BONNERS FERRY FLOOD LEVEL STUDY REPORT

Includes Kootenai River Channel Capacity Study Report

Authors: US Army Corps of Engineers, Seattle District
Don Bisbee, Carolyn Fitzgerald, Lynne Melder, Pat Wheeler

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EXECUTIVE SUMMARY

This study is in response to the U.S. Fish and Wildlife Service 2000 Biological Opinion for the Kootenai River in Montana and Idaho. The main purposes is this study are 1) to evaluate the impacts of raising the flood stage at Bonners Ferry to elevation 1,770 and 2) estimate the channel capacity of the Kootenai River between Libby Dam and Troy, MT.

For the purposes of this study the evaluation of impacts is executed through the estimation of flood risk from Kootenai River bank or levee overtopping or failure at 25 locations in the lower Kootenai valley. Groundwater seepage is not evaluated as part of this study. The main flood risk parameters computed are the annual probability of flooding and expected annual damages for each discreet leveed area in the Kootenai Valley. The computation of these parameters typically is associated with a federal flood damage reduction study. While this is not a flood damage reduction study, these parameters are useful in evaluating the impacts of increasing the flood stage given the current (based on a 2004 inspection) condition of the locally owned levees.

This study evaluated four different Libby Dam operating scenarios:

1. Operating Libby Dam with a Bonners Ferry flood stage of elevation 1,764 and providing powerhouse capacity (approximately 25,000 cfs) sturgeon flows (1764PH)

2. Operating Libby Dam with a Bonners Ferry flood stage of elevation 1,764 and providing powerhouse capacity plus 10,000 cfs (approximately 35,000 cfs) sturgeon flows (1764PH10)

3. Operating Libby Dam with a Bonners Ferry flood stage of elevation 1,770 and providing powerhouse capacity (approximately 25,000 cfs) sturgeon flows (1770PH)

4. Operating Libby Dam with a Bonners Ferry flood stage of elevation 1,770 and providing powerhouse capacity plus 10,000 cfs (approximately 35,000 cfs) sturgeon flows (1770PH10)

The findings of this study indicate that given the 2004 condition of the levees, under a 1764PH10 operation, flood risks would increase at the wildlife area adjacent to the Canadian Border. Under a 1770PH operation flood risk increases at 14 areas when compared to 1764PH. Under 1770PH10 these 14 areas all experience an additional increase in flood risk. The degree of flood risk increase varies from area to area. In addition, the annual probability of flooding/expected annual damage relationships vary as well. In general the areas where flood risk is increased under a 1,770 flood stage are agricultural areas. In the Bonners Ferry area the Kootenai River Inn appears to be at most risk in terms of a 1,770 flood stage.

Given that the levees are locally owned there no authorization, other than local participation in the PL84-99 program, for federal involvement in the maintenance or repair.

The channel capacity in the Libby Dam to Troy, MT reach of the river was estimated based on the 2002 high flow event and available data. This event produced stages which were in close proximity to a number of dwellings located along the river. While the discharge data from Libby Dam is reliable, lack of other mainstem Kootenai River gauges in the reach make it difficult to estimate the local flow present. Based on calculating the local flow a couple of different ways, this study estimates the channel capacity at Libby, MT to be approximately 42,700 cfs and the channel capacity at Troy, MT to be approximately 45,000 cfs. Depending on the timing and magnitude of flow on local tributaries, given the current channel capacity, the potential exists, especially in May, for the local flow to constrain the release of a powerhouse capacity plus 10,000 cfs sturgeon flow.

SECTION 1 INTRODUCTION

1.1 INTRODUCTION

The Kootenai River White Sturgeon was listed as an endangered species on September 6, 1994, under the Endangered Species Act. Critical habitat for Sturgeon on the Kootenai River was designated on September 6, 2001. The 2000 U.S. Fish and Wildlife Service (USFWS) Biological Opinion (2000 BiOp) recommends higher spring Kootenai River flows to enhance spawning habitat and aid sturgeon recruitment. This study addresses the Reasonable and Prudent Alternatives (RPA) 8.2a2, 8.3.a and b, presented as follows:

> 8.2a2. The report of the proposed Kootenai River channel capacity investigation shall include or append all site-specific elevation data gathered on structures which could be impacted and data on the defined 100-year floodplain. Should the evaluations of channel capacity study determine that structural floodplain encroachment may constrain the increased release capacities at Libby Dam (specified herein, up to 35,000 cfs at Libby Dam), the December 30, 2001 report shall also include any remedies necessary to restore this channel capacity, the means available to effect those remedies, and a schedule to do so.
>
> 8.3.a. By spring 2001, the Corps shall evaluate flood levels and public safety concerns along the banks of the Kootenai River below Libby Dam, and the feasibility of increasing releases above any identified channel capacity constraints through structural or nonstructural means. A report shall be provided to the Service by December 1, 2001.
>
> 8.3.b. By May 2004 the action agencies shall seek means to restore, maintain, or enhance levees throughout the Kootenai Valley to the greater of: 1) the PL 84-99 Corps' 1961 levee specifications, or 2) the levee elevations needed to contain the flows/river stages of the 100 year event as authorized for the Libby Project, which is now defined as 1770 feet at Bonners Ferry. The action agencies shall also seek means to incorporate conservation measures for sturgeon, including self maintaining rocky spawning substrates, as a component and Federal purpose of any new levee project above.
>
> In the interim, the Service and Corps will coordinate efforts to attempt to limit sturgeon spawning flows so they do not exceed a levee elevation of 1764 feet at Bonners Ferry. (Note: This may not always be possible

during periods of unusual local runoff which may be beyond control of Libby Dam.).

The current National Weather Service (NWS) flood stage at Bonners Ferry is 1764 feet, based on the NGVD1929 datum. The Corps currently operates Libby Dam in such a manner as to not exceed elevation 1764 feet at Bonners Ferry if at all possible.

1.2 Basin Description

At its confluence with the Columbia River in British Columbia Canada, the Kootenai River (spelled Kootenay in Canada) drains an area of approximately 19,300 square miles. (USACE 1984) The basin is located in southwest British Columbia, northwest Montana, and northern Idaho (See Figure 1). The Kootenai River originates near the continental divide in Canada and flows south into Montana, past the towns of Canal Flats and Fort Steele, B.C., and Eureka, MT. At Canal Flats, the Kootenay is within one to two miles of the Columbia River source. At approximately river mile (RM) 222, in Montana, the Kootenai River is impounded by Libby Dam, a 2,887 foot wide, 432 foot high structure completed in 1972 (USACE 1984). The impoundment behind Libby Dam, known as Lake Koocanusa, is approximately 90 miles long and extends approximately 42 miles into British Columbia at full pool.

Major tributaries to the Kootenai above Libby Dam include the Tobacco River in Montana and the Elk, Bull, White, and St. Mary rivers in British Columbia. From Libby Dam, the Kootenai flows in a northwesterly direction through the towns of Libby, MT (RM 204) and Bonners Ferry, ID (RM 153), past the U.S./Canada international boundary (RM 106), to Kootenay Lake in southwestern British Columbia. Major tributaries along this reach of the Kootenai include the Fisher and Yaak Rivers in Montana, the Moyie River in Idaho, and the Goat River in British Columbia.

About 25 miles north of the international boundary the Kootenai River joins Kootenay Lake at its south end, near the town of Kuskonook, B.C. Kootenay Lake is long and narrow, and aligned in roughly a north-south orientation. Other major tributaries to Kootenay Lake besides the Kootenai River include the Duncan and Lardeau Rivers, which join the Lake at its north end. Flow from the Duncan River is regulated by Duncan Dam. Kootenay Lake drains through the West Arm, near Nelson, B.C., where it becomes the lower Kootenay River, eventually reaching the Columbia River near Castlegar, B.C. Kootenay Lake elevation is regulated by Corra Linn Dam at the lake's outlet. Depending on how Corra Linn is operated, the hydraulic control for the lake outlet can either be the dam itself or a natural constriction known as the Grohman Narrows located approximately 7 miles upstream of the dam. Other dams on the lower Kootenay River besides Corra Linn include Upper Bonnington, Lower Bonnington, South Slocan, and Brilliant Dams.

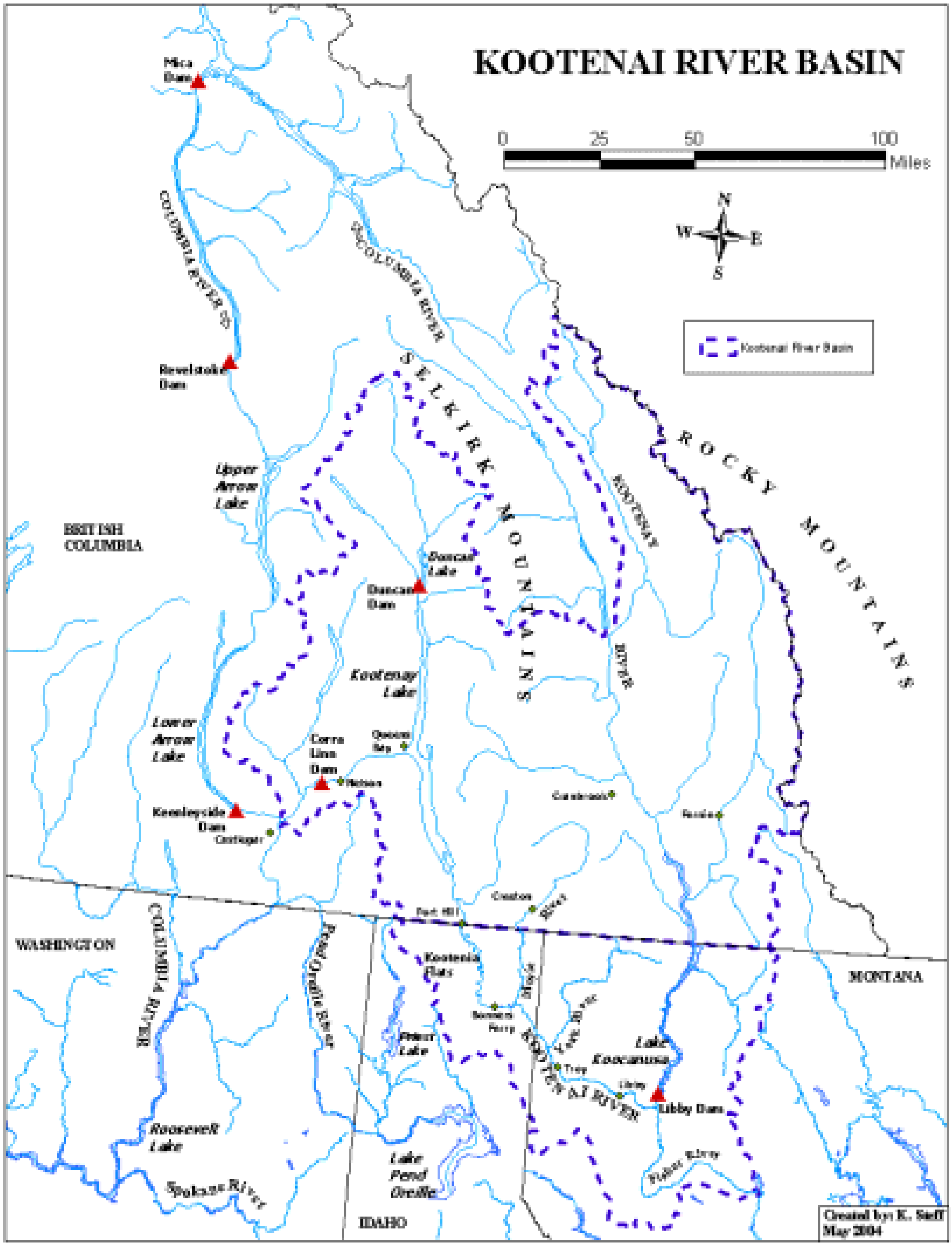


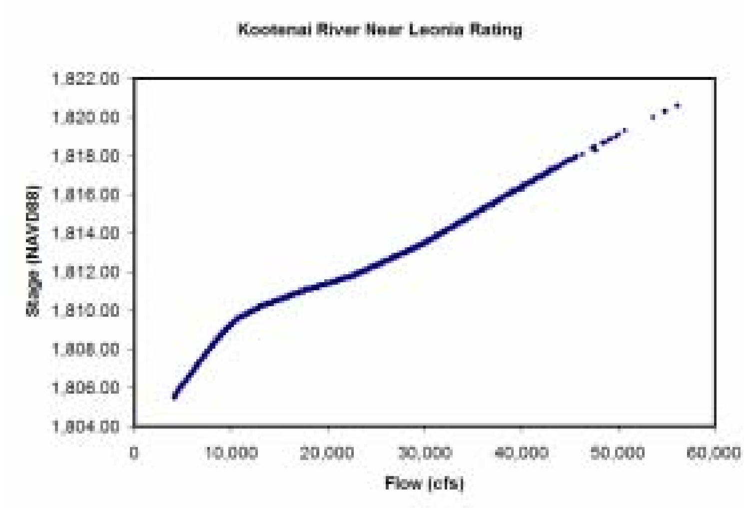
Figure 1. Kootenai River Basin

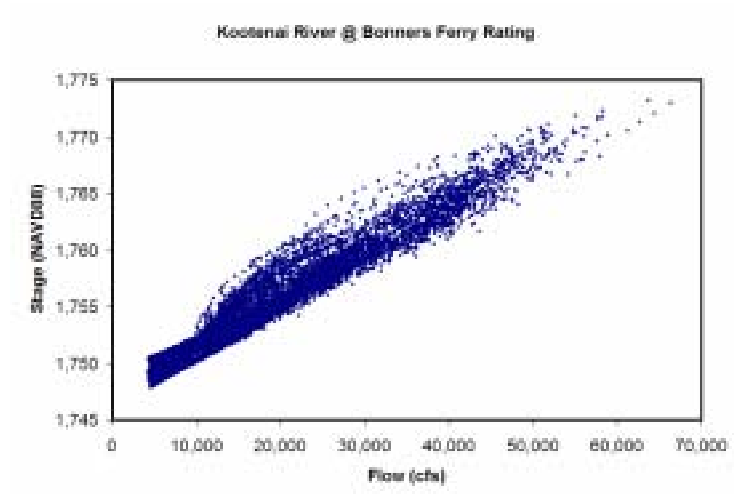
The Kootenai Basin lies in a rugged, mountainous region. Elevations range from approximately 1,500 feet above sea level near the Kootenay's confluence with the Columbia River to more than 11,000 feet in the vicinity of the river's source (USACE 1984). Near its headwaters, the Kootenay is bordered by the Kootenay and Park Ranges of the Canadian Rocky Mountains (Daley 1981). Between Canal Flats and Libby Dam the Kootenai River flows in a geological formation known as the Rocky Mountain Trench. Downstream of Libby Dam the river is located in a more confined valley, located between the Cabinet Mountains to the south and the Purcell Mountains to the north. At approximately RM 193 the Kootenai passes Kootenai Falls, near the town of Troy, MT. At Bonners Ferry, Idaho, the river enters a much wider valley, commonly referred to as Kootanai Flats, which extends to the river's entrance to Kootenay Lake. At this point the river valley is bordered by the Purcell Mountains to the east and the Selkirk Range to the west.

The reach extending from below Kootenai Falls to approximately the confluence with the Moyie River is located in a narrow canyon with very little flood plain area. This reach is relatively steep with an average gradient of about 3-1/3 feet per mile of river. Downstream of this section, between Bonners Ferry, Idaho, and Kootenay Lake, the gradient of the river is much less, only about 1/3 of a foot vertical drop per mile of river. Here the river meanders in a wide valley. Historically this reach of river had a large area of floodplain available. However, an extensive system of levees constructed starting in the early 1900s has for the most part confined the river to its main channel. The transition area between the steeper canyon reach and the lower meandering reach is known as the braided reach. This short reach (approximately 3 miles) upstream of Bonners Ferry is characterized by shallow, braided channels cut through deposited gravels.

In the meandering reach downstream of Bonners Ferry, river stages can be the result of various combinations of flow and Kootenay lake elevation. For a given inflow and lake elevation, there is a point along the reach, typically in the braided reach, where river stages become strictly a function of river flow. The point where this occurs is the upriver extent of the Kootenay Lake backwater influence. The location where this occurs is variable with time (depending on the rate of change of flow) and is a function of many factors such as lake elevation, inflow to the reach, local inflow, rate of change of inflow, duration of a flood peak, channel gradient, and channel geometry.

For the section of river within the backwater influence, the degree of both the flow and lake elevation components on stage vary as well. River stages at locations close to the Canadian border are more sensitive to lake elevations than locations upstream, closer to Bonners Ferry. To illustrate this, Figure 2 shows plots of flow versus stage for three locations on the river.





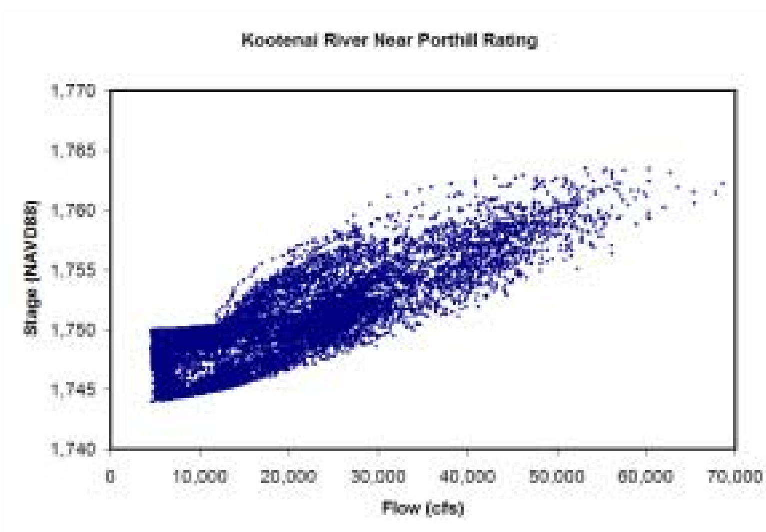


Figure 2. Selected Kootenai River Rating Curves

The underlying data used in these plots is from a hydraulic simulation of 52 years of Kootenai River hydrologic data. The top graphic is for a location near Leonia, on the Montana-Idaho border. This location is well out of the backwater influence of the lake. This plot shows very little scatter in the data. The middle plot is from the Bonners Ferry area, which is under the lake backwater influence most of the time and the bottom plot is from a location at Porthill, at the international boundary, closer to the lake. The increase in the scatter of the plots closer to Kootenay Lake indicates the increasing influence of lake elevation on river stages for locations farther downstream.

The principal mechanism driving large floods in the Kootenai Basin is spring snowmelt. Since the development of the lower valley in the late 1800s, the basin has experienced several large floods. The largest on record is the flood of 1894. This was a very large flood throughout the entire Columbia Basin and, system-wide, is thought to be a flood event with an annual probability of occurrence of 0.5%[1], often referred to as a "200-year" event. Additional large floods in the valley occurred in 1916, 1948, 1956, and 1961. Other large runoff years were 1974 and 1997. The presence of Libby Dam significantly mitigated the downstream impacts from runoff during these two years. 1961 is the last year where any appreciable levee failures occurred. While other floods have had larger flood volumes and higher estimated peak flows, changes in the configuration of the local levees, which resulted in more floodwater confinement within the main river channel, resulted in the highest recorded peak stages on record at Bonners Ferry (USACE 1961). During the 1961 flood, 7,021 acres of leveed land in the lower Kootenai valley were flooded due to levee failures (USACE 1961).

Since 1972, Kootenai River flows have been regulated by the Libby Dam Project, a multi-purpose project authorized to provide flood control benefits both locally and on the lower Columbia River as well. The project provides local flood control benefits via a combination of reservoir storage and locally owned levees in the lower Kootenai River valley below Bonners Ferry. In addition, the Libby project has a power generation facility with a current capacity of 600 MW. The project has a usable storage capacity of 4.98 million acre-feet and regulates 8,985 square miles of the Kootenai basin. This leaves approximately 3,700 square miles of the basin between Libby Dam and Bonners Ferry that is unregulated.

Prior to the Libby regulation of the Kootenai River, the annual hydrograph of the river was comprised of low flows through the fall and winter and high flows during the spring snowmelt period. With Libby Dam regulation, the hydrograph is comprised of higher fall and winter flows and lower spring flows. Figure 3 shows summary hydrographs for the Kootenai River at Leonia computed from 1929 to 2002 daily flow data. It contains plots for both pre- and post-Libby Dam years, as well as for the years

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[1] A 0.5%-chance-exceedance flood has a 1 in 200 chance of being equaled or exceeded in any given year. It is sometimes called a "200-year flood".

1992 to 2002, to show how additional spring flow, provided for sturgeon, has impacted the summary hydrograph. The plots were constructed for each category by averaging all the daily flows for each day in the category and plotting the values.

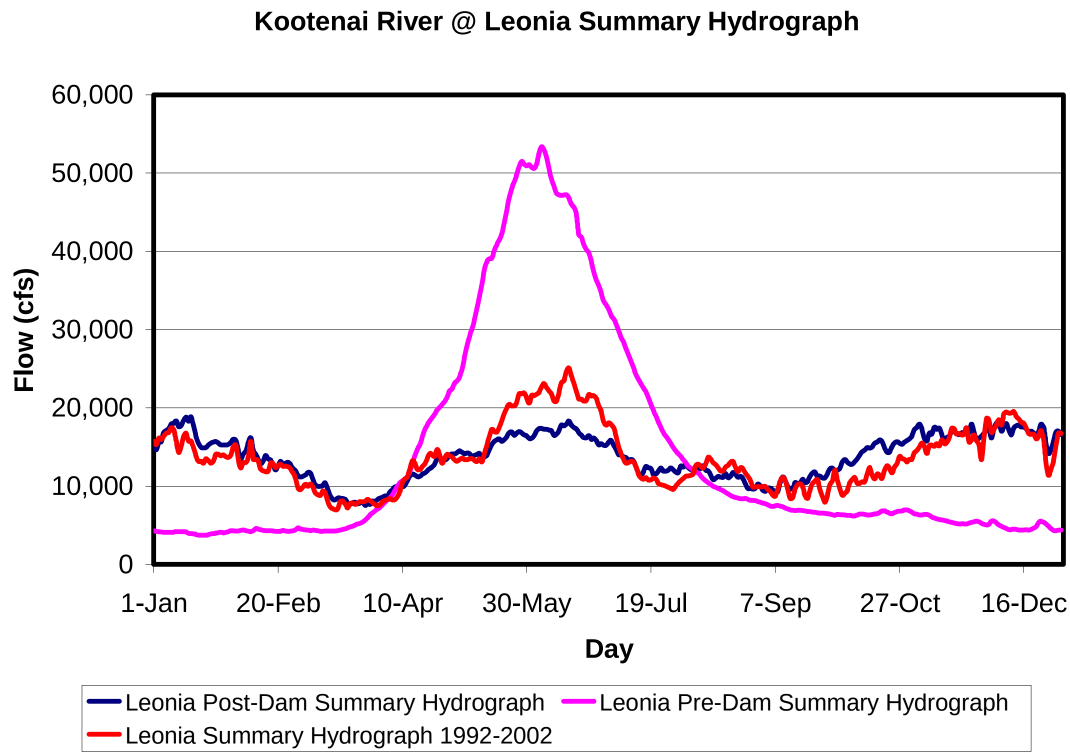

Figure 3. Leonia Summary Hydrograph (Average)

The current objective of the Libby Dam Project is to regulate outflows such that elevation 1,764 feet at Bonners Ferry, ID will not be exceeded, if possible. If an operation to provide additional flow for sturgeon would result in exceeding elevation 1,764 feet, the operation would be modified and project outflow would be reduced. It is recognized that conditions may be such that it is not possible to regulate the Bonners Ferry stage at or below 1,764 feet.

It is difficult to measure discharge at Bonners Ferry due to Kootenay Lake backwater influence. Various combinations of lake elevation and flow can produce damaging stages at Bonners Ferry. As a result a stage-based regulation target has subsequently been adopted. In the 1970s it was determined that elevation 1770 feet at Bonners Ferry was the elevation at which damage in the Bonners Ferry area would begin.

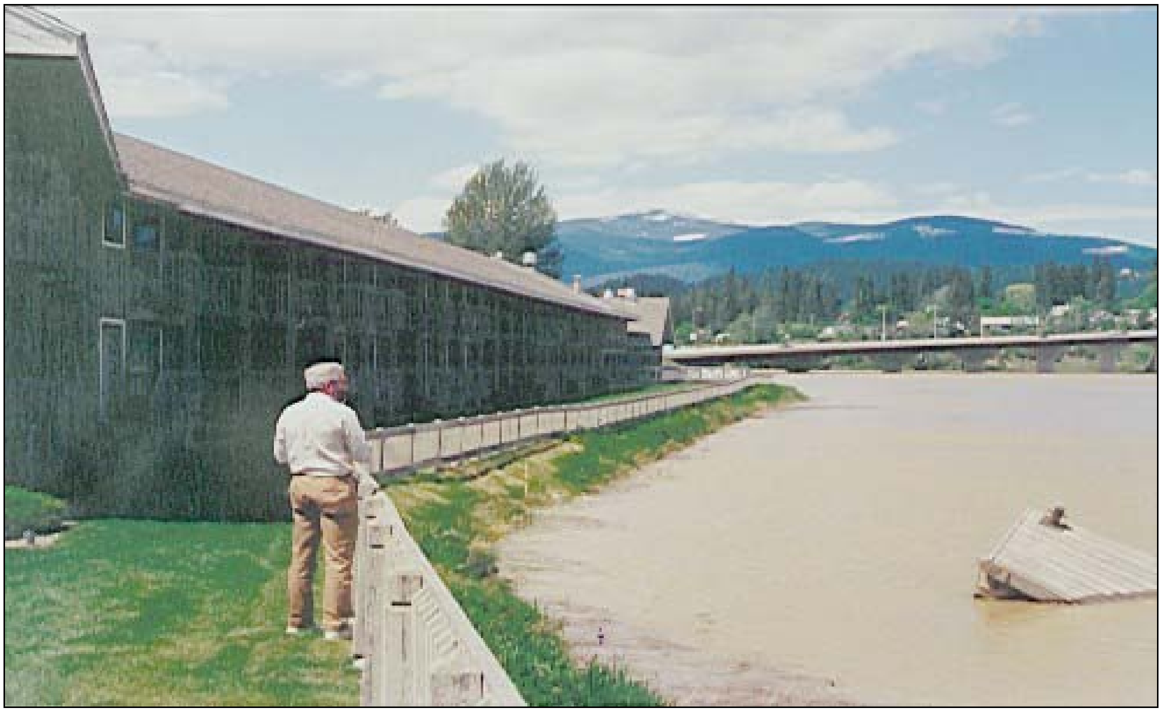

Figure 4. 1997 photo of Kootenai River at 1764.67 ft at Kootenai Tribe casino.

In the mid 1990s, due to concerns about the condition of the local levees, it was determined that Libby Dam would be operated so that stages at Bonners Ferry would be kept below elevation 1764 feet if possible. This change in operation recognized there might be instances where this is not possible; elevations between 1764 and 1770 feet at Bonners Ferry might result from floods more frequent than the 0.5% chance exceedance (200-year) Kootenai River flood. Figure 4 is a photograph from 1997 of the Kootenai River adjacent to the Kootenai Tribe casino at a stage of 1764.67 feet.

1.3 OBJECTIVES

The objective of this study is to respond to the 2000 USFWS BiOp RPAs 8.2a2, 8.3a, and 8.3.b to a sufficient level of detail. The study will:

- Revisit the effective flood protection level implied in the Libby Project authorization language. The implied level of protection is that the project be capable of regulating the 0.5% chance exceedance (200-year) Kootenai River flood to elevation 1770 at Bonners Ferry.

- Use a risk-based analysis to compare four Libby Dam operating scenarios: 1) operating the project with flood stage of elevation of 1764 feet at Bonners Ferry and a spring fish flow capped at powerhouse capacity (noted as 1764PH); 2) operating with a flood stage of 1764 at Bonners Ferry and a spring fish flow capped at powerhouse capacity plus 10 kcfs (noted as 1764PH10); 3) operating with a flood stage of 1770 at Bonners Ferry and a spring fish flow capped at powerhouse capacity (noted as 1770PH); and 4) operating with a flood stage of 1770 at Bonners Ferry and a spring fish flow capped at powerhouse capacity plus 10 kcfs (noted at 1770PH10). These scenarios will be compared based on differences in flood risk at locations between the Bonners Ferry area and the Canadian border. Flood risk will be quantified based on the current condition of the levees. Any levee failures are assumed to occur only at levee locations adjacent to the Kootenai River and not along tributary levees such as those along Deep Creek.

- Update off-channel depth-damage data for the valley.

- Update the lower valley river levee condition by identifying probable failure locations and failure probability-elevation relationships for each drainage district or off-channel area protected by a river levee. Levee failure is only based on Kootenai River stage. Duration of stages is not considered.

- Determine Libby Dam to Troy, MT, channel capacity. This portion of the study is intended to bring to closure a report from 2004 (see **Appendix D**).

All four of the Libby Dam operating scenarios include a spring sturgeon flow component. The mechanism for comparing these operating scenarios will be the computation of expected annual damages (EAD), conditional non-exceedance probabilities, and annual exceedance probabilities. These parameters are computed using the HEC-FDA computer program. Inputs to HEC-FDA include the results from a reservoir and hydraulic modeling effort (to obtain stage-frequency relationships and associated uncertainty), levee condition data, and stage-damage data with uncertainty.

This study looks at flooding and resulting damages from surface water flow only. Issues regarding seepage are part of a separate study. In addition, this study looks only at flooding in the Kootenai River Valley. The Libby Dam Project is part of the larger Columbia River basin system. For this study, hydrologic/ hydraulic simulation modeling for Libby Dam release decisions are based only on local conditions. The project's role in the larger Columbia River system and how it might impact project operations, and possibly the results of this study, are not taken into account.

The analyses of elevation 1,770 at Bonners Ferry as a regulation stage and a sturgeon flow of 10 kcfs above powerhouse capacity is only done as part of this study's response to the USFWS 200 BiOp. There is currently no federal action to increase the regulation stage at Bonners Ferry above 1,764 and/or provide a sturgeon flow in excess of current powerhouse capacity.

1.4 STUDY OVERVIEW

Following is a brief outline of the steps, tasks, and products required for execution of this study. In-depth discussion of the technical aspects and assumptions made to execute this study are described in later sections of this document. The basic tasks/products required for the risk and uncertainty (R&U) analysis are as follows:

- **Economic Data: Elevation-Damage Function --** Quantification of monetary damages resulting from various Kootenai River stages at flooding locations of interest. This results in a depth-damage or stage damage function with uncertainty for areas of interest within the basin.

- **Levee Analysis --** Quantification of geotechnical uncertainty through the identification of levee probable failure locations for each flooding area of interest, and estimation of probable non-failure point (PNP) elevations and probable failure point (PFP) elevations. As per U.S. Army Corps of Engineer's publication EM 1110-2-1619 (USACE 1996), a PNP elevation is defined as the "water elevation below which it is highly unlikely the levee would fail" and a PFP is the "water elevation above which it is highly likely the levee would fail". For the purposes of this study, the PNP elevation carries a probability of levee failure of 0.0 (0%) while the PFP elevation carries a probability of levee failure of 0.85 (85%).

- **Hydrology and Hydraulics: Stage-Frequency Curves --** Development of a function for each levee failure location which describes the annual probability of the river attaining a given elevation for a homogeneous set of Libby Dam outflow data. Each outflow data set represents a particular Libby Dam operation type. There is not an observed record of stages at every point of interest. Even if there were such a record, it would not accurately describe the stage-frequency function due to the fact that part of the period of record is during unregulated conditions while another part is during regulated conditions. Under regulated conditions, the project has been operated under different operational rules over the years. As such, it is necessary to recreate this stage data at both gaged and ungaged locations using Libby Dam outflows based on a desired operating method. This is accomplished using appropriate hydrological data and determining what type of Libby Dam releases would be appropriate based on a given operation. These releases are then input into a hydraulic model, which also includes estimated natural local flow as inputs, and resulting stages at various locations along the reach are computed for the period of record. To better estimate the configuration of the extreme event portion of the stage-frequency curve, more work is done with available hydrology to estimate what the hydrology of these events would look like for further hydraulic simulation. Along with sensitivity analysis, the resulting stage data is used to create a stage-frequency curve, with uncertainty, which represents the annual probability of the river reaching a particular stage given a Libby Dam operating scheme. The underlying assumption with this product is that historic hydrological events are representative of what might occur in the future.

> It is important to note that the frequency curves developed as part of this study were constructed only to facilitate the execution of this study and are not intended to replace any existing frequency curves currently used by local, state, federal, tribal, or private entities.

The hydrology and hydraulic model developed for construction of the stage-frequency functions will also be used for analysis of the Kootenai River channel between Libby Dam and Troy, MT.

SECTION 2 LIBBY DAM HYDRO-REGULATION

Simulated hydro-regulations were used to provide Libby outflows for the upper boundary conditions for the hydraulic modeling. The outcome of the hydro-regulations can be affected by many factors, including (but not limited to): the assumed water supply forecasts, the rules used to trigger operational changes, and the shape and timing of the fish-related flows. To minimize bias, a consistent set of rules regarding flood control drafts, operational foresight, and fish-related flows were applied in all model simulations.

Modeling of the Kootenai River basin was conducted using the Corps SSARR and AutoReg computer programs. The modeling was conducted using a daily time step, providing daily output of parameters such as reservoir elevations, Libby Dam outflows, Kootenay Lake elevations, and Bonners Ferry stages. This output was then used in the hydraulic modeling phase of this study.

2.1.1 Period of Record/Extreme Events Modeling

A 52-year record (1948-1999) was used in this study. This period of time encompasses a wide variety of water years, and therefore provides a good data set describing a wide range of hydrology. However, the data set is still limited, as it is not large enough to produce frequency curves that depict the probability of extremely rare events having probabilities of 1% or less. Hydrographs were created for the 1%[2], 0.5% and 0.2%[3]-chance exceedance floods. These hydrographs were used to create a set of inflows and forecasts for the hydro-regulation modeling.

2.1.2 Water Supply Forecasts

In the Columbia River basin, the quantity of runoff from snowmelt is highly variable from one year to the next. Due to this variability, flood control operations at large storage projects like Libby Dam are guided by a SRD (Storage Reservation Diagram). The SRD is used in combination with a seasonal water supply forecast to determine how much space is needed for flood control. The use of forecast data in the hydro-regulations, as opposed to observed volumetric runoff, adds the element of

[2] A 1%-chance-exceedance flood has a 1 in 100 chance of being equaled or exceeded in any given year. It is sometimes called a “100-year flood”.

[3] A 0.2%-chance-exceedance flood has a 1 in 500 chance of being equaled or exceeded in any given year. It is sometimes called a “500-year flood”.

uncertainty that is experienced in real-time water management and is a more rigorous test of a flood control operation.

The water supply forecasts used for this study are a combination of derived and actual water supply volume forecasts for the 1948-1999 period. In 1986, Wortman and Morrow of the Corps' North Pacific Division developed a forecasting procedure to predict seasonal runoff based on observed conditions in and around the Kootenai basin. The Libby forecast procedure for runoff volume was revised in 2003. The new forecast is referred to as the PCReg (Principal Components Regression Equation) and incorporates the Southern Oscillation Index (SOI) as part of runoff forecasts issued in November and December. The December 1 forecast is used to determine the December 31 draft requirement. For the January and subsequent draft requirements, the Wortman-Morrow forecasts were used.

2.1.3 The 1938 IJC Order on Kootenay Lake

In the model simulations, Kootenay Lake, located in British Columbia at the lower end of the Kootenai basin, is regulated according to rules defined by the International Joint Commission (IJC) Order of 1938. When a conflict existed in meeting the 1938 Order at Kootenay Lake, Duncan Reservoir release was reduced to passing no more than inflow and Libby Dam was allowed to continue to draft, if possible. At no time were Libby or Duncan Dams required to pass less than inflow by this order. Throughout the simulations, Corra Linn Dam at the outlet of Kootenay Lake operated according to its upper rule curve, unless the outflow required by the rule curve exceeded the hydraulic capacity at Grohman Narrows. Grohman Narrows is a natural constriction in the channel located upstream of Corra Linn Dam. At this location, the channel has a relatively small cross-sectional area, creating a "pinch point" that physically limits the amount of flow that can pass through the opening. Once the spring rise of Kootenay Lake has commenced, the rule curve is no longer fixed and is instead determined by a "lowering" formula. During this "lowering" period, the modeled results for Kootenay Lake are based on the hydraulic capacity of Grohman Narrows.

2.1.4 Upper Rule Curves

As a prerequisite to performing flood control simulations for the Kootenai basin, Upper Rule Curves (URCs) that guide seasonal reservoir flood control operations during the evacuation period were developed for storage projects in the basin (Libby Reservoir, Kootenay Lake , and Duncan Reservoir). URCs were developed by using a project's SRD in conjunction with seasonal water supply forecasts for the project, on a month-by-month basis, to calculate the winter and early spring reservoir levels required to provide adequate flood control that year.

The Libby Reservoir URCs for this study were developed using seasonal runoff forecasts and the VARQ SRD. VARQ is a flood control method where flood control space is reduced at Libby Dam during the winter months, and a variable flow is released during the refill period. Libby began operating according to VARQ flood control in 2002 and will continue using VARQ an interim basis until a final decision on permanent implementation is made. The URCs reflect a variable end-of-December flood control requirement based on each year's 1 December PCReg forecast.

URCs from high runoff forecast years from the period of record modeling were used to develop the URCs for the hypothetical floods (1%, 0.5%, and 0.2%-chance-exceedance floods). For Libby's URC for the hypothetical events, the year 1974 was used because of its consistently greater than 8 maf (million acre-feet) forecasts. The flood control draft for this year was 2287 feet. For Duncan Reservoir, 1956 was chosen for its greater than 2.3 maf forecasts. New URCs were developed for Libby and Duncan Reservoirs only. The URCs for Kootenay Lake are identical for every year.

In a truly single-purpose flood control simulation, Libby would operate to its URC and would deviate from it only due to a minimum flow requirement, a flood emergency requiring temporary impoundment of water above the URC, or to prevent an IJC violation at Kootenay Lake (as discussed in the previous section). Until the Commencement of the Spring Rise in Kootenay Lake, Libby would be operated to its URC unless this results in a violation of the IJC order, in which case Libby would be limited to passing inflows. After the commencement of the spring rise, Libby may resume drafting to its flood control target (URC). Further drafting at this time depends on the forecast, reservoir elevation, and when refill begins. Drafting Libby after the commencement of the spring rise was only done for the extreme hypothetical events, not for the period of record modeling. For the period of record modeling, an additional assumption was used such that Libby would not release more than powerhouse capacity (i.e., would not spill) to reach its flood control target elevations. For the extreme events modeling, this assumption was removed so that Libby could release more than powerhouse capacity to reach its flood control target elevation (i.e., draft to 2287 feet). .

2.1.5 Powerhouse Capacity

The hydro-regulation model assumes a powerhouse capacity ranging from 19,000 cfs to 27,600 cfs, depending on reservoir pool elevation (head). For planning purposes for fish-related flows in the springtime, a powerhouse capacity of 25,000 cfs was used. The powerhouse capacity-head relationship used for modeling is based on historic data from the project. This was deemed to be the most realistic choice for estimating powerhouse capacity, rather than assuming a full wicket gate opening where the maximum powerhouse capacity was as high as 29,000 cfs. The hydro-regulation modeling for this study assumed that all five generating units at Libby Dam were available.

2.1.6 Local Flood Control and Refill

A difference among the four Libby Dam project operation scenarios compared in this study (see Section 1.3) is the elevation to which river stages are limited at Bonners Ferry by regulating outflow from the dam. Flood stage is defined as the level or stage at which a stream overflows its banks or the stage at which damage or public safety concerns arise. In 1997, the National Weather Service established a flood stage at Bonners Ferry of 1764 feet. Prior to this, the Bonners Ferry flood stage was 1766.5 feet for several years after being reduced from elevation 1770.

