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TECHNICAL EVALUATION REPORT

HYDRAULIC AND HYDROLOGIC EVALUATION OF PAR POND DAM

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**M. Reich, P. C. Wang,
R. Khanbilvardi and P. Bezler**

October 1993

**DEPARTMENT OF ADVANCED TECHNOLOGY
BROOKHAVEN NATIONAL LABORATORY, ASSOCIATED UNIVERSITIES, INC.
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TECHNICAL EVALUATION REPORT
HYDRAULIC AND HYDROLOGIC EVALUATION
OF PAR POND DAM

**M. Reich, P. C. Wang,
R. Khanbilvardi and P. Bezler**

October 1993

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ABSTRACT

The PAR Pond Dam was constructed in 1958-1959. Seepage, depressions, boils and spring flows were observed at varying locations on the dam in the last few years. Comprehensive geotechnical and hydraulic investigations pertaining to the effects of the above observations on the abilities of the dam to withstand future floods were made in 1991 and 1992 by the Westinghouse Savannah River Company (WSRC) and Bechtel Savannah River, Inc. (BSR). Additional studies were performed by the EBASCO Engineering Corporation in 1992 and early 1993 where dam capacity to survive flooding and seismic events were evaluated.

Brookhaven National Laboratory (BNL) was asked by the Department of Energy (EH) to carry out an independent review of the PAR Pond Dam response to future flooding and seismic events. This report addresses the studies made to evaluate the capacity of the dam to survive floods. A companion report will summarize the evaluations performed to assess the seismic capacity of the dam.

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1.0 INTRODUCTION

The PAR Pond Dam is an earthen dam constructed in 1958-1959 across the Low Three Runs Creek located on the U.S. Department of Energy (DOE) Savannah River Site (SRS) in South Carolina, (Fig. 1-1). The dam has a 30-foot-wide crest at a design elevation of about 211 feet above mean sea level (MSL). It has a height of 66 feet and a length of 4500 feet. At the spillway elevation (200.00 feet MSL) the reservoir has a surface area of 7640 acres and an impoundment of approximately 57000 acre feet (Fig. 1-2).

The embankment cross section is essentially homogeneous consisting of silty and claying sands with relatively low permeability, flanked on the downstream side by a pervious zone of silty sand with a width about one-fourth of the total embankment width. The foundation is relatively pervious. A reinforced concrete outlet conduit, with internal sectional dimensions of 8 feet by 8 feet, was constructed at the base of the dam near its maximum section to serve as a sluiceway and spillway (Fig. 1-3).

Since its construction, seepage problems arose. A depression in the vicinity of the outlet conduit was discovered, together with flow from beneath the conduit and associated downstream wet areas. In addition, boils and springs flow from within the riprap in the tailwater pool. Consequently, DOE undertook the following remedial measures:

1. An earthen blanket was placed over the downstream toes of the dam, and
2. Six relief wells were constructed in the vicinity of the outlet conduit.

Because of the adverse conditions, Westinghouse Savannah River Company (WSRC) and Bechtel Savannah River, Inc. (BSR) performed comprehensive geotechnical and hydraulic investigations in 1991 and 1992. Further studies to determine the capacity of the dam to withstand flooding and earthquake events were performed in 1992 and early 1993 by EBASCO, Incorporated. In addition, DOE/EH funded an independent safety evaluation performed by Brookhaven National Laboratory (BNL). This report is a summary of the BNL hydraulic and hydrologic studies. The report includes: an evaluation of probable maximum flood, flood frequency based on analysis of existing rainfall data, and an evaluation of the probability of dam failure due to flood overtopping.

2.0 PROBABLE MAXIMUM FLOOD

2.1 Introduction

According to available information (Bechtel, 1992), the existing spillway was designed to withstand a "1500-year" flood which was estimated to have a peak inflow of 11,400 cubic feet per second (cfs).

The Federal Interagency Committee on Dam Safety states that the Probable Maximum Flood (PMF) should be used as the inflow design flood for dams that represent a significant hazard to the public in the event of failure.⁽¹⁾ Probable Maximum Flood results from Probable Maximum Precipitation (PMP), which is defined as the ultimate "catastrophic" precipitation event that could ever be created under conceivably the most unfavorable combination of the specific meteorological conditions. Since the PAR Pond Dam was not designed to withstand the PMF, BNL conducted a study based on the Probable Maximum Precipitation (PMP) and the Probable Maximum Storm (PMS). Subsequently, BNL computed the PMF inflow hydrograph for the PAR Pond Dam. In addition, based on recorded precipitation data, BNL also performed a probabilistic flood study. The drainage areas of the PAR Pond, L Lake and portions of Steel Creek and Lower Three Runs are shown in Figure 2-1. PAR Pond basin is designated as Basin 3.

2.2 Probable Maximum Precipitation (PMP)

PMP is defined as "the theoretically greatest depth of precipitation for a given duration that is physically possible over a particular drainage area at a certain time of year." The PMP values are generalized estimates which were obtained by analysis of over 500 storms to determine the depth-area-duration relationships (U.S. Army Corps of Engineers, 1945).

In this analysis, three operations were considered to produce generalized all-season PMP maps covering durations of 6, 12, 24, 48 and 72 hours for areas of 10, 200, 1,000, 5,000, 10,000 and 20,000 square miles applicable to nonorographic regions of the United States east of 105 meridian. These three operations consist of:

- a. Moisture maximization - increases the storm precipitation to a value that is consistent with the maximum moisture in the atmosphere for the storm location and time of occurrence.
- b. Transposition - relocates the storm precipitation to a region that is homogeneous relative to terrain and meteorological features important with respect to the particular storm rainfall.

- c. Envelopment - smoothly interpolates between the maxima from a group of values for different duration and/or areas.

The PMP maps are published as Hydrometeorological Report No. 51: (HMR51) "Probable Maximum Precipitation Estimates, United States East of the 105th Meridian", Office of Hydrology, National Weather Service.⁽²⁾

2.3 Probable Maximum Storms (PMS)

To develop the PMS for a particular location, area and duration, NOAA Hydrometeorological Report No. 52: (HMR52) "Application of Probable Maximum Precipitation Estimates"⁽³⁾ may be used. The procedure of utilizing the PMP map as input to produce the PMS for the PAR Pond Dam is as follows:

- a. Obtain depth-area/duration, from HMR51 map, yielding,

Area (mi ²)	6 hr (Precipitation-in.)	12 hr	24 hr	48 hr	72 hr
10	31.0	37.1	43.8	48.2	51.4
200	23.0	28.0	34.6	38.2	42.2
1000	16.9	22.5	29.4	33.0	35.6
5000	9.7	14.0	19.4	24.0	27.0
10000	7.4	11.2	15.8	20.1	23.4
20000	5.4	8.8	12.6	16.4	19.3

- b. Obtain preferred orientation of rainfall from HMR51 map. Preferred orientation is 207°. The actual orientation based on selected optimized isohyetal pattern placement (see Figs. 2-1 and 2-2) is found to be 214°. Since the difference is less than 19°, then, according to HMR51, no adjustment factor is applied.
- c. For a basin area of 31.7 mi² (Figs. 2-1 and 2-2), the total average precipitation in each isohyet area (A, B, and C) is computed to be 47.8 in., with and without adjustment.
- d. The temporal distribution of the rainfall, assuming that the maximum precipitation occurs at the seventh 6 hr. period, is plotted as shown in Figure 2-3.
- e. Incremental and total rainfall for 5 minute intervals is also computed. The total rainfall-time or probable maximum storm (PMS) is used to compute the maximum flood (PMF) as described below.

2.4 Probable Maximum Flood (PMF)

The PMF computation is based on the input of PMS, which is the precipitation data shown in Figure 2-3. The runoff is computed using the computer program, 4EC-1, FLOOD HYDROGRAPH PACKAGE, developed by the Hydrological Engineering Center of the U.S. Army Corps of Engineers.⁽⁴⁾ The runoff is computed separately for the land and reservoir portions and then combined to arrive at the dam flood stage elevation with the pre-rain stage assumed to be at elevation 200 ft.

