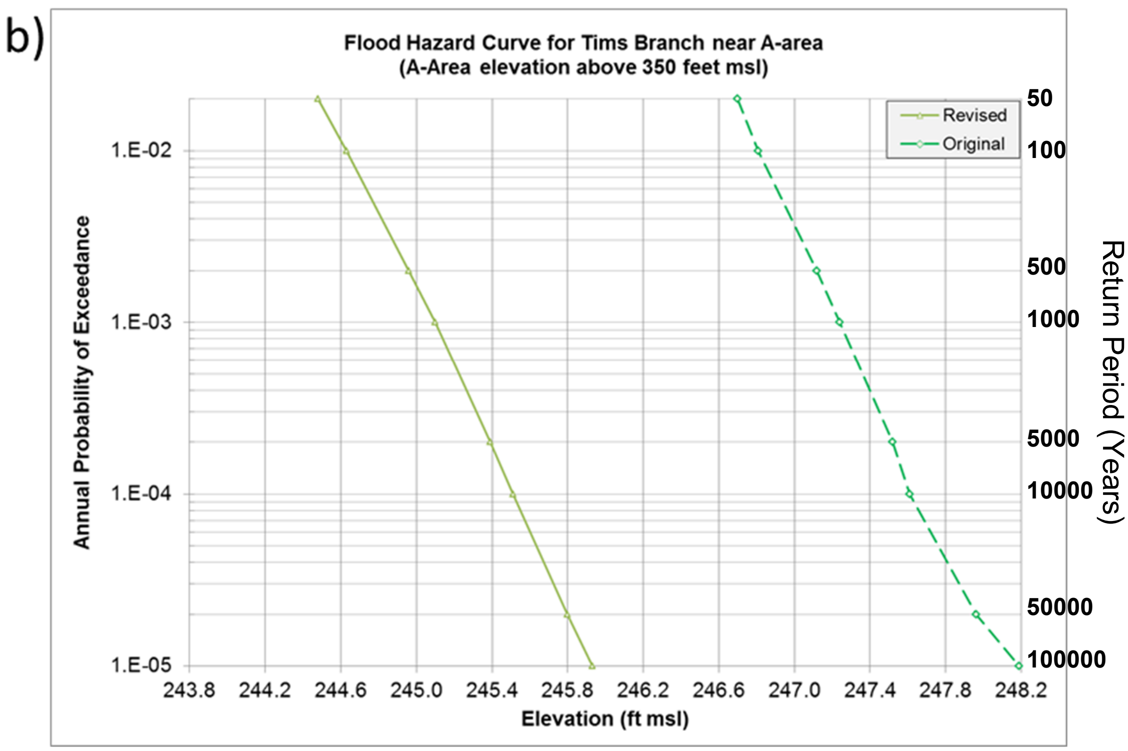


Figure 13: Flood hazard curves for Tims Branch basin near A-area for a) 6-hr accumulation, and b) 24-hour accumulation. Data for the prior C00 study is shown in b). Note that no data were generated in the C99 study for Tims Branch basin. The A-area elevation is above 350 feet msl.