Operation of Libby Dam includes an evacuation phase and a refill phase. With VARQ Flood Control, the release during refill varies with the runoff volume forecast and is further refined depending on reservoir elevation. Refill begins 10 days before the forecasted exceedance of the Initial Controlled Flow (ICF) at The Dalles, Oregon. The Dalles, located approximately 80 miles east of Portland on the Columbia River, is the control point used to provide flood control to the Portland/Vancouver, WA area. The VARQ outflow during refill was determined according to VARQ Operating Procedures (USACE 1999).

2.1.7 Fish Flow Template

In general for this evaluation, between October and April, Libby Dam operations are driven by flood control requirements. Special operation of Libby Dam to provide fish flows downstream is not required until the late spring and summer. The fish flow proposal, developed based on discussions between the Corps and USFWS suggests a minimum requirement for sturgeon and bull trout flows. A fish flow template (FFT) was developed to define the timing and shape of fish flows for modeling purposes. Although this FFT is different for each year due to the difference in the May forecasts, the shape is similar for all years. The fish flows are abandoned when necessary for flood control at Bonners Ferry.

Beginning in mid to late May, the requested volume of sturgeon water is released. The volume is based on the May water supply forecast. Immediately following the sturgeon flow augmentation at powerhouse capacity, the outflows from Libby are ramped down to the bull trout minimum flows ranging from 6,000 to 9,000 cfs. Then, before August 31, a portion of the water stored behind Libby Dam must be released for the benefit of salmon in the lower Columbia River.

It should be noted that the modeling of fish flows for this study differs from that used in the Upper Columbia Alternative Flood Control and Fish Operations EIS (UCEIS). At the time simulation modeling was done for the UCEIS, a FFT with variable start dates based on runoff forecast was mutually agreed to by the USFWS and the Corps.

By contrast, the FFT for this study is based on an updated proposal to deliver sturgeon water so that 45% is released in May, 45% in June, and 10% in July. In both cases, the best available information at the time was used in the modeling. In reality, the shaping and timing of sturgeon flows varies from one year to the next and depends on several factors, including water temperature, which cannot be accounted for in a modeling template.

2.1.8 Regulation Rules

The hydro-regulation model runs were performed with consistent modeling rules. Although the actual hydrograph for each historic water year is known to modelers, the modeling was conducted with limited foresight, assuming regulators would make decisions based on a 10-day streamflow forecast, and no greater. These rules are related to:

- flood control draft;
- departure from URCs;
- avoiding outflows above an assumed powerhouse capacity or assumed powerhouse capacity plus 10 kcfs;
- regulation to Bonners Ferry flood control elevation; and
- following the Fish Flow Template.

2.1.8.1 General Assumptions

Generally, Libby was drafted to follow its URC during the evacuation phase, unless a flow reduction was required to prevent an IJC violation. If following this URC would have required flows above powerhouse capacity, discharge was limited to powerhouse capacity. In some years, this resulted in Libby being above its URC before refill. Although the URC required a draft to 2287 feet in some years, for the Period of Record modeling, Libby did not discharge above powerhouse to achieve this. However, for the 0.5%-and the 0.2%-chance-exceedance hypothetical floods, Libby Dam was drafted to the minimum flood control pool of 2287 feet. The volume of the 0.5%-chance exceedance hypothetical flood is about 82% of the estimated 1894 flood event and is higher than any other in operational history. It warranted special consideration in reservoir operations by drafting down to minimum flood control pool during the two-week period extending from April 15 to 29, which coincides with the typical time window for the IJC “spring rise” operations for Kootenay Lake. The potential for major flood damages is very high for not only the Kootenai Basin, but also for the Columbia River system as a whole, extending from the project to damage centers downstream of The Dalles, Oregon. It was agreed upon with USACE’s Northwestern Division Reservoir

Control Center that, for these types of forecasts, drafting Libby to 2287 feet was a valid assumption. Releases in excess of powerhouse capacity were required to accomplish this.

A potential conflict existed between filling Libby to 2459 feet and minimizing the double peak resulting from the salmon draft to 2439 feet by August 31. Releasing higher flows from mid to late July through August 31 for the salmon draft was highly unlikely to result in exceeding the flood stage at Bonners Ferry. A greater risk of exceeding the assumed flood control stage at Bonners Ferry existed when Libby‘s elevation approached 2459, and larger releases were required to control the rate at which the reservoir filled. Therefore, this study used a more conservative approach by following the ramp-down from the sturgeon flows to bull trout minimum flows (BT minimums) in the FFT, thereby increasing the probability of refilling and managing the diminishing amount of reservoir storage.

From October through December Libby Dam was modeled to achieve the variable end-of-December elevation. From January until the start of the refill phase, Libby was operated to follow the URC as previously described. When the refill phase began, ramping rates from the re-consultation between the Corps and USFWS were followed unless there was a flood control emergency. For regulating, changes to flow in increments of 5,000 cfs were used, which represents the hydraulic capacity of one generating unit at Libby Dam and corresponds to the maximum daily ramp-down rate for discharges from 16,000 cfs up to powerhouse (25,000 cfs)

2.1.8.2 Modeling Rules

Specific rules for modeling Libby Dam operations are as follows:

- Begin VARQ refill on refill start date. When VARQ refill flows were calculated, the elevation at that time, along with water supply forecast, determined the discharge. However, for this study, if there is a conflict between the higher FFT flows and the lower VARQ flows, the FFT is followed.

- One day before the assumed flood control stage (1764 or 1770) would be exceeded at Bonners Ferry (BF), reduce flow by 5,000 cfs and hold for 3 days. If on third day, BF stage was 1 foot below target elevation or lower (i.e., for the 1764 regulation, if BF was 1763 or less), resume VARQ or FFT. If less than one foot below target elevation (1763+ or 1769+), hold flow for 3 more days before resuming previous flows.

- If this reduction in flow for Bonners Ferry interferes in releasing powerhouse capacity for the required volume for the FFT, recalculate volume and extend fish flows at powerhouse to accomplish this release.

Libby outflows during refill were increased in increments of 5,000 cfs (one unit) to no higher than powerhouse capacity 10 days prior to when the reservoir would otherwise fill and spill.

In some years, it was necessary to spill water from Libby during the late stages of refill to avoid uncontrolled spill. In these instances, Libby outflows were increased by 1,200 cfs above powerhouse capacity 5 days prior to when the reservoir would otherwise fill and spill. A spill of 1,200 cfs was selected for modeling purposes because Montana State Water Quality Standards for Total Dissolved Gas (TDG) are exceeded for spill amounts greater than 1,200 cfs. When additional outflows were needed to prevent uncontrolled spill, Libby outflows were further increased in increments of 5,000 cfs above powerhouse 2 days prior to when the reservoir would otherwise reach full pool. Outflows were increased in increments of 5,000 cfs until inflows began receding. Inflows were passed until inflows decreased below 25,000 cfs (assumed powerhouse capacity at full pool of 2459 feet).

2.1.9 Initialization

The FFT requires an end of August elevation of 2439 feet for the salmon draft. It was assumed that project outflow would be about equal to inflow during the month of September. Therefore, Libby Dam was re-initialized at 2439 feet at the beginning of each water year (a water year begins on October 1 and ends on September 30.) Model simulations are re-initialized each year, rather than run in a continuous mode, so that one year's operation is independent of conditions in the previous year. The other reservoirs in the Kootenai basin were re-initialized at full pool.

SECTION 3 STUDY METHODOLOGY

3.1 LEVEE ANALYSIS

Most of the lower Kootenai River valley below Bonners Ferry is protected by levees. In most locations the levees are adjacent to the river but there are some sections where the levees are setback. For this study, only levees along the Kootenai River were inspected and are considered in the study. Also for this study, it is assumed that any flooding from levee failure/ overtopping would occur via levees located along the mainstem Kootenai, not along tributaries such as Deep Creek. In addition, structures such as elevated road or railway rights-of-way were not considered to provide any flood protection, although these types of structures could in fact have an impact on off-channel flooding. Notable areas are at downtown Bonners Ferry and the Crossport area upstream of Bonners Ferry.

The levees along the Kootenai River were inspected in September 2004 to estimate PNP/PFP (probable non-failure point/probable failure point) elevations. An elevation associated with the PNP represents a river stage where the levee in question would have a 0% chance of failure. An elevation associated with the PFP represents a river stage where the levee in question would have an 85% chance of failure. Table 1, below, is a tabulation of PNP/PFP elevations at locations along the river in the study reach.

3.2 STAGE-DAMAGE RELATIONSHIPS

Stage damage relationships are discussed in detail in the report, Economic Flood Depth-Damage Analysis for the Kootenai River, included here as **Appendix C**.

3.3 HYDROLOGY AND HYDRAULICS

For most risk and uncertainty-type analyses (R&U) involving levees, the typical approach would be to develop a stage-damage function with uncertainty, a discharge-frequency function with uncertainty, and a discharge-stage rating curve with uncertainty for an area of interest. Expected annual damages would be computed for both the without-project, or existing condition, and for the various alternatives involving levees and/or storage projects. The discharge-frequency and the discharge-stage rating curves are developed based on the application of hydrologic and hydraulic analyses.

Table 1. PNP/PFP Elevations (NGVD29) at Various Locations in the Study Reach

Off-Channel Area	Hydraulic Model Storage Area	PNP Elevation	PFP Elevation	Top of Levee
District 8	18	1759	1762	1769
Left Bank at Int. Boundary	21	1756	1758	1768
Left Bank RM 110.4	20	1760	1762	1769
District 10	13	1761	1763	1768
District 10	19	1760	1764	1770
District 6	14	1762	1763	1772
District 13	12	1763	1765	1772
District 9	8	1764	1767	1772
District 4	7	1768	1770	1773
Left Bank RM 126	10	1768	1770	1776
Left Bank RM 126	11	1766	1770	1776
District 16	9	1765	1767	1776
Right Bank RM 135	15	1768	1770	1773
District 14	16	1768	1771	1772
District 12	17	1765	1768	1776
Districts 5 & 11	4	1770	1775	1778
District 3	5	1768	1770	1777
District 7	3	1770	1772	1778
District 1 N of UPRR	1	1773.85	1773.85	1773.85
District 1 S of UPRR	2	1773.85	1773.85	1773.85
Downtown Bonners Ferry West of U.S. 2/95	2	1773.85	1773.85	1773.85
Bonners Ferry North of Kootenai River	22	1770	1775	1780
[1]Kootenai River Inn	2C	1769	1769	1769
Bonners Ferry East of U.S. 2/95 Except KRI	2B	1773.85	1773.85	1773.85
District 2	23	1773	1775	1776
[1]Below Grandview Cemetery	26	1776	1776	1776
[1]Crossport	24	1776	1776	1776

[1]No formal levee at this location; referenced elevation is top of bank.

To gain insight as to the risk associated with various alternatives, a probability would need to be assigned to various magnitudes of flow which might be expected in the area of study. For the typical without-project condition flow data would likely result from natural or unregulated flows. When following accepted guidance (USACE 1993, IACWD 1981) pertaining to assigning exceedance probabilities to natural river flows, typically it is assumed that these flows follow a known statistical distribution, such a log Pearson Type III, and, as such, consistent frequency curves can be constructed using analytical methods. Furthermore, since a known distribution is assumed, the estimation of extreme events, the magnitude of which might not be present in the flow data record, can be estimated. An uncertainty band can be applied to this curve based on the length of record and what is known about the skew of the flow data in the basin or from other similar basins in the region.

A discharge-stage rating curve can also be developed based on known hydraulic principles and local factors such as channel geometry, roughness, expansion and contraction coefficients, etc. Typically construction of such a rating curve would be accomplished using a hydraulic model such as HEC-RAS (HEC 2004). The discharge-frequency curve and the rating curve comprise the basic H&H elements of a typical risk and uncertainty analyses.

For this particular study there are two main obstacles with using the typical R&U study methodology described above to construct the required H&H products. One is the influence of Kootenay Lake elevations on Kootenai River stages in the lower Kootenai Valley. The presence of the lake causes water surfaces in portions of the lower valley to be higher than they would be if the lake were not present. When performing water surface elevation computations on a river such as the Kootenai with subcritical flow, the computations start at some downstream boundary (either a known, assumed, or computed water surface elevation) and proceed upstream. The water surface elevation at a downstream location has an impact on the water surface elevation at an upstream location. When the downstream boundary (in this case Kootenay Lake), has a much higher elevation for a given flow than would be present in the river had the river elevation been computed based strictly on hydraulic factors such as channel roughness, slope, geometry, etc. (i.e., without the lake or using a normal-depth calculation), then elevated water surfaces, or a backwater envelope curve (Chow 1959), will persist for some distance upstream. In hydraulic terms this backwater envelope curve is analogous to a M1 backwater curve (Bakhmeteff 1932) associated with steady flow in prismatic channels. This lake backwater influence makes it difficult to develop a discharge-stage rating curve at a location of interest because a given stage can be achieved with different combinations of lake elevation and flow. As such, stage-frequency curves (rather than discharge-frequency curves) were developed using output from unsteady flow hydraulic simulations.

The second main issue is the lack of homogeneous flow data on the mainstem Kootenai River. Prior to 1972 all flows in the Kootenai were natural, or unregulated in nature. Post 1972, flows have been regulated by Libby Dam. As can be seen from Figure 3, operation of the project has had a significant impact on flows below the dam when compared to pre-dam conditions. In addition, Libby Dam has been operated differently over the years, making even the post-1972 outflow data inconsistent. Barring climate change or significant basin changes such as urbanization, extensive logging, paving. etc., a record of natural flows can be considered homogeneous. For this study, in order to obtain homogeneous data and accurately quantify the probability of a given river stage occurring at a given location (i.e., develop consistent stage-frequency curves for a given Libby Dam operation), stages must be based on a consistent project operating methodology. This makes it impossible to use historic observed stages in the lower valley. Some type of methodology must be employed to take available flow data and/or stage data and adjust it to reflect the desired operation of Libby Dam. This study employed a reservoir model which computed Libby Dam outflows and Kootenay Lake elevations based on Libby Dam operations where the flood stage at Bonners Ferry is elevation 1764 feet and 1770 feet respectively and flows provided for sturgeon are powerhouse capacity and powerhouse capacity plus 10 kcfs..

3.3.1 Hydraulic Model Construction

The simulation of Kootenai River stages was executed using the HEC-RAS version 3.1.2 (HEC 2004) one-dimensional computer model in unsteady flow mode. The model extends from Kuskonook B.C. to the USGS gaging site below Libby Dam. The assumption here is that the hydraulics in the Kootenai River can be reasonably simulated using the one-dimensional flow assumption. It is felt this is a reasonable assumption for most locations. One area where this assumption is likely the weakest (at low flows) is in the area known as the braided reach, upstream of Bonners Ferry. Channel geometry for the model was obtained from USGS surveyed cross sections from 2002 (USGS 2004). At locations where overbank geometry was needed (for instance in the cases where levees are set back from the river bank), this information was obtained from available GIS data, USGS topographic mapping, and/or cross section and overbank geometry (available in graphical hard copy form) which was available in-house. For example, at Shorty's island, downstream of Bonners Ferry, only the main channel was surveyed by the USGS. In an effort to capture the channel on the opposite side of the island which is activated by high flows, the in-house cross section geometry was used.

While the hydraulic model extends from Libby Dam to Kooteany Lake, water surface elevations are only needed in the reach extending from just upstream of Bonners Ferry to the Canadian Border. The reach upstream of Bonners Ferry is used for the routing of Libby Dam outflows and various local flows. The goal of the hydraulic model is to represent the variability of stages in the lower Kootenai valley resulting from varying combinations of Kootenay Lake backwater influence and flow.

The off-channel areas protected by levees below Bonners Ferry, ID, were modeled using the storage area feature to represent the various drainage districts, urban areas, and other areas. Any exchange of flow between the river and these off-channel areas are modeled using lateral weirs to represent the existing levees. Figure 5 shows the location of storage areas along the Kootenai River.

Bridge data was obtained from a field survey and construction drawings obtained from local and state agencies.

The hydraulic model uses Libby Dam daily average outflows and Kootenay Lake elevations, simulated using the reservoir model, as upstream and downstream boundary conditions respectively. Intermediate inflows for the Fisher, Yaak, Moyie, and Goat Rivers are included as well as ungaged local inflow between Libby Dam and Leonia,

Leonia and Bonners Ferry, and Bonners Ferry and Kuskonook. The ungaged local flows between Libby Dam and Leonia, Leonia and Bonners Ferry, and Bonners Ferry and Kootenay Lake, are incorporated in the model using the feature that allows an intermediate hydrograph to be uniformly distributed over a given distance along a reach of river.

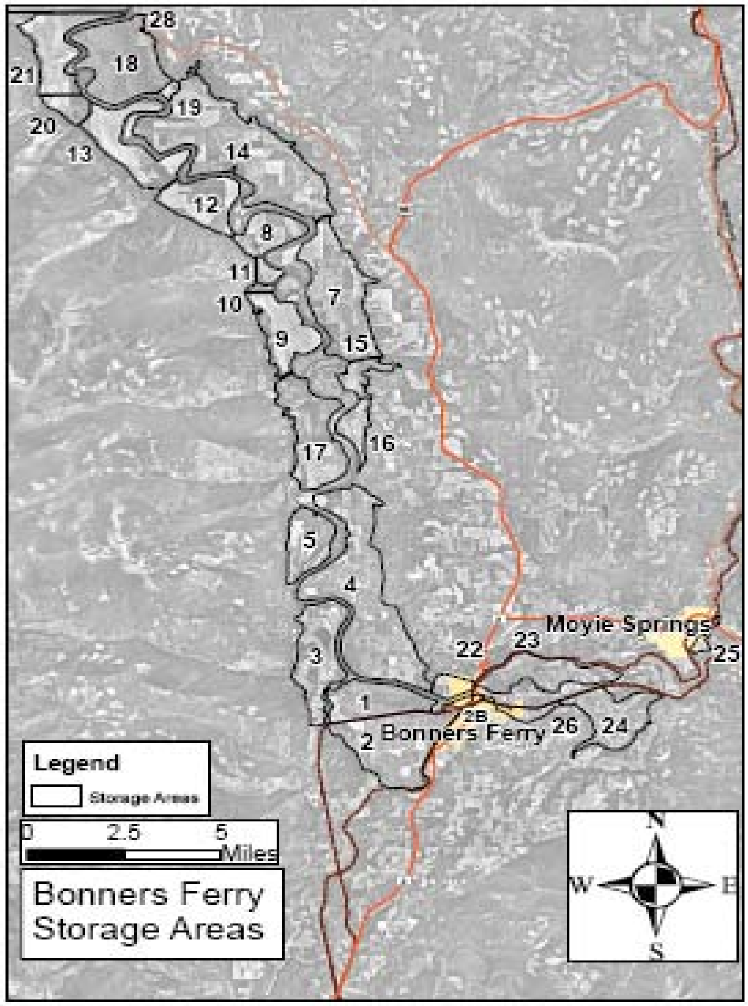

Figure 5. Schematic of Hydraulic Model Storage Areas

The model is run using a six minute time step. This time step seems to be the largest increment which allows for stable solutions for a wide range of flow conditions.

The hydraulic model allows period of record simulation of flow data for Libby Dam operations where elevations 1764 and 1770 are the regulating elevations at Bonners Ferry. This allows for a homogeneous set of stage data throughout the lower valley for use in constructing index location stage-frequency functions. Figure 6 shows the layout of the study area and the various hydrologic inputs to the hydraulic model.

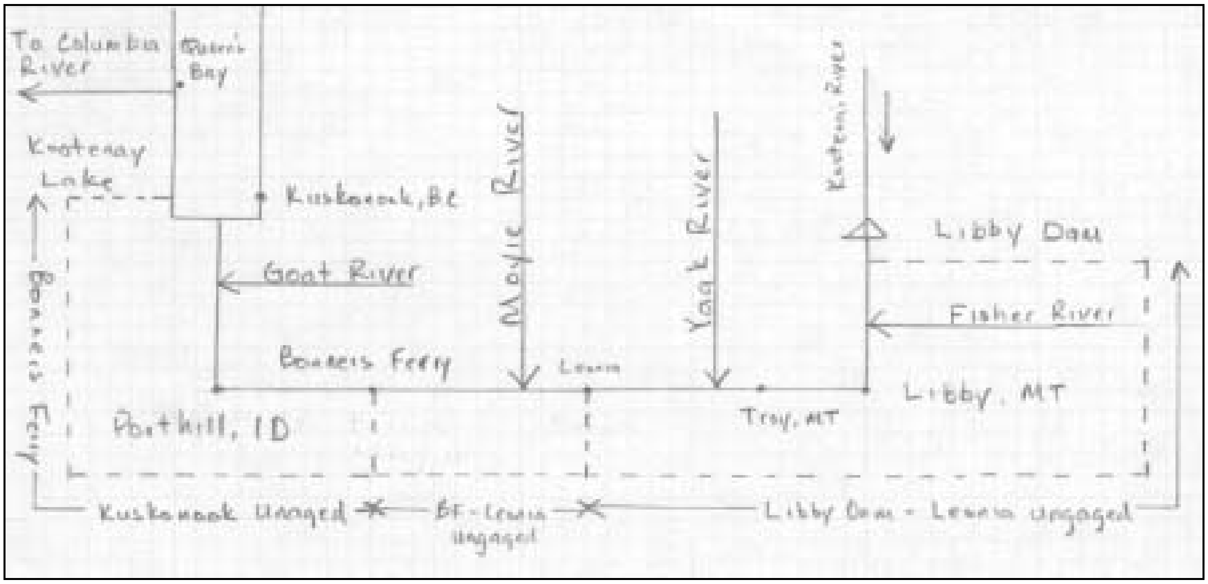

Figure 6. Basin Schematic

Most of the lower Kootenai valley is protected by levees. However, there are some locations where there is not a formal levee but an elevated railroad grade or road alignment which may offer some amount of flood protection. No known geotechnical analysis is available quantifying what type of flood protection, if any, these structures might provide. As such, these structures are not considered when modeling off-channel flooding in these areas. One location where there is not a levee but a railroad and/or road alignment is the area upstream of Bonners Ferry, on the left bank (looking downstream), near an area known as Crossport. This area appears to be a historic channel of the Kootenai River. Figure 7 identifies this location and the elevated road and railroad alignments which, for the purposes of this study are assumed to provide no flood protection to these areas. When the hydraulic model computes water surface elevations in the vicinity of these areas in excess of the left bank elevation, these areas are shown to be inundated.

Another area with a similar situation is the downtown Bonners Ferry area. Located on the west side of the downtown area is a system of elevated railroad alignments which form a barrier between the downtown area and District 1 to the southwest (see Figure 8).

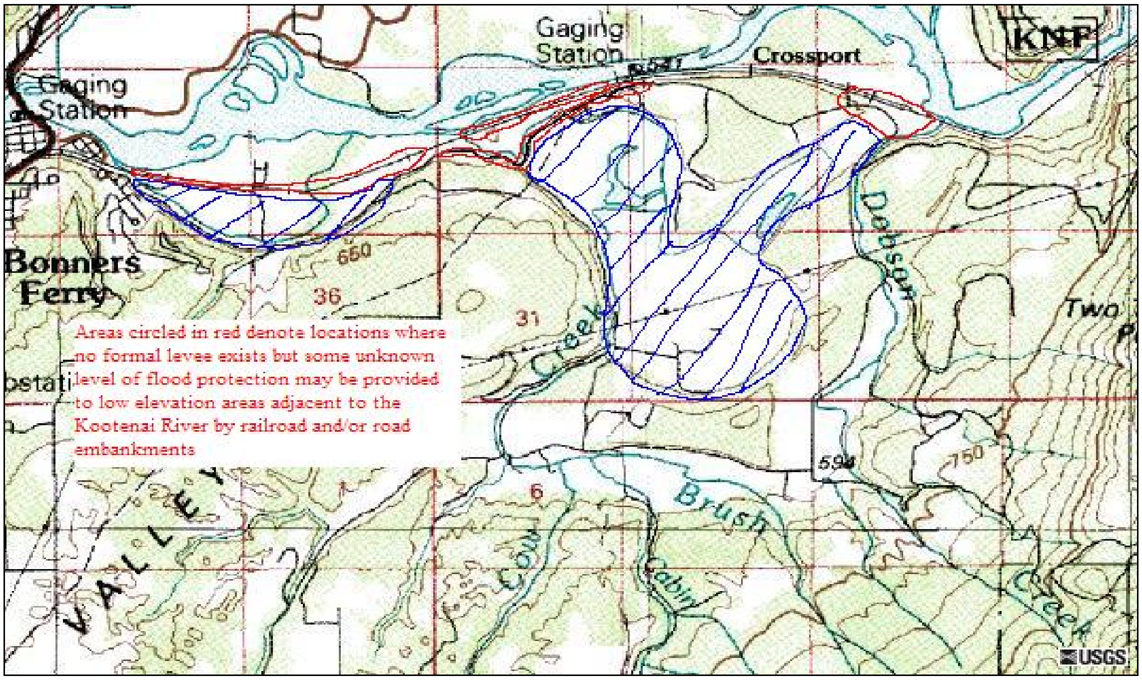

Figure 7. Locations between Bonners Ferry and Crossport without a Formal Levee

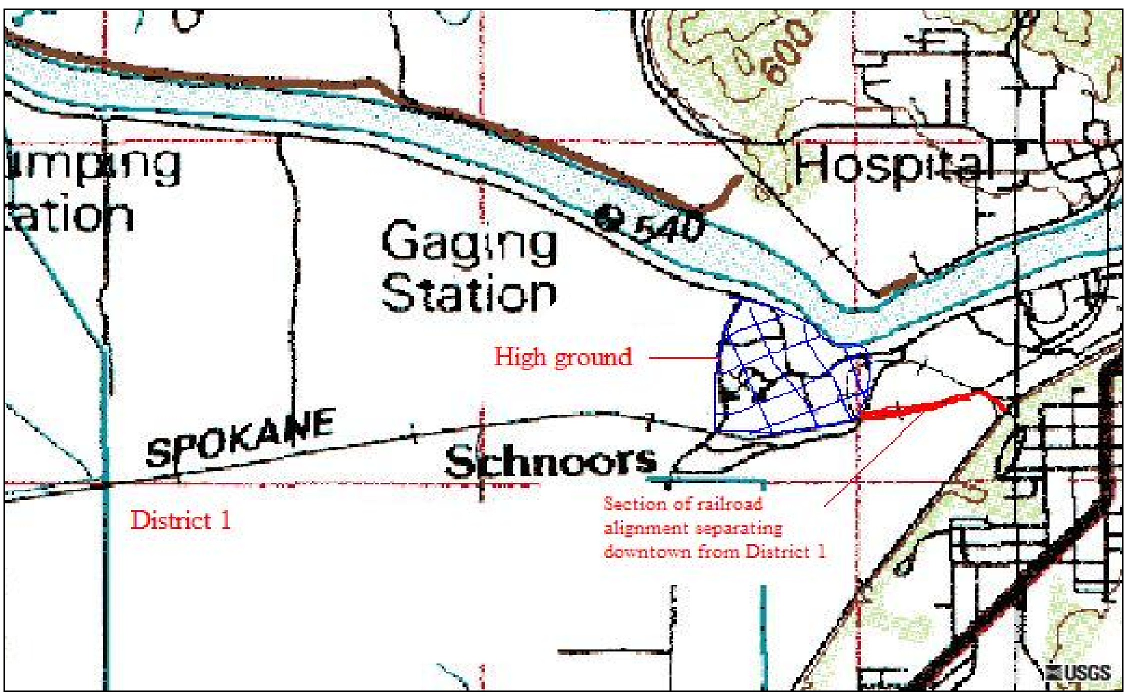

Figure 8. Railroad Right of Way Barrier between District 1 and Downtown

As with the area near Crossport, for the purposes of this study, the railroad alignment was not considered to provide any flood protection to downtown Bonners Ferry in the event District 1 flooded from a levee failure or levee overtopping.

3.3.2 Hydraulic Model Flow Data

There are a number of gaging sites within the Kootenai River basin. In addition, some computed flow data is also available. Some data of interest to this study includes:

- Observed Libby Dam outflow (1972-present)
- Estimated Libby Dam Inflow (1929-present)
- Kootenay Lake at Queens Bay elevation (1931-present)
- Kootenay Lake at Kuskonook, B.C. elevation
- USGS flow and stage data at Porthill, ID (1929-present)
- USGS observed stage data at Bonners Ferry, ID (1929-present, gage number 12309500, DA 12,690 mi^2)
- Kootenai River at Klockmann Ranch stage data (USGS gage 12314000, DA 13,300 mi^2 continuous for water years 1931 to present)
- USGS flow and stage data at Leonia, ID (1929-present, USGS 12305000)
- USGS flow data for the Fisher River. Both nr Libby, MT (12302055 DA 838 mi2) and near Jennings, MT (12302000 DA 780 mi2).
- USGS flow data for the Yaak River (USGS 12304500 DA 766 mi2)
- Moyie River flow at Eastport, ID (1929-present, USGS 12306500 DA 570 mi2)
- 2000 Level Modified streamflow (natural) at Bonners Ferry, ID generated by BPA (1929-1999) (BPA 2004)
- 2000 Level Modified Libby-Bonners Ferry local inflow generated by BPA (1929-1999) (BPA 2004)
- 2000 Level Modified local inflow above Corra Linn Dam generated by BPA (1929-1999) (BPA 2004)

The hydrologic data developed by BPA is used by both BPA and the Corps of Engineers for system-wide Columbia River studies. This data has been adjusted to represent a level of irrigation depletions consistent with those from 2000.

For this study, average daily flows are used. While the instantaneous peak flows tend to be higher than the average daily flow on the day the instantaneous annual peak occurs, the difference is minimal. Figure 9 shows a plot of Leonia instantaneous annual peak flow versus the annual peak one-day flow to illustrate this.

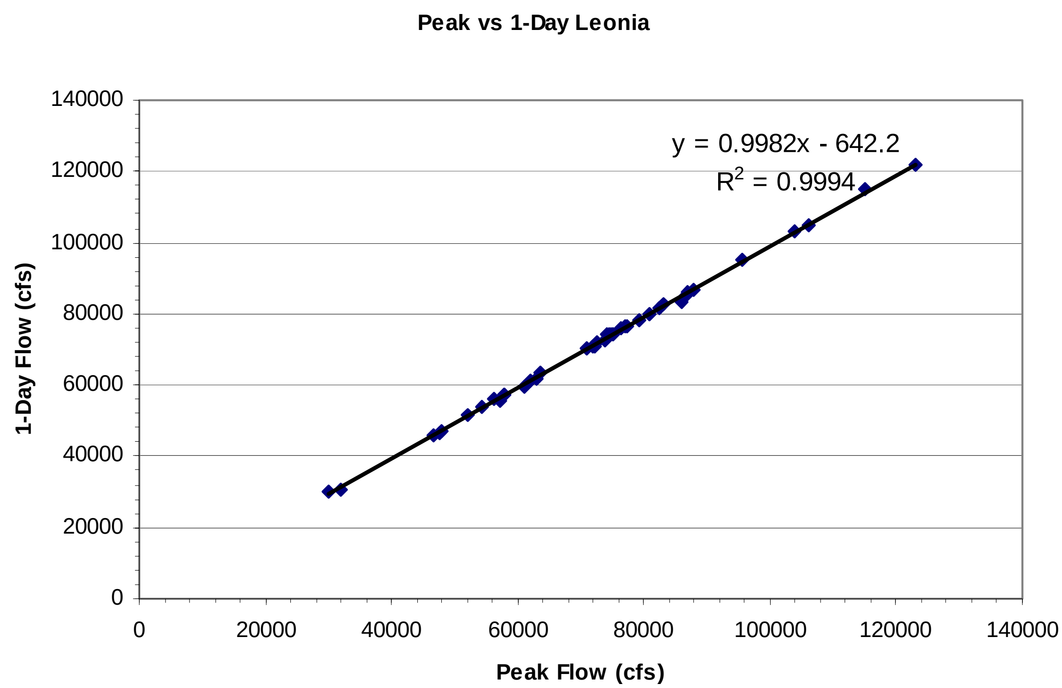

Figure 9. Peak vs. 1-Day Leonia

The various hydrologic inputs to the hydraulic model, and the derivation of these inputs, as well as other required hydrologic data, are as follows:

1. **Libby Dam Outflow –** Determined using the reservoir model. Flow values from observed stages at USGS gage number 12301933 (below Libby Dam gage) used for hydraulic model calibration purposes.
2. **Libby Dam Inflow** – Calculated for water years 1929 through present. Estimated for water years 1911 through 1928 using Kootenai River at Libby MT. Gage (USGS 12303000) and applying monthly adjustment coefficients derived by BPA (BPA 2004). Also available is an estimate of Libby Dam inflow generated by the 1894 flood.
3. **Kootenai River at Libby, MT. –** Flow data from USGS gage number 12303000. Flow data is available for water years 1911 through 1991. This data was used to estimate Libby Dam inflows prior to 1929. Used for model calibration and to estimate Libby Dam inflow data prior to 1929.
4. **Leonia to Bonners Ferry Local Inflow** – Computed using BPA equation (BPA 2004) for this parameter. The equation is Leo-BF Local=Moyie River @ Eastport, ID*a+b. Coefficients a and b are values based on month. This value is the entire Leonia-Bonners Ferry local flow, including the Moyie River, which is gaged.

5. **Leonia to Bonners Ferry Ungaged Local Inflow** – The corresponding Moyie River gage data is subtracted from the value representing the entire Leonia-Bonners Ferry local to arrive at the ungaged value.
6. **Libby Dam to Leonia Local Inflow –** Using the BPA Libby Dam to Bonners Ferry local flow value (adjusted for any irrigation depletion), the Leonia to Bonners Ferry local flow (step 2 above) is subtracted out.
7. **Libby Dam to Leonia Ungaged Local Inflow –** The corresponding values of the Fisher and Yaak River flow data are subtracted out of the Libby Dam to Leonia Local from above.
8. F**isher River Flow –** For the water years 1968 through 1999, the data from USGS gage 1230255 is used directly. For the period 01 Jan 1951 through water year 1967, data from the Fisher River near Jennings, MT is scaled to drainage area of USGS gage 1230255. For water year 1948 through 31 December 1950, Leonia to Libby Dam local inflow data is scaled based on the drainage area ratio.
9. **Yaak River Flow –** For the period 01 March 1956 through water year 1999, flow data based on average daily flows recorded at USGS gage number 12304500. For water year 1948 through February 1956, flows are based on Libby Dam to Leonia local flow (see number 2 above) scaled based on the drainage area ratio.
10. **Moyie River Flow –** Based on daily data from USGS gage 12306500.
11. **Bonners Ferry to Kuskonook Local Inflow –** Computed using the BPA local flow above Corra Linn Dam and below Bonners Ferry and Duncan Dam. This data was scaled using the drainage area ratio. This value excludes the Goat River.
12. **Goat River Flow –** Based on the Corra Linn local flow scaled using the drainage area ratio.
13. **Kootenay Lake Elevation –** Computed using the reservoir model. The reservoir model computed Kootenay Lake elevation at Queen's Bay B.C. Observed elevations are available as well for hydraulic model calibration purposes.
14. **Simulated Kootenay Lake Elevation at Kuskonook, B.C. –** Based on regression analysis of observed Kootenay Lake at Queen's Bay vs. observed Kootenay Lake at Kuskonook elevations. 0.5 feet is added to the Queen's Bay elevations to account for a difference in elevation between the two locations.

3.3.3 Extreme Event Hydrology

Due to limitations with the period of record simulations to estimate stages beyond about the 2% exceedance probability flood event, hypothetical hydrographs for the 1%, 0.5%, and 0.2% exceedance probability floods were developed. These hydrographs were simulated using the reservoir and hydraulic models to compute corresponding stages at various locations along the lower river.

Much thought and discussion was put into estimating what these events might look like and what parameters would drive peak stages in the Bonners Ferry to Canadian border reach of the Kootenai. Due to differences in climate, topography, and other factors, there can be a substantial amount of variability between both timing and magnitude of inflow to the reservoir and that of the local runoff below the dam. In certain years peak stages can be driven more by what is happening in terms of reservoir inflow, while other years, peak stages can be more of a function of the local unregulated runoff below the dam. For instance, there can be conditions where both the watershed upstream of Libby Dam and the region below the dam experience significant snow accumulation during the winter. For these conditions, spring weather patterns can greatly influence the timing of the runoff from these two sub-basins. If this local snowmelt was significant, peak stages in the lower valley could be more influenced by the local contribution than the releases from Libby Dam. Conversely, conditions can be such that lower elevation precipitation during the winter is made up of more rain than snow, resulting in lower spring runoff volumes resulting from the portion of the basin below the dam.

For conditions where the project has the most impact on stages, other factors, such as forecast accuracy and fish flows, can impact the annual peak stage at a given location. An advantage of simulating the period of record hydrology is that, to some degree, all of the variation in coincidence, timing, and other factors regarding reservoir inflow and local inflows is inherently captured in the resulting period of record stage data. However, as already discussed, due to the fact that only 52 years of data is available, it is very difficult to extrapolate beyond about the 2% exceedance event, especially when dealing with stages, and regulated flows, which typically do not follow any known distribution. For this reason, it was necessary to develop hypothetical events to better estimate what the stage-frequency curve looks like beyond the 2% exceedance flood event.

In reality, the probability of exceeding a given stage each year at a given location is a conditional probability problem with a large number of parameters, such as magnitude and timing of the Libby inflow and local inflow hydrographs, forecast accuracy, real time operational obstacles, etc. In addition, the various parameters have varying degrees of dependence or independence which would be impossible to quantify. After much discussion it was decided that the main parameters which would be focused on were 1) the appropriate recurrence interval local flow hydrograph and reservoir inflow hydrograph to combine and 2) the appropriate timing of these hydrographs.

Based on examination of existing data, there is not a clear pattern as to the combinations of reservoir inflow and local inflow recurrence intervals which occur each year. In addition, there is no clear pattern as to the timing of these events. The degree of dependence between the coincidence of local and reservoir flows is difficult to quantify from the period of record data. These events do not appear to be entirely independent nor entirely dependant with respect to each other. It appears quite likely that in a given year a rather extreme local runoff event occurs while a relatively frequent reservoir inflow event occurs. The opposite appears to be true as well. Assuming these hydrologic parameters are perfectly correlated with respect to timing and recurrence interval would likely result in an overestimation of stages in the study reach. Linsley et al (Linsley 1986) discusses the issue of hydrologic events and conditional probabilities of occurrence and offers a caution regarding the assumption of complete correlation of hydrological events.

To develop the extreme event hydrographs, two cases were considered for both the 1764 and 1770 operations. One case has peak river stages driven primarily by reservoir inflow and the other has unregulated local inflow below the dam as the driver of peak stages. For each recurrence interval, the case producing the highest stage at a given location was used as the stage corresponding to that recurrence interval on the stage-frequency curve. Furthermore, it is assumed that for the 0.5% and greater flood events, the 1770 operation will be used for both the 1764 and 1770 operation stage-frequency curves. Due to the magnitude of these events, it is really impractical to attempt to regulate to 1764 at Bonners Ferry. Given available forecasts, and the fact that the project is technically authorized to operate to 1770 at Bonners Ferry, it is assumed that project operators would have knowledge that an extreme event spring runoff scenario is developing and would adjust operations accordingly, even under a 1764 operation scheme. Both operations were simulated for the 0.01 flood event and the output for each operation was used on the respective stage-frequency curves. Below are the steps for the development of these extreme events.

Coincident Event Determination Frequency curves were developed (using annual peaks) for the following hydrologic categories: Annual peak Libby Dam inflow, Libby Dam inflow April to August volume, annual peak aggregated local inflow between the dam and Bonners Ferry, the annual peak aggregated local inflow below Bonners Ferry, and the sum of the latter three categories. Since all of these categories are made up of natural flows, they are assumed to follow a log-Pearson III distribution and as such, procedures outlined in Bulletin 17b (IACWD 1981) are followed. For the local inflow and the summed categories, the water years 1948 through 1999 were used as the data set as that constitutes the available data. For the reservoir inflow, data was available for the years 1911 through 2002. In addition, an accepted estimate of what peak reservoir inflow would have been for the 1894 flood is available. This value was included as a historic event. The five frequency curves described above are shown in **Appendix B**.