The land portion of PAR Pond Reservoir basin is 27.32 square miles in area. The basin land is classified as 60% forest, 30% grange and 10% impervious. The soil type is classified as 60% A, 35% B and 5% C, thereby corresponding to Soil Conservation Services (SCS) curve number 57. Based on this information, the precipitation loss due to infiltration is computed and plotted as shown in Figure 2-4. Deducting the infiltration from the PMS precipitation yields the excess precipitation. The SCS dimensionless unit hydrograph is constructed based on a time lag of 1.07 hours between the center of mass of rainfall excess and the peak of the unit hydrograph. The hydrograph is computed and plotted as shown in Figure 2-5.

BNL also computed the runoff at the reservoir portion (4.38 mi² area) assuming impervious conditions. The combined hydrograph of the land and reservoir portions is shown in Figure 2-6.

Finally, BNL computed the route of the run-off to the dam for the PMF. Assuming a pre-rain water level of 202.0 ft., a maximum reservoir water surface elevation of 213.56 ft. (3.56 ft. above the top of the dam) is predicted. Table 2-1 shows the computation results.

3.0 STATISTICAL FLOOD ANALYSIS

The Probable Maximum Storm (PMS) is used as the input since in the Probable Maximum Flood (PMF) analysis, it is implied that the "theoretically greatest depth of precipitation" has been combined with the worst (i.e., most damaging) chance of occurrence. An alternate approach is to use recorded precipitation data with a statistically based computational method.⁽⁵⁾ Many processes in hydrology can be analyzed and explained in a probabilistic (statistical) sense owing to their inherent randomness. It is not possible to predict future rainfall on a purely deterministic basis since it is difficult to quantify all the causal mechanisms with any degree of confidence. Statistical methods are available to organize and reduce observed data to a form that facilitates their interpretation and evaluation. There are many methods by which the uncertainty in hydrologic data may be quantified and presented in a standard probabilistic framework. The most common methods are based on the "normal," "gamma" (Pearson Type 3), "Log-normal," "Log-gamma" (Log Pearson Type 3), and "Gumbel" (extreme value Type I) distributions. In the sections that follow, a description of each of these methods is provided.

3.1 The Normal Distribution

The normal distribution is also known as the Gaussian distribution. It is fundamental to the realm of probability and statistics. According to the Central Limit Theorem, under very general conditions, as the number of variables in a sum becomes large, the distribution of the sum of a large number of random variables will approach the normal distribution regardless of the underlying distribution.⁽⁶⁾

In normal distributions, the Probability Density Function (PDF) is a bell-shaped curve with the following equation:

$$f(x) = \frac{1}{\sqrt{2\pi} \sigma} e^{-(1/2) [(x-\mu)/\sigma]^2} \quad -\infty \leq x \leq \infty \quad (1)$$

where:

μ = mean, and
 σ = variance.

Estimate of the mean can be obtained as follows:

$$\mu = \bar{x} = \frac{1}{n} \sum_{i=1}^n X_i \quad (2)$$

For the variance, an unbiased estimate is:

$$\sigma^2 = S_x^2 = \frac{1}{n-1} \sum_{i=1}^n (x_i - \bar{x})^2 = \frac{\sum x_i^2 - n\bar{x}^2}{n-1} \quad (3)$$

Random variables having the distribution of Equation (1) are denoted by $N(\mu, \sigma^2)$. The numerical value of the PDF is not normally needed since the PDF itself is not a probability. In fact, it is the Cumulative Density Function (CDF) that is of interest since it is a probability. The continuous CDF is defined as:

$$F(x_1) = P(-\infty \leq x \leq x_1) = \int_{-\infty}^{x_1} f(x) dx \quad (4)$$

where:

$P(-\infty \leq x \leq x_1)$ is the probability that the discrete random variable X takes on values between $-\infty$ and x_1 is the area under the PDF curve between $-\infty$ and x_1 .

The PDF and CDF are related by:

$$\frac{dF(x)}{dx} = f(x) \quad (5)$$

The following equations define other properties of $F(X)$

$$0 \leq F(x) \leq 1.0 \quad (6)$$

and

$$P(x_1 < x \leq x_2) = F(x_2) - F(x_1) \quad (7)$$

Using Equation (1), the CDF could be evaluated after a change of variable to

$$Z = (x - \mu) / \sigma \quad (8)$$

where Z is normally known as the standard normal variate and is $N(0,1)$.

The CDF is then written as:

$$F(Z) = \int_{-\infty}^Z \frac{1}{\sqrt{2\pi}} e^{-\frac{u^2}{2}} du \quad (9)$$

This integration cannot be performed analytically. However, tables of $F(Z)$ vs. Z are readily available in Statistics Texts.⁽⁶⁾

The most common means used in hydrology to indicate the probability of an event is to assign a return period or so-called "recurrence interval" to the event. An annual maximum event has a return period of T years if its magnitude is equalled or exceeded on average, once every T years. The $1/T$ is the exceedance probability of the event, which is the probability that the event is equalled or exceeded in any one year.

The magnitude of X for a given return period, T , can be found by using the following equation:

$$F(Z) = 1 - \frac{1}{T} \quad (10)$$

and then,

$$X = \bar{X} + Z S_x \quad (11)$$

The value of Z can be obtained from normal probability function tables using the $F(Z)$ value from Equation (9).

3.2 The Log Normal Distribution

Generally, if a random variable X results from the product of a large number of other random variables (the so-called "multiplicative mechanism"), the distribution of the logarithm of X will approach a normal distribution. In rainfall/runoff analyses, many factors that are inherently random and about which there is little or no deterministic information, contribute to the flow. Therefore, it is reasonable to consider a multiplicative mechanism for rainfall prediction.

A random variable has a log-normal distribution if the log of the random variable is distributed normally. Therefore, if $Y = \log X$ and Y is distributed normally, then X has a log-normal distribution. The procedure for obtaining the magnitude of X for a given return period, T , for a log-normal distribution is the same as for a normal distribution using the following change of variables:

$$Y_i = \log X_i \quad (12)$$

Then, replacing X 's by Y 's to obtain $\bar{Y}$ and S_y and using Equation (11), Equation (12) will be:

$$Y = \bar{Y} + Z S_y \quad (13)$$

The magnitude of X for a given return period, T , will be:

$$X = 10^Y \quad (14)$$

The value of Z can be obtained from statistics tables.⁽⁶⁾

3.3 The Gamma (Pearson Type 3) Distribution

This type of distribution has well-known mathematical properties and is used extensively in hydrology. The three parameters of PDF are simple functions of the mean, variance and skewness. The skewness, which is the third central moment normalized by dividing by the cube of the standard deviation (S), is a shape parameter. The skewness presents special problems because it involves a summation of the cubes of deviations from the mean. Therefore, it is subjected to larger errors in its computation. An unbiased estimate of the skewness is given by:

$$C_{s1} = \frac{n}{(n-1)(n-2)} \frac{\sum (x_i - \bar{X})^3}{S_x^3} \quad (15)$$

Tasker and Stedinger (1986) suggest the following correction to C_{s1} is suitable for most hydrologic application:

$$C_{s2} = \left(1 + \frac{6}{N}\right) C_{s1} \quad (16)$$

It is common in hydrologic applications to evaluate the CDF with frequency factors. The frequency factor "K" is a function of skewness and return period (or CDF). The values of K are tabulated in statistical hydrology texts such as Bedient and Huber.⁽⁷⁾ To obtain the T-year rainfall, the moments of the data (X) are computed from:

$$X = \bar{X} + K (C_{s2}, T) S_x \quad (17)$$

To obtain values of K for return periods beyond those available in published tables, the magnitudes vs. return period is plotted and interpolated to obtain the desired value. It must be noted that the gamma distribution is directly related to the chi-squared distribution tabulated in Benjamin & Cornell (1970).