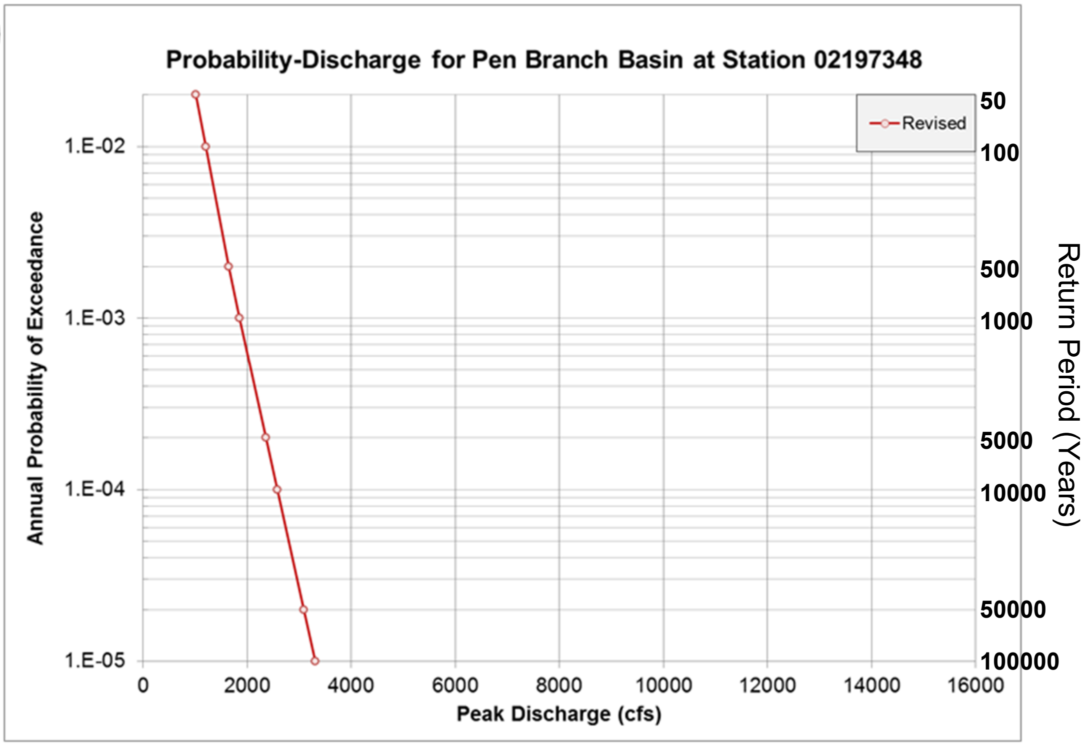
3.4 Pen Branch Basin

The Pen Branch basin drainage area is about 22 square miles. Pen Branch follows a path roughly parallel to Fourmile Branch until it enters the Savannah River swamp (Figure 2a, 2b). The only significant tributary to Pen Branch is Indian Grave Branch, which flows into Pen Branch about 5 miles upstream from the swamp. Pen Branch enters the swamp about 3 miles from the Savannah River, flows directly toward the river for about 1.5 miles, and then turns and runs parallel to the river for about 5 miles before discharging into Steel Creek at about 0.5 mile from its mouth. A USGS flow recorder was installed in November 1976 at SRS Road A-13.2 on Pen Branch (station 02197348). During the period between 1976 and 1986, the flow at this station ranged from a minimum of about 1 cfs during a K-Reactor outage to a maximum of 750 cfs during simultaneous K-Reactor operation and a heavy precipitation event.

A storm event that occurred on January 6, 1995 was selected to determine the HEC-HMS input parameters that characterize the Pen Branch basin (C00). Data at the CLM was used to allocate the daily accumulated rainfall into an hourly time series, using the average from the four rain gauges (CLM, 100-K, 100-L, and 100-P) that cover the Pen Branch basin as the average rainfall of that storm for the basin. The peak flows at station 02197348 for various return-period storms were calculated using data from W13 (Fig. 14).

The peak discharge for the 100-year return period and a 6-hr accumulation period is 1207 cfs for gauge 02197309, while the 100,000-year return period discharge is 3304 cfs (Fig. 14a). Similarly, for the 24-hr accumulation period, the peak discharge (Fig. 14b) for the 100 and 100,000-year return periods is 2384 cfs and 6794 cfs, respectively. Flood elevation levels near K-area for both Pen Branch and Indian Graves Branch basins using WSPRO are indicated in Figure 15. The estimated flood level at Pen Branch for a 100,000-year return period is 175 ft for a 6-hour accumulation period (Fig. 15a), 177 ft for a 24-hour accumulation period, and 181 ft for Indian Graves Branch for a 24-hour accumulation period. All of these are significantly less than the 260 ft msl elevation at K-area.

a)



b)