In addition to the frequency curves, regressions were performed of daily peak flow vs. 3-, 7-, 15- and 30-day average flows for each parameter. These regressions were used as an aid in shaping the hypothetical hydrographs. Flow hydrographs from 1974 were used as an initial starting point for constructing the reservoir and local flow hypothetical hydrographs. These hydrographs were scaled upward to the peak flow corresponding to the 1%, 0.5%, and 0.2% events. Using an iterative process, the hydrographs were adjusted to reflect the peak flow versus 3-, 7-, 15- and 30-day averages.

Once the peak to 3-, 7-, 15- and 30-day averages had been prepared, the following procedure was used (the 0.5%, or 200-year event is used as an example):

Case 1 Reservoir Inflow Based

- From the combined and the Libby inflow frequency curves, pick off the 0.5% flow values. The value of the Libby inflow will then be the peak daily flow around which the 0.5% hypothetical Libby inflow hydrograph will be shaped.

- Subtract the Libby inflow out of the combined value. The remainder will be the aggregate Libby Dam to Bonners Ferry and below Bonners Ferry locals.

- Of the aggregate local value, using the respective frequency curves, determine a value for both the Libby Dam-Bonners Ferry and below Bonners Ferry locals that have approximately the same recurrence interval and add up to the starting aggregate local value.

- Shape the respective frequency curves based on the daily peak regressions. For Libby inflow, the 0.5% April-August volume is assumed.

Case 2 Local Inflow Based

- From the combined, below Bonners Ferry, and below Libby Dam frequency curves, pick off the 0.5% values. The values of these locals will be the peak daily flow around which the 0.5% hypothetical below Bonners Ferry and below Libby Dam hydrographs will be shaped.

- Subtract out the below Bonners Ferry and the below Libby Dam values from the 0.5% combined. The remainder is the peak daily inflow around which the reservoir inflow hydrograph will be shaped.

- Shape the respective frequency curves based on the daily peak regressions.

Coincident Event Timing The other main parameter considered was the timing of the three hydrographs. This was mainly based on judgment. As discussed elsewhere, stages in the lower Kootenai valley are a function of Kootenay Lake elevation as well as flow. In addition, lake elevations typically would be expected to rise as inflow volume to the lake rises. Given this, the 0.5% stage is actually some type of very complicated conditional probability problem with many parameters of varying degrees of independence or dependence, of which one would be timing. For the hydrology, the timing of actual large flood events was investigated. **Table 2** shows the ten largest inflow events in terms of peak daily inflow and the date the peak inflow occurred. While based on these numbers the “average” date would be 06 June, it is clear that the system does have the capability to generate large inflow events later in June. Typically when inflows peak later into June, the elevation of Kootenay Lake would be higher than it would be in late May or early June, producing a greater backwater influence on stages in the lower valley.

Table 2. Ten Largest Historical Flow Events (Annual Peak Daily Flow, cfs)

	Libby Inflow (1911-1999)		Below Libby Dam Local (1948-1999)		Below Bonners Ferry Local (1948-1999)		Duncan Inflow (1929-1999)	
Rank	cfs	date	cfs	date	cfs	date	cfs	date
1	115200	6/21/16	47571	5/21/56	86478	6/17/74	33281	7/12/83
2	110529	6/18/74	45898	5/19/54	80723	6/01/72	30936	6/24/55
3	103456	5/28/48	43824	5/17/97	79141	6/01/97	27041	6/29/84
4	92388	5/23/56	38521	5/23/48	78538	6/07/61	25547	6/18/61
5	92386	6/03/72	36069	5/11/76	77545	6/03/68	24756	6/11/61
6	90277	5/29/61	34266	5/23/67	75458	6/22/67	24605	6/04/60
7	85294	5/31/86	34200	5/14/50	73564	6/24/55	24529	7/10/64
8	84944	6/07/95	34077	4/28/52	72511	6/01/56	24422	5/31/36
9	82242	6/18/33	33090	6/17/74	72157	6/17/99	24214	7/15/53
10	82149	6/09/96	32671	5/13/71	70301	6/02/86	24140	6/09/48

Keeping in mind the goal is to estimate what a 0.5% stage might be at locations along the river, and since lake elevation is a factor in ultimate peak stage, the goal was to select a date for the inflow hydrograph to peak that would be representative of a date that could be reasonably expected, but not so late (i.e. extreme), that when combined with an inflow magnitude that has been estimated to have a 0.5% probability of occurring each year, would result in a stage somewhere along the lower river with a recurrence interval even more extreme than the 0.5% stage. With this in mind, and noting that the two largest inflow events occurred on 18 June and 21 June, June 20th was selected for the Libby inflow to peak. The dates for the below Libby Dam and below Bonners Ferry locals to peak were selected using this type of reasoning as well. 13 June was used for the below Libby Dam local flow and 17 June was used for the below Bonners Ferry local flow.

The hydrographs for the Libby Dam inflow, below Libby Dam local, and below Bonners Ferry local were then assembled using the methods described above. In addition, the reservoir model required a Duncan Dam inflow hydrograph. For simplicity, the Duncan inflow was not considered in the above methodology. The contributing drainage area to Duncan is much smaller than Libby Dam's contributing area-833 mi^2 versus 8,985 mi^2, and as such, so is its contribution to total Kootenay Lake inflow volume. Based on data from 1948 through 1999, Duncan inflow also typically peaks later than Libby inflow. The 1961 Duncan inflow hydrograph was used as the basis for the reservoir model. Peak Duncan inflow for this year is 25,500 cfs on 18 June. This is the second largest Duncan inflow in the 1948 to 1999 period. In addition, the timing of the 1961 Duncan hydrograph with respect to the other hydrographs was deemed to be reasonable. Figure 10 shows the 0.5% hypothetical hydrographs for Libby Dam inflow, below Libby

Dam local flow (also called the ‘Bonners Ferry local’), and below Bonners Ferry local flow (also called the ‘Corra Linn local’).

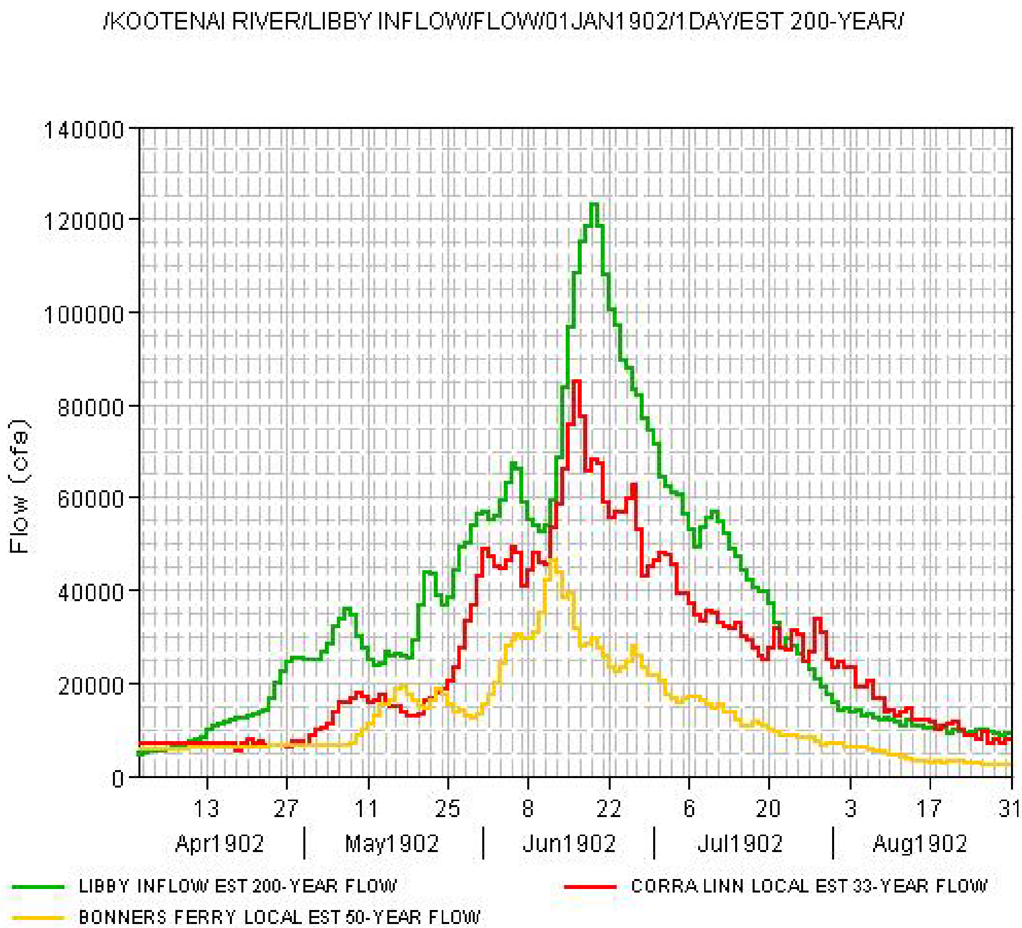

Figure 10. 0.5% Kootenai River Hypothetical Hydrographs

Hydrographs of various exceedance probabilities were scaled up or down from the 0.5% hydrographs using a factor based on peak inflow. Table 3 contains selected data regarding the extreme events.

Table 3. Extreme Event Data

Event	0.01A	0.01B	0.5%A	0.5%B	0.2%
Hydrograph					
Libby Peak Inflow/E.P.	115000 cfs/0.01	96000 cfs/~0.05	123000 cfs/0.5%	100400 cfs/~0.033	132000 cfs/0.2%%
Peak Date	20-Jun	20-Jun	20-Jun	20-Jun	20-Jun
Libby Apr-Aug Vol.	10.3 MAF	9 MAF	10.8 MAF	7 MAF	11.2 MAF
Below Libby Local Peak/E.P.	42700 cfs/~0.033	50000 cfs/0.01	46200cfs/~0.02	53600 cfs/0.5%	50000 cfs/~0.01
Peak Date	13-Jun	13-Jun	13-Jun	13-Jun	13-Jun
Below Bonners Ferry Peak/E.P.	82300 cfs/~0.033	94000 cfs/0.01	84800cfs/~0.033	100000 cfs/0.5%	90000 cfs/~0.013
Peak Date	17-Jun	17-Jun	17-Jun	17-Jun	17-Jun
Duncan Peak Inflow	25500 cfs	25500 cfs	25500 cfs	25500cfs	28000 cfs
Peak Date	18-Jun	18-Jun	18-Jun	18-Jun	18-Jun

'A' denotes case where Libby inflow is emphasized; 'B' denotes case where local flow is emphasized; E.P.-Exceedance Probability

More data used in the development of these extreme events can be found in **Appendix A**, Extreme Events.

3.3.4 Hydraulic Model Calibration

The hydraulic model is intended to serve two main purposes. For the reach from approximately Bonners Ferry, Idaho to the Canadian border the model was used in unsteady flow mode for simulating period of record stages for different Libby Dam operations. For the reach between Libby Dam to just above Bonners Ferry, the model was used essentially as a flow routing tool. The basic parameter used to calibrate the model is Manning's roughness coefficient (n-value). Figure 11 contains representative photographs of the river at various locations.

Kootenai River below Bonners Ferry, ID

Kootenai River at Bonners Ferry, ID

Kootenai River below Bonners Ferry, ID

Kootenai River below Troy, MT

Kootenai River near Troy, MT

Kootenai River below Libby, MT

Kootenai River Above Libby, MT

Kootenai River @ Libby, MT

Figure 11. Representative photos of the Kootenai River at various locations.

The unsteady flow model will be used to compute period of record stages along this reach. The goal for this reach is to configure the model with appropriate roughness values that can be justified in literature and that also reasonably reproduce observed stages at calibration points for a wide range of flow and backwater conditions. For this portion of the study, the section of the model above the braided reach is essentially used for the routing of Libby Dam outflows and associated local flows through the system. Given the confined nature of the river and lack of appreciable flood plain upstream of the braided reach, routed flows to Bonners Ferry were not very sensitive to roughness coefficient variation. As such, not a lot of effort was put into calibrating the unsteady flow model upstream of the braided reach. Flow data from 1997 was used for calibration. 1997 was a recent large runoff year throughout the Kootenai Basin.

Observed stages at Bonners Ferry slightly exceeded elevation 1764 feet in 1997. The model was calibrated to observed 1997 stages at Porthill, Klockmann Ranch, Bonners Ferry, and Leonia (see Figure 6) by adjusting Manning's roughness coefficients. It is assumed that these locations are representative of the reach between Bonners Ferry and Kootenay Lake, and as such, these locations are used to identify appropriate roughness coefficients for the entire reach. Care was taken to ensure that realistic n-values were used and that adjustments to this parameter were applied as uniformly as possible between calibration locations to improve stage computation accuracy at intermediate points (Cunge et al 1980-a source of general calibration considerations). The model was then run using flow data from other years to verify that the model was able to reproduce observed stages resulting from hydrographs of various magnitudes and durations. After the first verification runs some minor n-value adjustments were made to configure the model such that it provided, based on judgment, the best overall results. Unfortunately, there is not any recent observed stage data available upstream of Bonners Ferry, in the braided reach. The most recent stage data dates back to the late 1950s. This necessitated the use of published data and judgment to arrive at roughness coefficients. As discussed earlier, this portion of the river is different in terms of slope, bed material, and cross section uniformity, compared to the river below Bonners Ferry.

To begin, roughness Manning's coefficients found in literature (Chow 1959, Acrement 1989) were used as a starting point. Initial values were chosen based on field observations and comparison with photographs and tables found in Chow. For the reach below Bonners Ferry, initial starting values were 0.075 for areas of the channel cross section near the banks or levees, to represent the brush, occasional trees, grass, and riprap, and 0.035 for the main channel. This horizontal variation in roughness attempts to account for not only the increased roughness at the river bank due to the aforementioned factors but the reduced velocities that are typically found at the channel boundaries as well (Chow 1959, Figure 2-4). Ultimately, varying the roughness coefficients with respect to river stage provided the best reproduction of observed stages. Final main-channel n-values used ranged from 0.036 for lower stages to 0.022 at some locations for higher stages. For the braided reach, main-channel n-values were varied with stages as well. In most locations, low-stage n-values were selected in the 0.045 range and decreased to approximately 0.034-0.036 for higher stages.

During the calibration process for the lower valley portion of the model, n-values which were lower than those which could be justified in literature had to be used to reproduce some of the higher observed stages. The concern was that the roughness value was being used to adjust some other parameter. Given that there was a high degree of confidence in the channel cross section geometry, the hydrology was examined-particularly the local below Bonners Ferry. The aggregate local flow (Corra Linn local) used to estimate the Bonners Ferry to Kuskonook B.C. component encompasses a wide area around Kootenay Lake. Daley (1981) indicated that the flow volume yield on a drainage area basis was much larger for the Duncan and Lardeau River basins than for the middle Kootenai Basin. The Lardeau and the portion of the Kootenai below Bonners Ferry both are part of the Corra Linn local. Given this information, and the difficulties in matching observed stages with the hydraulic model, it appears that basing the Bonners Ferry to Kuskonook local inflow on the drainage area ratio over estimates this hydrological component. The local flow hydrographs for the Bonners Ferry to Kuskonook local flow were reduced by 40 percent across the board to reflect the apparent difference in yield between different geographic areas which contribute to the Corra Linn local inflow. With this adjustment, roughness coefficients for the lower valley reach were within the range justified by available literature.

Figure 12 is a plot of simulated vs. observed stages at Bonners Ferry for the years 1997 through 1999.

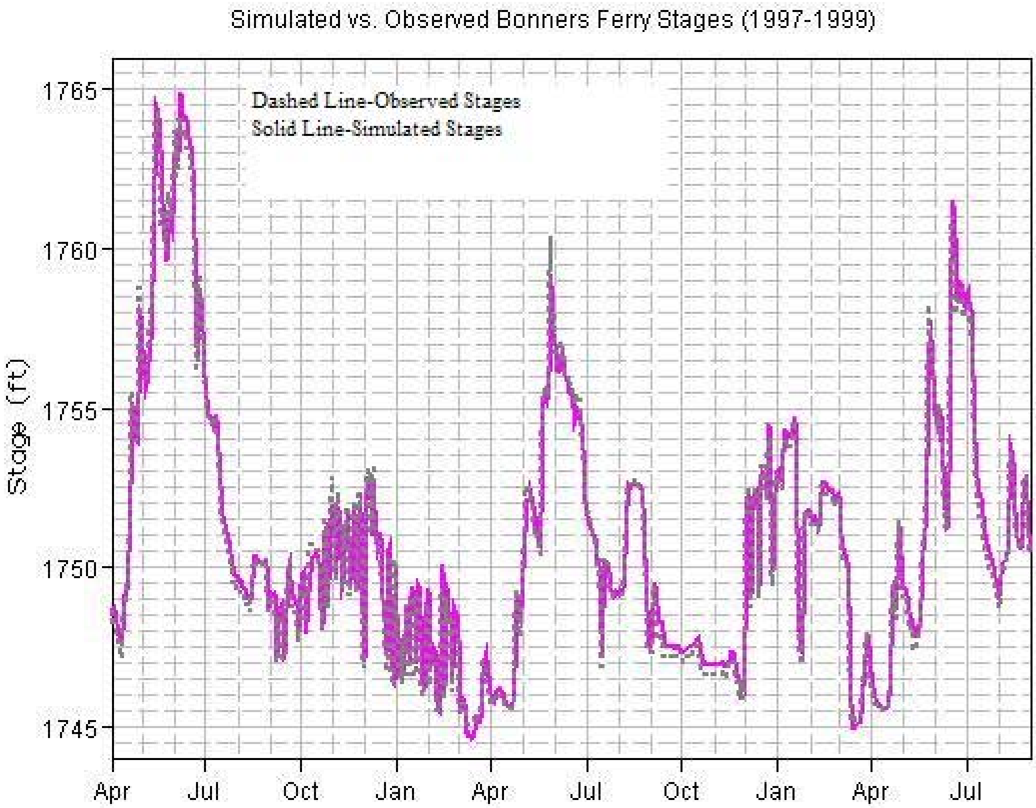

Figure 12. Simulated vs. Observed Bonners Ferry Stages (1997-1999)

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3.3.5 Hydrologic/Hydraulic Simulations

3.3.5.1 Lower Kootenai Valley

The period of record hydraulic model simulations were performed using the unsteady flow version of the HEC-RAS model. The purpose of these simulations is to create a period of record data set of Kootenai River stages corresponding to the two Libby Dam operating schemes. This data set will then be used to construct graphical stage-frequency curves at locations of interest along the river. Levee failures have the potential to influence peak stages at locations along the river channel by opening up large areas of off-channel storage. Due to large uncertainties as to how levees might fail in a large flood in terms of combinations and locations, any off-channel flooding simulated by the hydraulic model is the result of levee overtopping only, or for the locations where no levee is present, overbank flooding only.

These simulations were performed for Libby Dam operations where 1764 is the flood stage at Bonners Ferry and where 1770 is the flood stage at Bonners Ferry. In addition, separate simulations were performed for the two cases for each of the extreme events discussed previously in this document. Upstream and downstream boundary conditions were based on output from the reservoir model. The downstream boundary is the Kootenay Lake elevation at Queen's Bay as computed by the reservoir model and adjusted to Kootenay Lake at Kuskonook. The upstream boundary condition in the hydraulic model is the daily outflow from Libby Dam as computed by the reservoir model.

Simulated Libby Dam releases using the reservoir model are based in part on stages the reservoir model computes at Bonners Ferry. The reservoir model utilizes a rating table which contains Bonners Ferry stages based on Kootenay Lake elevation and flow at Bonners Ferry. The flow at Bonners Ferry is a combination of the Libby Dam outflow routed to Bonners Ferry combined with the local inflow between the dam and Bonners Ferry. By contrast, the hydraulic model computes stages at each cross section based on the approximation of one-dimensional unsteady-flow equations (HEC 2002). In addition, the hydraulic model incorporates local flows below Libby Dam both as discrete sources (such as the Moyie River) and distributed sources (such as the Leonia to Bonners Ferry ungaged local), whereas the reservoir model treats local flows in a more "bulked" fashion. Given the different methods between the two models for computing stages, there is bound to be discrepancies in computed Bonners Ferry stages. For this study, greater confidence was placed on stages computed by the hydraulic model. Any discrepancy was further investigated when the reservoir model produced a data set where Bonners Ferry stages were regulated to flood stage (for example 1764 or 1770, depending on the operation) and the hydraulic model computed a higher stage at Bonners Ferry using the same data set. For years where this occurred, the regulation used in the reservoir model was re-examined and a determination made as to whether or not Libby outflow could have been further reduced to control Bonners Ferry river elevations to flood stage, based on how the hydraulic model was computing stages. If this was determined to be the case, then the peak stage for this particular year was considered to be flood stage, either 1764 or 1770. For years where this was determined not to be the case, the stage computed by the hydraulic model was used. To then adjust stages computed by the hydraulic model at other locations, the year in question was re-run with adjusted Libby outflows such that Bonners Ferry elevation was forced to the target elevation. Table 4 and Table 5 show the hydraulic model output, the reservoir model output and any adjustment made to the data set for the 1764PH and 1770PH Libby Dam operations. The same procedure was followed for 1764PH10 and 1770PH10. Elevations in bold type indicate years that were re-evaluated. The next column over shows whether or not the peak stage for this year was adjusted down to the operation target stage. The far right column is the final data set used to construct the stage-frequency curves for this operation. As shown in Table 4, eight years were adjusted for the 1764 operation while no years were adjusted for the 1770 operation.

Table 4. Simulated Peak Annual Bonners Ferry Stages-1764PH Operation

Year	Bonners Ferry Peak Annual Simulated Stage (Hyd. Model)	Bonners Ferry Peak Annual Simulated Stage (Res. Model)	Re-Regulate to 1764?	Adjusted 1764 Data Set
1948	**1764.94**	**1763.97**	Yes	1764.00
1949	1763.76	1762.99		1763.76
1950	1763.42	1763.43		1763.42
1951	1762.88	1762.08		1762.88
1952	1760.94	1760.29		1760.94
1953	1761.55	1760.96		1761.55
1954	1764.61	1763.59		1764.61
1955	1762.99	1762.64		1762.99
1956	**1765.02**	**1763.95**	No	1765.02
1957	1762.81	1761.82		1762.81
1958	1763.24	1762.43		1763.24
1959	1763.63	1762.99		1763.63
1960	1762.19	1761.88		1762.19
1961	**1764.97**	**1764.11**	No	1764.97
1962	1760.80	1760.29		1760.80
1963	1760.06	1759.26		1760.06
1964	1763.40	1763.30		1763.40
1965	1760.50	1760.04		1760.50
1966	1762.10	1761.25		1762.10
1967	**1764.16**	**1763.86**	Yes	1764.00
1968	1762.10	1761.08		1762.10
1969	1763.92	1763.43		1763.92
1970	1761.48	1760.86		1761.48
1971	**1764.06**	**1763.12**	Yes	1764.00
1972	**1764.95**	**1763.92**	Yes	1764.00
1973	1758.55	1758.06		1758.55
1974	**1764.45**	**1763.62**	Yes	1764.00
1975	**1764.26**	**1763.66**	Yes	1764.00
1976	1760.87	1760.33		1760.87
1977	1751.17	1750.79		1751.17
1978	1761.03	1760.52		1761.03
1979	1759.91	1759.31		1759.91
1980	1760.36	1759.47		1760.36
1981	1762.24	1761.27		1762.24
1982	1763.79	1763.39		1763.79
1983	**1764.28**	**1763.26**		1764.00
1984	1759.90	1759.63		1759.90
1985	1761.41	1760.43		1761.41
1986	1761.92	1760.67		1761.92
1987	1759.02	1758.53		1759.02

Year	Bonners Ferry Peak Annual Simulated Stage (Hyd. Model)	Bonners Ferry Peak Annual Simulated Stage (Res. Model)	Re-Regulate to 1764?	Adjusted 1764 Data Set
1988	1755.39	1754.55		1755.39
1989	1760.32	1759.63		1760.32
1990	1761.16	1760.51		1761.16
1991	1763.30	1762.21		1763.30
1992	1756.78	1756.39		1756.78
1993	1755.92	1754.93		1755.92
1994	1757.61	1756.89		1757.61
1995	1761.13	1761.04		1761.13
1996	**1764.25**	**1763.58**	Yes	1764.00
1997	**1764.75**	**1763.95**	Yes	1764.00
1998	1763.43	1763.35		1763.43
1999	1763.94	1763.01		1763.94

Table 5. Simulated Peak Annual Bonners Ferry Stages-1770PH Operation

Year	Bonners Ferry Peak Annual Simulated Stage (Hyd. Model)	Bonners Ferry Peak Annual Simulated Stage (Res. Model)	Re-Regulate to 1770?	Adjusted 1770 Data Set
1948	1766.34	1766.78		1766.34
1949	1763.73	1762.97		1763.73
1950	1764.09	1764.27		1764.09
1951	1762.84	1762.04		1762.84
1952	1760.91	1760.26		1760.91
1953	1761.53	1760.94		1761.53
1954	1768.27	1769.05		1768.27
1955	1762.95	1762.61		1762.95
1956	1769.36	1769.54		1769.36
1957	1762.76	1761.76		1762.76
1958	1763.18	1762.37		1763.18
1959	1763.61	1762.97		1763.61
1960	1762.18	1761.87		1762.18
1961	1766.95	1766.05		1766.95
1962	1760.73	1760.21		1760.73
1963	1760.02	1759.22		1760.02
1964	1763.37	1763.26		1763.37
1965	1760.44	1760.15		1760.44
1966	1762.24	1761.38		1762.24
1967	1764.20	1763.89		1764.2
1968	1762.10	1761.10		1762.1
1969	1763.81	1763.32		1763.81
1970	1761.43	1760.83		1761.43
1971	1764.16	1763.22		1764.16
1972	1766.49	1765.64		1766.49
1973	1758.48	1757.99		1758.48
1974	1765.82	1766.18		1765.82
1975	1764.18	1763.58		1764.18
1976	1760.82	1760.44		1760.82
1977	1751.17	1750.96		1751.17

Year	Bonners Ferry Peak Annual Simulated Stage (Hyd. Model)	Bonners Ferry Peak Annual Simulated Stage (Res. Model)	Re-Regulate to 1770?	Adjusted 1770 Data Set
1978	1760.98	1760.46		1760.98
1979	1759.93	1759.32		1759.93
1980	1760.32	1759.44		1760.32
1981	1762.18	1761.21		1762.18
1982	1764.97	1764.84		1764.97
1983	1764.22	1763.20		1764.22
1984	1759.89	1759.61		1759.89
1985	1761.38	1760.40		1761.38
1986	1761.99	1760.74		1761.99
1987	1759.05	1758.56		1759.05
1988	1755.44	1754.60		1755.44
1989	1760.27	1759.57		1760.27
1990	1761.10	1760.45		1761.1
1991	1763.36	1762.30		1763.36
1992	1756.82	1756.43		1756.82
1993	1755.90	1754.91		1755.9
1994	1757.58	1756.85		1757.58
1995	1761.25	1760.51		1761.25
1996	1764.35	1763.75		1764.35
1997	1768.54	1768.72		1768.54
1998	1763.42	1763.34		1763.42
1999	1765.16	1764.69		1765.16

Once the period of record simulations were complete, the hydrology developed for the extreme events was simulated. The hypothetical hydrographs were first simulated using the reservoir model. The resulting Libby Dam outflows and Kootenay Lake elevations were then used as upstream and downstream boundary conditions in the hydraulic model. Table 6 and Table 7 show the Bonners Ferry peak stages computed by both models for the extreme events for 1764PH and 1770PH operations.

Table 6. Simulation Results for Extreme Flood Events-1764PH Operation

1764 Operation	1% Flood Event Peak Stage		0.5% Flood Event Peak Stage		0.2% Flood Event Peak Stage
Model	**Extreme Libby Inflow**	**Extreme Local Inflow**	**Extreme Libby Inflow**	**Extreme Local Inflow**	**Extreme Libby Inflow**
Reservoir (1764)	1763.86	1766.32	1769.57	1768.23	1769.65
Hydraulic (1764)	1765.26	**1766.67**	**1770.00**	1767.96	**1770.80**

Table 7. Simulation Results for Extreme Flood Events-1770PH Operation

1770 Operation	1% Flood Event Peak Stage		0.5% Flood Event Peak Stage		0.2% Flood Event Peak Stage
Model	**Extreme Libby Inflow**	**Extreme Local Inflow**	**Extreme Libby Inflow**	**Extreme Local Inflow**	**Extreme Libby Inflow**
Reservoir	1768.85	1769.36	1769.57	1768.23	1769.65

(1770)					
Hydraulic (1770)	1768.66	**1769.24**[1]	**1770.00**	1767.96	**1770.80**

[1] This value was revised upward to **1769.36** to match the peak stage computed for 1956

The 0.01 (100-year) peak stages for the 1764PH10 and 1770PH10 operations were slightly higher (see Figures 15-18). The values in bold shown in Table 6 and Table 7 were the values used on the frequency curves for 1764PH and 1770PH Libby Dam operations for the 1%, 0.5%, and 0.2% annual probability of exceedance flood events. All operating scenarios have the same 0.5% (200-year) and 0.2% (500-year) flood events.

Figure 13 shows the differences between Libby Dam outflow for both the 0.5% (red line) and 0.2% (green line) hypothetical flood events and the resulting stages (0.5%-gray line, 0.2%-purple line) at Bonners Ferry. While the 0.2% peak stage is only 0.8-foot greater in terms of stage at Bonners Ferry than the 0.5% event, notice how the 0.2% flood has a second peak which is nearly as high as the first one. Likely both of these peaks are large enough to cause damages to be incurred. This second peak is not present on the 0.5% flood event. Attempting to regulate Bonners Ferry to 1770 feet necessitated reducing Libby Dam outflows more than was required for the 0.5% flood. This in turn used up reservoir space, resulting in the need to spill when the reservoir was full.

An assumption for the R&U portion of the study is that leveed off-channel areas (drainage districts or storage areas) fill up to the same elevation as the river stage. Figure 14 is a plot of an HEC-RAS modeled levee failure at Drainage District 11 (storage area 4). Drainage District 11 is one of the larger off-channel areas in the valley. For this simulation, the levee failure was simulated as occurring close to the top of the hydrograph. The breach geometry assumed a trapezoidal breach with a 300-foot bottom elevation, 4:1 side slopes and a failure time of five hours. In addition the geometry of the failure breach was simulated such that the minimum breach elevation was only several feet below the failure trigger elevation. The off-channel area was increased by approximately double to produce a large storage volume relative to other areas in the valley.

The purpose of this simulation was to verify the assumption that the off-channel areas fill to the same elevation as the river when a levee fails. This was initially done by using failure parameters and an off-channel area volume which would represent the case where the storage area would be the least likely (or at least no the best case) to attain the same elevation as the river; kind of a worst-case scenario. As can be seen from Figure 14, this failure has an impact on both the river stage (they are reduced) at the failure location as well as the stage at Bonners Ferry. In this case it is clear that the Drainage District quickly fills to the same elevation as the river. The dark blue line represents the water surface elevation of the drainage district, the light blue line represents the river stage at the failure location before during and after the failure, and yellow line represents the stage at Bonners Ferry before, during and after the failure. For comparison purposes plots of river stage at Bonners Ferry (gray line) and at the failure location (purple line) without levee failure are shown.

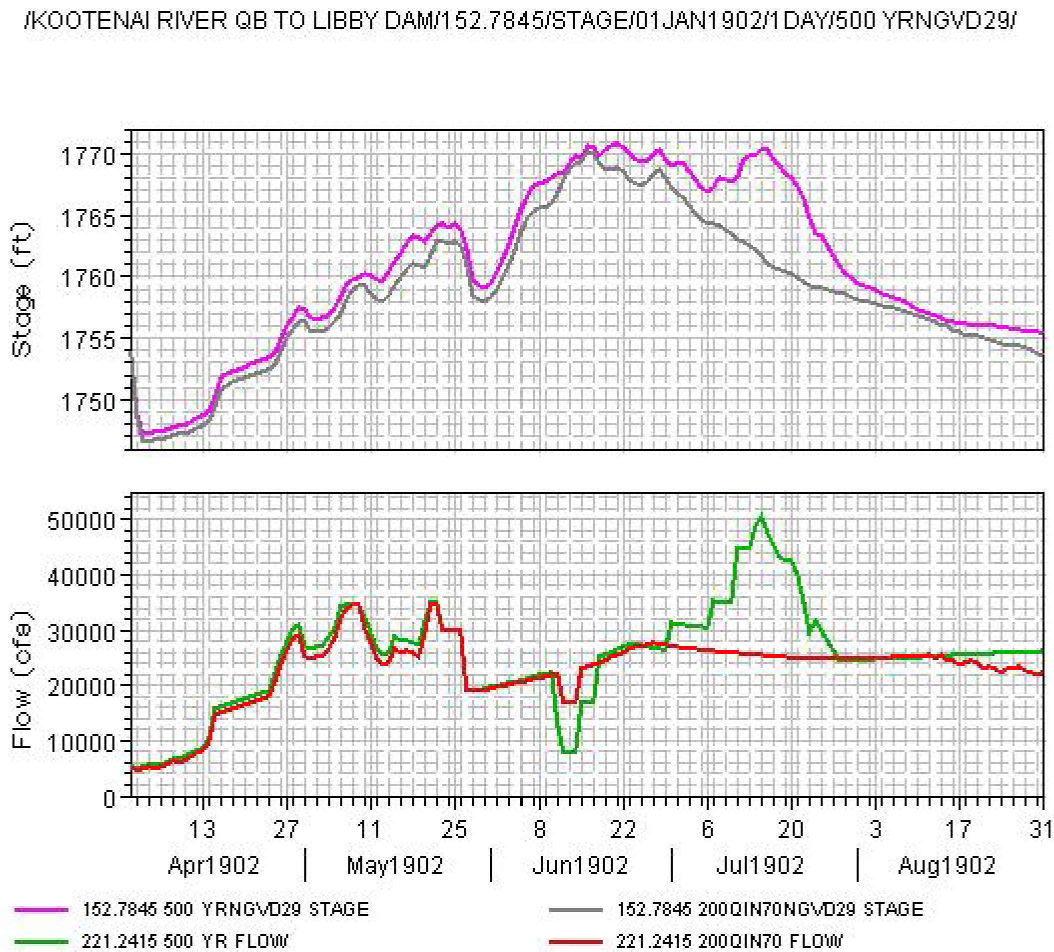

Figure 13. 0.5% and 0.2% Hypothetical Flood Event Hydrographs

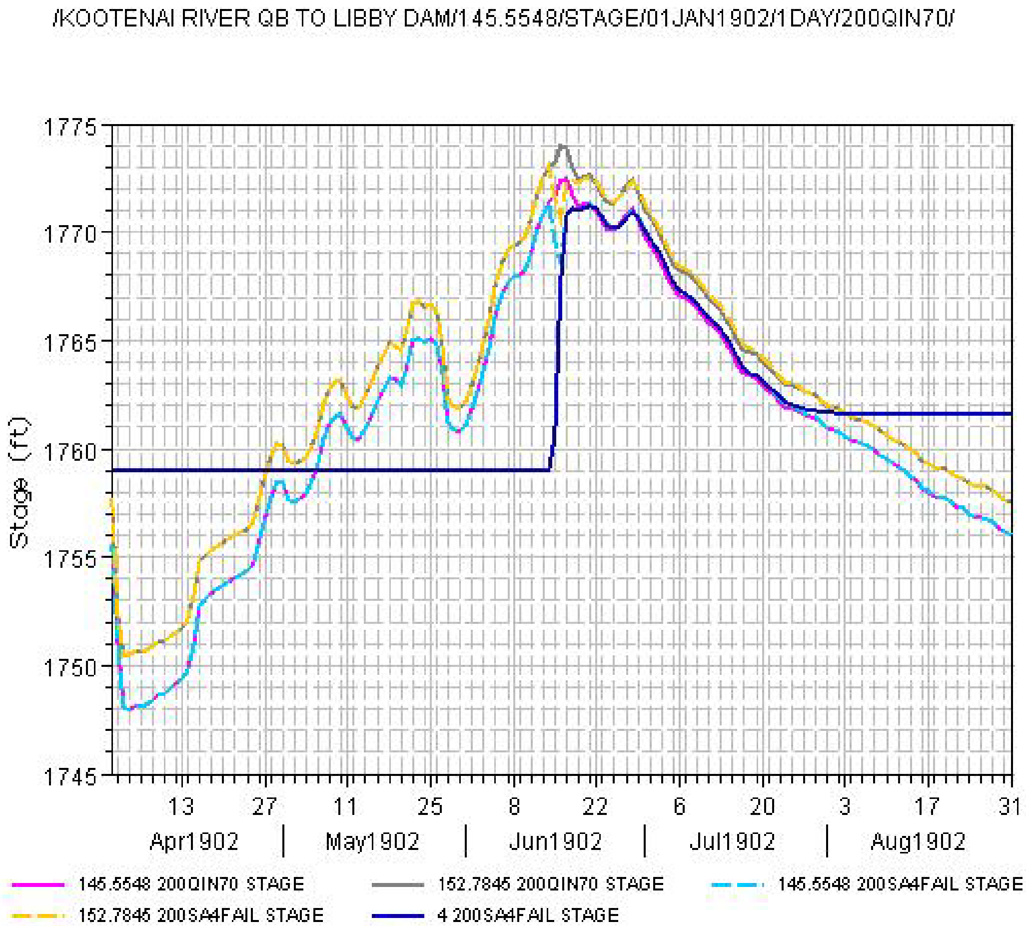

Figure 14. Plot of an HEC-RAS modeled levee failure at Drainage District 11 (storage area 4).

The assumption that drainage districts or storage areas attain the same water surface elevation as the river in the event of a levee failure makes the execution of this study much easier, since hydraulic modeling of levee failures for every off-channel area under a wide range of flow conditions is not required. This assumption is justified for several reasons. One is that sensitivity hydraulic model runs indicate that, for most of the off-channel areas, except for floods which just briefly bump a failure elevation, resulting water surface elevations in these areas are very close to the river stage. Another reason for this assumption is that for situations where there might be an exterior/interior difference, the flow has to flow overland to fill the low area. In doing this, it is entirely possible that damages will be incurred from this overland flow. However, this study does not look at this type of flow and the resulting damages. It is felt that for these instances, this type of damage is accounted for by possibly over estimating the stillwater elevation in these areas.

3.3.6 Stage-Frequency Curves

Stage-frequency curve sets intended to be representative of the four scenarios were constructed form the period of record hydraulic simulations of the period of record hydrologic and the extreme event hydraulic data. Curves were constructed for twelve locations from river mile 105.6 (Canadian border) to river mile 156.6 (between Bonners Ferry and the Moyie River confluence). As discussed elsewhere, levee failures, and the impact they might have on stages throughout the reach were not modeled and, as such, are not reflected in the stage-frequency curves. Given that it is difficult to determine a levee failure scenario, off-channel flooding through a breached levee(s) was not incorporated into the underlying data making up the frequency curves. For these simulations any off-channel flooding only occurs through levee or river bank overtopping.

Peak annual stages from the period of record simulations were used to define the curve for recurrence intervals more frequent than about the 2% annual exceedance probability flood event. Each of the 52 annual peak stage values (Table 4 and Table 5) were assigned plotting positions based on Weibull's plotting position formula (Linsley et al. 1986, USACE 1997), and plotted on a probability-normal scale plot. In addition, the extreme event stage values from Table 6 and Table 7 were plotted at their respective recurrence intervals on the same graph. The curves were graphically fitted to the period of record data and transitioned to extend directly through the extreme event stages entered on the plot. Figure 15, Figure 16, Figure 17 and Figure 18 show the curves at Bonners Ferry for the 1764PH, 1764PH10, 1770PH, and 1770PH10 operations respectively. The plots constructed for the remaining locations are located in **Appendix B**.