3.4 The Gumbel (Extreme Value Type I) Distribution

The Gumbel Distribution is based on the Theory of Extremes developed by Gumbel.⁽⁸⁾ This PDF is unbounded on both the lower and upper ends.

The CDF has the double exponential form:

$$F(X) = e^{-e^{-\alpha(X-u)}} \quad -\infty \leq X \leq \infty \quad (18)$$

F(X) could be readily evaluated without tables. Parameter α (a scale parameter) and u (a location parameter) are related to the Mean (μ) and Variance (σ^2) by

$$\mu = u + \frac{0.5772}{\alpha} \quad (19)$$

and

$$\alpha = \frac{\pi}{\sqrt{6}\sigma} \quad (20)$$

Kite (1977) derived a frequency factor K^1 by using the above CDF to solve for X in terms of F or T combined with the moment relationship. The K^1 factor can be obtained from:

$$K^1 = -0.7797 \left\{ 0.5772 + \ln \left[\ln \left(\frac{1}{F} \right) \right] \right\} \quad (21)$$

while:

$$F = 1 - \frac{1}{T} \quad (22)$$

where T = desired return period.

The frequency factor K^1 is then used to calculate X from

$$X = \bar{X} + K^1 S_x \quad (23)$$

3.5 The Log-Pearson Type 3 Distribution

The Log-Pearson Type 3 distribution differs from the Pearson Type 3 distribution in that the log variable (Y) distributes in Pearson Type 3. If we assign

$$y = \log X \quad (24)$$

then Y has the Pearson Type 3 distribution

$$\bar{Y} = \frac{\sum y}{n}, \quad S_y = \left[\frac{\sum (y - \bar{y})^2}{n-1} \right]^{0.5} \quad (25)$$

$$C_{s1} = \frac{n}{(n-1)(n-2)} \frac{\sum (y - \bar{y})^3}{S_y^3} \quad (26)$$

The skew coefficient C_{s1} can be adjusted based on overserved generalized skew coefficient C_{s1} based on the Mean Square Errors $MSE(C_{s1})$ and $MSE(\bar{C}_{s1})$ as discussed in Reference 9,

$$C_{s2} = \frac{MSE(\bar{C}_{s1}) C_{s1} + MSE(C_{s1}) \bar{C}_{s1}}{MSE(\bar{C}_{s1}) + MSE(C_{s1})} \quad (27)$$

Then $y = \bar{y} + K(C_{s2}, T) S_y$ and $X = \log^{-1} y$

4.0 AVAILABLE DATA

To use the methodologies mentioned in the previous section, actual rainfall data were needed. The available data sets were grouped into two categories: (a) regional data, and (b) local data. The following subsections describe each category.

4.1 Regional Data

Regional data in this report, refers to the data sets which belong to sites near the PAR Pond Dam. An extensive amount of data for nearby sites is available through the National Climatic Center in North Carolina. BNL obtained two data sets from this office. The sources of these data sets were the Augusta, Georgia, and Savannah, Georgia stations. Figure 4-1 shows the location of these stations with respect to the PAR Pond Dam.

Rainfall data has been collected at the Augusta station since 1949. The data includes the daily rainfall (inches) for every day in the period 1949-1991. This 43-year data set was reduced to obtain the annual maximum rainfall in 24-hour, 48-hour, and 72-hour intervals. These intervals were selected for comparison with the 72-hour PMP obtained in Chapter 2 of this report. The 24-hour and 48-hour rainfall were needed to provide information on the distribution of the 72-hour rainfall event. Table 4-1 shows the annual maximum rainfall values (inches) in 24-hour, 48-hour, and 72-hour intervals for the period 1949-1991. Statistical parameters needed for flood frequency analysis are provided in the same table. Figures 4-2 through 4-4 show the variation of these values with time. Table 4-2 shows the log-values of the annual maximum rains for the same period with the corresponding statistical parameters.

Rainfall data was also available for a site in Savannah, Georgia. The data for this site is available for the period of 1949-1986. Rain data during 1969, 1970, and 1971 are missing from this data set. This 36-year data set was also reduced to obtain the maximum rainfall in 24-hour, 48-hour, and 72-hour intervals. Table 4-3 shows the annual maximum rain for these intervals during 1948-1986. Figures 4-5 through 4-7 plot these values versus time. Table 4-4 provides log-values of annual maximum rain for the same period and the corresponding statistical parameters.

4.2 Local Data

In addition to the regional, rainfall amounts and durations were measured since 1961 at 11 locations within the Savannah River Site adjacent to PAR Pond Dam. From this data set, maximum daily rain values were extracted. These values were then used to pick the annual maximum daily (24-hour) rainfall for each year. The same analysis was repeated to obtain the annual maximum

48-hour and 72-hour rainfalls for the period of 1961 to 1990. Table 4-5 shows these values with the corresponding statistical parameters. Figures 4-8 through 4-10 plot the variations of these values with time. Log-values of the annual maximum rains with the related statistical parameters are shown in Table 4-6.

5.0 FLOOD FREQUENCY ANALYSIS

Values obtained from regional data sets were used first for the statistical analysis. The methodologies discussed in Chapter 3 of this report were used to calculate the 24-hour, 48-hour, and 72-hour rainfall events with probabilities of occurrence of 10^{-2} , 10^{-3} , 10^{-4} , and 10^{-5} . A rainfall event with a probability of occurrence of 10^{-2} is an event that is expected to happen once in every 100 years (i.e., return period = 100 years).

Table 5-1 shows the result of the frequency analysis for the Augusta Station data, while Table 5-2 shows the result of the frequency analysis for the Savannah Station data.

A weighting factor was assigned for each statistical method to calculate a weighted average. The weighting factors were 0.05 for the normal, 0.3 for the log normal, 0.05 for the gamma, 0.3 for the Log Pearson Type 3 and 0.3 for the Gumble distributions.⁽¹⁰⁾ Since the distances between these two stations and the PAR Pond Dam are not the same, the following equation was used to obtain a distance weighted average:

$$R_p = \frac{\left(\frac{R_a}{d_a}\right) + \left(\frac{R_s}{d_s}\right)}{\left(\frac{1}{d_a}\right) + \left(\frac{1}{d_s}\right)}$$

where:

R_p = weighted rain (inch) for PAR Pond Dam,
 R_a = rain (inch) at Augusta Station,
 R_s = rain (inch) at Savannah Station,
 d_a = distance (mi) between Augusta Station to PAR Pond Dam,
 d_s = distance (mi) between Savannah Station and PAR Pond Dam.

Table 5-3 shows the distance weighted rain values for the two stations where $d_a/d_s = 1/7$. Using the 24-hour and 48-hour values, the distribution of a 72-hour rainfall that could occur once every 100,000 year (i.e., probability of occurrence = 10^{-5}) was obtained. Figures 5-1 through 5-4 show the rain distribution for a 72-hour storm for 100-year, 1,000-year, 10,000-year, and 100,000-year return periods, respectively. Figure 5-5 plots rain values for varying probability and rain durations.

Using data obtained within the Savannah River Site (SRS) and conducting flood frequency analysis, maximum rainfall events were calculated for return periods of 100-years, 1,000-years, 10,000-years, and 100,000-years. These values and the weighted average values are shown in Table 5-4. Figures 5-6 through 5-9 depict

the distributions of a 72-hour rainfall for 100-year, 1,000-year, 10,000-year, and 100,000-year return periods, respectively. The relationship between total rain values for varying probability and rain durations are shown in Figure 5-10. The rainfall distributions shown in Figures 5-4 and 5-9 were used to develop runoff and spillway flow for the PAR Pond Dam. This is discussed in detail in the next chapter.