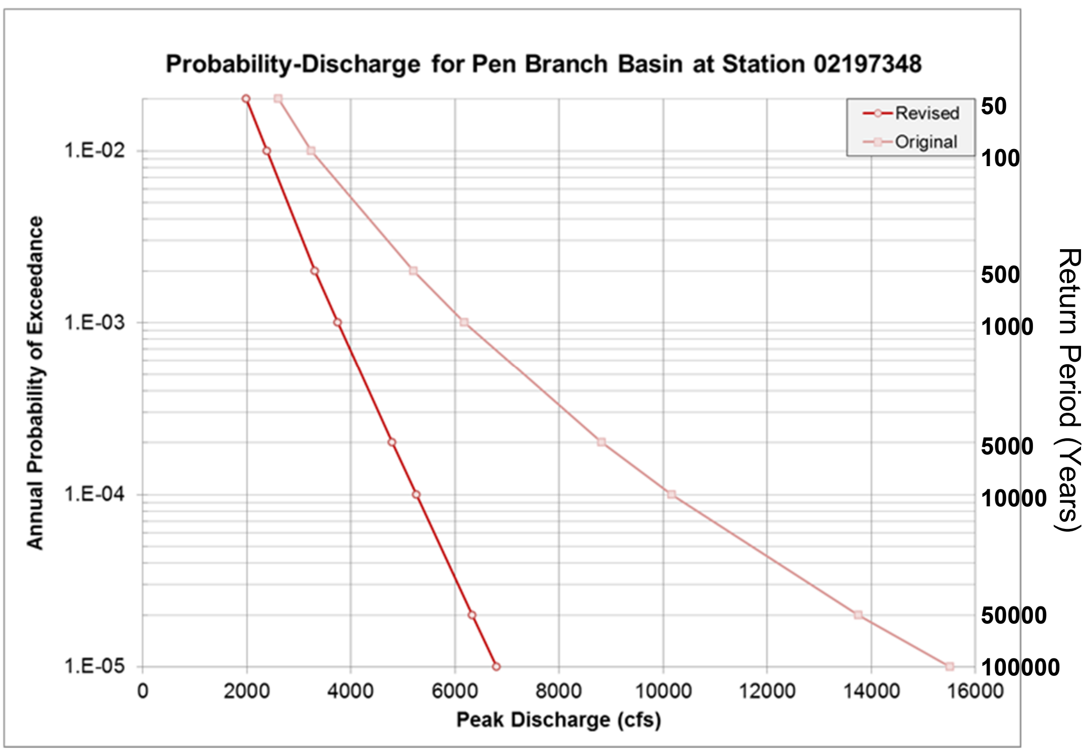


Figure 14: Probability of exceedance versus peak discharge for a) 6-hour accumulation, and b) 24 hour accumulation for Pen Branch basin. Data for the prior C00 study is shown in b). Note that no data were generated in the C99 study for Pen Branch basin.