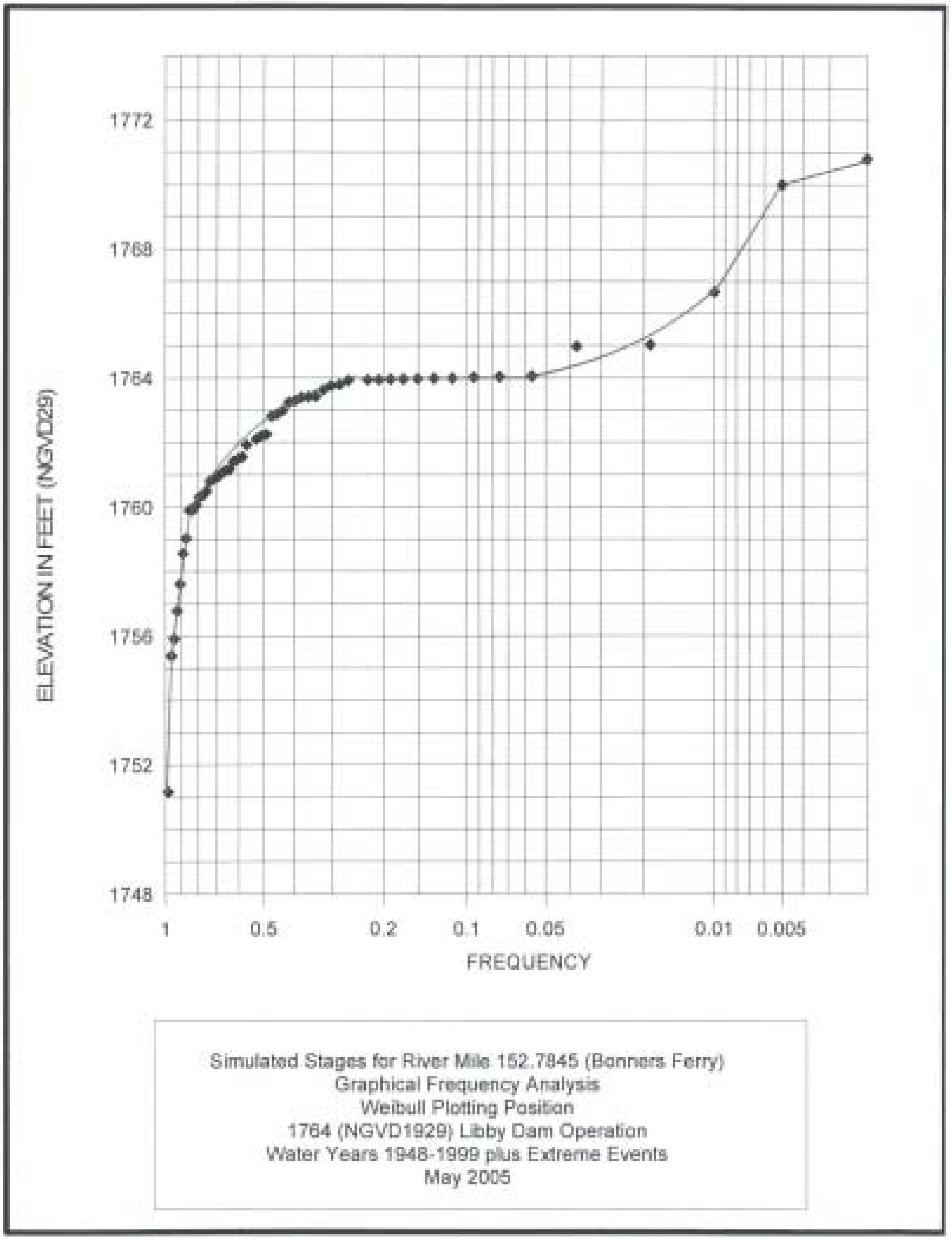

Figure 15. Stage-Frequency Curve Kootenai River at Bonners Ferry, 1764PH

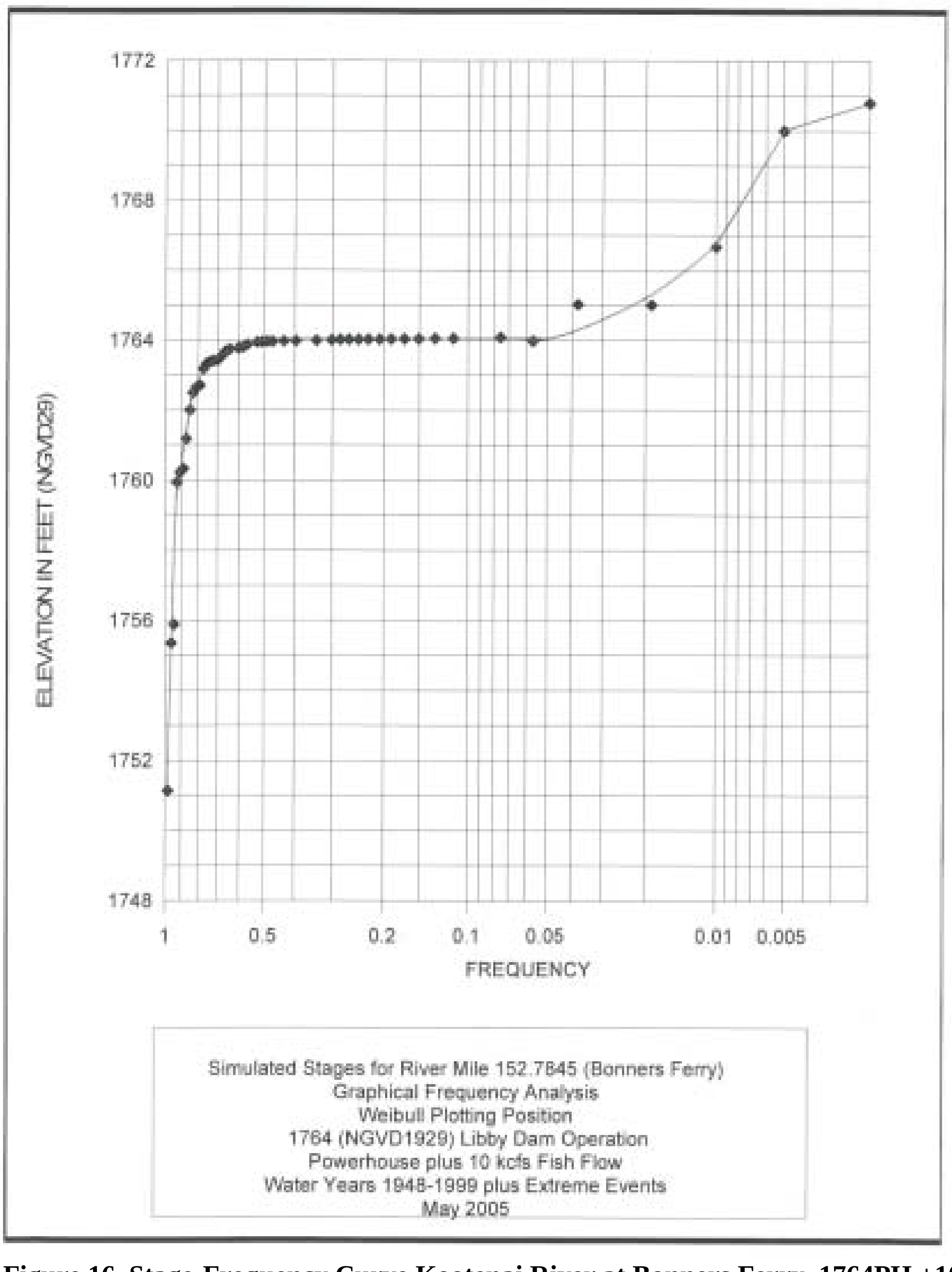

Figure 16. Stage-Frequency Curve Kootenai River at Bonners Ferry, 1764PH +10k

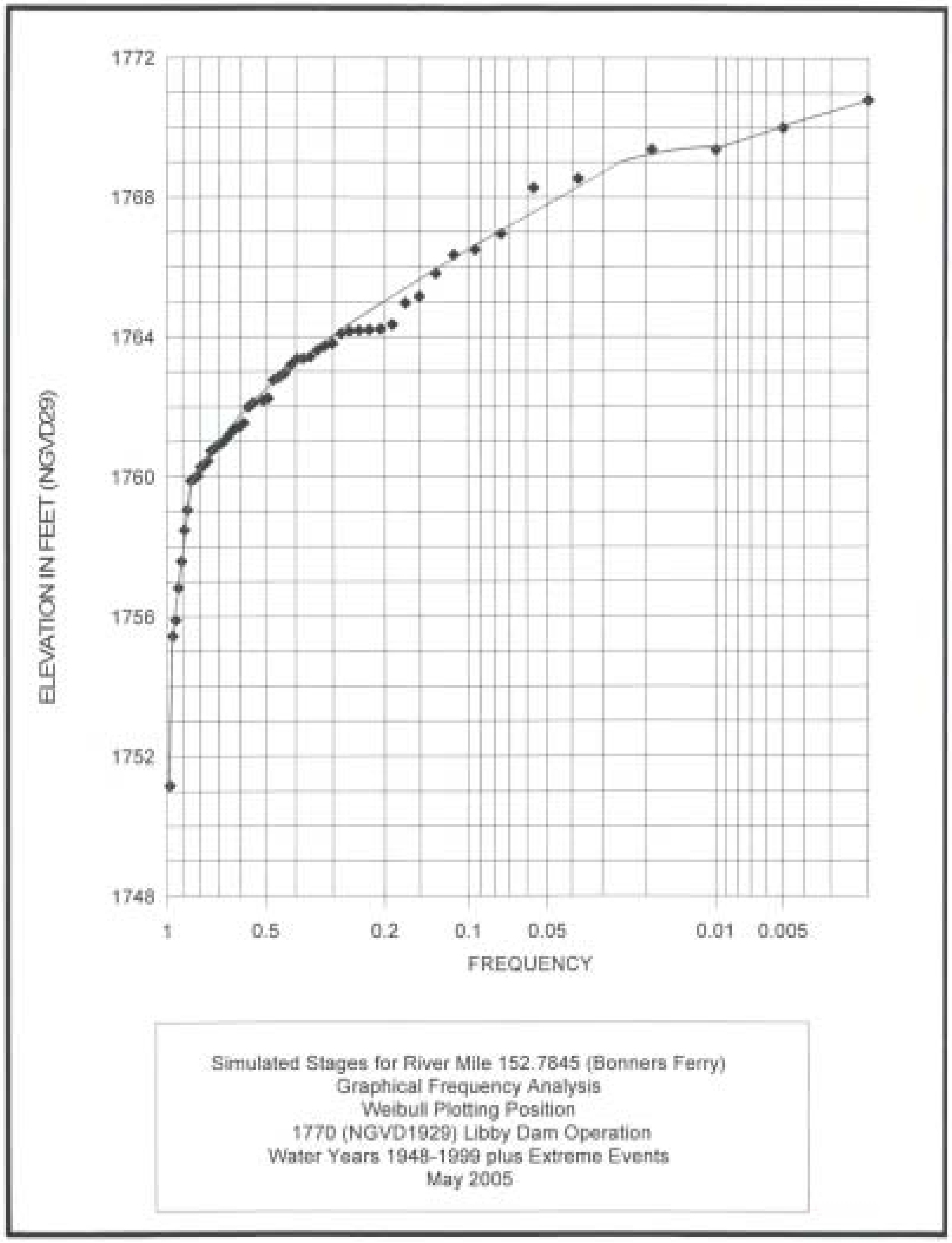

Figure 17. Stage-Frequency Curve Kootenai River at Bonners Ferry, 1770PH

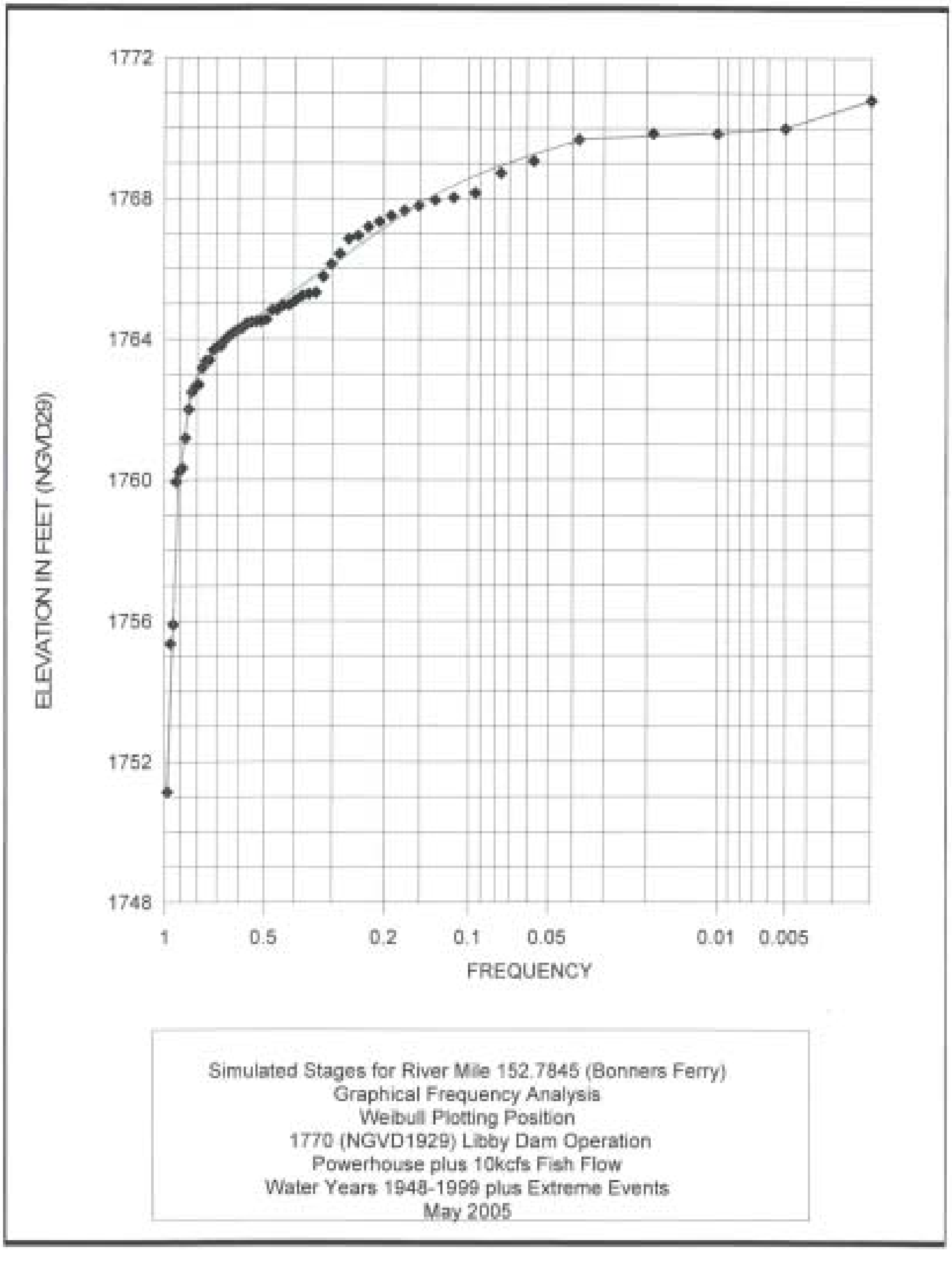

Figure 18. Stage-Frequency Curve Kootenai River at Bonners Ferry, 1770PH +10k

From these hand drawn plots, stages at each location corresponding to the recurrence intervals which adequately define the shape of each frequency curve were pulled off and tabulated for use in the computer program HEC-FDA. The uncertainty associated with each curve is not shown on the curve. The uncertainty is computed within HEC-FDA.

For potential levee failure locations between frequency curve locations, the tabular frequency data was interpolated to the potential failure location of interest based on the slope of the water surface between two frequency curve locations and the distance of the failure point from one of the bounding frequency curves. This data was entered into the HEC-FDA program at each potential levee failure or bank overtopping location.

The final parameter regarding the frequency curves is the assignment of the uncertainty for the stage-frequency curve. This is done within the HEC-FDA program based on selecting an appropriate Equivalent Record Length (ERL). The program uses the ERL number and order statistic methods outlined in publication ETL 1110-2-537 (USACE 1997). Corps publication ETL 1110-2-537 (USACE 1997) provides guidance for selecting an appropriate ERL, depending on how stage data is derived. If the frequency curves used in this study were constructed from a long record of observed stages representing a consistent Libby Dam operation, then the ERL would be the actual record length in years. Since this study manipulated the hydrological data via modeling to obtain a consistent set of stage data relative to a particular Libby Dam operation, it would be expected that the ERL would be something less than the number of years simulated with the reservoir and hydraulic models.

The portion of the Kootenai River in this study is comprised of two different geomorphic reaches. Below Bonners Ferry the river has a very low gradient, and meanders, with bed material comprised of finer type materials. Upstream of Bonners Ferry, the braided reach has a steeper channel with coarser bed material. Downstream of Bonners Ferry observed stages at three locations were available for model calibration. This allowed roughness values to be adjusted to get the best match to the observed stages at these locations. In the braided reach there was no observed stage data available for calibration. Manning’s roughness coefficient selection for this reach was limited to values suggested by literature (Chow 1959) and judgment. As such, the ERL used for frequency curves at locations below Bonners Ferry should be greater than the ERL used at locations in the braided reach.

In addition, the frequency curves were constructed by piecing together the period of record simulations with the hypothetical extreme events. Most likely a different ERL would apply to the two portions of the curve. The mechanism within HEC-FDA for quantifying the uncertainty about a graphical frequency curve does not allow for using two different ERLs on the same curve. This needs to taken into account when selecting an appropriate ERL value.

Based on Table 1 of ETL 1110-2-537, for a model calibrated to a long-period gage within a watershed, the suggested adjustment to the period of record would be to use something between 50% to 90% of the number of years in the period of record used. Since this criterion appears to closely match the conditions of the simulation below Bonners Ferry, an ERL of 35 years was selected. While there are three locations used for hydraulic model calibration which are thought to be representative of the river below Bonners Ferry, simulated stages from locations other than the three calibration locations were used for stage-frequency construction. During the calibration process the model reproduced observed stages better for some flood events than for others. This could be a result of changes in river bed, the uncertainty of the hydrology inputs (there is a significant amount of ungaged flow), and floodplain changes. As such, it was determined, based on judgment, that reducing the 52 years of simulated record to about 70% (approximately the middle of the range suggested by ETL 1110-2-537) provided a realistic uncertainty. Thus 35 years was selected as the ERL for frequency curves located at Bonners Ferry and points downstream.

For situations where no observed stage data is available for calibration, published roughness coefficients and judgment are the only options available. For these conditions, ETL 1110-2-537 suggests an ERL of 10 to 15 years. This situation corresponds to conditions under which simulation occurred for locations adjacent to the braided reach portion of the river. For frequency curves in this reach 10 years was selected as the ERL due to lack of calibration data.

SECTION 4 RISK BASED ANALYSIS

4.1 INTRODUCTION

The risk-based analysis portion of this study is intended to quantify the relative differences in Kootenai Valley flood risk between the four Libby Dam operation scenarios evaluated given the current state of the locally owned levees adjacent to the Kootenai River. This analysis combines the stage-frequency curves developed for each of the Libby Dam operation scenarios with the levee failure probability data and the depth-damage relationships (along with associated uncertainties), to compute various project performance and economic parameters. These parameters allow for the relative comparison of each of the scenarios.

The computer program HEC-FDA (Flood Damage Analysis) is used for this analysis. This program is typically used in flood damage reduction studies to formulate and compare various flood damage reduction measures. The program computes performance parameters such as conditional non-exceedance probability by event and annual non-exceedance probability. The former is essentially the probability of not getting wet given a specific flood event occurs and the latter is the probability of not getting wet during any given year in the future. In addition, when depth-damage data is added, an economic parameter called expected annual damage (EAD) can be computed and used in a comparative manner.

HEC-FDA uses Monte Carlo simulation to compute various performance and economic parameters. Monte Carlo simulation allows for the inclusion of uncertainty in the program inputs, such as the stage-frequency function.

4.2 RISK-BASED ANALYSIS

Within HEC-FDA, the stage-frequency curves, the levee stage-failure probability data, and the depth-damage functions for each damage area were entered. As discussed earlier, the uncertainty associated with the stage-frequency curve was computed within HEC-FDA based on an adjusted record length. At some locations, where the levee failure-probability curve was clearly located relative to the hypothetical events (the 0.01, 0.005, and 0.002 probability), the record length was reduced further, to 15 years, to better account for the increased uncertainty about these events. This increased uncertainty is due to factors such as hydrologic data uncertainty, lack of extreme calibration events, and the results of hydraulic model sensitivity runs.

The parameters of interest from the HEC-FDA simulations for each off-channel area are the annual exceedance probability (the probability of getting wet during any given year), the conditional non-exceedance probability by event (the probability of containing a specific flood event, should said flood event occur), and the expected annual damage (the mean, or average, of all damage values computed through Monte Carlo sampling). Table 8, Table 9, Table 10 and Table 11 show the computed values by location for annual exceedance probability (the probability of getting wet in any given year), conditional non-exceedance probability (the probability of not having flooding occur given a particular flood occurs) for the 0.01 (100-year) flood event, conditional non-exceedance probability for the 0.005 (200-year) flood event, and the expected annual damage respectively.

Table 8. Annual Exceedance Probability by Damage Location (%)

Off-Channel Area	Hydraulic Model Storage Area	Annual Exceedance Probability (%)			
		1764PH	1764PH10	1770PH	1770PH10
District 8	18	1.1	1.1	3.1	6.9
Wildlife Area @ Border	21	17.5	22	23	37
Left Bank @ Smith Creek	20	0.7	0.7	2.3	5.6
District 10	13	0.6	0.6	1.6	4.2
District 6	14 & 19	0.6	0.6	1.7	4.7
District 13	12	0.4	0.4	0.5	1.1
District 9	8	0.3	0.3	0.4	0.9
District 4	7	0	0	0	0
Left Bank RM 126	10	0	0	0	0
Left Bank RM 126	11	0.1	0.1	0.1	0.1
District 16	9	0.4	0.4	0.6	1.4
Right Bank RM 135	15	0	0.1	0.1	0.1
District 14	16	0.1	0.1	0.1	0.1
District 12	17	0.5	0.5	1.8	3.5
Districts 5 & 11	4	0.1	0.1	0.1	0.1
District 3	5	0.2	0.2	0.4	0.5
District 7	3	0.1	0.1	0.1	0.1
District 1 N of UPRR	1	0.1	0.1	0.1	0.1
District 1 S of UPRR	2	0.1	0.1	0.1	0.1
Downtown Bonners Ferry West of U.S. 2/95	2	0.1	0.1	0.1	0.1
Bonners Ferry North of Kootenai River	22	0.1	0.1	0.2	0.4
Kootenai River Inn	2C	0.7	0.7	3.1	9.4
Bonners Ferry East of U.S. 2/95 Except KRI	2B	0.1	0.1	0.1	0.1
District 2	23	0.1	0.1	0.1	0.1
[1]Below Grandview Cemetary	26	0.2	0.2	0.6	0.8
[1]Crossport	24	0.7	0.7	5.5	13.3

[1]No Formal Levee at this location. Referenced elevation is top of bank.

Table 9. Conditional Non-Exceedance Probabilities for 0.01 Flood Event (%)

Off-Channel Area	Hydraulic Model SA	Conditional Non-Exceed. Prob. for 1% Event			
		1764PH	1764PH10	1770PH	1770PH10
District 8	18	80	80	31	31
Wildlife Area @ Border	21	0.3	0.3	0	0
Left Bank @ Smith Creek	20	92	92	30	30
District 10	13	98	98	40	40
District 6	14 & 19	93	93	26	26
District 13	12	100	100	79	79
District 9	8	100	100	86	0.86
District 4	7	100	100	100	100
Left Bank RM 126	10	100	100	100	100
Left Bank RM 126	11	100	100	100	100
District 16	9	100	100	78	78
Right Bank RM 135	15	100	100	100	100
District 14	16	100	100	100	100
District 12	17	100	100	51	51
Districts 5 & 11	4	100	100	100	100
District 3	5	100	100	92	92
District 7	3	100	100	98	98
District 1 N of UPRR	1	100	100	100	100
District 1 S of UPRR	2	100	100	100	100
Downtown Bonners Ferry West of U.S. 2/95	2	100	100	100	100
Bonners Ferry North of Kootenai River	22	100	100	94	94
[1]Kootenai River Inn	2C	99	99	14	14
Bonners Ferry East of U.S. 2/95 Except KRI	2B	100	100	100	100
District 2	23	100	100	97	97
[1]Below Grandview Cemetary	26	97	97	83	83
[1]Crossport	24	99	99	3	3

[1]No Formal Levee at this location. Referenced elevation is top of bank.

Table 10. Conditional Non-Exceedance Probabilities for 0.005 Flood Event (%)

Off-Channel Area	Hydraulic Model Storage Area	Conditional Non-Exceed. Prob. for 0.5% Event			
		1764PH	1764PH10	1770PH	1770PH10
District 8	18	18	18	18	18
Wildlife Area @ Border	21	0	0	0	0
Left Bank RM 110.4	20	14	14	14	14
District 10	13	17	17	17	17
District 6	14 & 19	7	7	7	7
District 13	12	58	58	58	58
District 9	8	70	70	70	70
District 4	7	100	100	100	100
Left Bank RM 126	10	100	100	100	100
Left Bank RM 126	11	97	97	97	97
District 16	9	55	55	55	55
Right Bank RM 135	15	99	99	99	99
District 14	16	98	98	98	98
District 12	17	32	32	32	32
Districts 5 & 11	4	97	97	97	97
District 3	5	75	75	75	75
District 7	3	91	91	91	91
District 1 N of UPRR	1	100	100	100	100
District 1 S of UPRR	2	100	100	100	100
Downtown Bonners Ferry West of U.S. 2/95	2	100	100	100	100
Bonners Ferry North of Kootenai River	22	90	90	90	90
Kootenai River Inn	2C	0	0	0	0
Bonners Ferry East of U.S. 2/95 Except KRI	2B	100	100	100	100
District 2	23	91	91	91	91
[1]Below Grandview Cemetary	26	68	68	68	68
[1]Crossport	24	0	0	0	0

[1]No Formal Levee at this location. Referenced elevation is top of bank.

Table 11. Expected Annual Damage Values by Damage Location

Off-Channel Area	Hydraulic Model SA	Expected Annual Damage ($1,000)			
		1764PH	1764PH10	1770PH	1770PH10
District 8	18	63	63	174	377
Wildlife Area @ Border	21	38	45	48	88
Left Bank @ Smith Ck	20	1	1	2	6
District 10	13	3	3	7	17
District 6	14 & 19	10	10	26	69
District 13	12	1	1	2	4
District 9	8	1	1	2	3
District 4	7	0	0	0	0
Left Bank RM 126	10	0	0	0	0
Left Bank RM 126	11	0	0	0	0
District 16	9	54	54	87	182
Right Bank RM 135	15	0	0	0	0
District 14	16	0	0	0	0
District 12	17	2	2	6	10
Districts 5 & 11	4	1	1	1	1
District 3	5	1	1	3	4
District 7	3	0	0	1	1
Downtown Bonners Ferry West of U.S. 2/95 & District 1	2	0	0	0	0
Bonners Ferry North of Kootenai River	22	1	1	2	3
Kootenai River Inn	2C	2	2	8	22
Bonners Ferry East of U.S. 2/95 Except KRI	2B	0	0	0	0
District 2	23	0	0	0	0
[1]Below Grandview Cemetery	26	0	0	0	0
[1]Crossport	24	0	0	0	0
Valley Total		179	186	369	789

[1]No Formal Levee at this location. Referenced elevation is top of bank.

It should be noted that many of the values in the preceding four tables were adjusted from the raw HEC-FDA output. Inspection of the HEC-FDA output revealed that in some cases the program was computing probabilities and EAD values that did not make sense. For instance, at the off-channel area adjacent to the left bank at river mile 110.4 (Storage Area 20), the conditional non-exceedance probability for the 0.01flood event was computed as 0.9169 under 1764PH. Under 1764PH10, the conditional non-exceedance probability was computed as 0.9350 at this location. The difference in these two scenarios is that 1764PH10 incorporates additional outflow from Libby Dam over 1764PH. This additional flow is only provided when conditions are such that the stage at Bonners Ferry would be at or below 1,764 feet. The difference this makes in the stage-frequency curves is that the 'more frequent' end of the 1764PH10 curve has higher stages. Another way to think of is that a given recurrence interval can have a higher stage under 1764PH10 than 1764PH. Due to the impacts of flood control regulation, the two curves converge somewhere toward the 'less frequent' end of the graph. Since the levee conditions are the same for all four scenarios, and it is assumed that each scenario has the same type of uncertainty, it makes sense then that 1764PH10 has to either have the same or greater flood risks associated with it when compared to 1764PH, not the other way around as the raw FDA data would indicate. This type of discrepancy was also found in some cases when comparing 1764PH10 with 1770PH (1770PH should pose the same or greater flood risk than 1764PH10), 1770PH with 1770PH10, etc.

This issue was traced to the way HEC-FDA applies the uncertainty band to a graphical frequency curve. The methodology used is based on the application of order statistics to calculate the uncertainty band about the curve. This method is sensitive to changes in slope of the frequency curve (USACE 1997). The flatter areas of the curve indicate lower variability in stage and as such the order statistic method will compute a smaller uncertainty than at the steeper areas. In the case of the 1764PH and 1764PH10 frequency curves, the 1764PH10 curve would have more 'flat' area than the 1964PH10 curve at the more frequent end due to the more frequent need to regulate Libby Dam outflows (for example compare Figure 15 with Figure 16). Even though above about a 0.2 annual probability event these two curves converge (i.e. the 0.005 stage is the same), evidently the narrower uncertainty band associated with the regulated portion of the 1764PH10 curve impacts the uncertainty band at the less frequent end as well. Depending on the levee failure/top of levee elevations, and their position relative to the frequency curve, this difference in uncertainty can result in a lower conditional non-exceedance probability value for a given event (the probability of passing a particular flood event without getting wet) being greater under 1764PH10 than under 1764PH. Since there is no known physically-based reason for the 1764PH10 curve to have a smaller uncertainty band than the 1764PH curve, it any risk-based parameter (such as EAD, annual exceedance probability, or conditional non-exceedance probability by event) for 1764PH should be either equal or more favorable than for the 1764PH10 scenario.

For the purposes of this study, some judgment was applied to the raw data in terms of appropriate values to report. Table 12 lists raw HEC-FDA output for Drainage District 14 (noted as Storage Area 16 for modeling purposes) as well as the adjusted values used for reporting. The location has a probable non-failure elevation of 1,768 feet, a probable failure elevation (defined as 85% probability of failure) of 1,771 feet, and a top of levee elevation of 1772 feet. The conditional non-exceedance values are for the 0.004 flood event. This is a very large flood and Libby Dam is not able to regulate it to an elevation at Bonners Ferry below elevation 1,770, event if the regulating stage is 1,764. Given this, the stages resulting from the 0.004 event are the same for all scenarios. Note that the uncertainty band (see Table 12) computed by HEC-FDA for all three scenarios are different even though the 0.004 stage is the same on all four frequency curves. Record lengths of 35 years were used for all four cases. 1764PH has a larger computed uncertainty than 1764PH10 and 1770PH has a larger uncertainty than 1770PH10. Given the elevation of the levee probability-failure elevation curve, it makes sense that a higher conditional non-exceedance probability would be computed for 1764PH10 than for

1764PH since the wider uncertainty band would allow for a greater chance of levee failure during the sampling within the uncertainty band that occurs during the Monte Carlo simulation process used in HEC-FDA.

Table 12. Example HEC-FDA Output Processing

HEC-FDA Raw Output for District 14 (SA 16)				
PNP=1768	PFP=1771	TOL=1772	0.004 w/s elevation=1767.10 ft	
	Annual exceed. Probability	**Conditional Non-Exceed Probability 0.004 Event**	**Expected Ann. Damage**	0.004 ± 2 SD Uncert. Band (ft)
1764PH	0.001	0.9816	0.07	4.7
1764PH10	0.001	0.9906	0.07	2.4
1770PH	0.001	0.9657	0.09	4.26
1770PH10	0.001	0.9884	0.07	2.84
Reported Values				
1764PH	0.001	0.97	0.07	
1764PH10	0.001	0.97	0.07	
1770PH	0.001	0.97	0.09	
1770PH10	0.001	0.97	0.09	

Since it is assumed that the 1764PH, 1764PH10, 1770PH, and 1770PH10 all have the same uncertainty at locations where the curves are the same (as occurs with the 0.004 flood event), all scenarios should have the same conditional non-exceedance probability. Given this, the lowest value among the four scenarios was reported for all scenarios. For conditional non-exceedance probabilities for events such as the 0.01 flood, where there is a difference between 1764PH or 1764PH10 and 1770PH or 1770PH10, then the same method was used to arrive at the reporting figure but instead of all four scenarios being looked at together, 1764 scenarios were treated separately from the 1770 scenarios. Had the raw conditional non-exceedance probability values in Table 12 been for the 0.01 event, then the reported values for 1764PH and 1764PH10 would have been 0.98 and the reported values for 1770PH and 1770PH10 would have been 0.97.

The raw expected annual damage numbers, while very small, raise a red flag as well. This value is based on a weighting of damages that can occur over all portions of the stage-frequency curve, and associated uncertainty, which overlap with the levee elevation-probability of failure data and/or the top of levee elevation. Visual inspection of the 1770PH and 1770PH10 frequency curves (Figures 17 and 18), and the assumption that uncertainties are essentially the same, would indicate that the expected annual damage value for 1770PH could not be higher than for 1770PH10. This is because there is no point on the 1770PH curve which is higher than that on the 1770PH10 curve. The 1770PH points are either the same or lower than on 1770PH10. Since the levee data is exactly the same between the two scenarios, the expected annual damage value discrepancy would then seem to be based on the wider uncertainty band for the

1770PH10 curve as computed by HEC-FDA. In this case, the reported expected annual damage number for both 1770PH and 1770PH10 is the greater of the two 1770 numbers. Had the raw values for 1770PH and 1770PH10 been 0.07 and 0.09 respectively, they would have been used as the reported expected annual damage numbers. For reporting purposes however, the expected annual damage numbers were rounded to the nearest $1000 so in this case they effectively become $0.

SECTION 5 FINDINGS AND IMPLICATIONS

5.1 BONNERS FERRY FLOOD LEVEL ASSESSMENT

The results of this study indicate that operating Libby Dam under a scenario where active flood control regulation would not occur unless elevation 1,770 at Bonners Ferry would be exceeded would result in an increase in flood risk and potential damages at various locations in the lower Kootenai Valley given the current condition of the locally owned levees. Results also indicate that providing a sturgeon flow which includes an additional 10 kcfs of flow over current powerhouse capacity while using the current regulating target of 1,764 feet at Bonners Ferry would only increase flood risk at one location as compared to the condition where sturgeon flows are capped at powerhouse capacity. This location is the wildlife refuge on the west side of the river adjacent to the Canadian border. Furthermore, results indicate that at the affected areas, providing a powerhouse capacity plus 10 kcfs sturgeon flow under a 1,770 flood stage would increase flood risk even more than under a powerhouse capacity sturgeon flow and a 1,770 flood stage. The amount of increase varies from area to area.

Table 13 lists the areas where 1764PH10, 1770PH, or 1770PH10 produced an increase in annual exceedance probability and/or expected annual damage. These

Table 13. Kootenai Valley Areas Experiencing a Flood Risk Increase

Area	Hydraulic Model Storage Area	Annual Probability of Flooding(%)/EAD ($1,000)				Levee Rehab Cost Est.
		1764PH	1764PH10	1770PH	1770PH10	
District 8	18	1/63	1/63	3/174	7/377	$4,865,000
Wildlife Area @ Border	21	18/38	22/45	23/48	37/88	$3,992,000
Left Bank @ Smith Creek	20	.7/1	.7/1	2/2	6/6	$500,000
District 10	13	.6/3	.6/3	2/8	4/17	$11,227,000
District 6	14 & 19	.6/10	.6/10	2/26	5/69	$11,726,000
District 13	12	.4/1	.4/1	.5/2	1/4	$874,000
District 9	8	.3/1	.3/1	.4/2	.9/3	$3,992,000
District 16	9	.4/54	.4/54	.6/87	1/182	$749,000
District 12	17	.5/2	.5/2	2/6	4/10	$7,111,000
District 3	5	.2/1	.2/1	.4/3	.5/4	$1,996,000
Bonners Ferry North of Kootenai River	22	.1/1	.1/1	.2/2	.4/3	$1,470,000
Kootenai River Inn	2C	.7/2	.7/2	3/8	9/22	no formal levee
[1]Below Grandview Cemetery	26	.2/0	.2/0	.6/0	.8/0	no formal levee
[1]Crossport	24	.7/0	.7/0	5/0	13/0	no formal levee

[1]No Formal Levee at this location. Referenced elevation is top of bank.

EAD values rounded to nearest $1,000

areas represent the areas impacted by either providing a 10 kcfs over powerhouse capacity sturgeon flow and/or increasing the regulation elevation at Bonners Ferry. The areas not listed did not experience an increase in risk based on this analysis. The Kootenai River Inn, the Crossport area, and the area below Grandview Cemetery do not have formal levees. Much of the Crossport area however appears to be ‘protected’ by railroad and road alignments. It is unknown what type of flood protection these structures would provide. As such, these embankments were assumed to provide no protection in this study.

Table 13 lists annual probability of flooding and expected annual damage values together for comparison and perspective purposes. While some areas, such as District 16, may not appear to have a very large chance of incurring flooding during any given year, the expected annual damage number is higher than other areas with a greater chance of incurring flooding during any given year, such as the Wildlife Area at the border. Table 13 also shows the variability in flood risk from area to area within the valley.

For comparative purposes only, Table 13 also includes a cost estimate to repair the levees at the listed areas. These estimates are based on data from 2004 levee inspections and are based on an estimate of the percentage of levee length at each area which is damaged. These estimates are included as another parameter to consider when examining the flood risk data. More detail on the cost estimate methodology can be found in **Appendix E**.

Many of the impacted areas are located toward the Canadian border end of the valley. Libby Dam does not have the ability to control stages at this end of the reach to the degree it does at Bonners Ferry mainly due to the increasing influence of Kootenay Lake elevation (local inflow below Bonners Ferry has some degree of impact as well) on stages. Examination of the 1764PH and 1764PH10 frequency curves in **Appendix B** bears this out. The portion of the curve where active regulation is limiting stages at Bonners Ferry to 1,764 feet is depicted with a noticeable ‘flat spot’. Looking at the curves farther downstream, this “flat spot” becomes less defined.

In terms of the conditional probability of containing a given flood event, should it occur, Table 14 and Table 15 show areas which have less than a 90% probability of containing the 0.01 and 0.005 floods respectively.

In Table 15 the conditional probabilities listed are the same for all scenarios. This is because at this extreme flood event, the regulation of each scenario essentially becomes the same.

The results of this study indicate that given the 2004 condition of the lower Kootenai Valley levees, increasing the regulating stage at Bonners Ferry to elevation 1,770 from elevation 1,764 would increase the flood risk and potential damages from overbank flooding at a number of areas in the lower Kootenai Valley. These increased flood risks appear to be variable by location. In addition, adding the additional component of increasing the sturgeon flow by 10 kcfs further increases the flood risk, particularly under the 1,770 regulating stage.