6.0 HYDROLOGICAL ROUTING

To investigate the probability of overtopping the dam, rainfall events must be routed through the reservoir behind the dam and spillway. An understanding of the routing is necessary in order to establish the response of the dam to a particular rain. The rainfall events selected for this purpose were the "worst" case scenarios. The "worst" rainfall in this report is referred to as the rainfall event with the lowest probability (10^{-5}) and longest duration (72-hr). Figures 5-4 and 5-9 show the "worst" rainfall events using regional and local data, respectively.

These rainfall events must then be routed through the conveyance system at the site. The conveyance system in the PAR Pond Dam Site includes the contributing drainage basin, the reservoir behind the dam, and the existing spillway. TR-20 (developed by USDA-Soil Conservation Services) and HEC-1 (developed by Hydraulic Engineer Center in Davis, CA) are among the existing computer models that are available for routing evaluations. HEC-1 is widely accepted by the engineering communities and was previously used by Bechtel Co. for the PAR Pond Dam. The following subsections will describe the HEC-1 program and input information for the PAR Pond Dam analysis.

6.1 HEC-1 Program

HEC-1 can simulate the surface rainfall excess by representing the basin as an interconnected system of different hydrologic and hydraulic components. Each component represents different aspects of the rainfall-runoff processes within a sub-basin. A component normally represents a sub-basin surface runoff, stream channels, and reservoirs. Each component is represented by a set of parameters that describes the particular characteristics of the component. These parameters are used in the mathematical relationships to describe the physical processes of the component. The modeling process will result in the development of a direct runoff hydrograph for various sub-basins and streamflow hydrographs at selected locations in the watershed.

A sub-basin (or sub-area) land surface runoff component is usually used to indicate the movement of water over the land surface and into channels. The input to the land surface runoff component is typically a rainfall hyetograph. Rainfall excess is then computed by subtracting infiltration and detention losses using one of the infiltration functions. The available infiltration loss functions in HEC-1 are: (1) initial loss and uniform loss rate, (2) general loss function, (3) SCS Curve Number, and (4) Holtan method.

In the HEC-1 model, rainfall and infiltration are assumed to be uniform over the sub-basin. The computed rainfall excesses

are then applied to the unit hydrograph to derive the sub-basin outlet runoff hydrograph. The available unit hydrograph options in HEC-1 model are: (1) Snyder's Unit Hydrograph, (2) SCS dimensionless unit hydrograph, or (3) a kinematic wave model.

Another component of the HEC-1 model is the stream routing component which is used to represent flood wave movement in a channel. The input to this component is an upstream hydrograph developed from individual or combined contributions of sub-basin runoff, streamflow routings, or diversions. This hydrograph is then routed to a downstream point, using the characteristics of the channel. The available routing techniques in HEC-1 model include (i) the Muskingum method, (ii) level-pool routing, and (iii) the kinematic wave method.

In the HEC-1 model, a rainfall-runoff and stream routing problem can be represented by a suitable combination of sub-area runoff and streamflow routing components. The order in which the input data components are arranged indicates the inter-connection of the stream network components. Simulation should always start at the uppermost sub-basin in a branch of the stream network. It then proceeds downstream, routing all the flows until a confluence is reached. The flows are combined at the confluence, and then the combined flow is routed downstream.

A reservoir component in the HEC-1 model is similar to that of a streamflow routing component. A reservoir component represents the storage and outflow characteristics of a reservoir or flood-reducing structure. This component functions by receiving upstream inflows and routing them through a reservoir using storage routing methods. The reservoir outflow is only a function of reservoir storage and is not dependent on downstream controls. Spillway characteristics and top-of-dam characteristics are also entered for overtopping analysis.

6.2 HEC-1 Analysis Using Regional Data

The soil characterization information used for the HEC-1 application was taken from the Bechtel studies. This information is summarized in Table 6-1.

The rainfall information was obtained from the BNL analysis described in Chapter 5, using data presented in Chapter 4. The 72-hour rainfall distribution with probability of occurrence of 10^{-5} is shown in Figure 5-4. Figures 5-1 through 5-3 show distributions of 72-hour rains for storms with probability of occurrence of 10^{-2} , 10^{-3} , and 10^{-4} , respectively.

This rainfall data was used as input in the HEC-1 evaluation of the PAR Pond site. The results of this evaluation are summarized in Table 6-2. Figure 6-1 shows the maximum height of water in the reservoir behind the dam for rains with varying

probability of occurrence. Figures 6-2 and 6-3 show the maximum outflow and storage behind the dam for varying probability of occurrence. Figures 6-4 through 6-7 show the input and output hydrograph for each rain.

6.3 HEC-1 Analysis Using Local Data

The local data, as described in detail in Chapter 4, was also used for HEC-1 application. The 72-hour rainfall distribution with probability of occurrence of 10^{-5} is shown in Figure 5-9. Figures 5-6 through 5-8 show distributions of 72-hour rains for storms with probability of occurrence of 10^{-2} , 10^{-3} , and 10^{-4} , respectively. These rainfalls were used in HEC-1 evaluation of the site. The results of this evaluation are summarized in Table 6-3. Figure 6-8 shows the maximum height of water in the reservoir behind the dam for rains with varying probability of occurrence. Figures 6-9 and 6-10 show the maximum outflow and storage behind the dam for varying probability of occurrence. Figures 6-11 through 6-14 show the input and output hydrograph for each rain.

6.4 Wave Impact

Waves, which normally are caused by winds, can damage dams. Major storms usually carry heavy winds. Therefore, it is important to consider the effect of wave height and its runup. The wave height as a result of 80 miles per hour wind was calculated to be 3.5 feet. In addition, this wave will have a runup on the dam face of about 1.5 feet. Some of this water would slosh over the dam. Although the assumption that heavy rainfall (a 100,000-year storm) with 80 miles per hour winds occurring at the same time is not realistic, this possibility should be considered.

The top of the dam is paved and the face of it has ripraps. The dam has a large well-protected freeboard. It is, therefore, unlikely that the dam will be damaged by wave action.

7.0 ANALYSIS OF RESULTS

An evaluation of the HEC-1 analysis results of the PAR Pond site using the regional and local data indicates that, regardless of data origin, rainfall with a probability of occurrence of 10^{-5} (i.e., a 100,000-year storm event) would not overtop the dam. The maximum elevation of water in the reservoir for the 100,000-year storm is 208.55 feet and 207.35 feet, for local and regional data, respectively.

The maximum elevation of the water in the reservoir, the maximum reservoir elevation, is a critical parameter. Figure 7-1 shows the variation of this elevation for rains with varying probability of occurrence. This figure shows that using regional data to make the estimates, yields less conservative results than those developed using local data. However, the difference between the maximum reservoir elevation based on these two data sets is not significant. Figure 7-1 also indicates that the maximum reservoir elevation varies linearly with the variation of the probability of occurrence. This allows an extrapolation of the results beyond the probability of occurrence of 10^{-5} . Based on extrapolation, it is reasonable to state that the probability of overtopping the PAR Pond Dam is less than 10^{-6} .

A 100,000 year storm event causes a maximum reservoir elevation which is 1 to 1.5 feet below the top of the dam. Because of this small margin of safety each input parameter plays an important role. A parametric study would identify the input parameters that have the greatest impact on the results. It is reported in the literature (Yucel, 1992) that the basin lag time and land use could impact the transformation of rainfall to runoff in a given watershed. Given their potential importance the accurate determination of these two parameters for the PAR Pond site is warranted.

A parametric study was performed using the HEC-1 model and a series of systematically varied lag-time (T-lag) and SCS runoff curve numbers (CN) to represent a range of land use conditions. The basin lag-time values of 0.5 hour, 1.07 hour, and 2.00 hours were used, each with a basin CN value of 30, 57, and 80. Figures 7-2 and 7-3 show the variations of the peak discharge and the maximum storage of the flood hydrographs, respectively, with the T-lag and CN parameters. Figure 7-2 indicates that variations of the T-lag parameters do not significantly affect the results. This conclusion is only applicable to the range of values considered and that are applicable to this site.