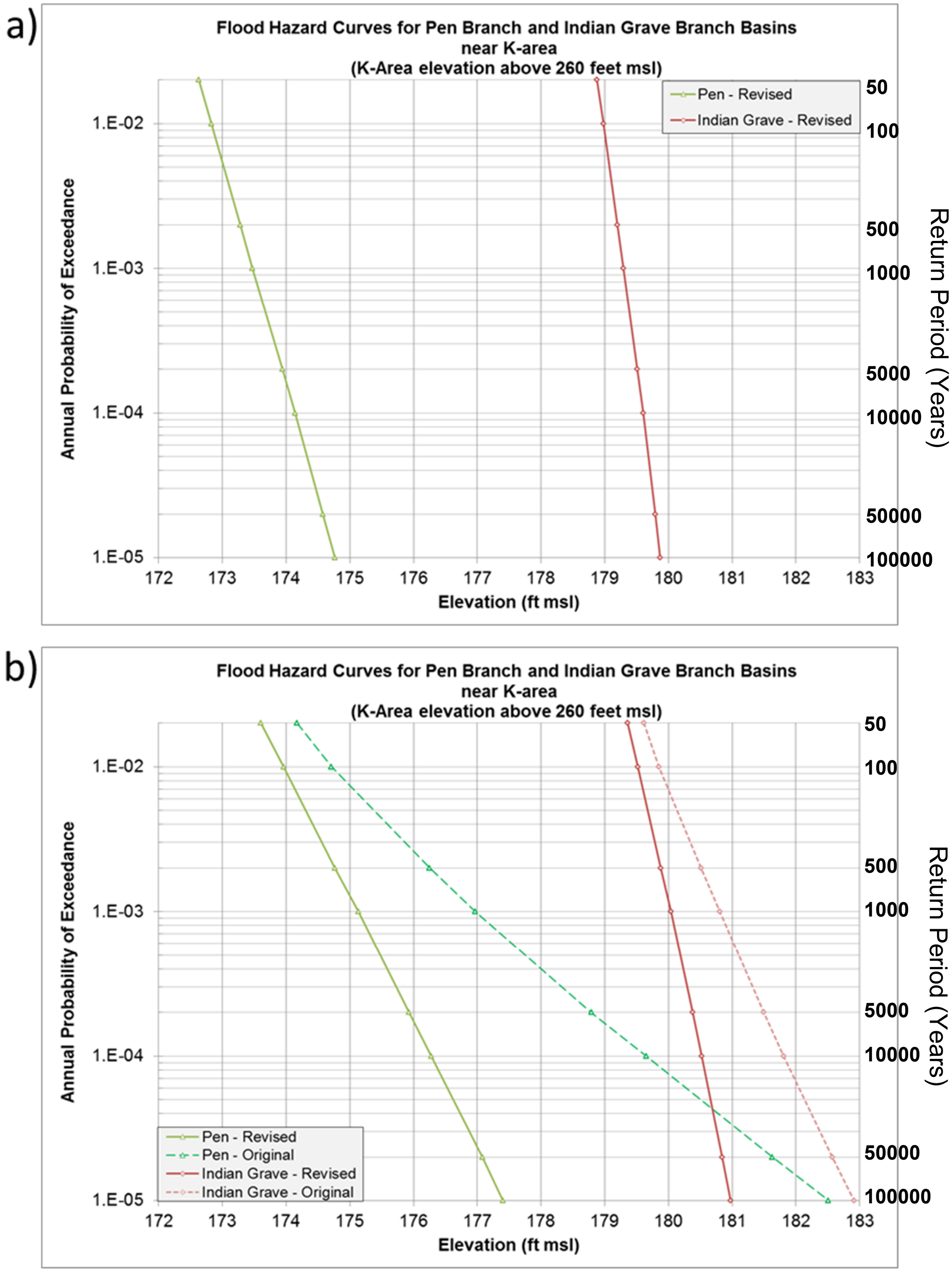
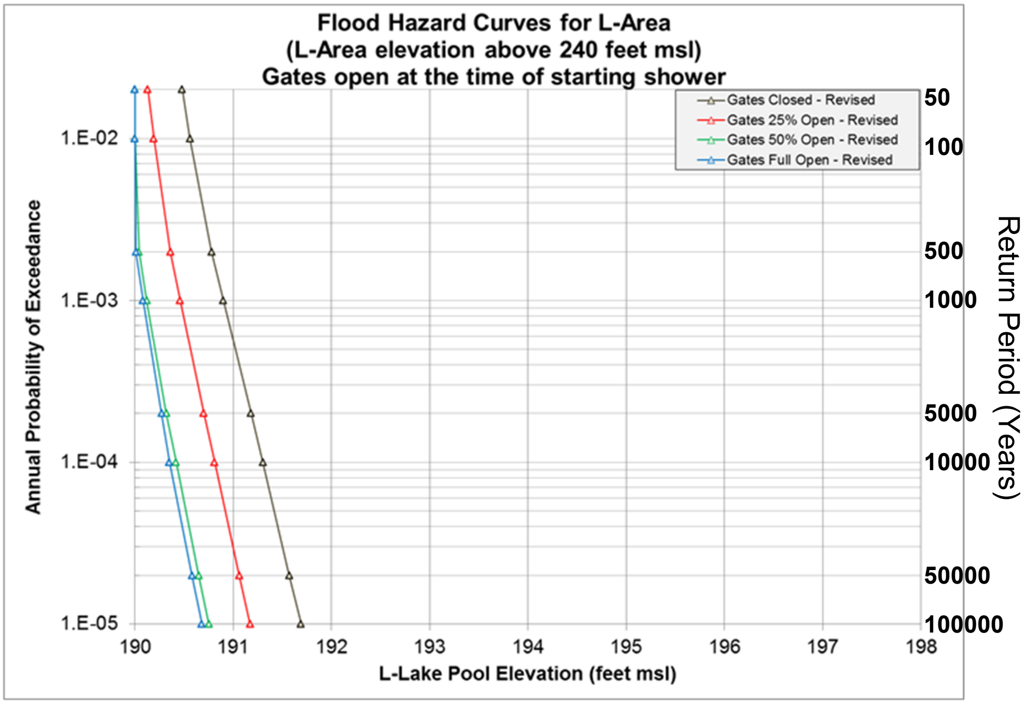


Figure 15: Flood hazard curves for Pen Branch basin near K-area for a) 6-hr accumulation, and b) 24-hour accumulation. Data for the prior C00 study is shown in b). Note that no data were generated in the C99 study for Pen Branch basin. The K-area elevation is above 260 feet msl.

a)



b)