Table 14. Areas with 0.01 (100-Yr) Flood Protection below 90% Reliability

Off-Channel Area	Hydraulic Model Storage Area	Conditional Non-Exceed. Prob. for 1% Event			
		1764PH	1764PH10	1770PH	1770PH10
District 8	18	80	80	31	31
Left Bank at Int. Boundary	21	0.3	0.3	0	0
Left Bank @ Smith Creek	20	92	92	30	30
District 10	13	98	98	40	40
District 6	14 & 19	93	93	26	26
District 13	12	100	100	79	79
District 9	8	100	100	86	86
District 16	9	100	100	78	78
District 12	17	100	100	51	51
Kootenai River Inn	2C	99	99	14	14
[1]Below Grandview Cemetary	26	97	97	83	83
[1]Crossport	24	99	99	3	3

Table 15. Areas with 0.005 (200-Year) Flood Protection below 90% Reliability

Off-Channel Area	Hydraulic Model Storage Area	Conditional Non-Exceed. Prob. for 0.5% Event			
		1764PH	1764PH10	1770PH	1770PH10
District 8	18	18	18	18	18
Wildlife Area @ Border	21	0	0	0	0
Left Bank RM 110.4	20	14	14	14	14
District 10	13	17	17	17	17
District 6	14 & 19	7	7	7	7
District 13	12	58	58	58	58
District 9	8	70	70	70	70
District 16	9	55	55	55	55
District 12	17	32	32	32	32
District 3	5	75	75	75	75
[1]Kootenai River Inn	2C	0	0	0	0
[1]Below Grandview Cemetery	26	68	68	68	68
[1]Crossport	24	0	0	0	0

[1]No Formal Levee at this location. Referenced elevation is top of bank.

This analysis indicates that at most locations the levees are of an adequate height to provide adequate protection under a scenario where 1,770 at Bonners Ferry is the regulating stage. The issue however is the condition of the levees. In many locations probable failure elevations well below the top of levee elevation have been identified. For the most part, these probable failure elevations have a great deal of influence on increasing the risk of flooding when comparing 1,770 with 1,764 as a regulation stage. Under a 1,770 regulating stage there would be concerns about increased flood risk at several areas without levees, such as the Kootenai River Inn, as well.

As these levees are not federal levees, the only means for federal involvement in their rehabilitation is through the PL84-99 program.

While increasing the regulating stage to elevation 1,770 at Bonners Ferry would increase the flood risks and potential damages, the 52 years of hydrologic data simulated did not result in many years where the stage actually approached elevation 1,770. In fact under 1770PH10, only three years (excluding the extreme events) recorded stages between 1,769 and 1,770 feet at Bonners Ferry. This may indicate that a Bonners Ferry regulation elevation somewhere between elevations 1,764 and 1,770 may prove to minimize the amount of levee rehabilitation required to maintain acceptable flood risk and associated potential damage while at the same time decreasing the number of years which the powerhouse or powerhouse capacity plus 10 kcfs sturgeon flow would need to be constrained to meet the Bonners Ferry regulation stage.

5.2 LIBBY DAM TO TROY, MT, CHANNEL CAPACITY ANALYSIS

Since Libby Dam began operating in 1972, peak flows in the Kootenai River below the dam (and for the purposes of this discussion between the dam and Troy, MT) have been dramatically reduced. As a result, there has been a significant amount of development that has occurred along the river. Development along this reach starts just above the Kootenai River/ Yaak River confluence below Troy at approximately river mile 178 and extends upstream to approximately the Kootenai River/ Fisher River confluence at approximately river mile 218. The Federal Emergency Management Administration (FEMA) has delineated a 0.01 annual chance exceedance (100-year) floodplain for the Kootenai River between Libby Dam and Troy, MT. This floodplain is based on a Libby Dam release of 60,500 cfs. In addition, several tributaries to the Kootenai have been mapped as well.

As per USFWS 2000 BiOp RPA 8.2a2, this portion of the study is intended to determine a reasonable channel capacity (in terms of flow) between Libby Dam and the Idaho border based on the current level of development along the river. No attempt is made to ascertain whether the level of development is appropriate or if it has resulted in reduced Libby Dam operational flexibility. In addition this exercise seeks to determine if the channel capacity below the dam would constrain additional project releases above powerhouse capacity (up to 10,000 cfs or approximately 35,000 cfs) in May and June.

In June and July of 2002 high inflows to Libby Dam resulted in unusually high project outflows as the reservoir neared its maximum elevation, forcing involuntary releases. Peak outflows of approximately 40,000 cfs occurred on 2 July 2002. Figure 19, Figure 20, and Figure 21 are photos of the Kootenai River during this high flow period relative to several structures along the river.

Figure 19. Structure along Kootenai River during 2 July 2002 high flow period.

Figure 20. Structure along Kootenai River during 2 July 2002 high flow period.

Figure 21. Structure along Kootenai River during 2 July 2002 high flow period.

While river stages did get close to several inhabited dwellings, none were flooded. From available data, photos and personal accounts, it appears that given the 2002 level of flood plain development, this event for all practical purposes reached the channel capacity of the river between Libby Dam and the Kootenai/Yaak confluence. As such, the 2002 event is the basis for estimating the channel capacity of the Kootenai River in this area.

Appendix D is a report on the channel capacity of the Libby Dam to Troy, MT reach of the Kootenai River prepared in 2004. This report outlines a hydraulic modeling effort which has been determined to be unwarranted. Given that, by all appearances, the 2002 flow event reached the channel capacity of the river relative to current development, it has been determined that the channel capacity estimate for the purposes of this study would be based on analysis of this flow event.

Other than the USGS stream flow gage immediately below Libby Dam, there are no other active gages in the reach. It is difficult to accurately determine what the flow was at various locations below the dam due to the ungaged local inflow. Some relevant gage data available from 2 July 2002 for estimating these values include:

- Peak Libby Dam outflow at USGS gage 12301933-40,400 cfs
- Peak Leonia flow from USGS gage 12305000-48,400 cfs
- Daily flow on the Fisher River from gage 12302055-653 cfs
- Daily flow on the Yaak River from gage 12304500-855 cfs

All the gages listed above have good ratings except for the Leonia gage which is rated fair for discharges above 25,000 cfs (USGS 2003). Based on the difference between the flow estimated at the below Libby Dam and the Leonia gages, the local between the dam and Leonia would be estimated to be about 8,000 cfs, or 2.90 cfs per square mile. Alternately the Libby Dam to Leonia local could be estimated using Fisher and Yaak River data and a drainage area ratio. This would yield a Libby to Leonia local flow of 2,920 cfs, or 1.06 cfs per square mile, a significantly lower value.

Since Troy is located just upstream of the Yaak/Kootenai confluence, the peak flow at Troy is estimated based on the Libby Dam outflow plus the Libby Dam to Leonia local flow minus the flow in the Yaak River. Using the high estimate for the local flow, a estimated peak flow of 47,500 cfs is computed. Using the low value for the local flow a value of 42,400 cfs is computed. The average of these high and low values is 45,000 cfs.

For the Libby, MT area, high and low estimates can be computed as well. Using the high local flow value of 2.90 cfs per square mile, a value of 44,000 cfs is computed. Basing the Libby Dam to Libby, MT local on the Fisher River drainage area ratio, a value of 0.78 cfs pe square mile is computed. When applied to the 1,255 square miles of drainage area between Libby Dam and Libby, MT, this equates to a local flow of about

980 cfs. Using this local flow value yields a flow estimate at Libby, MY of 41,400 cfs. The average of these high and low estimates is 42,700 cfs.

These estimates are for the channel capacity of the Kootenai River only. Local tributaries are not considered. Depending on the timing of flows from local tributaries, the backwater effect from high Kootenai River stages associated with channel capacity-type flows could possible pose flooding concerns on these streams.

During the May timeframe, using estimates of Libby Dam to Libby, MT local flow, based on the average channel capacity estimate of 42,700 cfs, there were 30 days out of the period from 1948 to 1999 where an outflow of 35,000 cfs would have exceeded this capacity. These 30 days were distributed among the years 1948, 1949, 1950, and 1954. It is interesting to note that these years form the early part of the period of record data used. It is unknown is this is a reflection of data quality or just chance. During the June timeframe, using the same methodology, there were four days were the 42,700 cfs channel capacity would have been exceeded with a 35,000 cfs outflow. These four days occurred in the year 1950.

It should be noted that depending on Lake Koocanusa pool elevation, powerhouse capacity plus 10,000 cfs can be more than 35,000 cfs. It could potentially be as much as 38,000 cfs. In this case, based on the estimated 1948 to 1999 local flows, there would be 169 days where the local flow would potentially constrain this type of operation.

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SECTION 7 APPENDICES

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7.1 APPENDIX A – EXTREME EVENTS

Figure 22 APPENDIX A
[See following pages]

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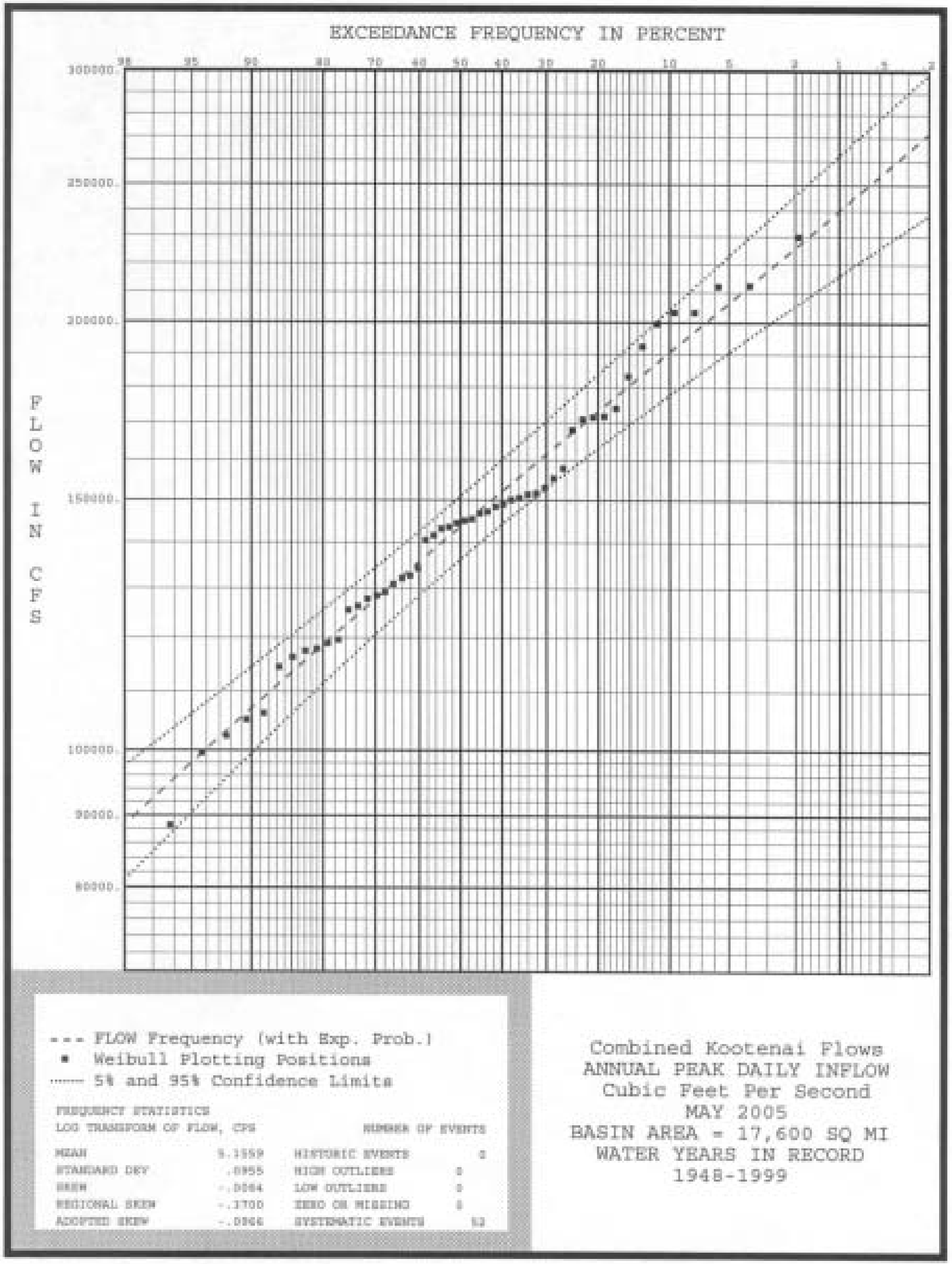