The results shown in Figure 7-3, however, clearly indicate that the basin curve number can significantly effect the results. The results shown in Chapter 6 of this report are based on a CN value of 57. The CN value is a function of soil type and soil cover. Due to the impact of the CN value on the final results of

the HEC-1 program, it is important to accurately estimate this value. The CN = 57 in this report is only used on the recommendation of Bechtel Co. BNL did not conduct an independent verification of the CN value in this study. Considering the marginal clearance for a 100,000-year storm event (i.e., 1 to 2 ft.), if further evaluations are performed, site inspection to clarify the CN value should be made.

Figures 7-2 and 7-3 can be used to establish a site-specific technical basis for a comprehensive sensitivity analysis aimed at reaching an optimum parameter estimation for the project at hand. Sensitivity analyses incorporating watershed-specific technical data on the design storm will provide more meaningful results than the often-implemented decision-making processes which are usually based on regional evaluations and overall risk and policy considerations.

Recently, a record of precipitation and reservoir elevation for the period 1988 to 1991, was obtained. This data was used to back calculate the CN values. The CN values obtained based on these computations, for this limited data set, are between 25 to 90 (Table 7-1). This is a wide range and, therefore, additional data is needed to accurately estimate the CN value for this site.

In obtaining all these numbers (reservoir elevation during and after the storm) the starting pool elevation was taken to be 202.00 feet which was the value used by Bechtel Co. to represent the operating condition of the reservoir behind the dam. However, the spillway crest is located at an elevation of 200.00 feet. If the initial or starting elevation of the pool is lowered to 200.00 feet to correspond to the spillway elevation, the final reservoir elevation would also be lowered. Figure 7-4 shows the final reservoir elevation, based on HEC-1 output and a starting initial elevation of 200 ft., for different CN values. Considering Figures 7-1 and 7-4, it is clear that by lowering the starting elevation of the reservoir, the final elevation of the reservoir would be lowered from 208.55 feet to 207.25 feet, for a CN value of 57. Figure 7-4 also shows the effect of varying CN values on the final reservoir elevation. Figure 7-5 indicates the effect of starting pool elevation for a 100,000-year rain. This information would help to formulate a decision to lower the reservoir elevation, if needed, to safely accommodate a 100,000-year storm.

Another important factor is the impact of sedimentation in the reservoir behind the dam. It is widely known that sedimentation exists in the dam's reservoir. The present sedimentation accumulated over the last decade. As discussed above, the water level for a 100,000-year storm will reach a height of 208.6 feet (if the starting reservoir elevation is 202.00 feet). This is about 1.4 feet lower than the top of the dam. If soil loss from the sub-basins has continued to accumulate in the reservoir

behind the dam, the 1.4 feet clearance would be reduced drastically. An analysis of the soil losses and sedimentation in this watershed is clearly needed. However, information on the rate and extent of soil loss from the contributing areas are not available. Further, detailed soil characteristics of the contributing sub-basins and soil cover conditions are not readily available, making the evaluation of soil loss and deposition not possible at this time.

It must be emphasized that a 100,000-year storm event is a storm that is expected to occur once every 100,000 years. Such a storm has a probability of occurrence of 10^{-5} for each year. Soil loss and sedimentation will definitely change with time. Therefore, there is a definite need to develop and update information regarding soil loss and sedimentation in this watershed. This information can be developed with site inspection, site monitoring and forecasting, to create a dynamic model of soil loss and sedimentation in the reservoir behind the PAR Pond Dam.

Finally, the results obtained from the HEC-1 program only predict the impact of a 100,000-year storm occurrence today. Any change in land use, land cover, soil topography, and other watershed characteristics could affect the results for future years. It is, therefore, important to consider future changes in watershed characteristics. Small changes can significantly effect the final water level behind the dam. To protect the dam, any natural or manmade changes to the watershed should be monitored and their effect on the hydraulics and hydrology of the site must be evaluated.

8.0 SUMMARY AND CONCLUSIONS

To investigate the probability of a flood overtopping the PAR Pond Dam, BNL conducted a hydrological and hydraulic study. The study was first carried out using rainfall data consistent with the Probable Maximum Precipitation (PMP). Using standard procedures the 72-hour PMP, a "catastrophic" storm, was found to be 47.8 inches for the Savannah River Site. This is the maximum amount of precipitation expected in any period of 72 hours, based on a symmetrical "center-peaking" distribution for the precipitation intensity during the period of the storm. The PMP distribution used throughout the study was developed for a representative watershed area of 31.7 square miles. The resulting Probable Maximum Flood (PMF) was computed using the PMP as input to the HEC-1 program.

The HEC-1 estimate of the PMF corresponding to the PMP indicated that the level of water would be 3.56 feet above the top of the dam. The concept of the PMP and the resulting PMF are often thought to be confusing and arbitrary criteria that cannot be substantiated for some applications. The PAR Pond Dam is considered to be one such application.

Real rainfall data has been collected at the Savannah River Site (11 rain gages) and in the regional area of the site (two raingage stations). These data sets are available for periods of 30 to 46 years. This rainfall data is clearly helpful in the rainfall analyses for this site. A flood frequency analysis based on these 13 raingage stations collecting data for 30 to 46 years is more useful than the PMP value obtained from the National Weather Service's HMR-51 and HMR-52. The PMP is a concept which is developed to simulate the ultimate "catastrophic" precipitation event that could ever be created under the most unfavorable combination of the specific meteorological conditions of a region involving parameters such as temperature, wind, and humidity. Although the impact of the 72-hour PMP was evaluated in this study, BNL contends that a 72-hour rainfall event with a probability of occurrence of 10^{-5} developed from actual site/region rainfall data is more meaningful and appropriate. The 13 raingage stations with 30 to 46 years of data produce a very specific rain distribution for the Savannah River Site. Applying probabilistic methods and using this data, the 72-hour 100,000-year storm was found to be approximately 1/2 of the PMP.

The HEC-1 evaluation for this 72-hour rainfall estimate indicated that the dam would not be overtopped by a 100,000-year storm. That is, a rainfall event with a probability of occurrence of 10^{-5} would not cause the PAR Pond Dam to be overtopped. However, this result is based on selected values of parameters related to the watershed characteristics for the site. The importance of accurately estimating the Curve Number, soil losses

from sub-basins, and sedimentation potentials in the reservoir behind the dam have been discussed earlier in this report and will impact on this result.

9.0 REFERENCES

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SECTIONAL ELEVATION
SPILLWAY AND SLUICWAY STRUCTURE
SCALE 1" = 50'

Figure 1-2 Spillway and Sluice for PAR Pond Dam

Figure 1-3 Plainview and Cross Section of PAR Pond Dam

Figure 2-1 Subbasin Boundaries, Rivers, and Dam Locations

• **ISOHYETAL AREA**

Figure 2-2 Isohyetal Pattern

Figure 2-3 Average Basin Rainfall

Figure 2-4 Rainfall and Losses, Based on PMF

Figure 2-5 Inflow Hydrograph, Based on PMF

Figure 2-6 Combined Outflow Hydrograph, Based on PMF

isolines are drawn through points of approximately equal values. Caution should be used in interpolating on these maps, particularly in mountainous areas.