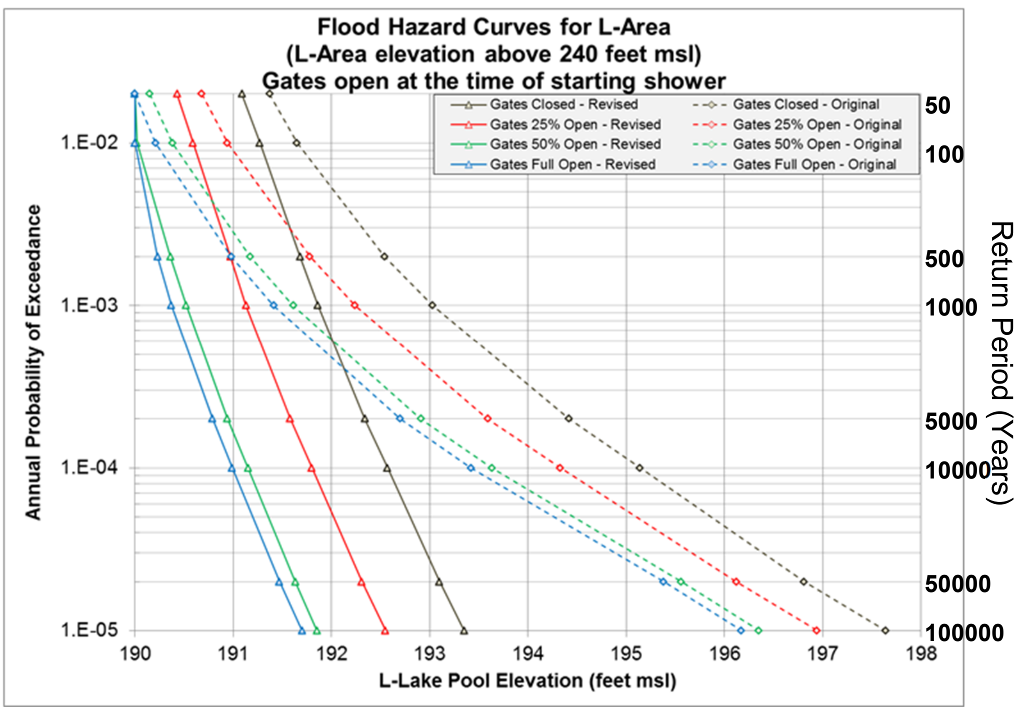


Figure 16: Flood hazard curves for L-area near L-Lake for a) 6-hour accumulation, and b) 24-hour accumulation as a function of gate status. The L-area elevation is above 240 feet msl. Data for the prior C00 study is shown b). Note that no data were generated in the C99 study for L-Lake.

3.5 L-Lake

Flood levels near L-area were also estimated in C00 using direct rainfall input to HEC-HMS, and making assumptions as to gate status (e.g., closed, partially open, and fully open) and pump flow rates. Assuming the gates are open at the starting time of the precipitation event, the flood hazard curves for L-area are shown in Figure 16 for both the 6-hr and 24-hr accumulation periods. Since the L-area elevation level is 240 feet msl, chances are quite small that this area would be flooded given a 100,000 year rain event, where the elevation level is estimated to be between 191 and 193 ft (depending on assumptions regarding the gate status and accumulation period).

4.0 Conclusions

Reports of flood hazard probabilities at various SRS facilities were generated approximately 15 years ago (C99, C00) using measured precipitation dating back to the 1950s and based on requirements set forth in the DOE 1020 technical standard (DOE, 1994), which has been revised from a prior version used by C99 and C00 (DOE, 2002, 2012). Recent work provided updated probabilistic hazard assessments for tornadoes, straight-line winds, and extreme precipitation events at the SRS (W13). Regarding the precipitation, more recent data were incorporated (through 2012), and modifications to the extreme-value distributions were explored. The resulting hyetographs (accumulated rainfall) for both 6-hr and 24-hr periods is lower than in the previous reports.

Note that prescriptive regulatory guidelines were used in this analysis and in the generation of the precipitation distributions of W13. There is much higher uncertainty associated with the longer return periods due to the relatively limited amount of observed precipitation. Estimation of these uncertainties is beyond the scope of this paper.

As in the C99 and C00 reports, the revised precipitation distributions were applied to time-periods ranging from 50 to 100,000 years and incorporated in a hydrologic modeling system to estimate the peak discharges from various basins at SRS (i.e. Upper Three Runs, Fourmile Branch, Tims Branch, and Pen Branch). In turn, the peak discharges were used as input to a water surface profile model to determine water surface elevations at numerous sites of interest (e.g. A, F, H, etc.). As would be expected with the lower cumulative rainfall estimates, the resulting peak discharge and flood elevation levels are also lower with the revised data. In all cases, chances for flooding even for the 100,000 year precipitation event are negligible.

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