Figure 23. Combined Peak Libby Inflow, Below Libby Local, and Below BF Local

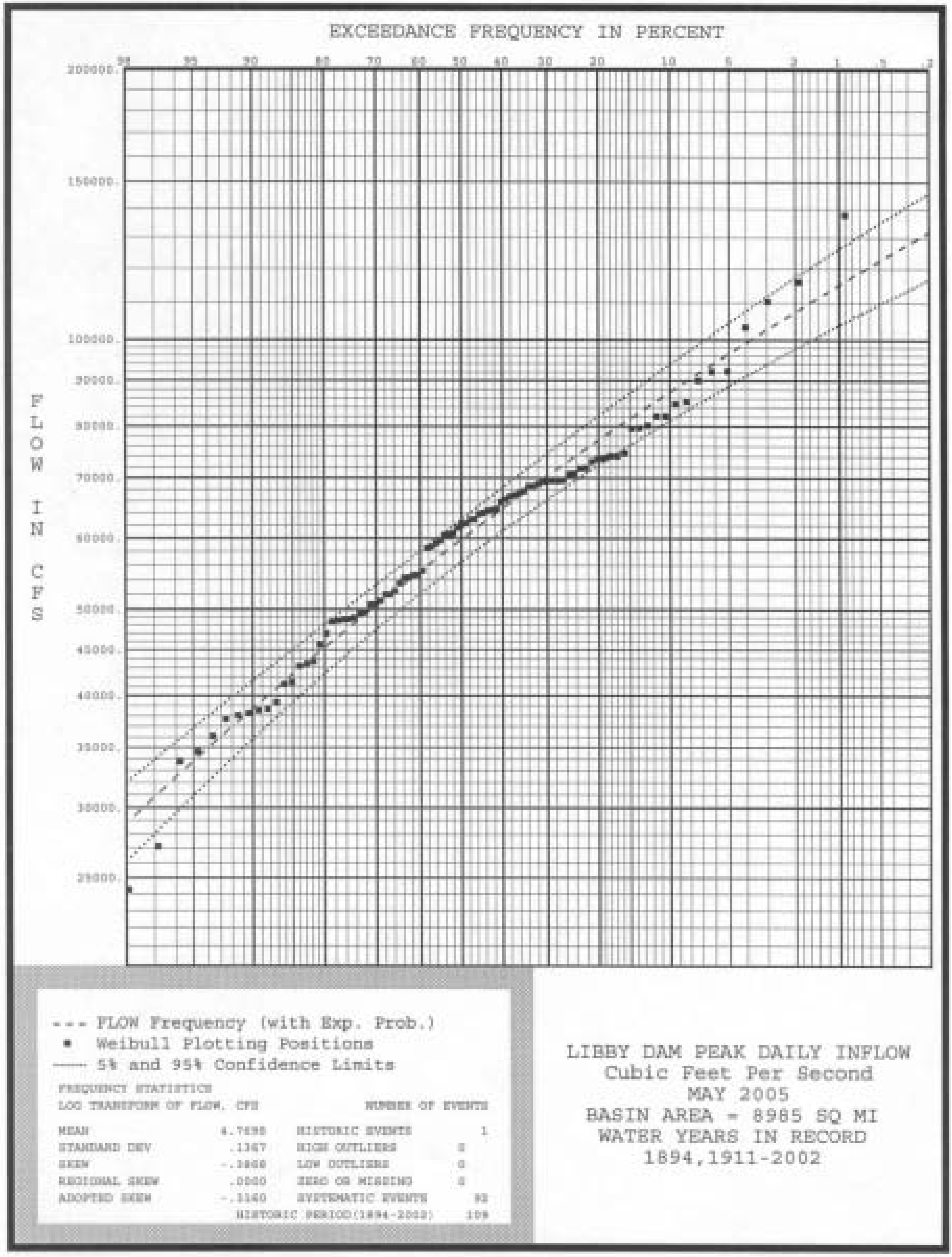

Figure 24. Libby Dam Inflow Frequency Curve

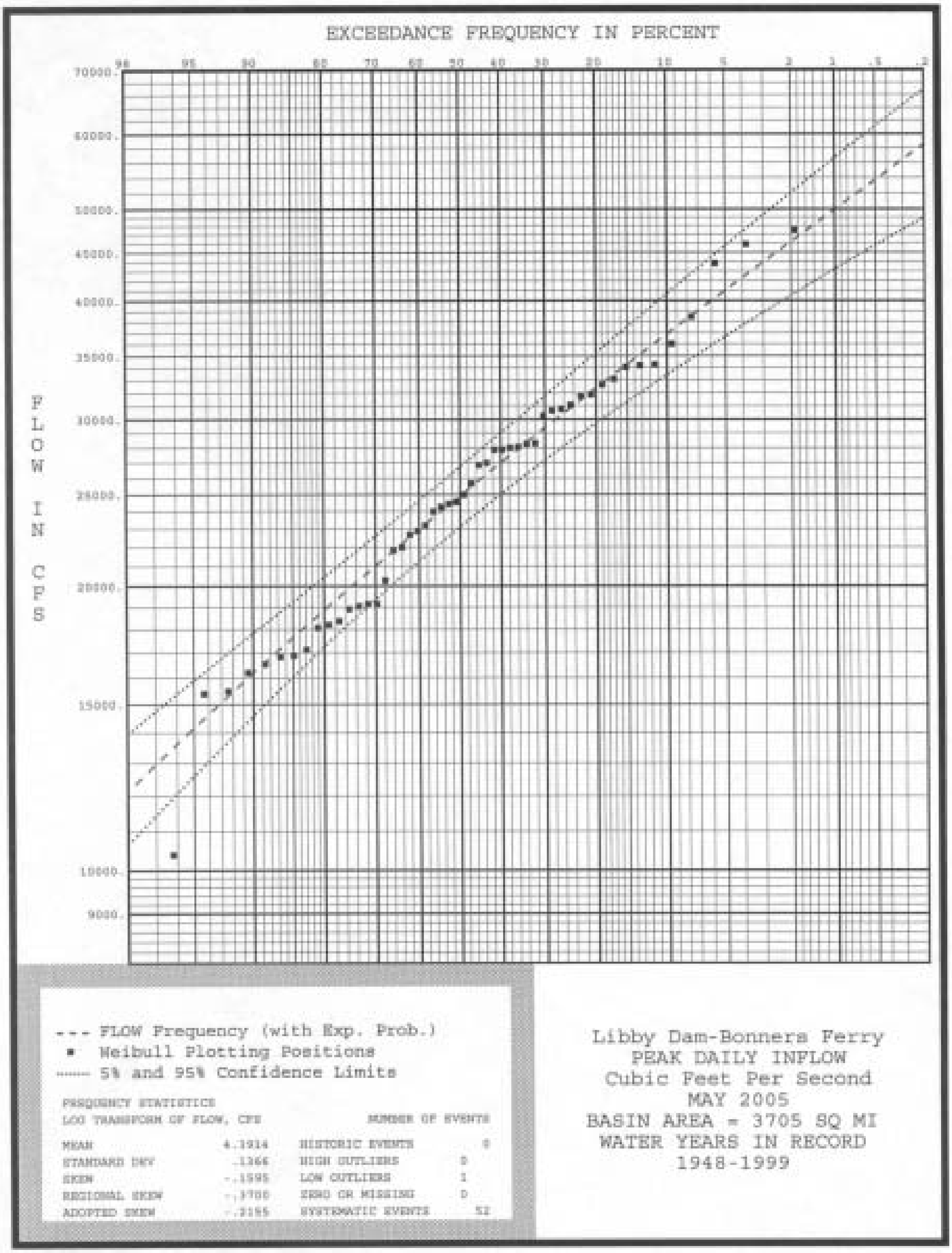

Figure 25. Below Libby Dam Local Flow Frequency Curve

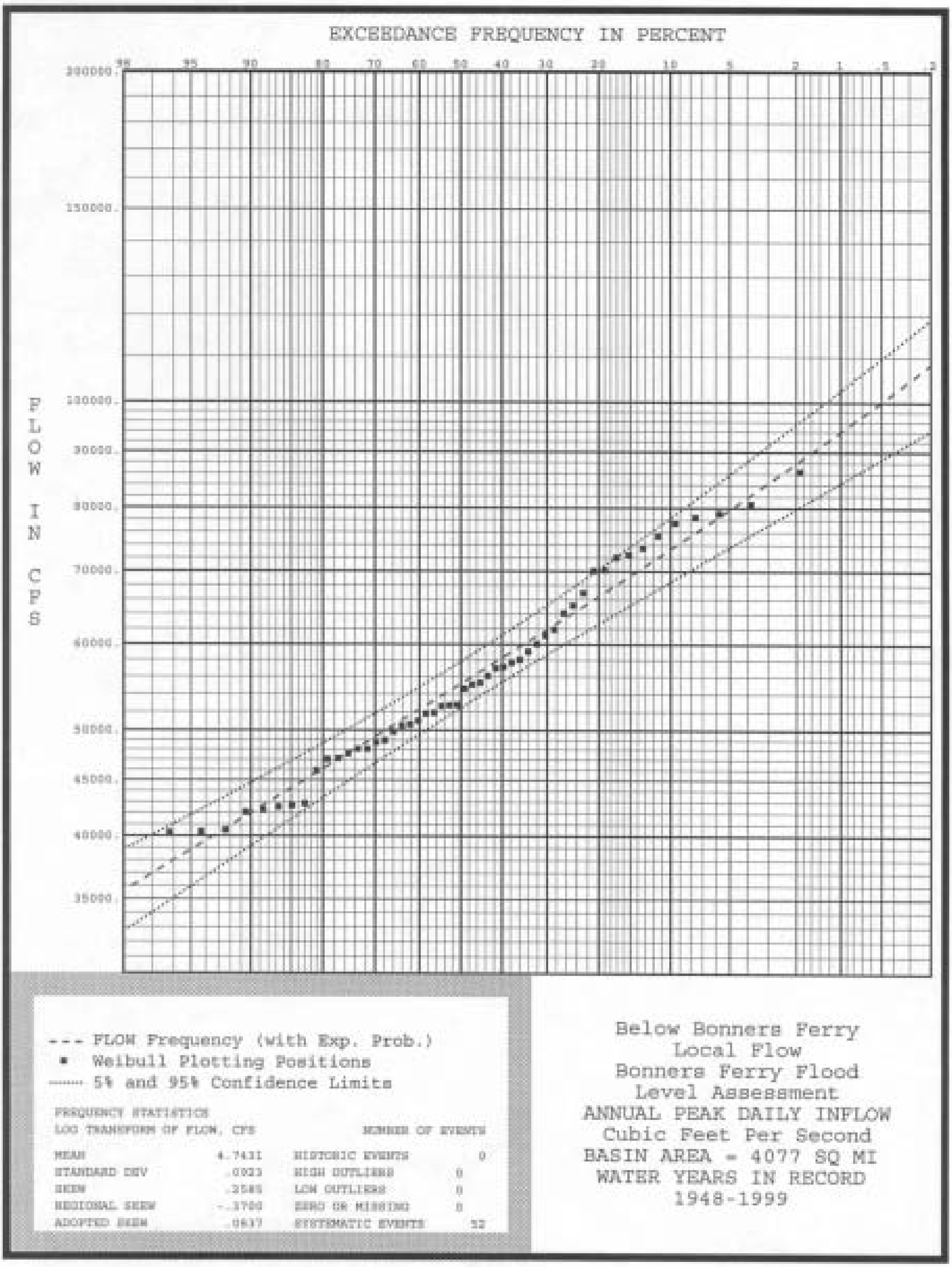

Figure 26. Below Bonners Ferry Local Flow Frequency Curve

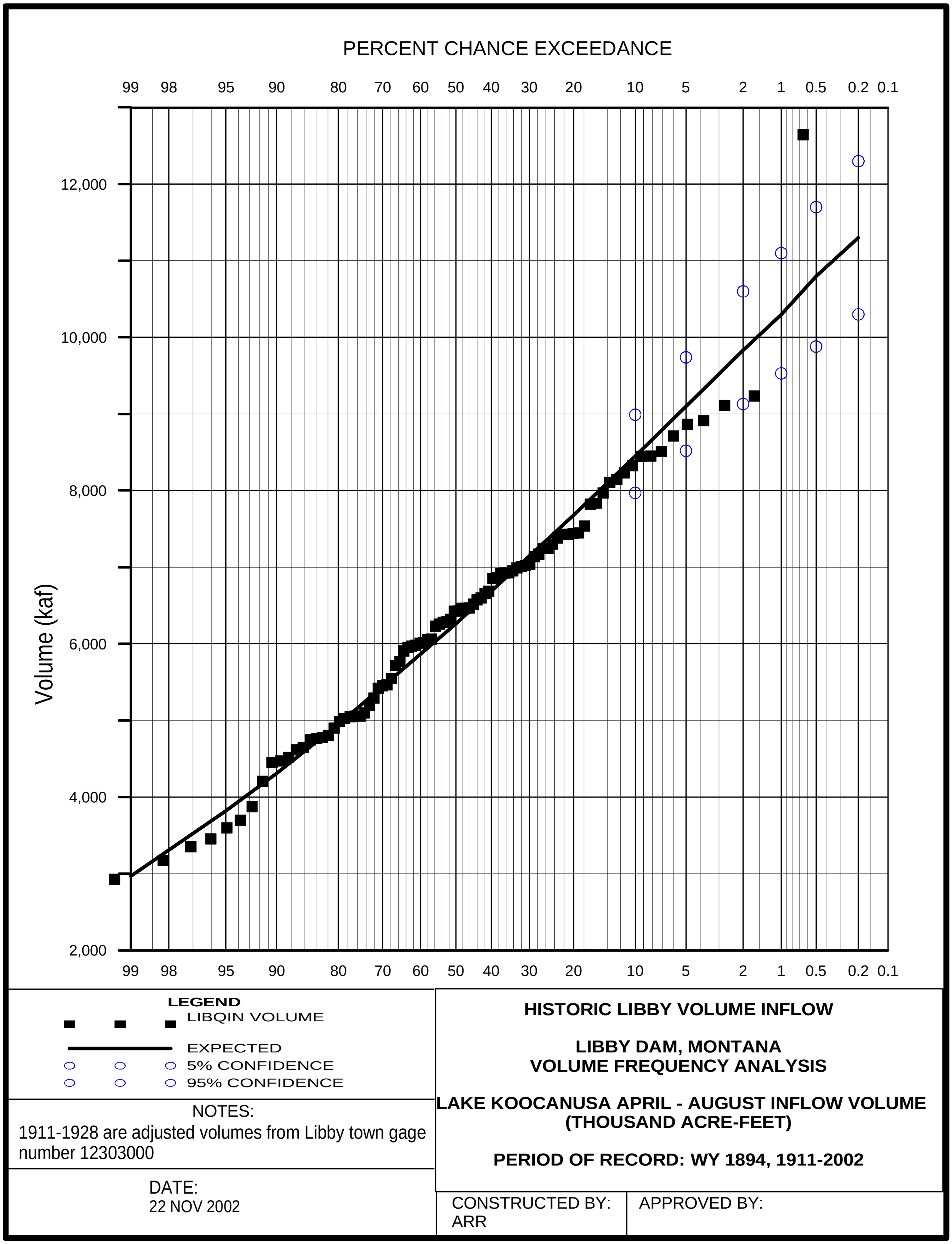

Figure 27. Libby Dam April-August Inflow Volume Frequency Curve

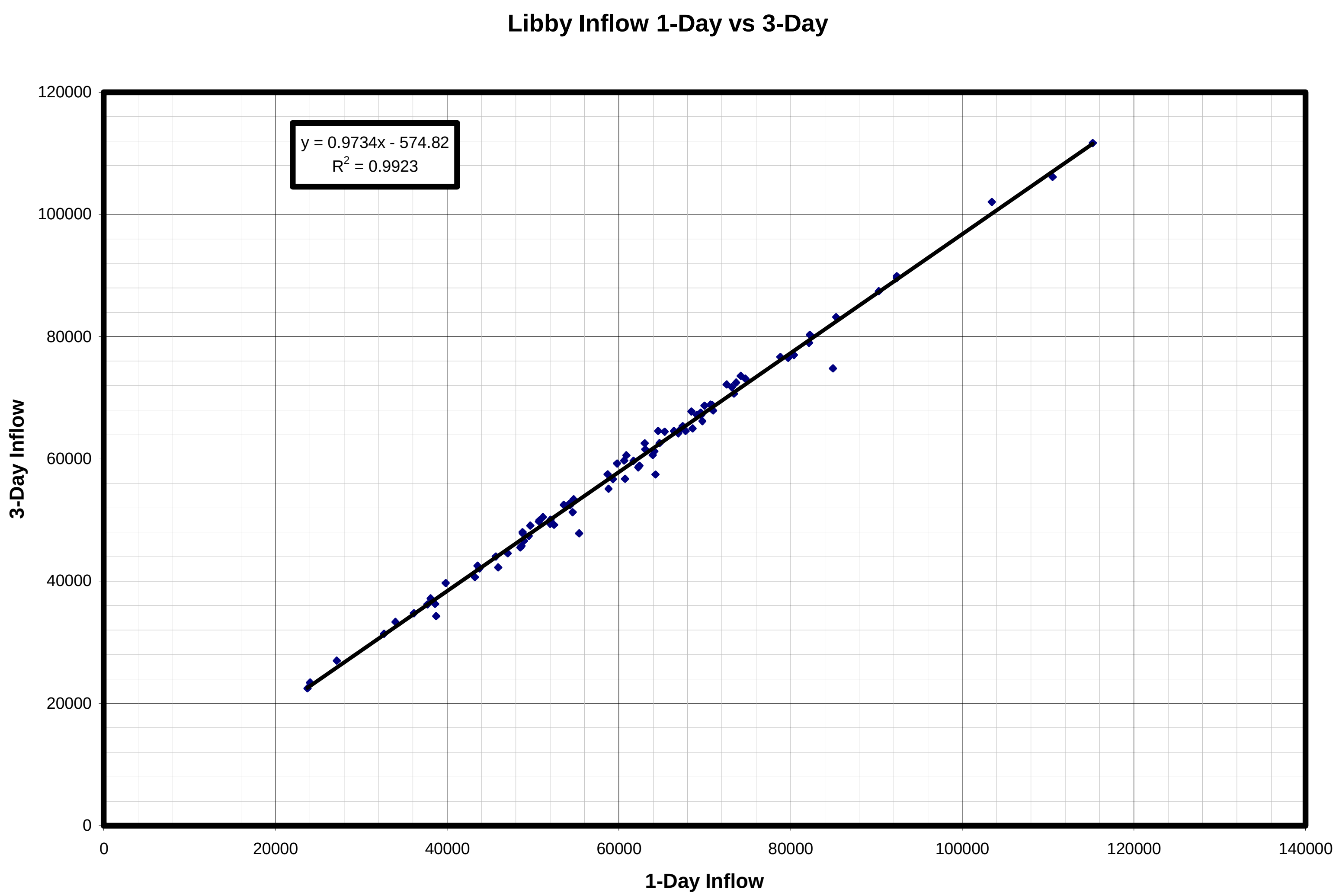

Figure 28. Libby Dam Inflow Peak 1-Day Flow vs. Peak 3-Day Flow

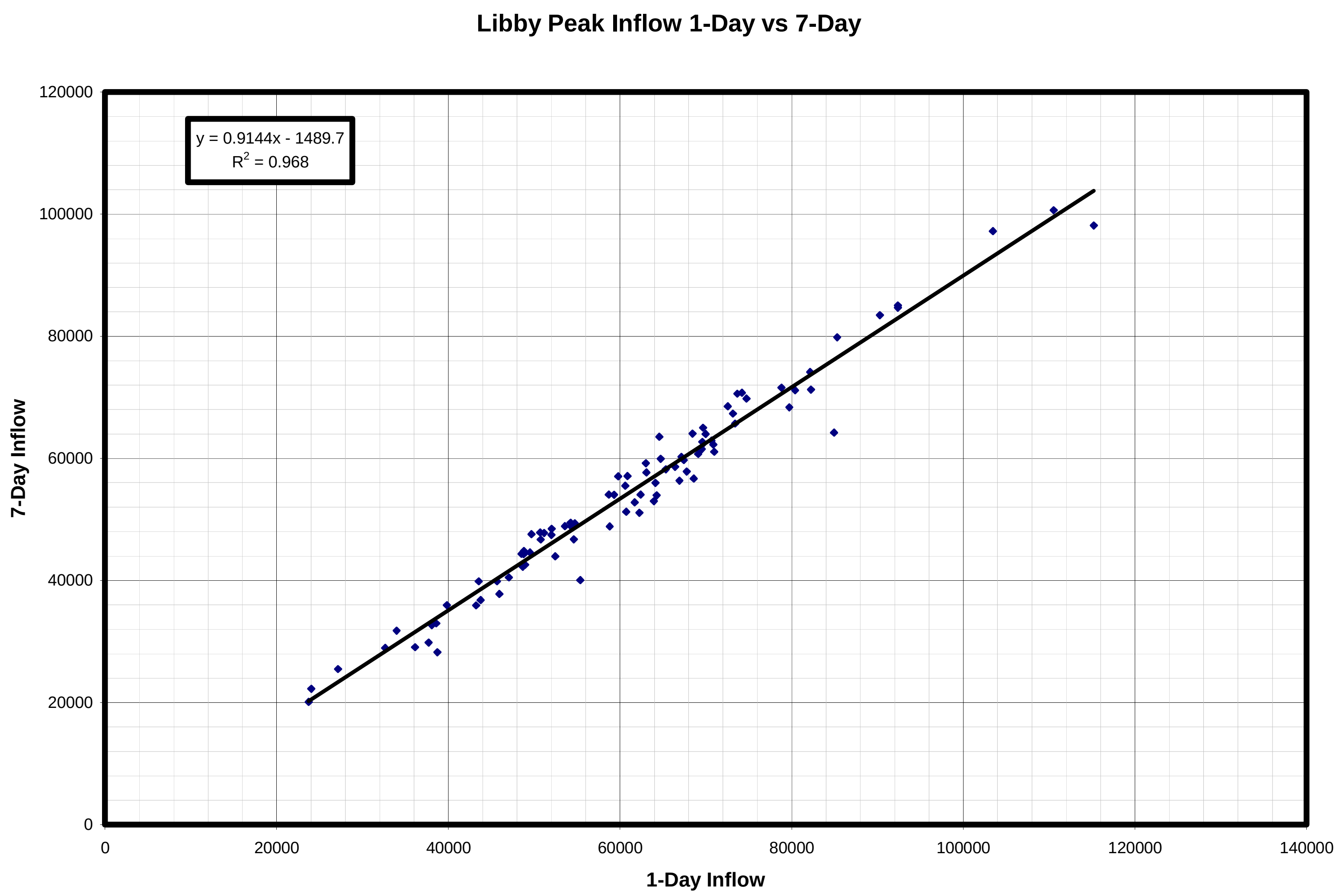

Figure 29. Libby Dam 1-Day Peak Inflow vs. 7-Day Peak Inflow

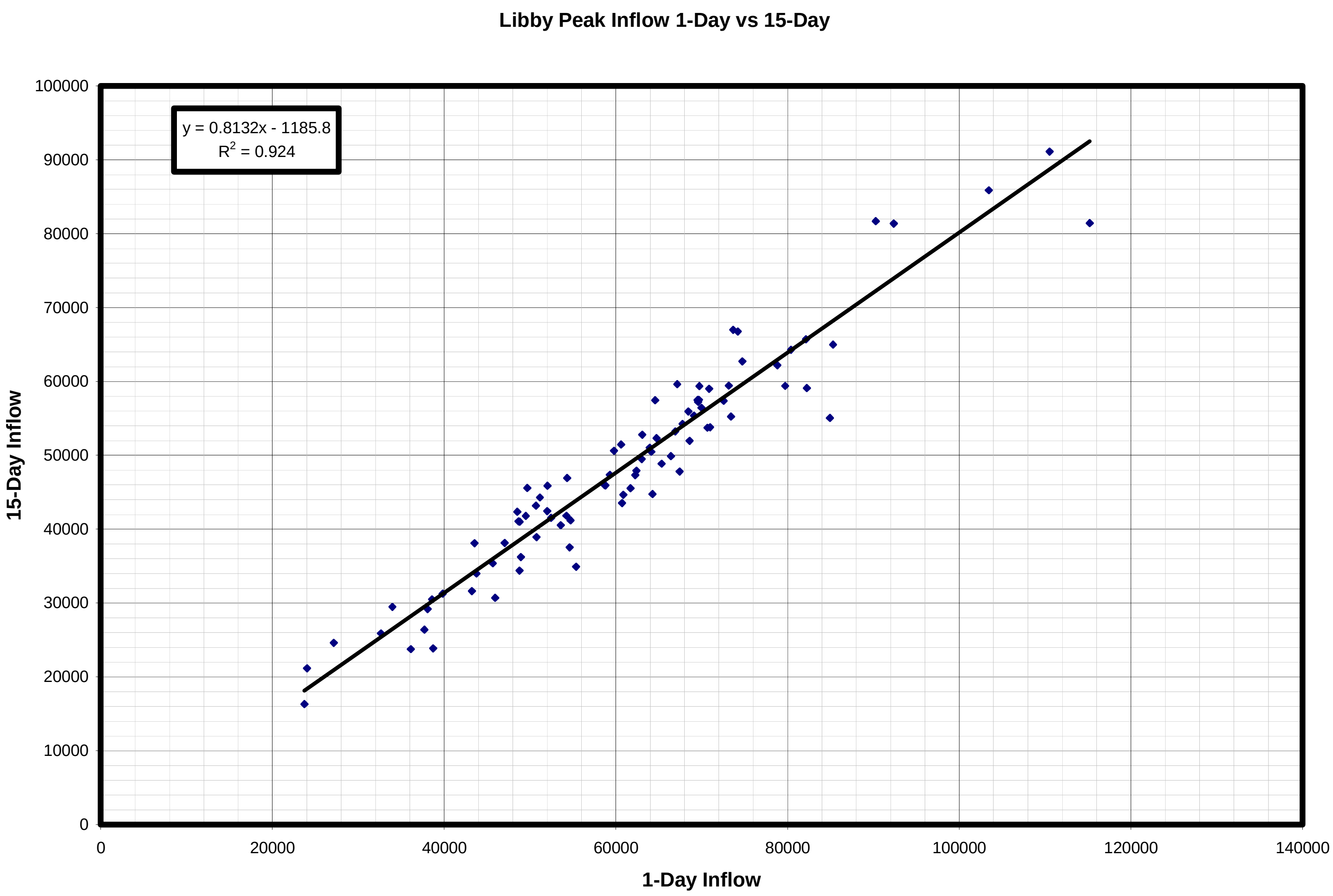

Figure 30. Libby Dam 1-Day Peak Inflow vs. 15-Day Peak Inflow

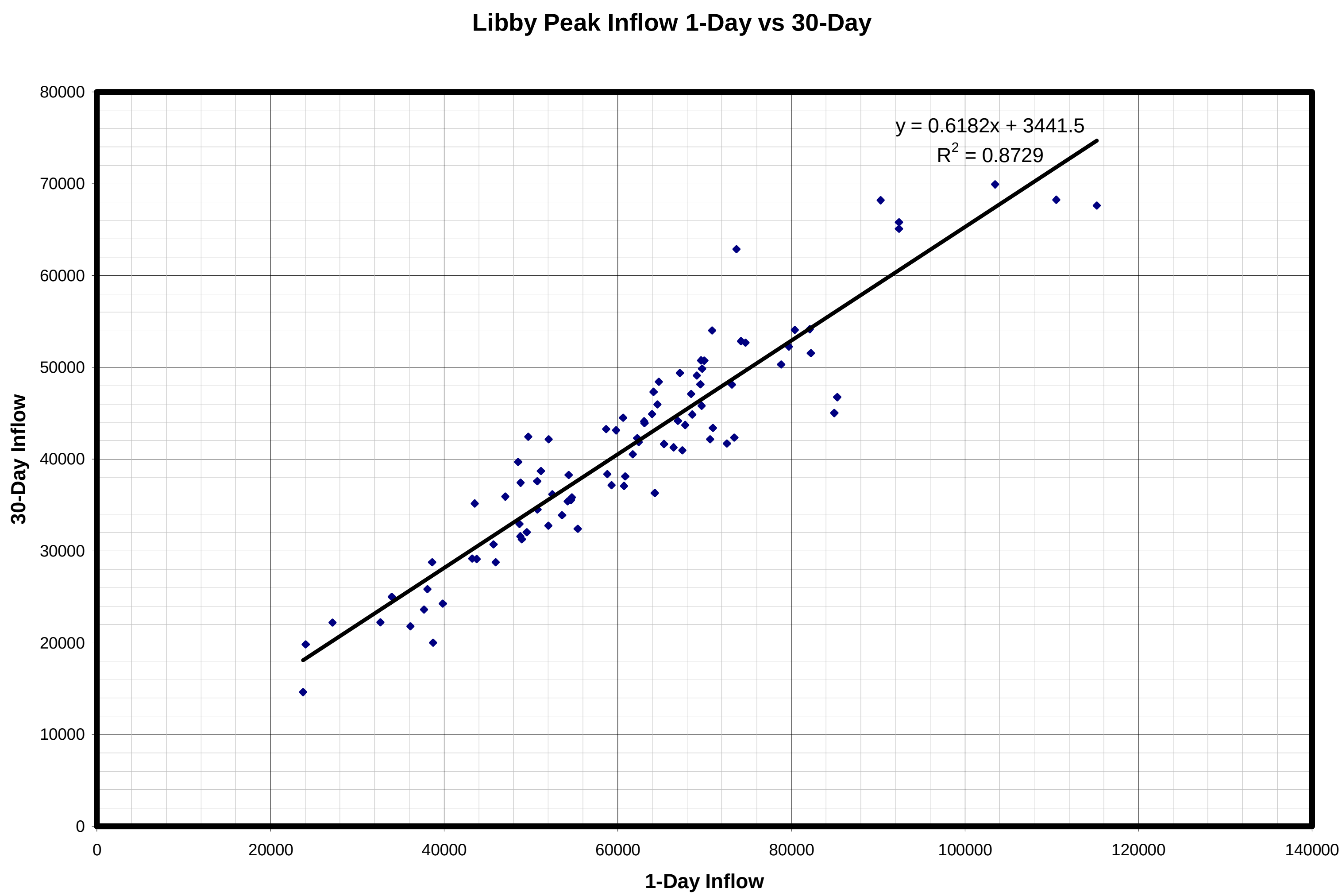

Figure 31. Libby Dam 1-Day Peak Inflow vs. 30-Day Peak Inflow

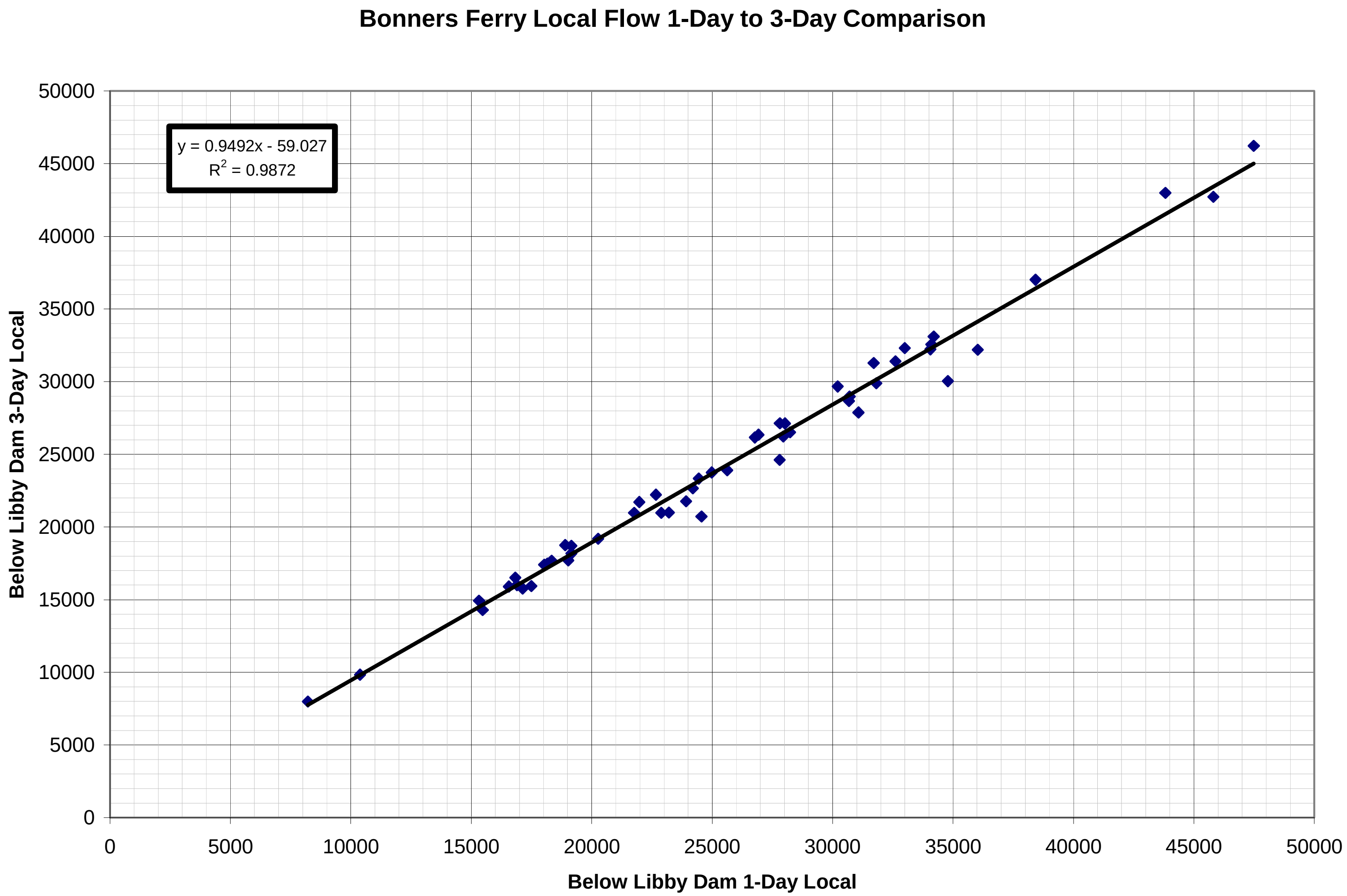

Figure 32. Below Libby Dam 1-Day Peak Local Inflow vs. 3-Day Peak Local Inflow

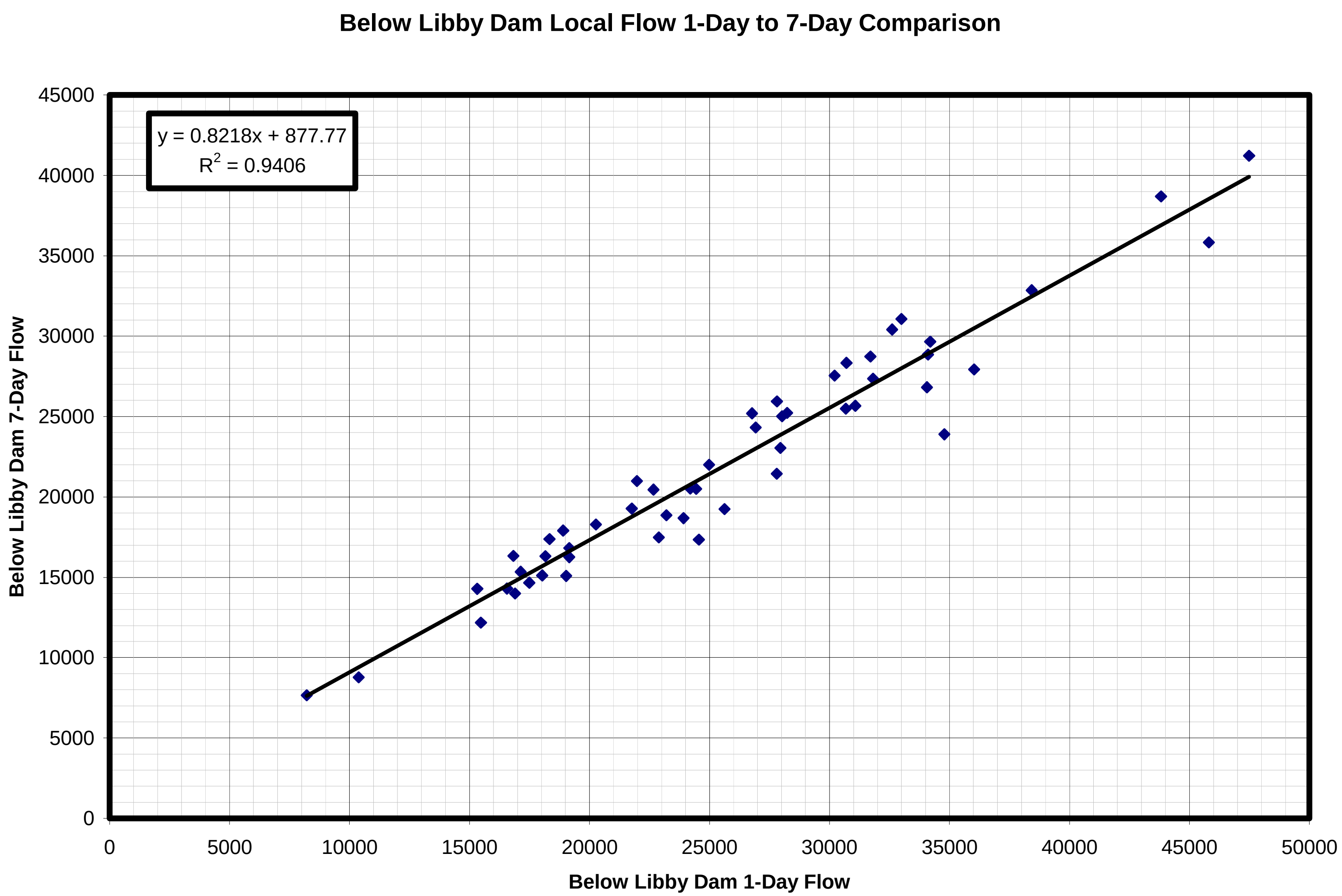

Figure 33. Below Libby Dam 1-Day Peak Local Inflow vs. 7-Day Peak Local Inflow

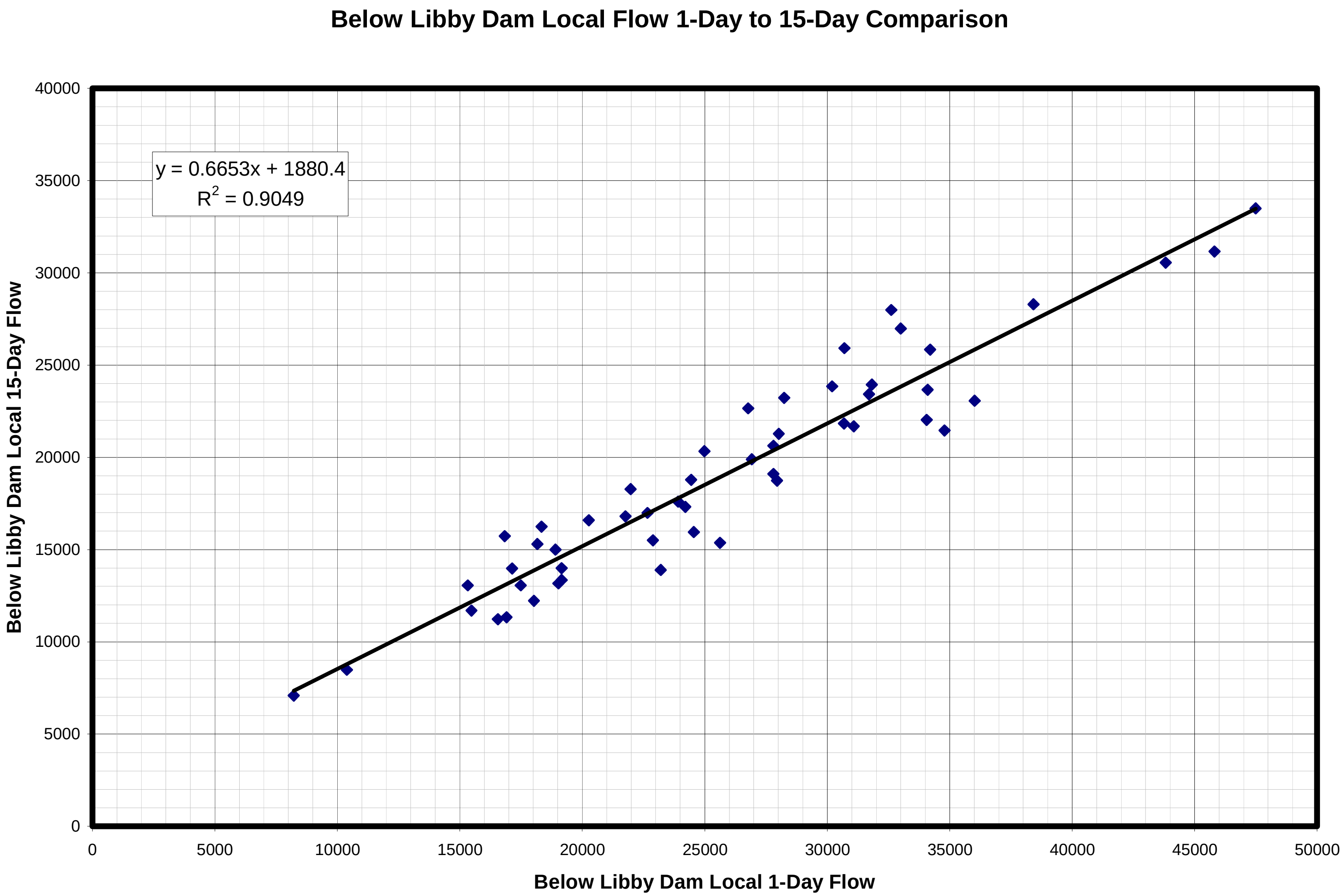

Figure 34. Below Libby Dam 1-Day Peak Local Inflow vs. 15-Day Peak Local Inflow

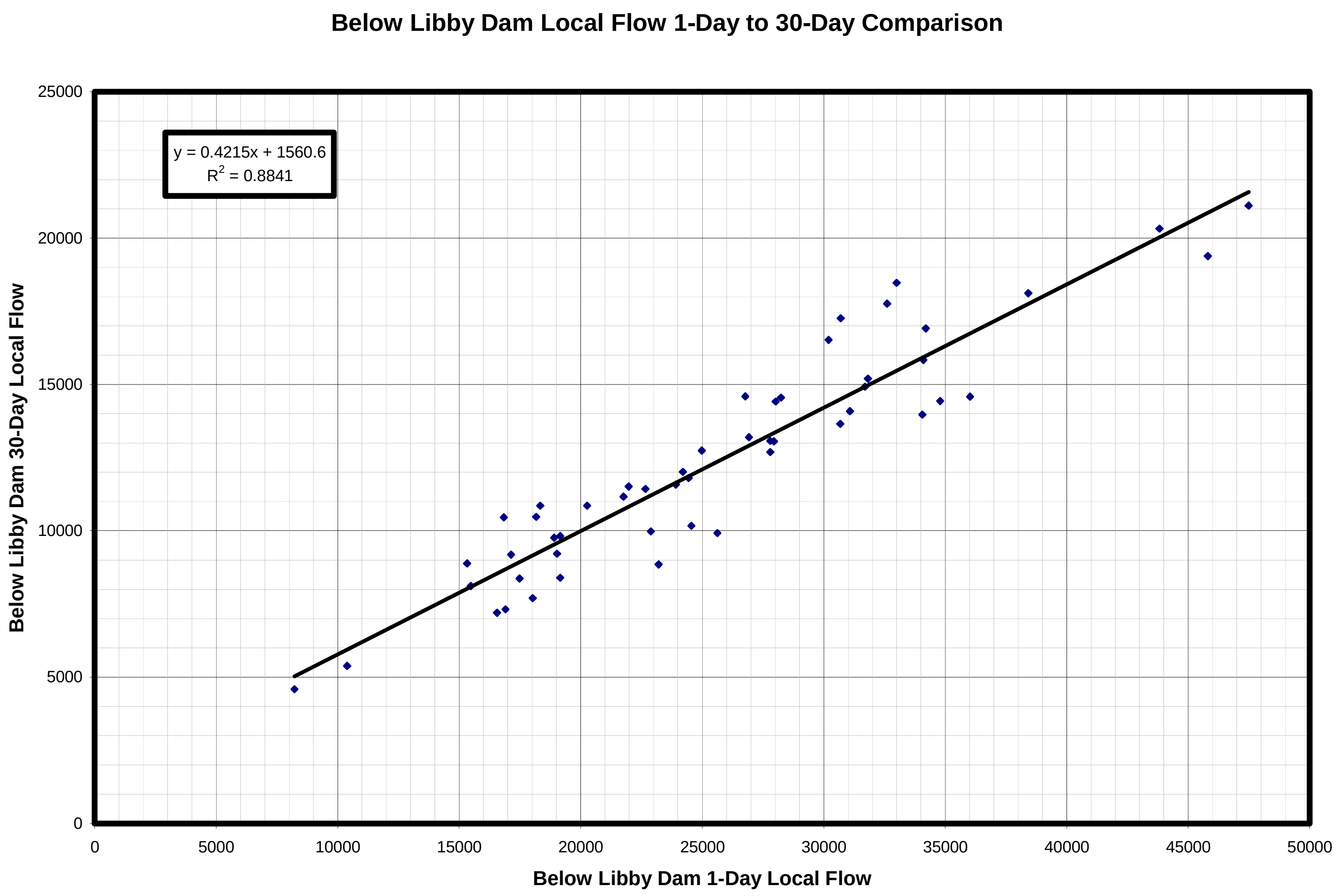

Figure 35. Below Libby Dam 1-Day Peak Local Inflow vs. 30-Day Peak Local Inflow

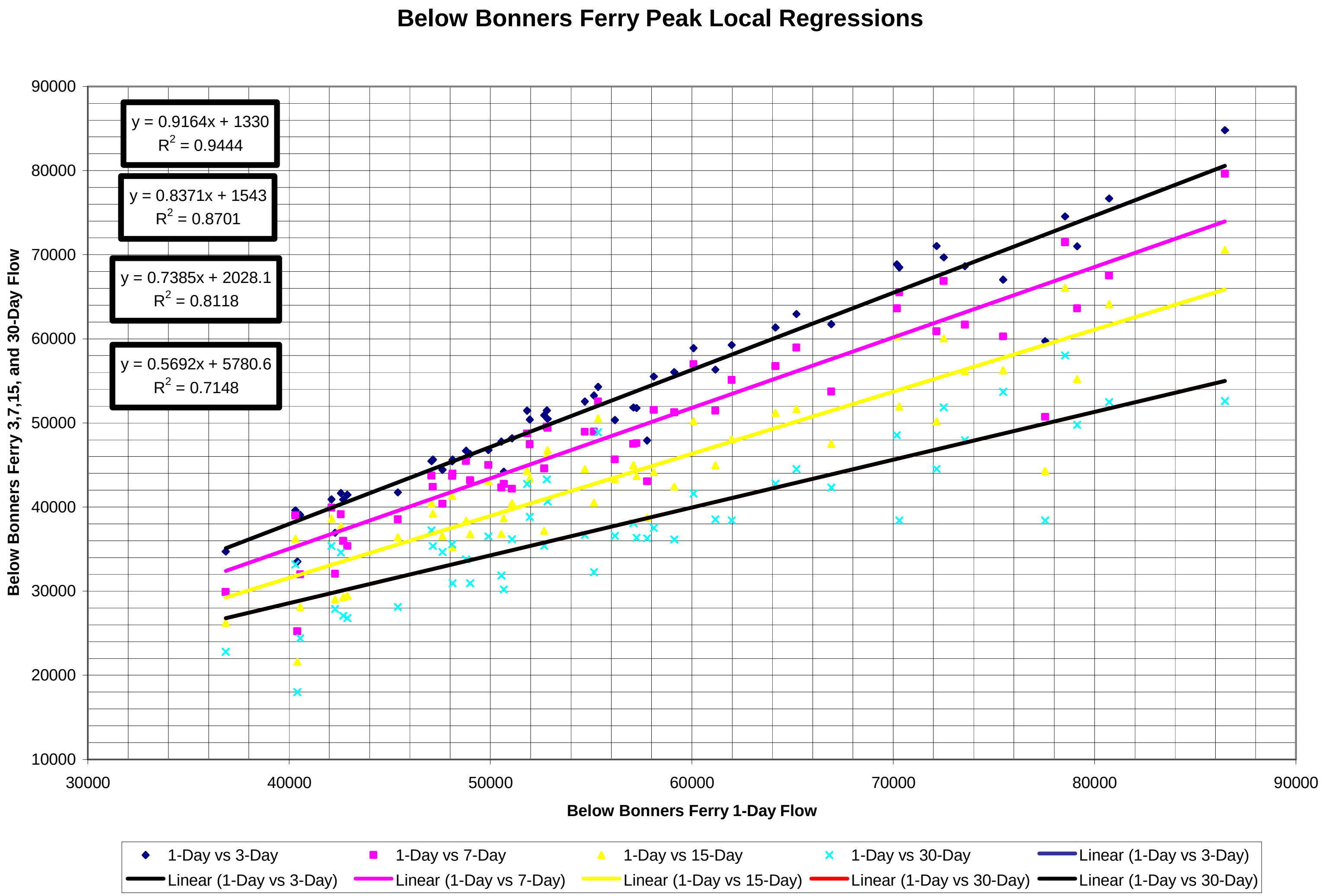

Figure 36. Below Bonners Ferry Peak 1-Day Local Flow vs. 3-, 7-, 15- and 30-Day Peak Local Flow

7.2 APPENDIX B – FREQUENCY CURVES

Figure 37 APPENDIX B

[See following pages]

Frequency curves were developed only to facilitate this study and are not intended to replace any existing frequency curves currently used by local, state, federal, tribal, or private entities.

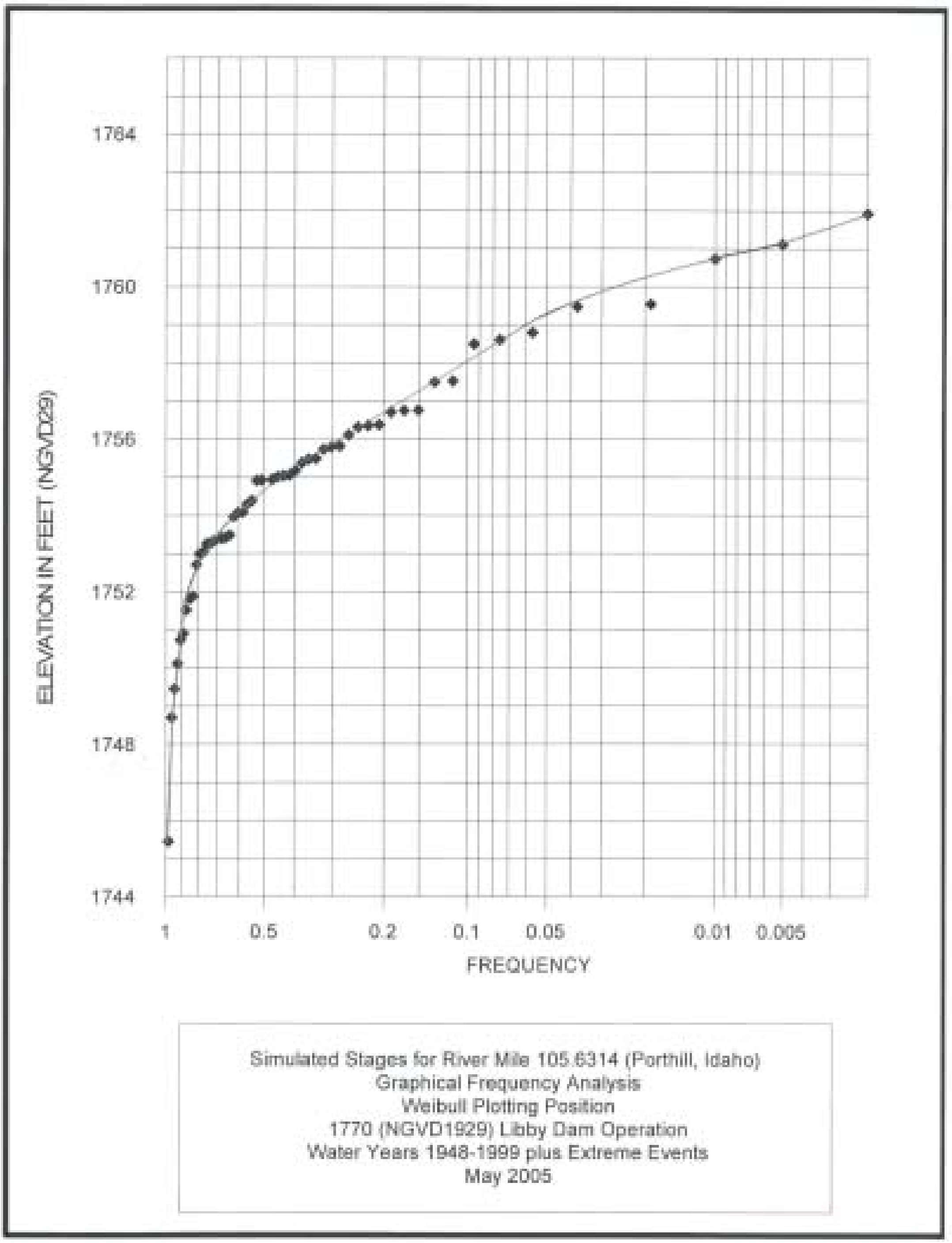

Figure 38. Frequency Curve 1

Frequency curves were developed only to facilitate this study and are not intended to replace any existing frequency curves currently used by local, state, federal, tribal, or private entities.

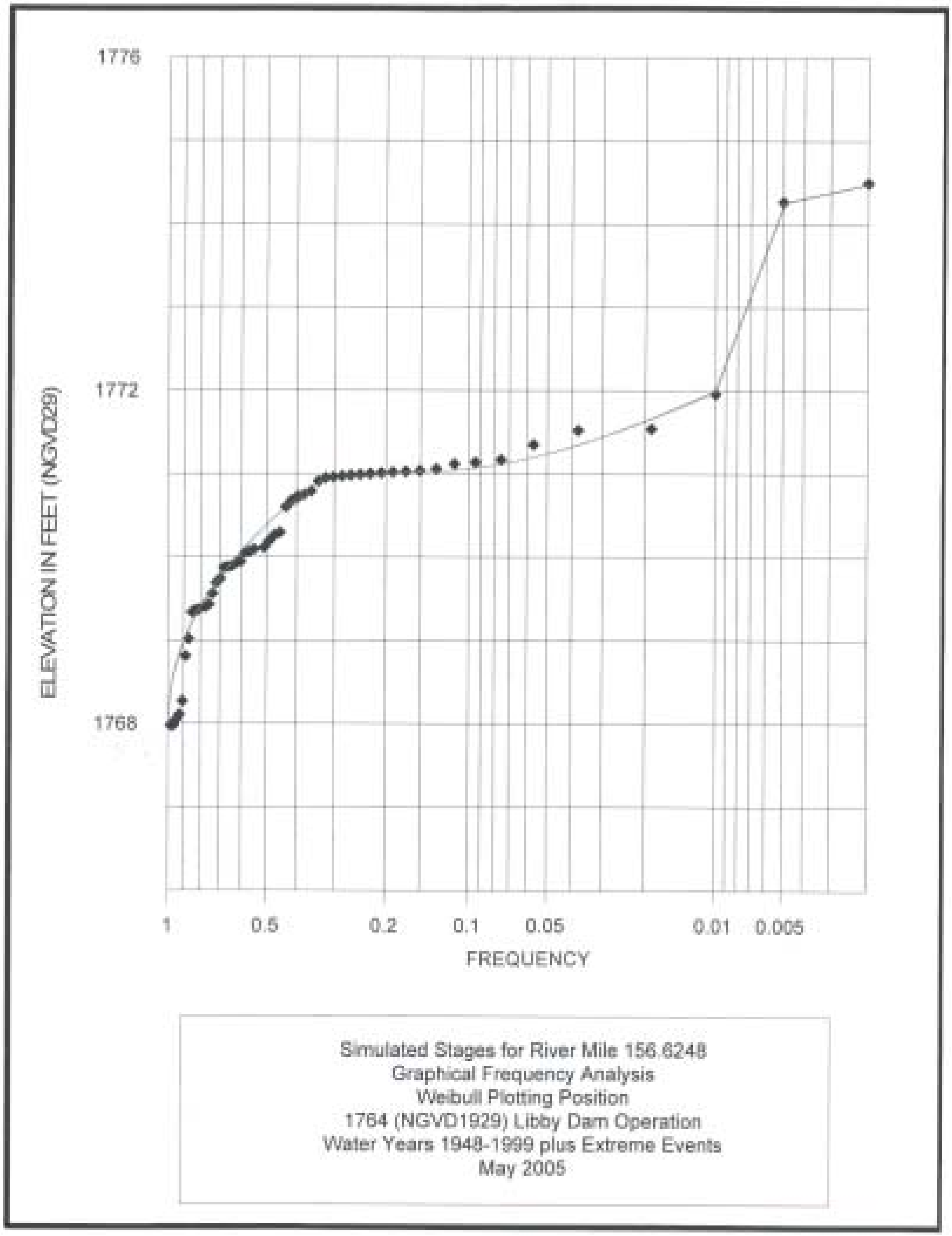

Figure 39. Frequency Curve 2

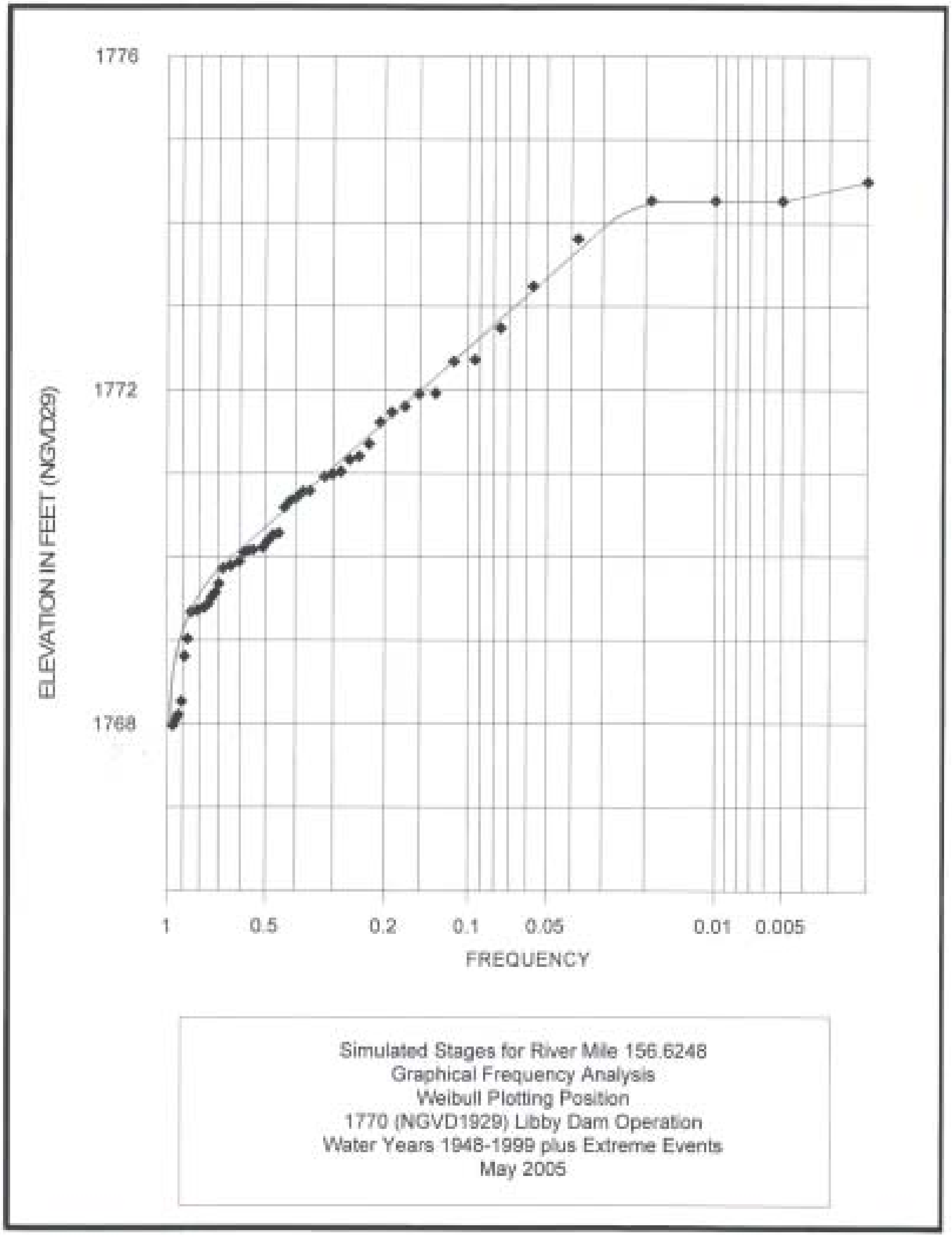

Figure 40. Frequency Curve 3

Frequency curves were developed only to facilitate this study and are not intended to replace any existing frequency curves currently used by local, state, federal, tribal, or private entities.