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Figure 4-2 Max.24 Hour Rain Records for Augusta, GA

Figure 4-3 Max.48 Hour Rain Records for Augusta, GA

Figure 4-4 Max.72 Hour Rain Records for Augusta, GA

MAX.24HRS.RAIN FOR SAVANNAH,GA

Figure 4-5 Max.24 Hour Rain Records for Savannah, GA

Figure 4-6 Max. 48 Hour Rain Records for Savannah, GA

Figure 4-7 Max.72 Hour Rain Records for Savannah, GA

Figure 4-8 Max.24 Hour Rain Records for the Site

Figure 4-9 Max. 48 Hour Rain Records for the Site

Figure 4-10 Max.72 Hour Rain Records for the Site

100 YRS RAIN DISTRIBUTION (10E-2 probability)

Figure 5-1 100 Year, 72 Hour Rain Distribution,
Based on Regional Data

1000 YRS RAIN DISTRIBUTION (10E-3 probability)

Figure 5-2 1000 Year, 72 Hour Rain Distribution,
Based on Regional Data

10,000 YRS RAIN DISTRIBUTION (10E-4 probability)

Figure 5-3 10,000 Year, 72 Hour Rain Distribution,
Based on Regional Data

100,000 YRS RAIN DISTRIBUTION (10E-5 probability)

Figure 5-4 100,000 Year, 72 Hour Rain Distribution,
Based on Regional Data

Figure 5-5 Rainfall Variation for Different Probabilities of Occurrence, Based on Regional Data

100 Yr. Rain Distribution
(0.01 probability) Based on Local Data

Figure 5-6 100 Year, 72 Hour Rain Distribution,
Based on Local Data

1000 Yr. Rain Distribution
(0.001 probability) Based on Local Data

Figure 5-7 1000 Year, 72 Hour Rain Distribution,
Based on Local Data

10000 Yr. Rain Distribution
(1E-04 probability) Based on Local Data

Figure 5-8 10,000 Year, 72 Hour Rain Distribution,
Based on Local Data

100000 Yr. Rain Distribution
(1E-05 probability) Based on Local Data

Figure 5-9 100,000 Year, 72 Hour Rain Distribution,
Based on Local Data

Figure 5-10 Rainfall Variation for Different Probabilities of Occurrence, Based on Local Data

Figure 6-1 Variation of Max. Reservoir Elevation
Using Regional Data

Figure 6-2 Variation of Max. Outflow
Using Regional Data

Figure 6-3 Variation of Max. Storage
Using Regional Data

Figure 6-4 Inflow and Outflow Hydrograph for a 100 Year Storm Event, Based on Regional Data

Figure 6-5 Inflow and Outflow Hydrograph for a 1000 Year Storm Event, Based on Regional Data

Figure 6-6 Inflow and Outflow Hydrograph for a 10,000 Year Storm Event, Based on Regional Data

Figure 6-7 Inflow and Outflow Hydrograph for a 100,000 Year Storm Event, Based on Regional Data

Figure 6-8 Variation of Max. Reservoir Elevation
Using Local Data

Figure 6-9 Variation of Max. Outflow
Using Local Data

Figure 6-10 Variation of Max. Storage
Using Local Data

Figure 6-11 Inflow and Outflow Hydrograph for 100 Year Storm Event, Based on Local Data

Figure 6-12 Inflow and Outflow Hydrograph for 1000 Year Storm Event, Based on Local Data

Figure 6-13 Inflow and Outflow Hydrograph for a 10,000 Year Storm Event, Based on Local Data

Figure 6-14 Inflow and Outflow Hydrograph for a 100,000 Year Storm Event, Based on Local Data

Figure 7-1 Variation of Max. Reservoir Elevation
Based on Regional and Local Data

Figure 7-2 Effect of Lag-time on Max. Storage Behind the Dam

Figure 7-3 Effect of Curve Number on Peak Discharge

Figure 7-4 Variation of Reservoir Elevation,
72 Hour Rain Distribution, Based on Local Data

Figure 7-5 Variation of Reservoir Elevation,
72 Hour Rain Distribution, Based on Local Data,
 10^{-6} Probability of Occurrence

Table 2-1 Summary of HEC-1 Output, Based on PMF

OPERATION	STATION	PEAK FLOW	TIME OF PEAK	TIME IN HOURS, AREA IN SQUARE MILES			BASIN AREA	MAXIMUM STAGE	TIME OF MAX STAGE
				AVERAGE FLOW FOR MAXIMUM PERIOD					
				6-HOUR	24-HOUR	72-HOUR			
HYDROGRAPH AT	Bas3	155055.	40.00	74057.	26005.	9705.	27.32		
HYDROGRAPH AT	PPres	43948.	39.00	13027.	4660.	1075.	4.38		
2 COMBINED AT	PResv	166703.	40.00	86381.	30594.	11581.	31.70		
ROUTED TO	PSp11	84076.	41.50	54389.	19923.	7313.	31.70	213.56	41.50

SUMMARY OF DAM OVERTOPPING/BREACH ANALYSIS FOR STATION PSp11
(PEAKS SHOWN ARE FOR INTERNAL TIME STEP USED DURING BREACH FORMATION)

PLAN 1	ELEVATION STORAGE OUTFLOW	INITIAL VALUE		SPILLWAY CREST		TOP OF DAM	
		202.00		210.00		210.00	
		60000.		85000.		85000.	
		206.		2278.		2278.	
RATIO OF PMF	MAXIMUM RESERVOIR W.S.ELEV	MAXIMUM DEPTH OVER DAM	MAXIMUM STORAGE AC-FT	MAXIMUM OUTFLOW CFS	DURATION OVER TOP HOURS	TIME OF MAX OUTFLOW HOURS	TIME OF FAILURE HOURS
1.00	213.56	3.56	97990.	84076.	32.50	41.50	.00

Table 4-1 Maximum Annual Rainfall Data Collected at Augusta, GA

Year	72 hours	48 hours	24 hours
	I N C H E S		
1949	3.72	3.72	3.24
1950	4.61	4.33	3.63
1951	1.92	1.75	1.45
1952	2.37	2.37	1.98
1953	3.17	3.03	2.66
1954	2.81	2.81	2.81
1955	4.54	4.02	3.16
1956	3.55	3.33	2.34
1957	3.30	2.55	1.68
1958	2.93	2.93	2.49
1959	4.32	4.18	2.71
1960	2.68	2.67	2.55
1961	3.59	3.59	3.01
1962	3.68	3.68	2.81
1963	4.09	2.93	2.43
1964	7.56	7.45	5.95
1965	2.36	2.36	2.10
1966	3.09	3.07	2.01
1967	5.31	5.31	5.31
1968	3.43	2.62	1.59
1969	5.31	5.08	4.18
1970	3.65	3.58	3.25
1971	5.85	4.88	2.68
1972	3.70	3.56	2.51
1973	2.67	2.67	2.67
1974	2.76	2.32	1.78
1975	4.81	4.57	4.55
1976	3.16	3.16	3.01
1977	3.40	3.30	2.61
1978	3.49	3.30	2.78
1979	4.84	3.2	3.67
1980	3.52	3.02	3.45
1981	5.75	3.08	4.44
1982	2.49	2.38	2.34
1983	3.17	3.13	2.71
1984	3.11	2.96	2.96
1985	4.55	3.83	3.50
1986	3.96	3.94	3.64
1987	4.08	3.68	2.60
1988	3.26	3.26	2.86
1989	3.44	3.00	2.51
1990	12.24	8.99	5.32
1991	5.16	4.21	3.73
Mean	3.99	3.66	3.01
$\sum x_i^2$	804.03	649.00	432.18
s^2	2.84	1.74	1.01
Variance	0.066	0.040	0.024
s (standard Deviation)	1.686	1.318	1.007
$\sum (x_i - \bar{x})^2$	591.97	196.53	44.124
C_A	3.084	2.143	1.079
C_u	3.515	2.443	1.230