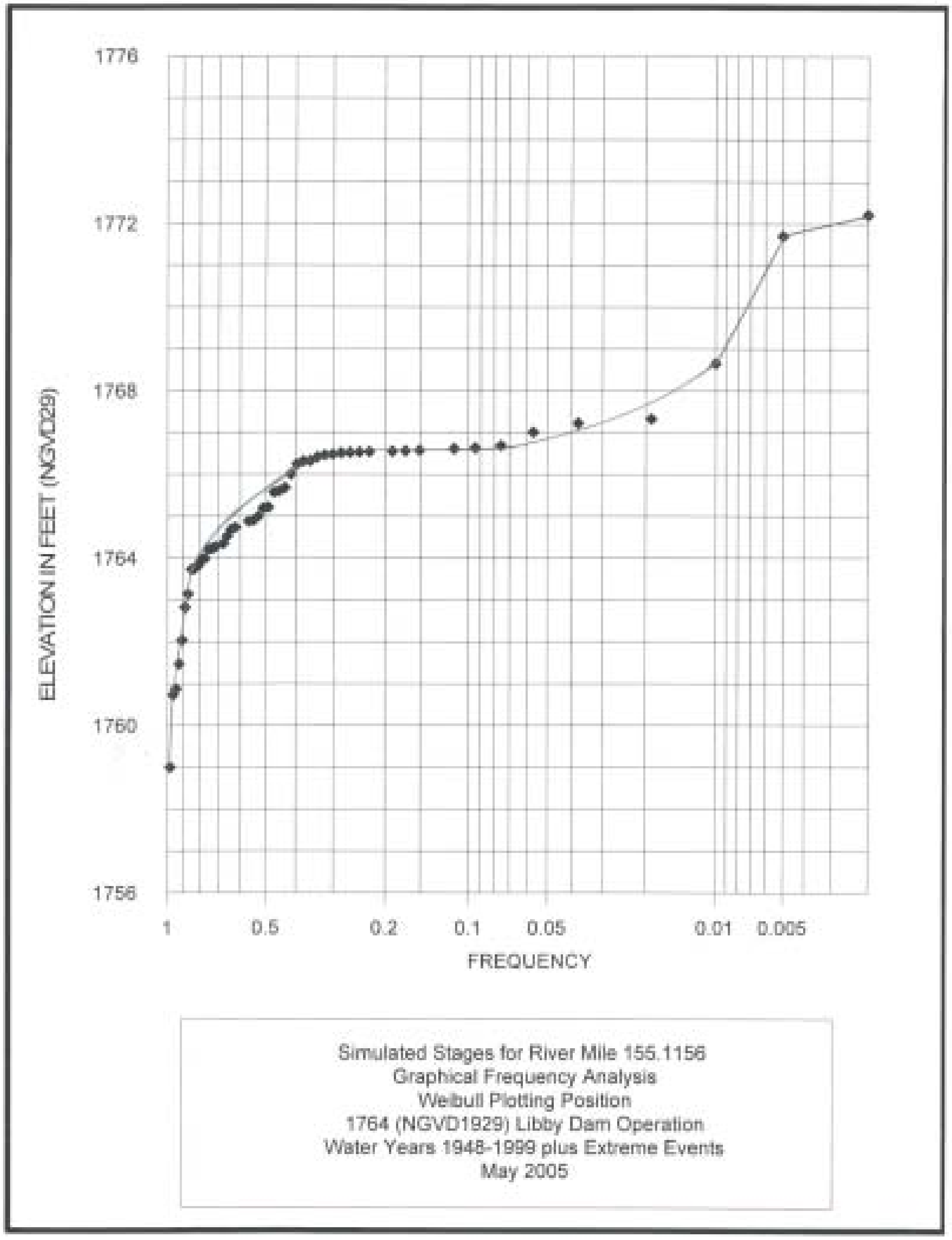

Figure 41. Frequency Curve 4

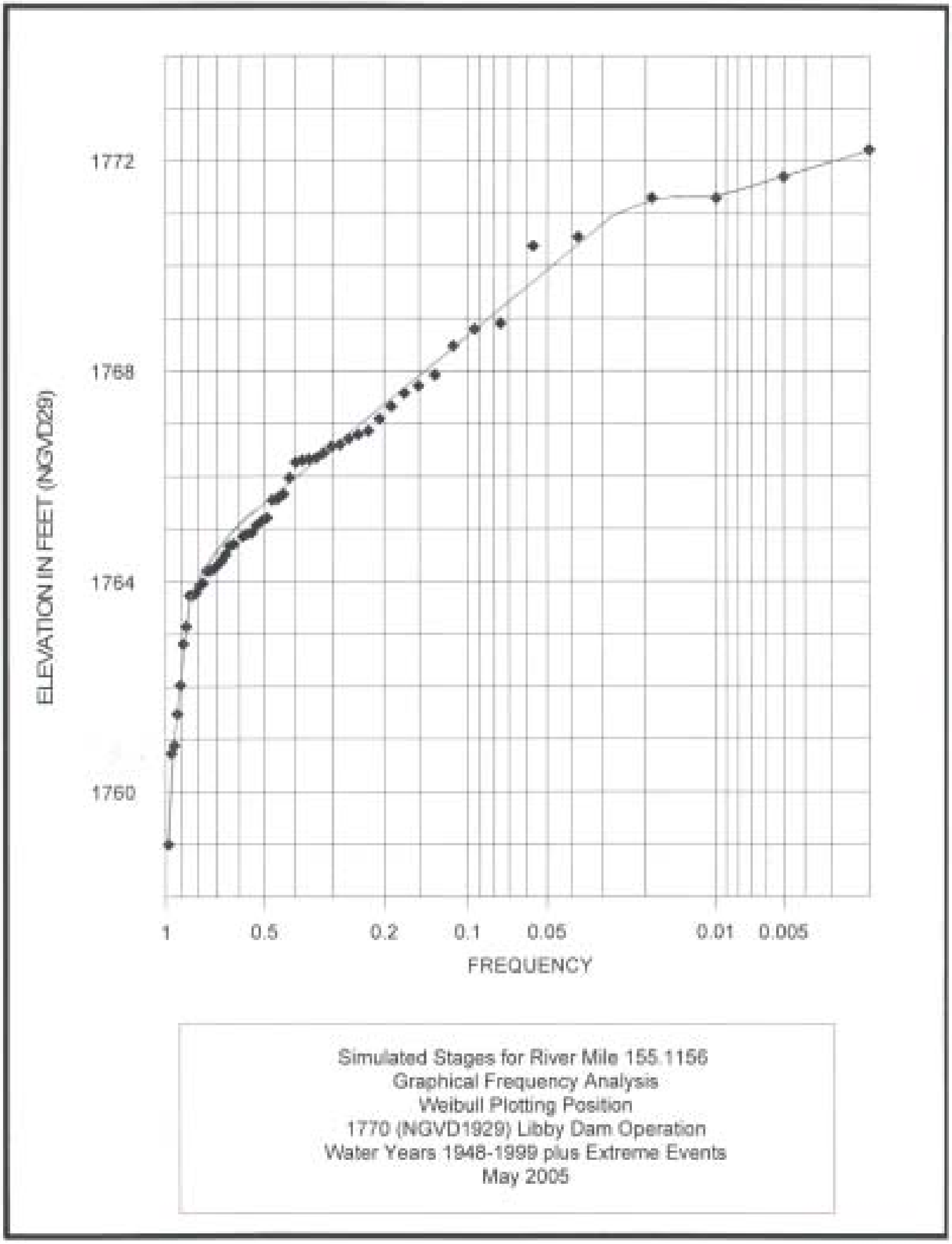

Figure 42. Frequency Curve 5

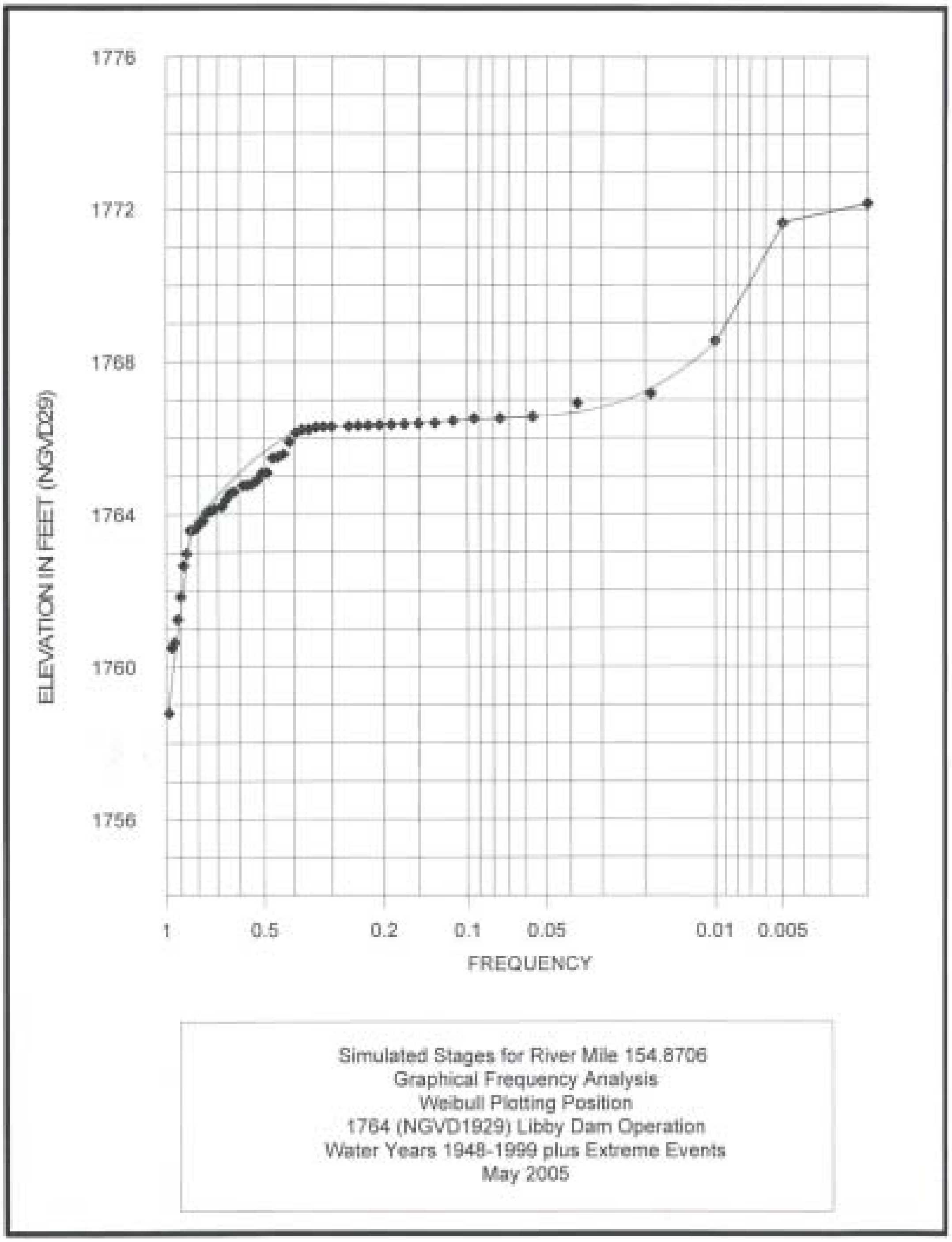

Figure 43. Frequency Curve 6

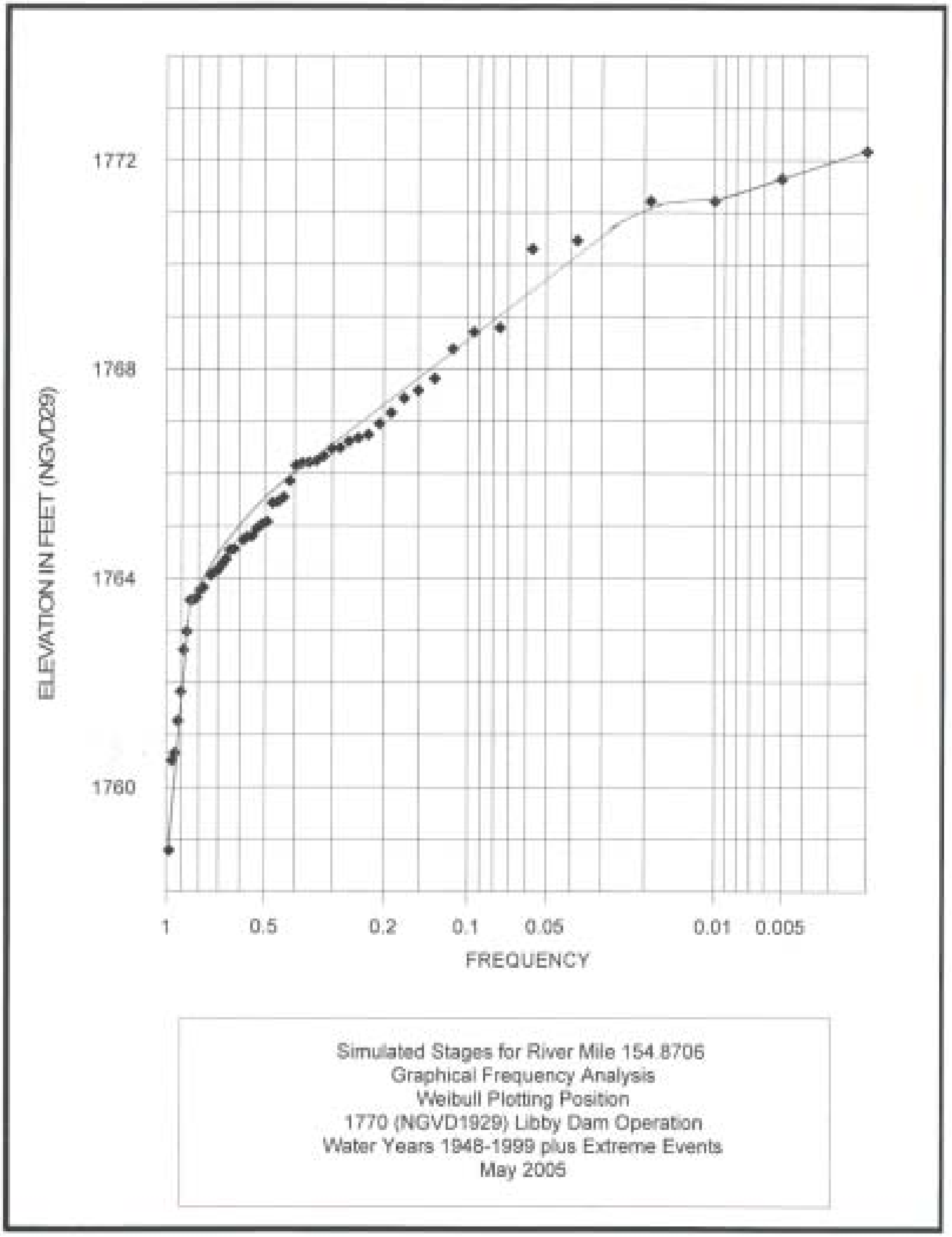

Figure 44. Frequency Curve 7

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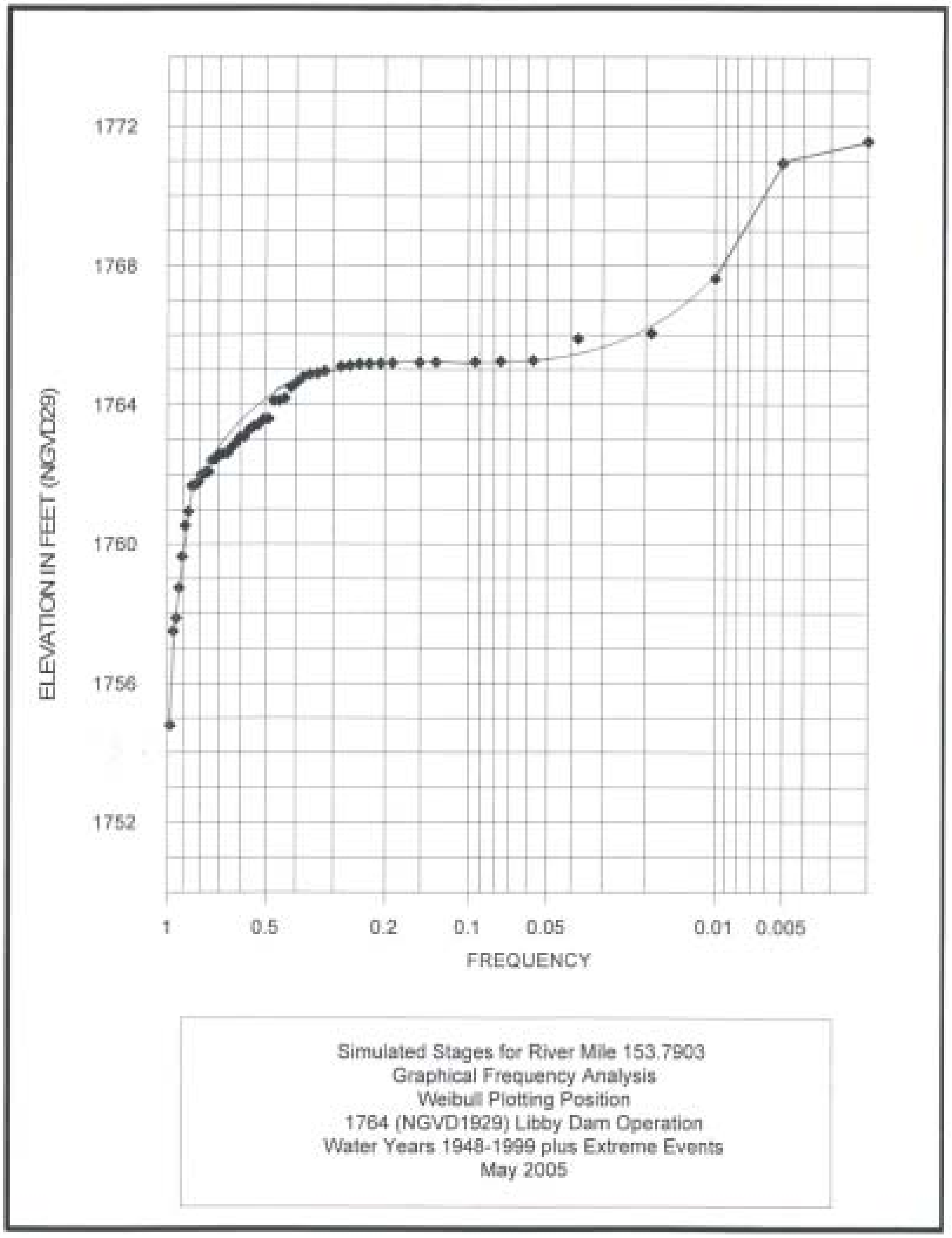


Figure 45. Frequency Curve 8

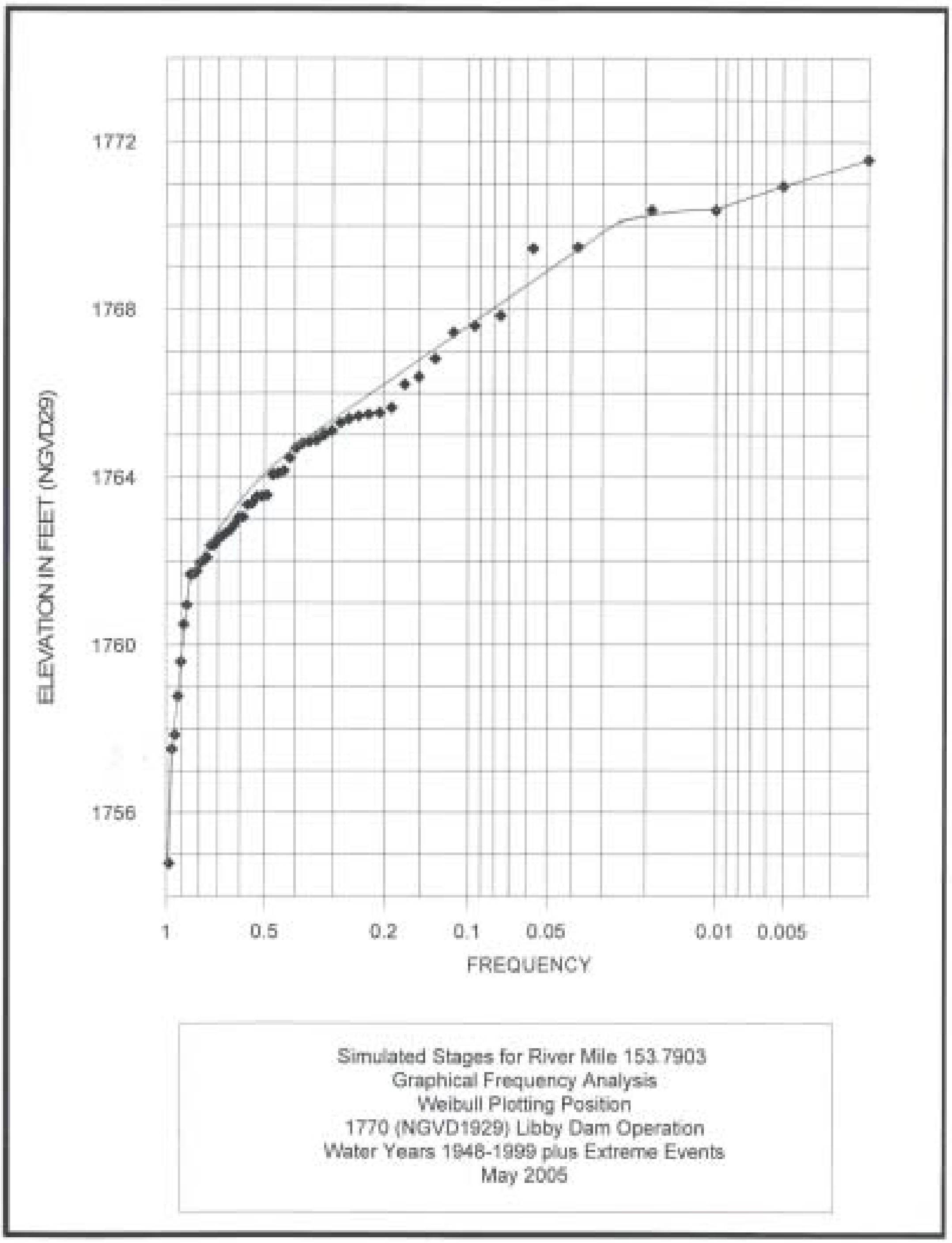


Figure 46. Frequency Curve 9

Frequency curves were developed only to facilitate this study and are not intended to replace any existing frequency curves currently used by local, state, federal, tribal, or private entities.

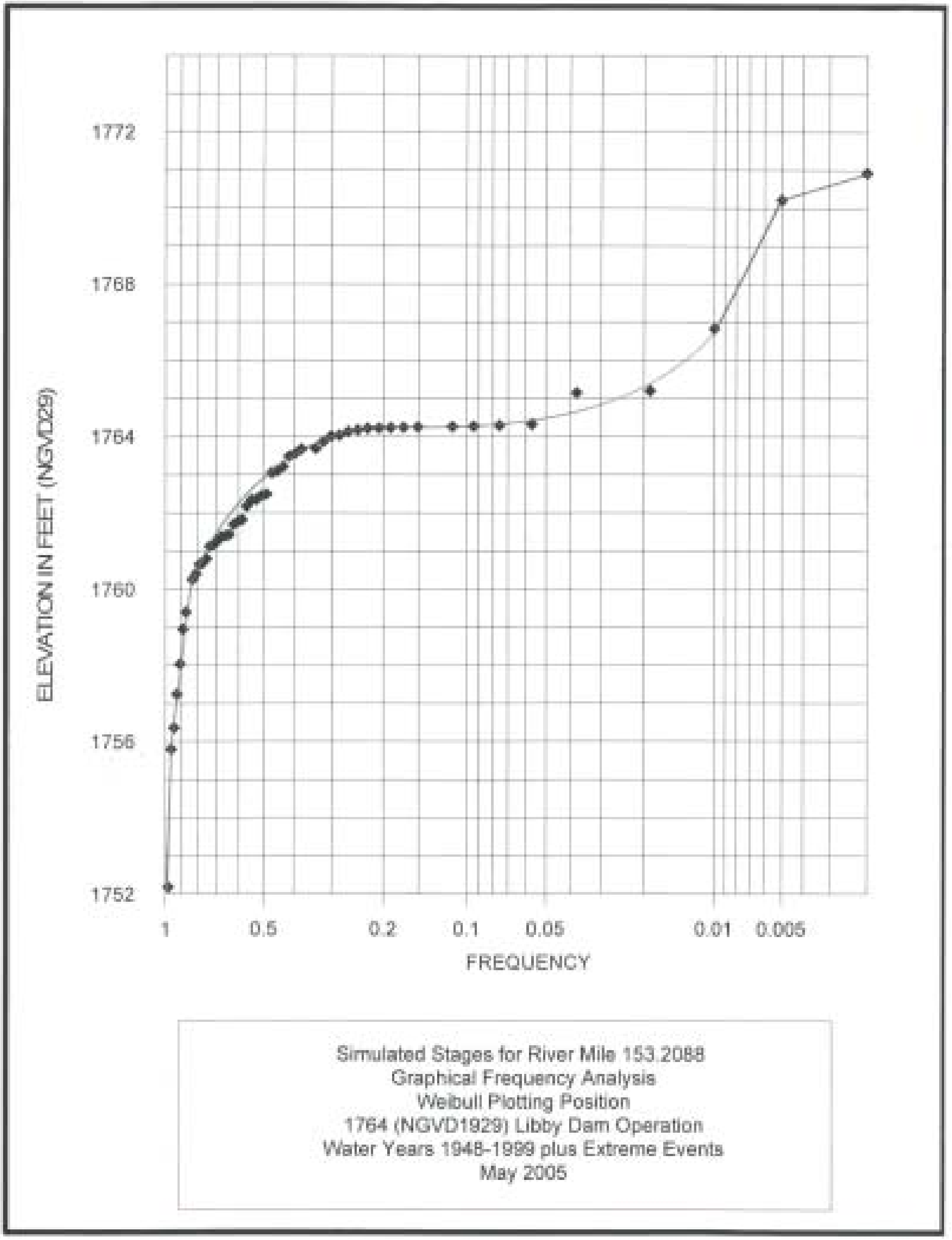


Figure 47. Frequency Curve 10

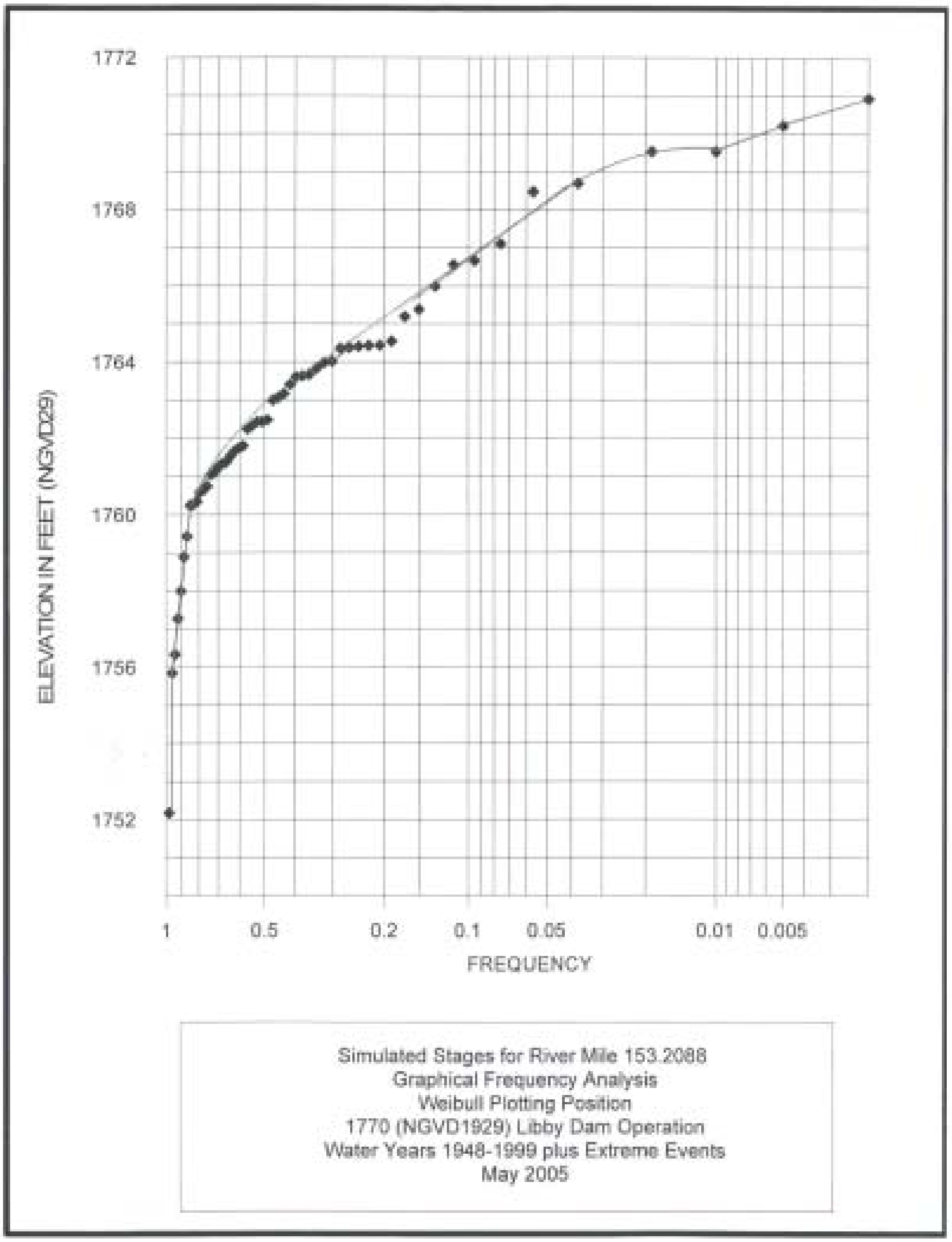


Figure 48. Frequency Curve 11

Frequency curves were developed only to facilitate this study and are not intended to replace any existing frequency curves currently used by local, state, federal, tribal, or private entities.

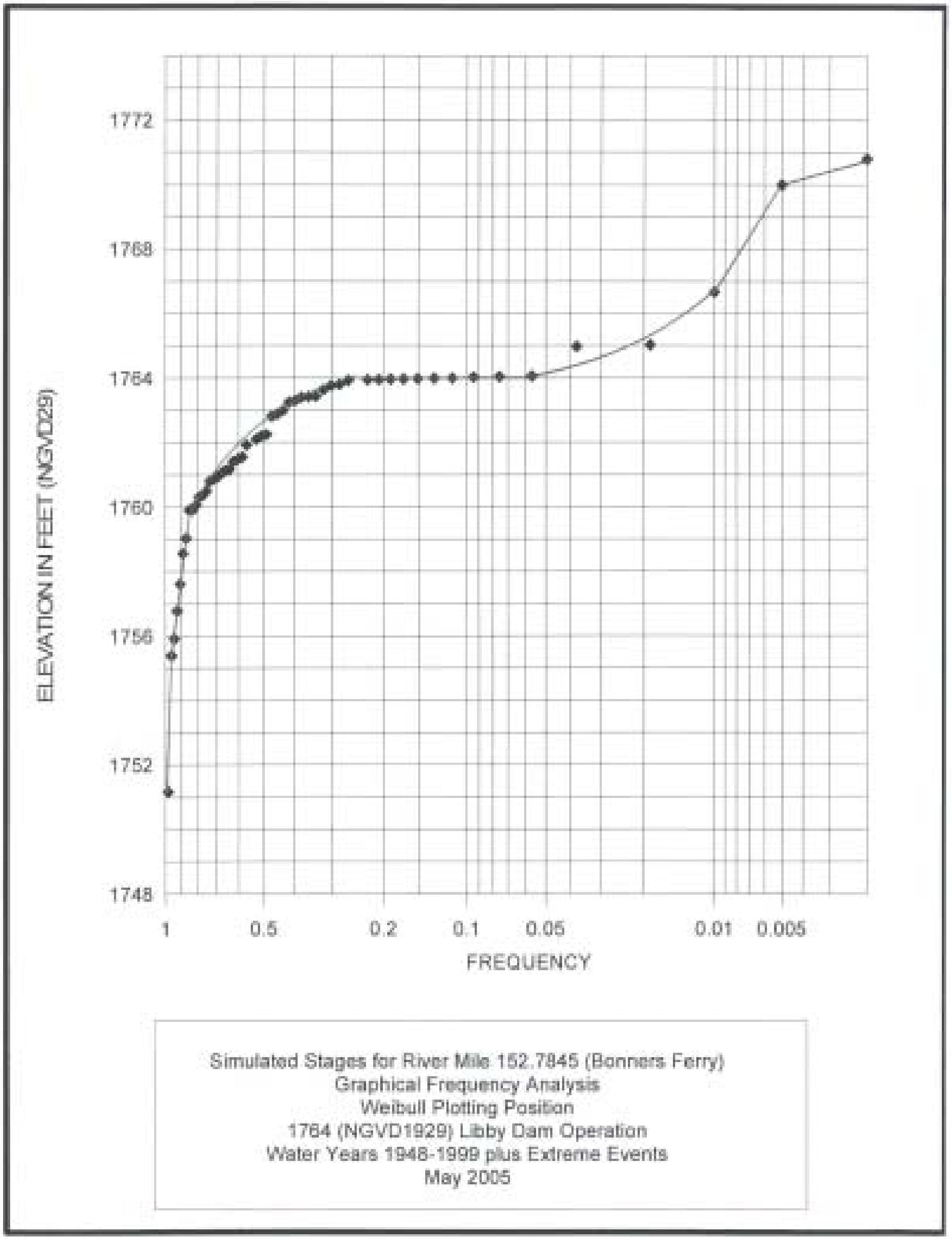
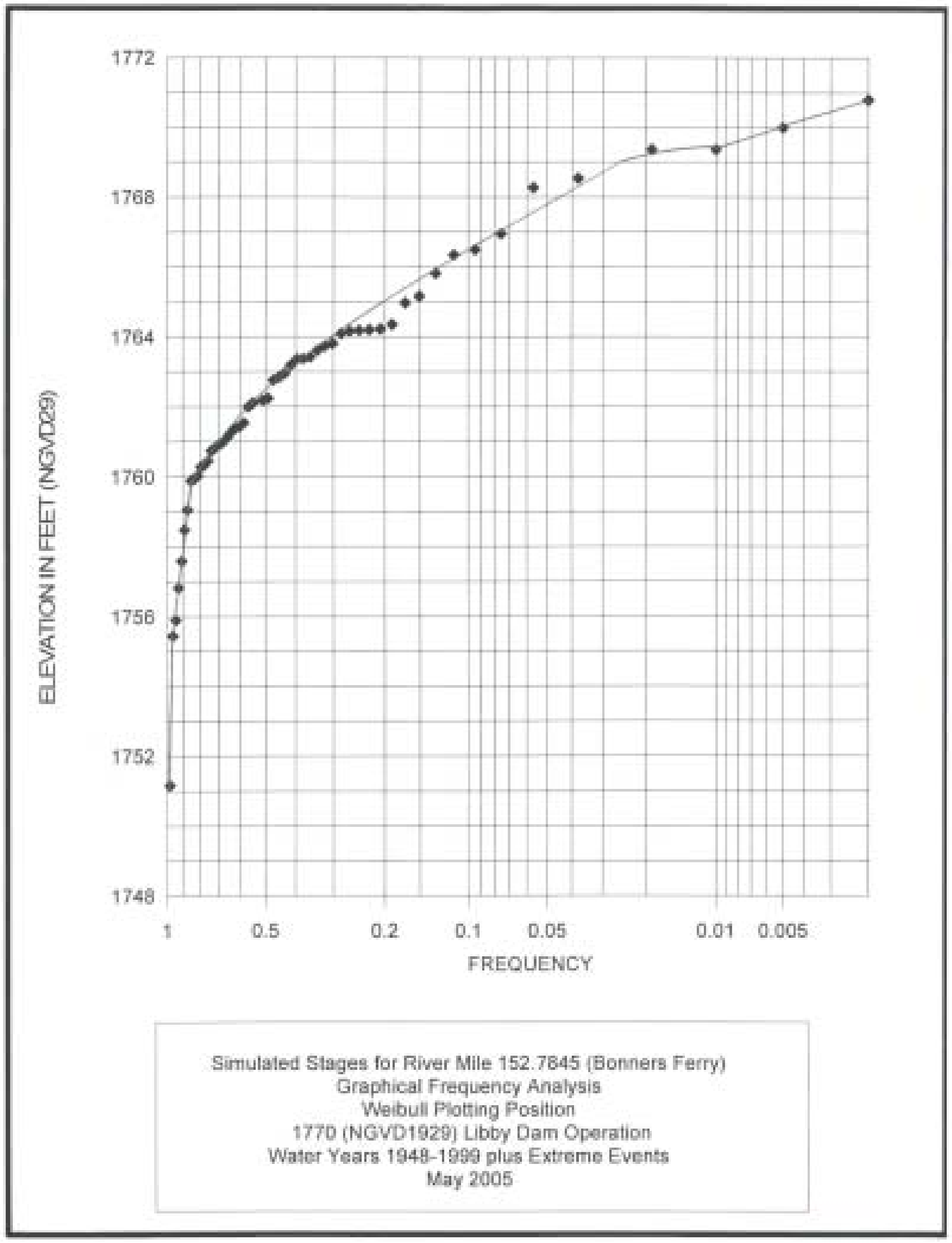


Figure 49. Frequency Curve 12

Frequency curves were developed only to facilitate this study and are not intended to replace any existing frequency curves currently used by local, state, federal, tribal, or private entities.



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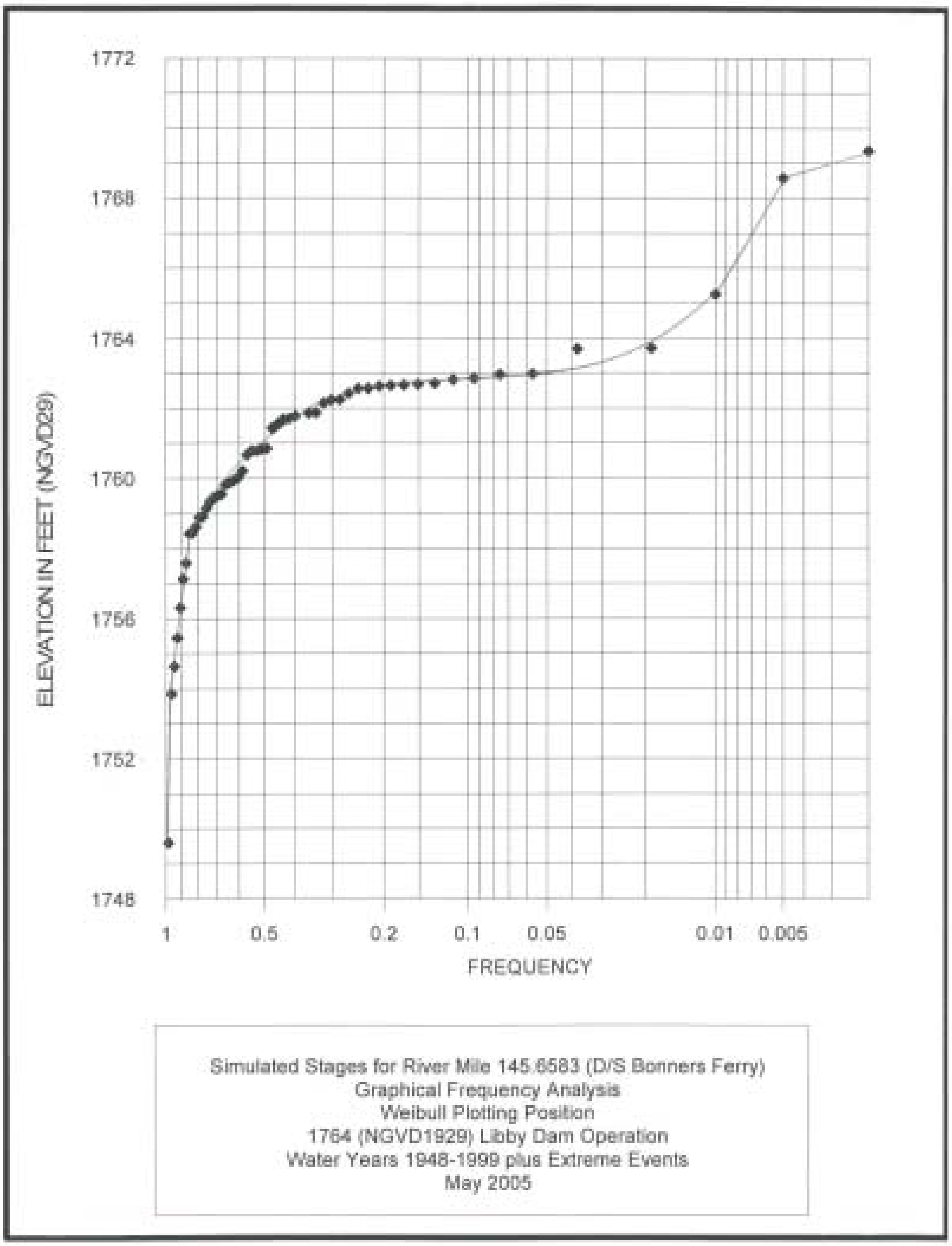
Frequency curves were developed only to facilitate this study and are not intended to replace any existing frequency curves currently used by local, state, federal, tribal, or private entities.