Table 4-2 Log-Maximum Annual Rainfall Data Collected at Augusta, GA

Year	72 hours	48 hours	24 hours
	I N C H E S		
1949	0.57	0.57	0.51
1950	0.66	0.64	0.56
1951	0.28	0.24	0.16
1952	0.37	0.37	0.30
1953	0.50	0.48	0.42
1954	0.45	0.45	0.45
1955	0.66	0.60	0.50
1956	0.55	0.52	0.37
1957	0.52	0.41	0.22
1958	0.47	0.47	0.40
1959	0.63	0.62	0.43
1960	0.43	0.43	0.41
1961	0.55	0.55	0.48
1962	0.57	0.57	0.45
1963	0.61	0.47	0.39
1964	0.88	0.87	0.77
1965	0.37	0.37	0.32
1966	0.49	0.49	0.30
1967	0.72	0.72	0.72
1968	0.53	0.42	0.20
1969	0.72	0.71	0.62
1970	0.56	0.55	0.51
1971	0.77	0.69	0.43
1972	0.57	0.55	0.40
1973	0.43	0.43	0.43
1974	0.44	0.36	0.25
1975	0.68	0.66	0.66
1976	0.50	0.50	0.48
1977	0.53	0.52	0.42
1978	0.54	0.48	0.44
1979	0.68	0.65	0.56
1980	0.55	0.55	0.54
1981	0.76	0.71	0.65
1982	0.40	0.38	0.37
1983	0.50	0.49	0.34
1984	0.49	0.47	0.47
1985	0.66	0.58	0.54
1986	0.60	0.60	0.56
1987	0.61	0.57	0.41
1988	0.51	0.51	0.46
1989	0.54	0.48	0.40
1990	1.09	0.95	0.73
1991	0.71	0.62	0.57
Mean	0.573	0.541	0.456
Σx_i^2	15.007	13.347	9.722
s^2	0.021	0.0181	0.0186
Variance	0.0005	0.0004	0.0004
S (standard Deviation)	0.145	0.134	0.136
$\Sigma (x_i - \bar{x})^2$	0.1338	0.0712	0.0179
C_u	1.0957	0.7389	0.1777
C_n	1.2486	0.842	0.2025

Table 4-3 Maximum Annual Rainfall Data Collected at Savannah, GA

Year	72 hours		48 hours	24 hours
	1st	2nd	I N C H E S	
	Inches			
1948	3.13	3.08	3.01	3.00
1949	4.39	4.29	4.30	3.30
1950	14.04	12.48	10.66	8.47
1951	4.60	3.33	4.60	4.59
1952	4.13	3.02	3.31	2.90
1953	6.51	3.35	5.48	2.92
1954	6.34	2.54	6.22	5.84
1955	3.05	2.44	2.73	2.72
1956	3.17	2.44	3.17	2.82
1957	8.16	3.32	7.52	3.78
1958	2.58	2.37	2.37	1.68
1959	4.78	4.68	3.58	3.58
1960	5.29	4.60	4.86	3.25
1961	3.96	3.66	3.66	3.07
1962	3.43	3.39	3.43	2.40
1963	5.84	4.02	4.84	3.60
1964	8.33	6.56	6.26	4.61
1965	2.35	2.20	2.02	1.76
1966	4.27	3.93	3.93	2.40
1967	4.17	2.63	3.34	2.77
1968	3.75	2.86	3.73	2.58
1969	--	--	--	--
1970	--	--	--	--
1971	--	--	--	--
1972	4.41	3.95	4.41	4.20
1973	4.98	2.61	4.02	3.29
1974	3.39	2.96	3.39	3.33
1975	3.50	3.50	3.50	3.50
1976	5.64	4.91	5.64	5.09
1977	2.92	2.68	2.92	2.92
1978	2.04	1.80	2.04	2.04
1979	6.90	6.40	6.88	6.79
1980	2.96	2.17	2.96	1.97
1981	5.85	3.99	5.25	4.63
1982	3.52	3.49	3.02	2.98
1983	3.80	3.17	3.80	3.17
1984	4.49	3.96	3.88	2.53
1985	4.08	3.04	3.93	3.62
1986	3.61	2.98	2.81	2.29
Mean	4.68		4.21	3.46
Σx_i^2	998.12		739.96	497.66
s^2	5.99		2.91	1.90
Variance	0.166		0.0809	0.053
S (standard Deviation)	2.447		1.706	1.380
$\Sigma (x_i - \bar{x})^2$	858.13		302.03	161.51
C_v	1.772		1.840	1.859
C_n	2.067		2.147	2.169

Table 4-4 Log-Maximum Annual Rainfall Data Collected at Savannah, GA

Year	72 hours	48 hours	24 hours
	I N C H E S		
1948	0.49	0.48	0.48
1949	0.64	0.63	0.52
1950	1.15	1.03	0.93
1951	0.66	0.66	0.66
1952	0.61	0.52	0.46
1953	0.81	0.74	0.46
1954	0.80	0.80	0.77
1955	0.48	0.44	0.44
1956	0.50	0.50	0.45
1957	0.91	0.88	0.58
1958	0.41	0.37	0.22
1959	0.68	0.55	0.55
1960	0.72	0.69	0.51
1961	0.60	0.56	0.49
1962	0.53	0.53	0.38
1963	0.77	0.68	0.56
1964	0.92	0.80	0.66
1965	0.37	0.30	0.24
1966	0.63	0.59	0.38
1967	0.62	0.52	0.44
1968	0.57	0.57	0.41
1969	--	--	--
1970	--	--	--
1971	--	--	--
1972	0.64	0.64	0.62
1973	0.70	0.60	0.52
1974	0.53	0.53	0.52
1975	0.54	0.54	0.54
1976	0.75	0.75	0.71
1977	0.46	0.46	0.46
1978	0.31	0.31	0.31
1979	0.84	0.84	0.83
1980	0.47	0.47	0.29
1981	0.77	0.72	0.67
1982	0.55	0.48	0.47
1983	0.58	0.58	0.50
1984	0.65	0.59	0.40
1985	0.61	0.59	0.56
1986	0.56	0.45	0.36
Mean	0.634	0.594	0.5097
Σx_i^2	15.46	13.568	10.467
s^2	0.0284	0.0247	0.031
Variance	0.0008	0.0007	0.0009
s(standard Deviation)	0.168	0.157	0.1784
$\Sigma(x_i - \bar{x})^2$	0.1204	0.0736	0.0717
C_n	0.768	0.575	0.3821
C_n	0.896	0.671	0.446

Table 4-5 Maximum Annual Rainfall From Data Collected at the Site

Year	72 hours	48 hours	24 hours
	I N C H E S		
1961	5.73	5.73	5.03
1962	4.95	4.95	4.90
1963	5.45	5.01	4.78
1964	10.88	9.72	7.05
1965	3.77	3.77	3.75
1966	4.90	4.90	4.00
1967	6.75	6.75	6.75
1968	6.92	4.95	4.35
1969	3.57	2.67	2.22
1970	4.25	4.22	4.02
1971	5.75	4.02	3.47
1972	4.72	3.53	3.12
1973	6.25	4.25	3.85
1974	4.89	4.55	3.45
1975	4.24	3.89	2.63
1976	7.21	7.21	6.53
1977	4.40	3.80	3.10
1978	4.03	4.03	2.48
1979	6.20	5.05	5.00
1980	3.90	3.81	3.78
1981	4.76	4.60	4.60
1982	4.66	2.80	2.30
1983	4.65	4.61	2.70
1984	6.33	5.13	4.50
1985	7.55	7.15	3.75
1986	5.40	5.30	5.00
1987	4.22	3.75	3.00
1988	4.45	4.35	2.75
1989	4.85	3.60	2.25
1990	13.34	9.65	7.15
Mean	5.63	4.92	4.07
Σx_i^2	1,076.73	810.19	556.40
s^2	4.34	2.89	2.05
Variance	0.145	0.096	0.068
S(standard Deviation)	2.083	1.70	1.431
$\Sigma (x_i - \bar{x})^2$	577.42	210.72	60.15
C_u	2.36	1.58	0.76
C_n	2.83	1.89	0.91

Table 4-6 Log Maximum Annual Rainfall from Data Collected at the Site

Year	72 hours	48 hours	24 hours
	I N C H E S		
1961	0.758	0.758	0.70
1962	0.695	0.695	0.69
1963	0.74	0.70	0.68
1964	1.04	0.99	0.85
1965	0.58	0.58	0.57
1966	0.69	0.69	0.60
1967	0.83	0.83	0.83
1968	0.84	0.69	0.64
1969	0.55	0.43	0.35
1970	0.63	0.62	0.60
1971	0.76	0.60	0.54
1972	0.67	0.55	0.49
1973	0.80	0.63	0.58
1974	0.69	0.66	0.54
1975	0.63	0.59	0.42
1976	0.86	0.86	0.81
1977	0.64	0.58	0.49
1978	0.60	0.60	0.39
1979	0.79	0.70	0.70
1980	0.59	0.58	0.58
1981	0.68	0.66	0.66
1982	0.67	0.45	0.36
1983	0.67	0.66	0.43
1984	0.80	0.71	0.65
1985	0.88	0.85	0.57
1986	0.73	0.72	0.70
1987	0.62	0.57	0.48
1988	0.65	0.64	0.44
1989	0.69	0.56	0.35
1990	1.12	0.98	0.85
Mean	0.730	0.671	0.585
Σx_i^2	16.46	14.01	10.88
s^2	0.016	0.017	0.021
Variance	0.0005	0.0006	0.0007
S(standard Deviation)	0.126	0.130	0.145
$\Sigma (x_i - \bar{x})^2$	0.0782	0.0467	0.0117
C_n	1.4443	0.7853	0.1418
C_n	1.7332	0.9424	0.1702

Table 5-1: Frequency Analysis on Augusta Data

HOURS	RETURN PERIOD (years)	PROBABILITY	NORMAL	LOG NORMAL	GAMMA	LP3	GUMBEL	WEIGHTED AVERAGE
72	100	10 ⁻²	7.91	8.13	11.15	9.46	9.28	9.01
	1000	10 ⁻³	9.21	10.53	16.63	13.75	12.31	12.27
	10,000	10 ⁻⁴	9.87	12.00	22.28	17.97	15.34	15.20
	100,000	10 ⁻⁵	11.21	15.62	27.77	23.48	18.35	19.18
48	100	10 ⁻²	6.73	7.12	8.69	8.00	7.79	7.64
	1000	10 ⁻³	7.75	9.05	12.09	10.88	10.16	10.02
	10,000	10 ⁻⁴	8.26	10.20	15.65	13.93	12.53	12.19
	100,000	10 ⁻⁵	9.30	13.02	19.08	16.76	14.89	14.82
24	100	10 ⁻²	5.35	5.92	6.21	6.15	6.17	6.05
	1000	10 ⁻³	6.13	7.54	8.05	8.29	7.98	7.85
	10,000	10 ⁻⁴	8.26	8.52	9.55	10.99	9.79	9.68
	100,000	10 ⁻⁵	9.30	10.92	11.17	13.68	11.59	11.88

Table 5-2 Frequency Analysis on Savannah Data

HOURS	RETURN PERIOD (YRS)	PROBABILITY	NORMAL	LOG NORMAL	GAMMA	LP3	GUMBEL	WEIGHTED AVERAGE
	100	10^{-2}	10.37	10.59	13.50	12.23	12.35	11.74
72	1000	10^{-3}	12.27	14.28	18.75	18.01	16.76	16.27
	10,000	10^{-4}	13.22	16.61	23.03	24.54	21.15	20.50
	100,000	10^{-5}	15.15	22.54	28.05	30.96	25.53	25.87
	100	10^{-2}	8.18	9.10	10.53	10.05	9.56	9.55
48	1000	10^{-3}	9.50	12.04	14.19	14.43	12.63	12.91
	10,000	10^{-4}	10.16	13.86	17.18	18.58	15.69	15.81
	100,000	10^{-5}	11.51	18.45	20.67	23.93	18.74	19.95
	100	10^{-2}	6.67	8.41	8.57	9.00	7.79	8.32
24	1000	10^{-3}	7.74	11.55	11.53	13.55	10.27	11.57
	10,000	10^{-4}	8.28	13.56	13.95	18.05	12.75	14.42
	100,000	10^{-5}	9.37	18.76	16.77	23.09	15.22	18.43

Table 5-3: Summary Space-Weighted Data

HOURS	RETURN PERIOD (YEARS)	RAIN (INCHES)
72	100	9.35
	1000	12.77
	10,000	15.81
	100,000	19.96
48	100	7.88
	1000	10.38
	10,000	12.64
	100,000	15.46
24	100	6.33
	1000	8.31
	10,000	10.27
	100,000	12.70

$$\text{Weighted (by space)} = (7W_{\text{AUG}} + W_{\text{SAV}}) / 8$$

Table 5-4: Frequency Analysis on Local Data

HOURS	RETURN PERIOD (YEARS)	PROBABILITY	NORMAL	LOG NORMAL	GAMMA	LP3	GUMBEL	WEIGHTED AVERAGE
72	100	10^{-2}	10.47	10.54	13.92	12.03	12.16	11.64
	1000	10^{-3}	12.09	13.20	19.79	16.65	15.92	15.32
	10,000	10^{-4}	12.99	14.78	24.58	21.00	19.65	18.50
	100,000	10^{-5}	14.54	18.59	28.97	26.48	23.38	22.71
48	100	10^{-2}	8.87	9.40	10.96	10.21	10.25	9.95
	1000	10^{-3}	10.19	11.86	16.48	13.77	13.31	13.01
	10,000	10^{-4}	10.85	13.33	18.86	17.50	16.36	15.64
	100,000	10^{-5}	12.19	16.88	22.77	20.95	19.40	18.92
24	100	10^{-2}	7.39	8.36	8.30	8.71	8.59	8.48
	1000	10^{-3}	8.51	10.83	10.94	11.97	11.13	11.15
	10,000	10^{-4}	9.06	12.33	12.80	14.38	13.70	13.22
	100,000	10^{-5}	10.19	16.06	14.52	18.16	16.26	16.38

Table 6-1 General Site Information

SCS curve number (CN)	57
Land use	60% forest
	30% grange
	10% impervious
	60% A
	35% B
	5% C
Antecedent soil moisture	II
Top of dam elevation, feet NGVD	210
Length of dam, feet	4470
Top width of dam, feet	30 - 45

Table 6-2: HEC-1 Output Results Using Regional Data

Probability of Occurrence	Max. Reservoir Elevation (Ft)	Max. Storage (Ac-Ft)	Max. Outflow (cfs)
10^{-2}	203.62	65066	768
10^{-3}	204.74	68566	1157
10^{-4}	206.13	72922	1696
10^{-5}	207.35	76715	2208

Table 6-3: HEC-1 Output Results Using Local Data

Probability	Max. Reservoir Elevation (Ft)	Max. Storage (Ac-Ft)	Max. Outflow (cfs)
10^{-2}	204.38	67427	1024
10^{-3}	205.73	71661	1525
10^{-4}	206.78	74950	1970
10^{-5}	208.55	80474	2251

Table 7-1 Recorded PAR Pond Dam Rain and Reservoir Elevation

Date	Rain (in.)	Reservoir Reading	CN (Estimated through HEC-1 Analysis)
Oct. 10, 1990	6.96 3.05 <u>1.56</u> 11.57	199.5 <u>200.8</u> 1.3	25
Oct. 23, 1990	4.152 0.168 <u>0.816</u> 5.136	200.2 <u>200.8</u> .6	50
June 26, 1988	2.09 .06 <u>.01</u> 2.16	199.6 <u>200.1</u> 0.5	72
Aug. 20, 1988	.75 .29 <u>.24</u> 1.28	200.0 <u>200.4</u> .4	90
Apr. 9, 1989	1.19 1.09 <u>1.75</u> 4.02	200.0 <u>200.5</u> .5	50
Jan. 28, 1991	0.091 0.89 <u>1.41</u> 3.21	200.1 <u>200.4</u> .3	60
Mar. 19, 1991	1.32 1.14 <u>2.28</u> 4.74	200.1 <u>200.4</u> .3	30
			<u>Av. 54</u>

END DATE
6/22/94