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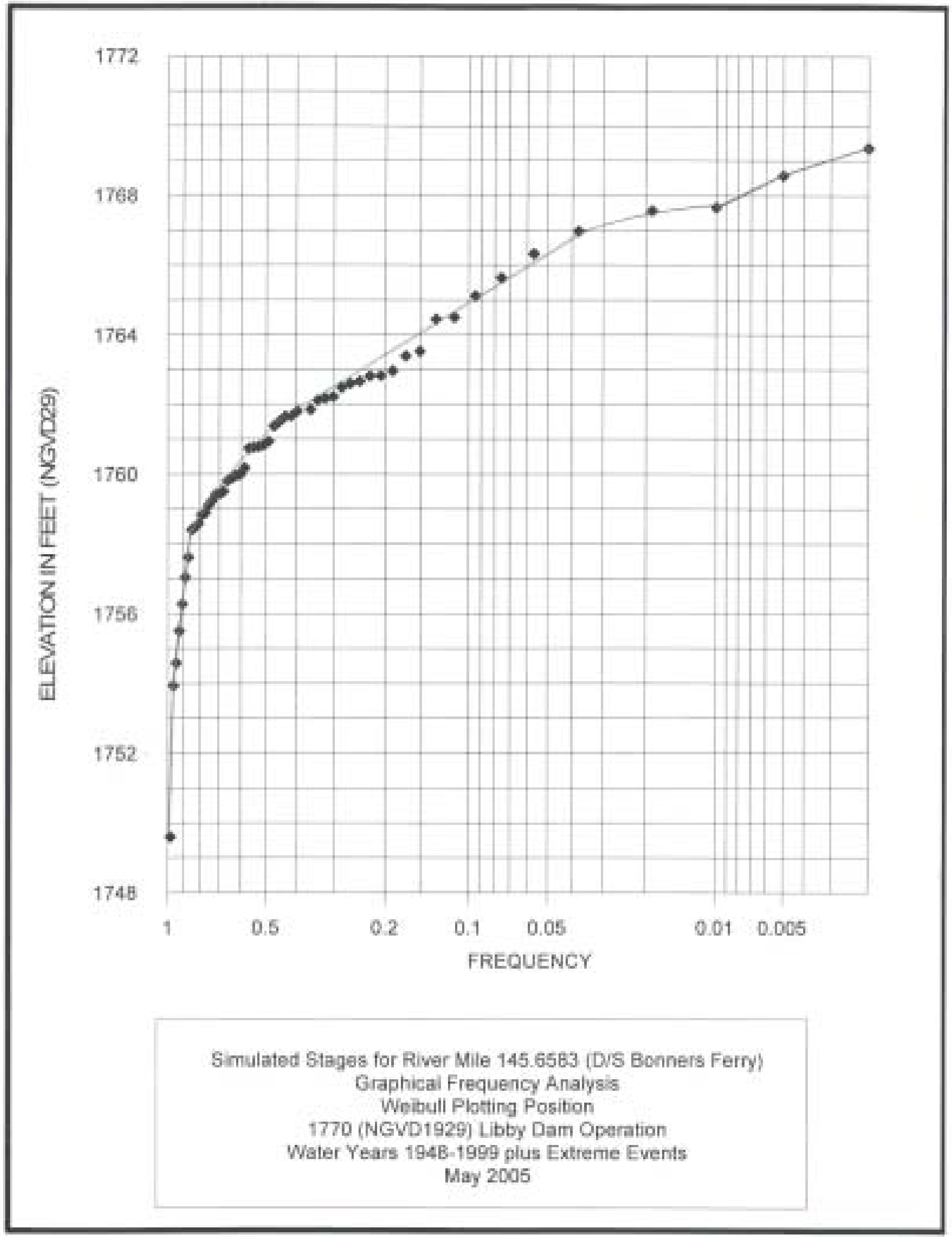

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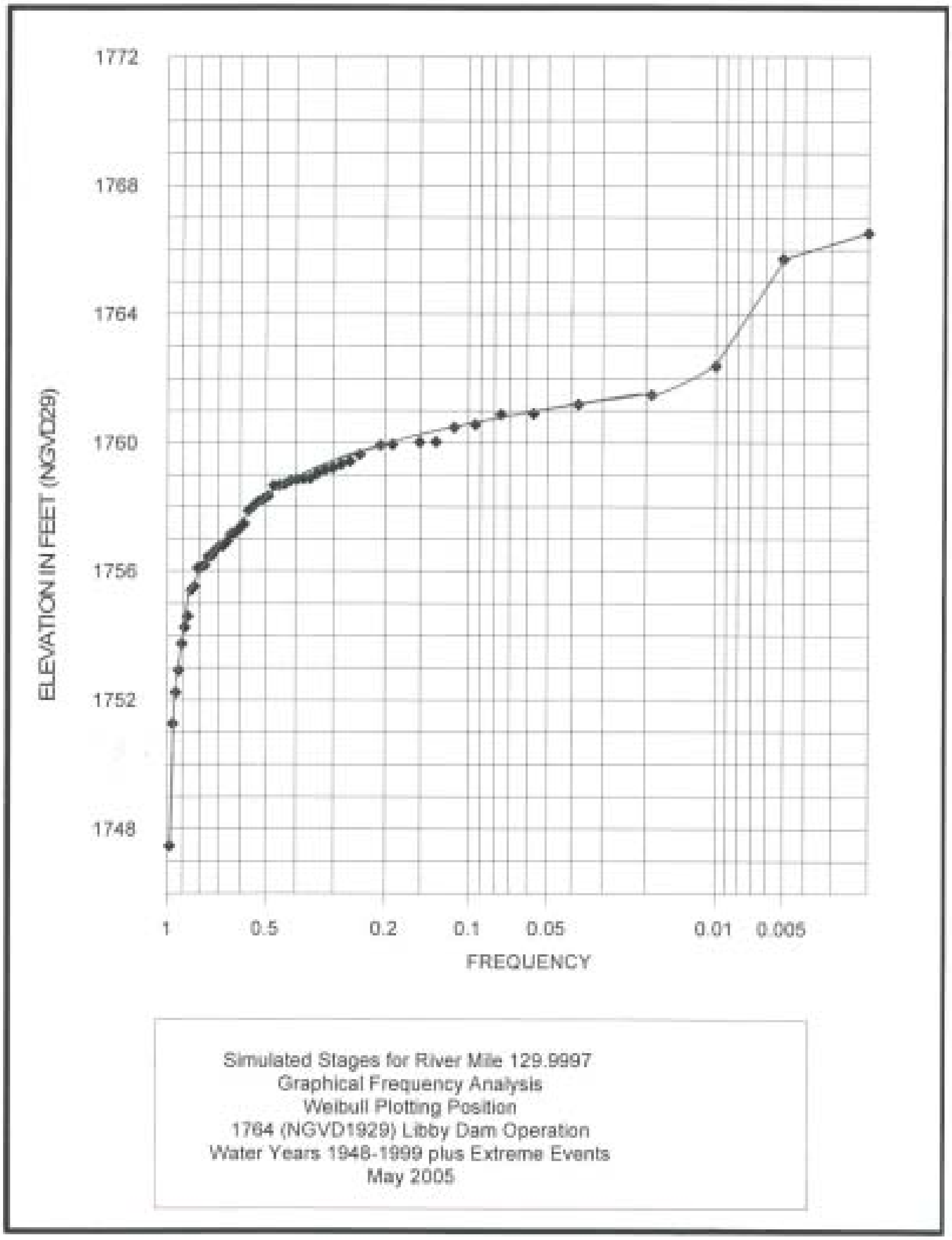

Figure 53. Frequency Curve 16

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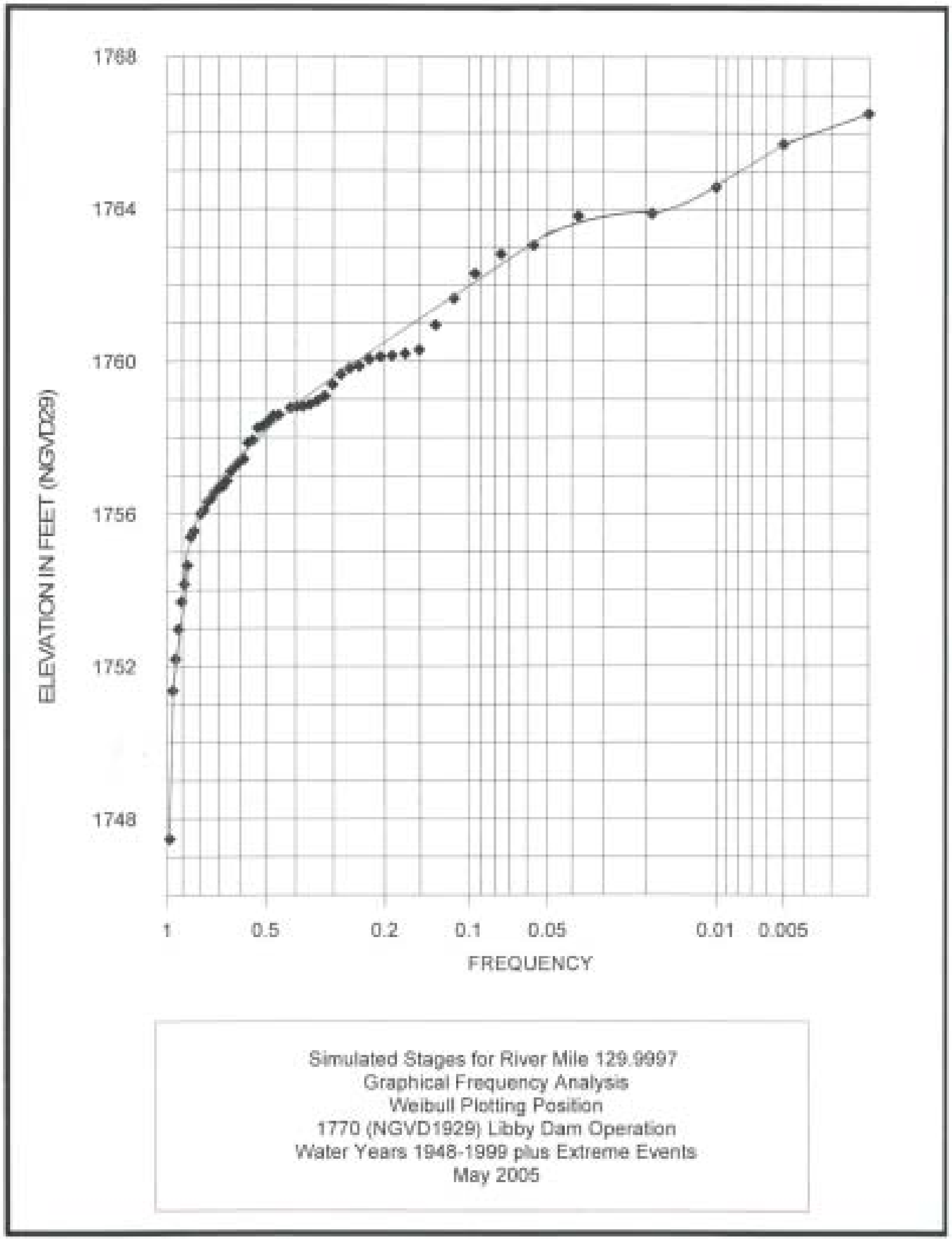

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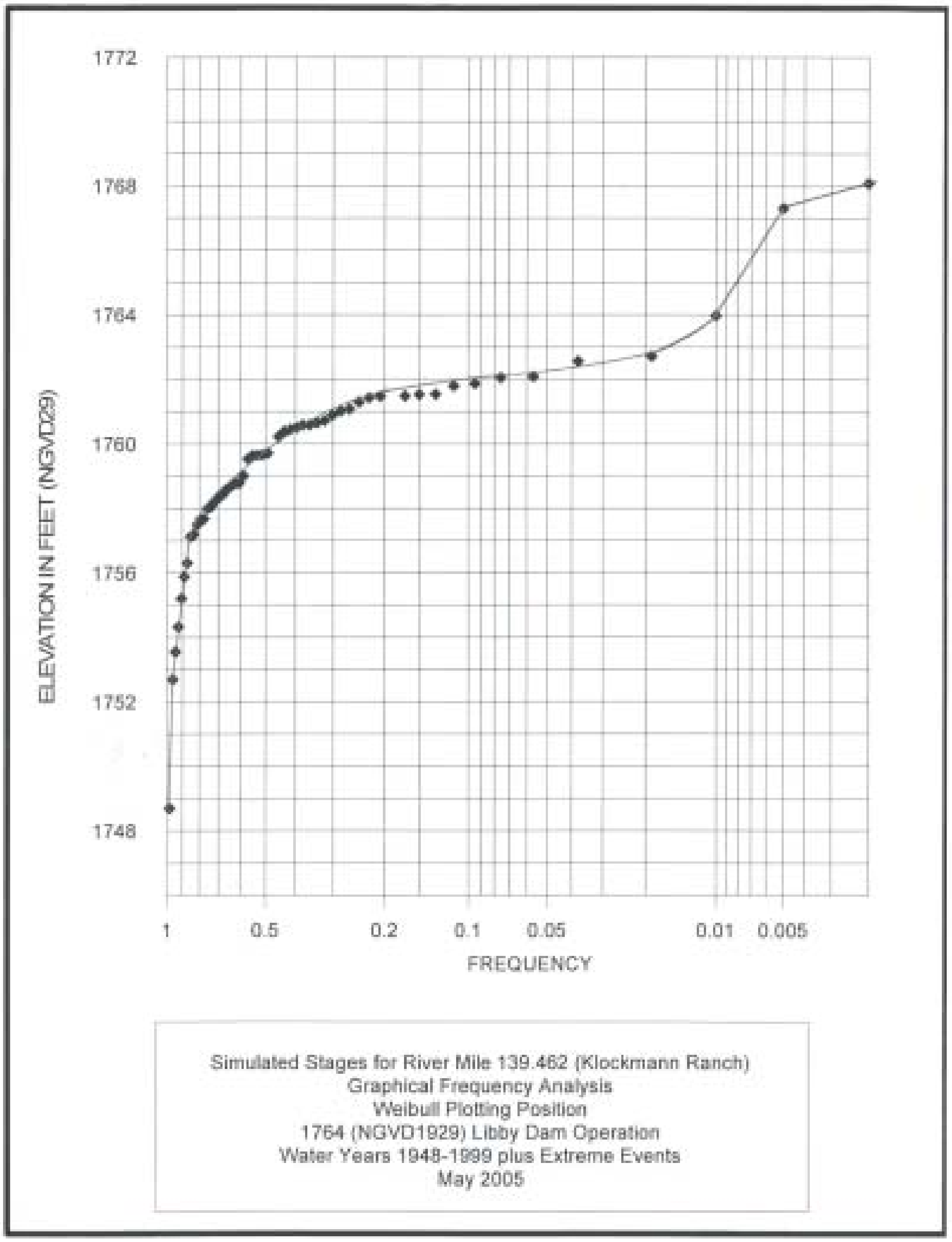

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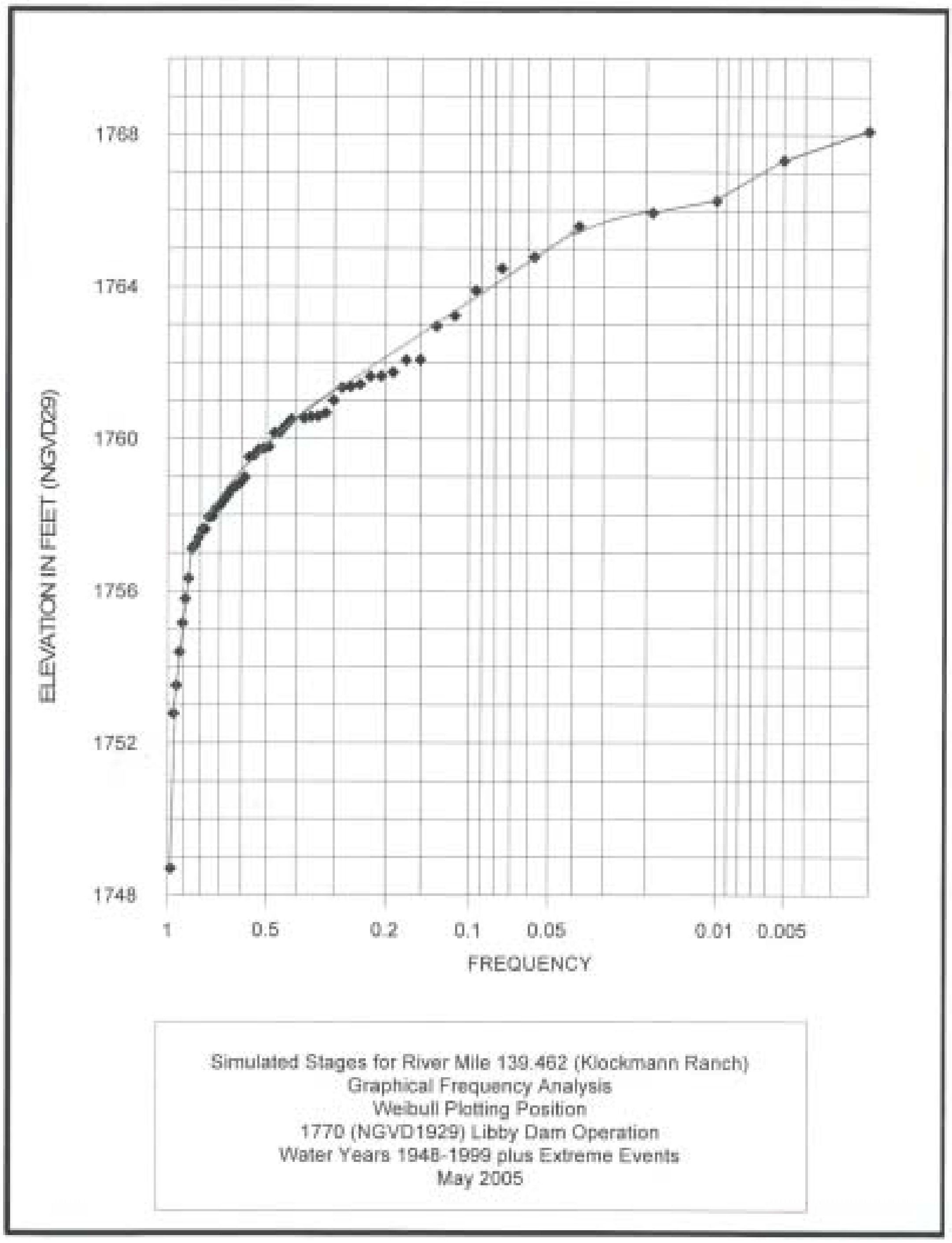

Figure 56. Frequency Curve 19

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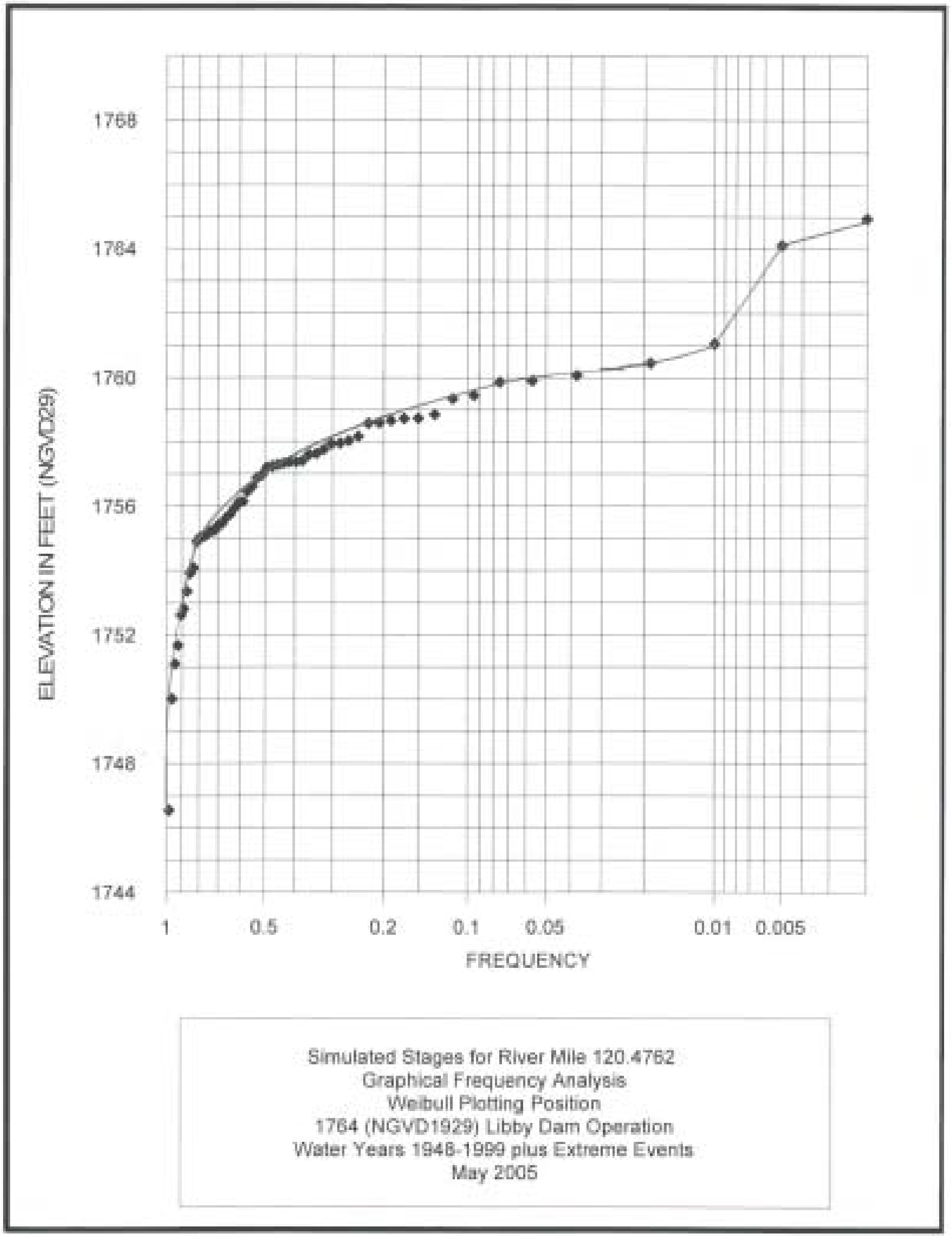

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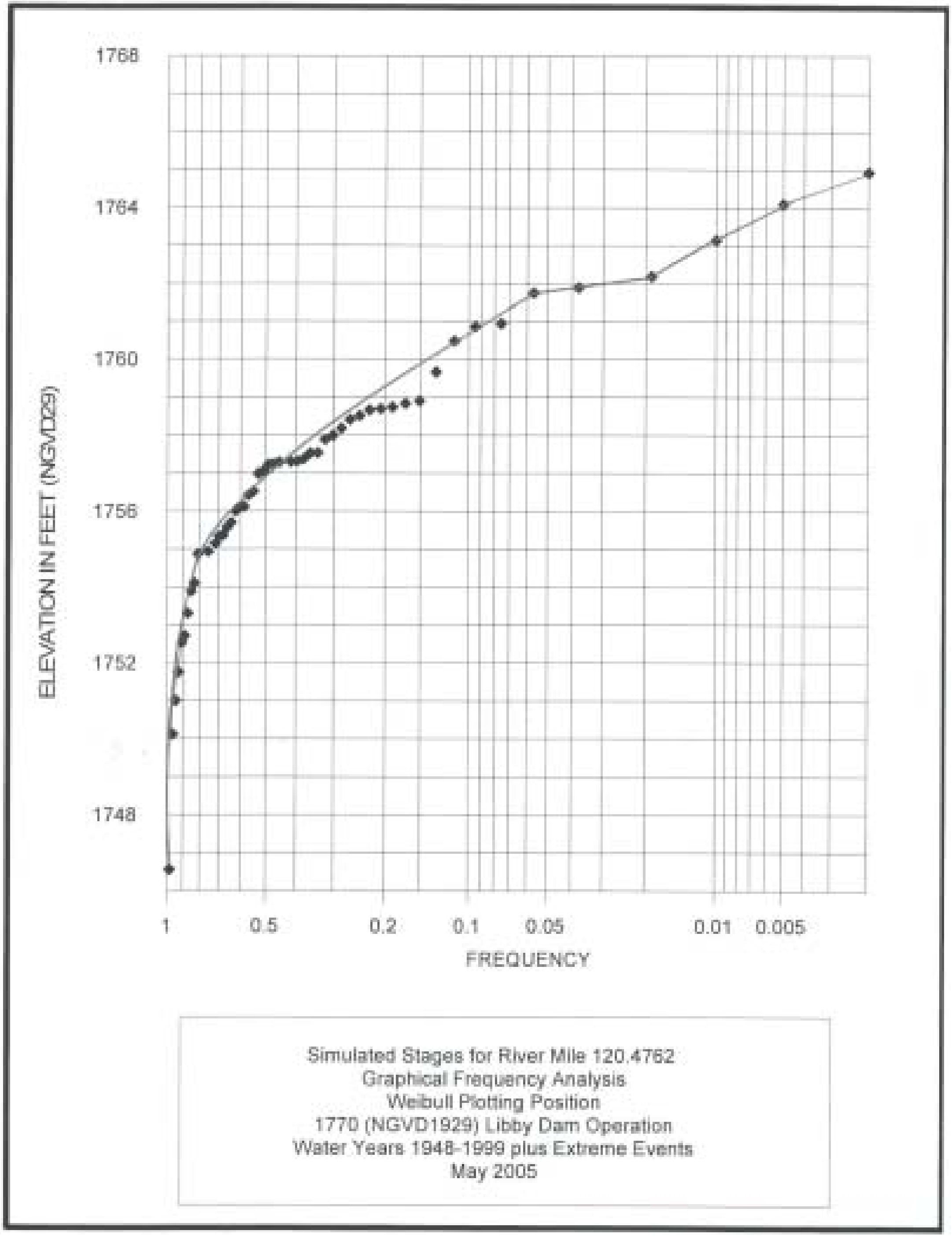

Figure 58. Frequency Curve 21

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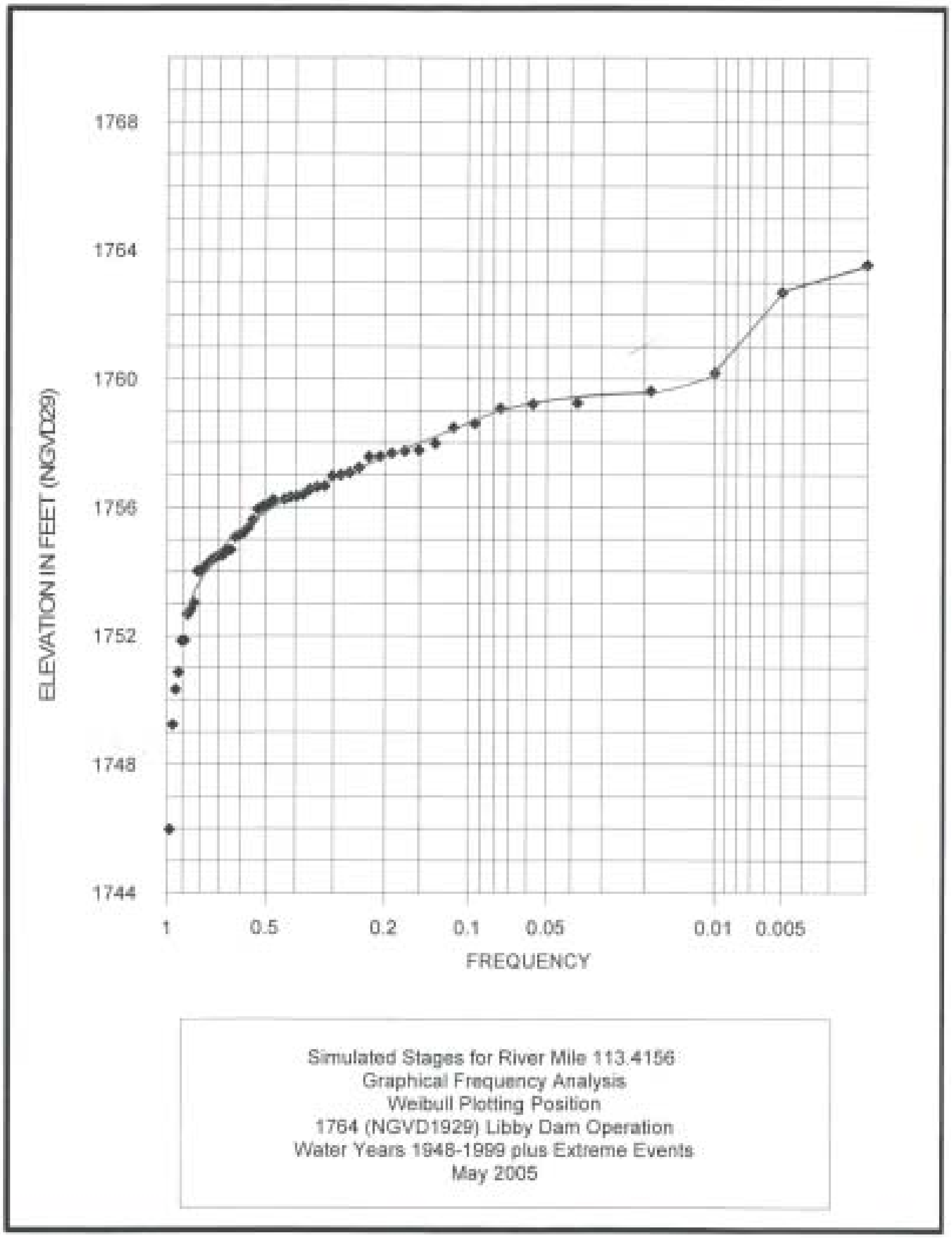

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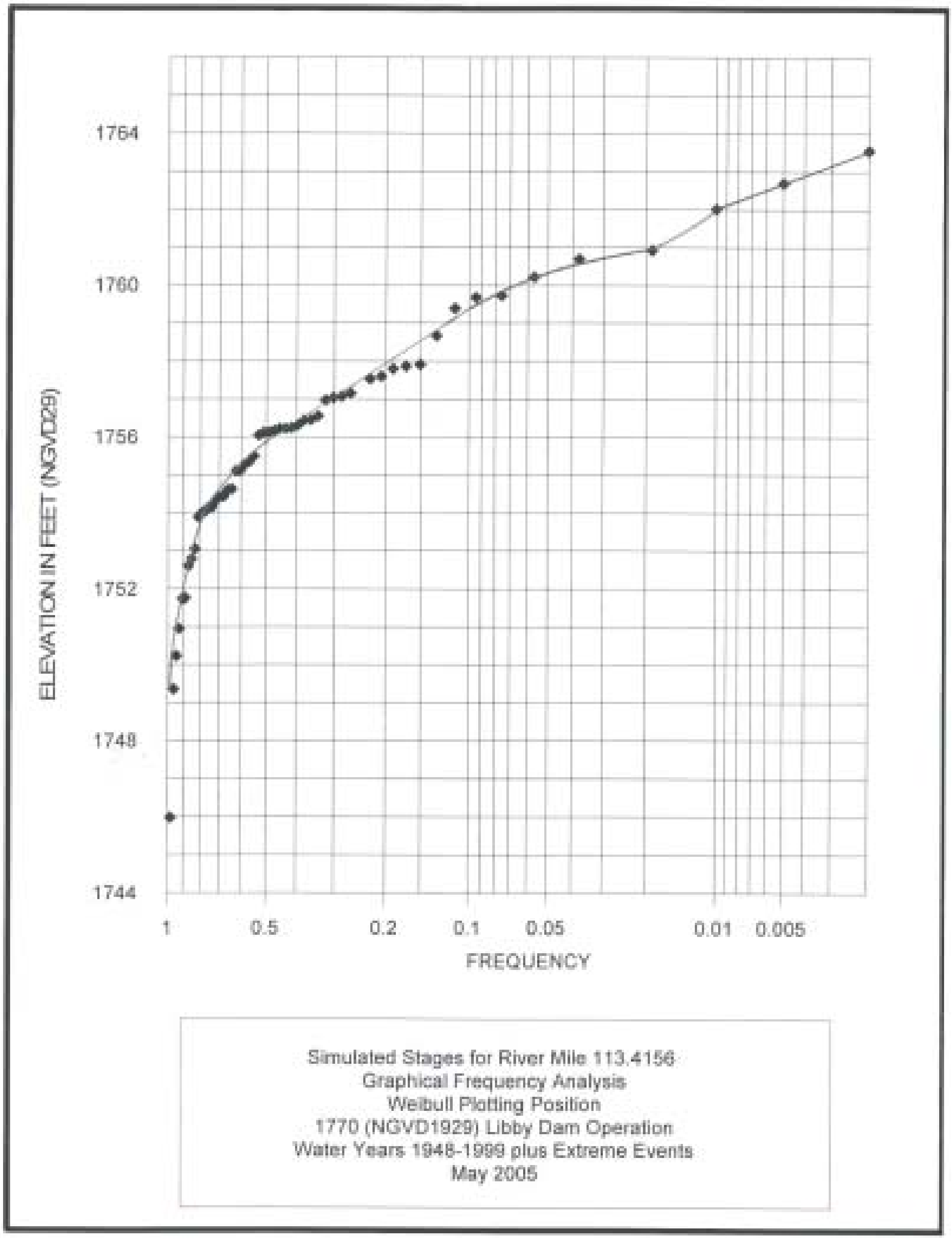

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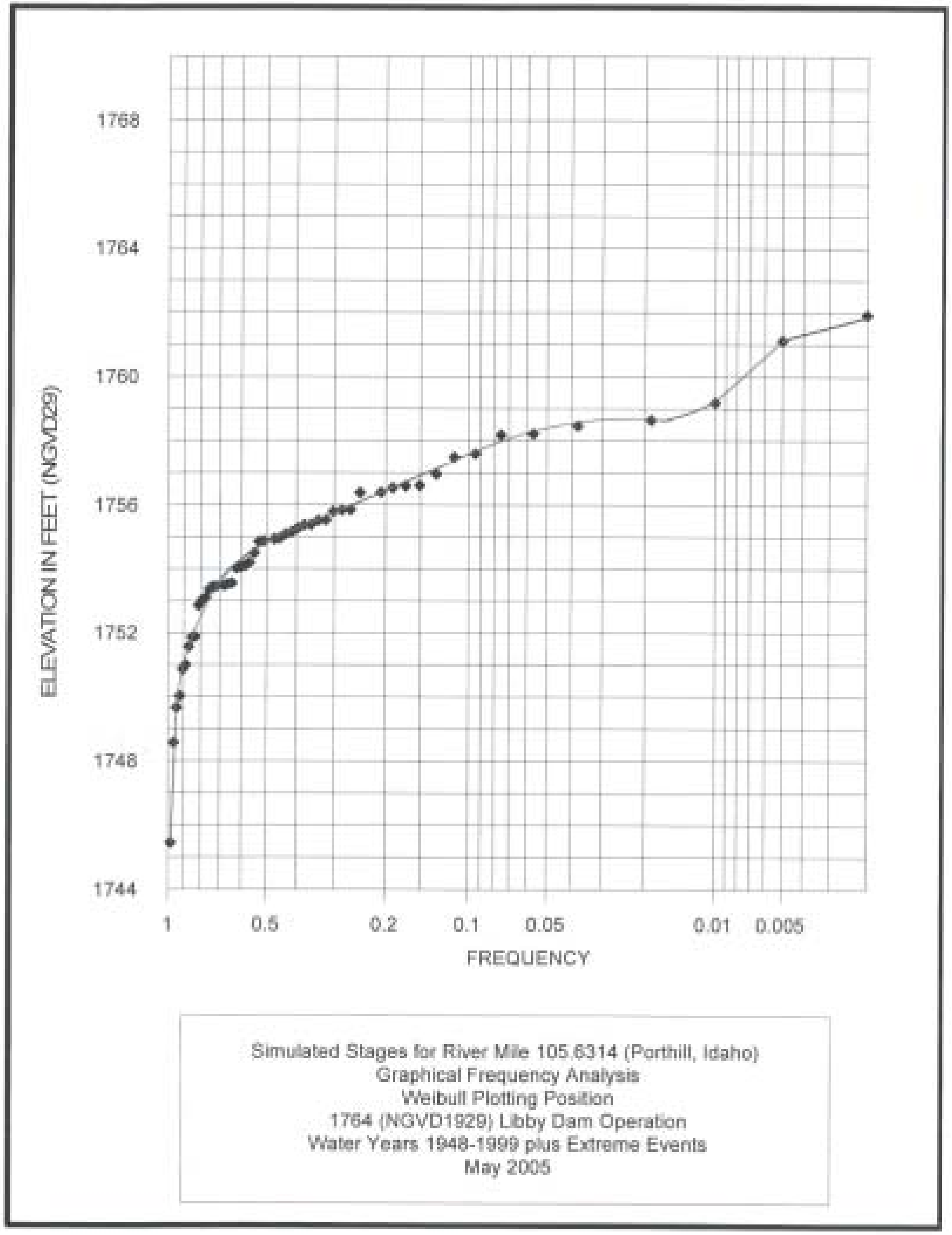

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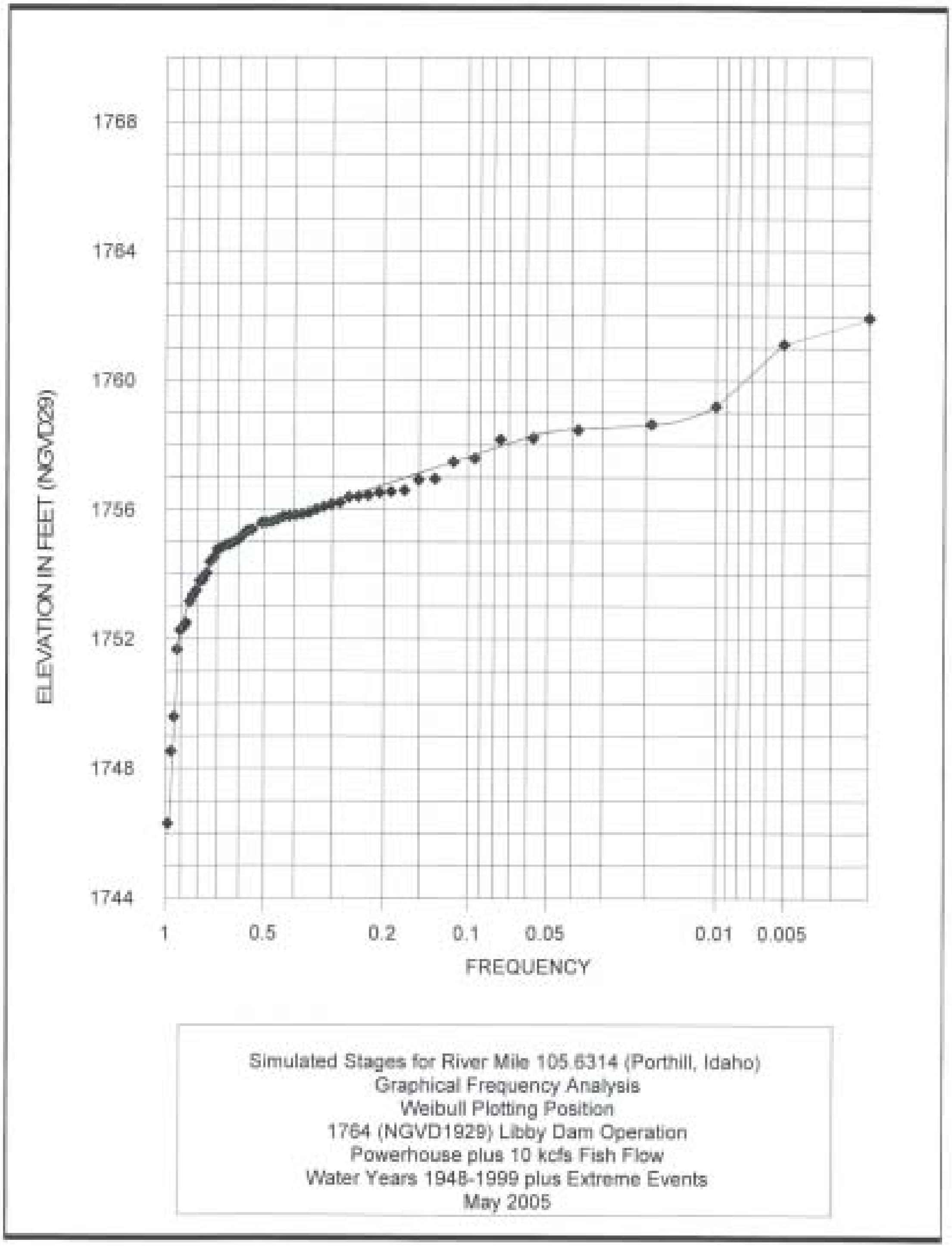

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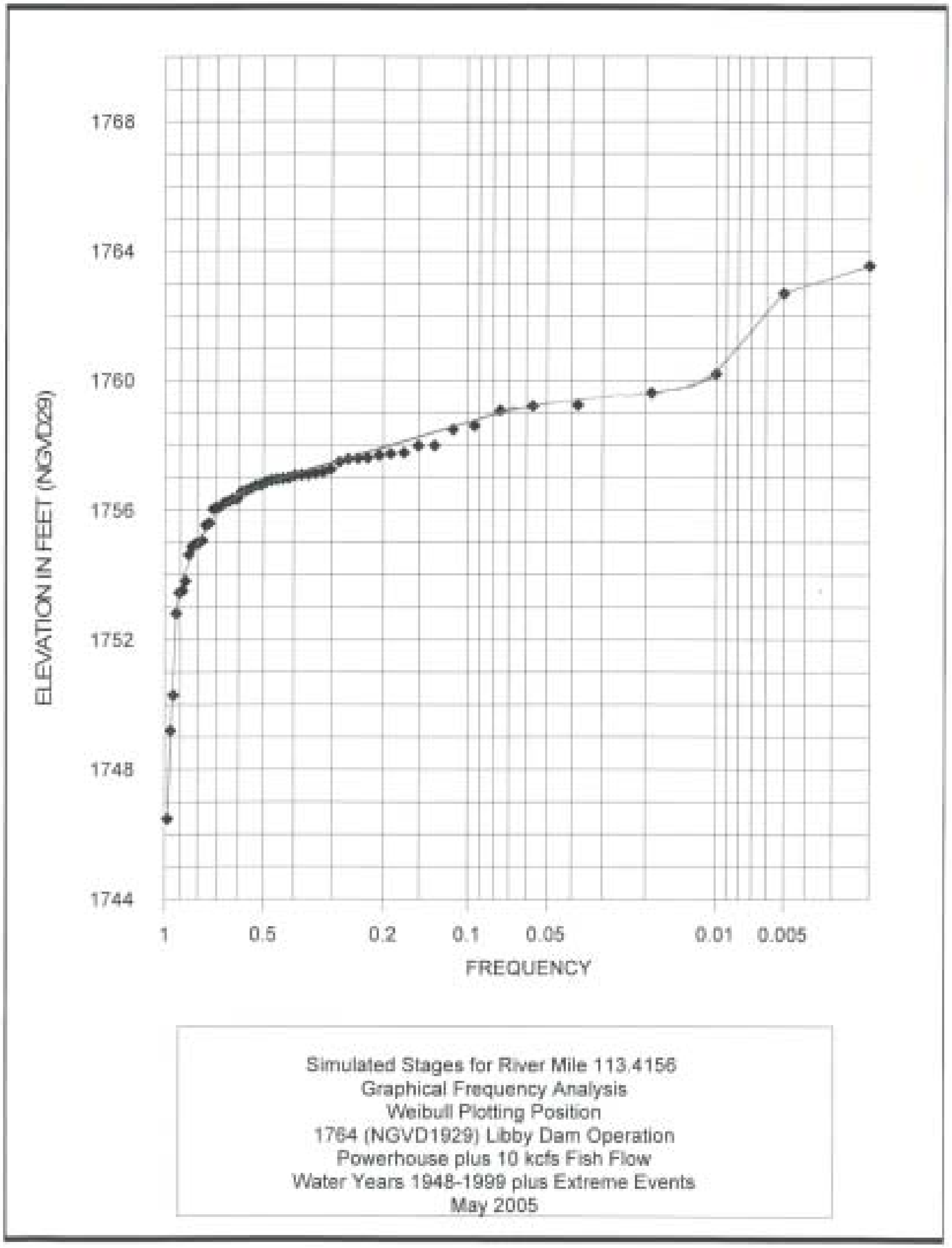

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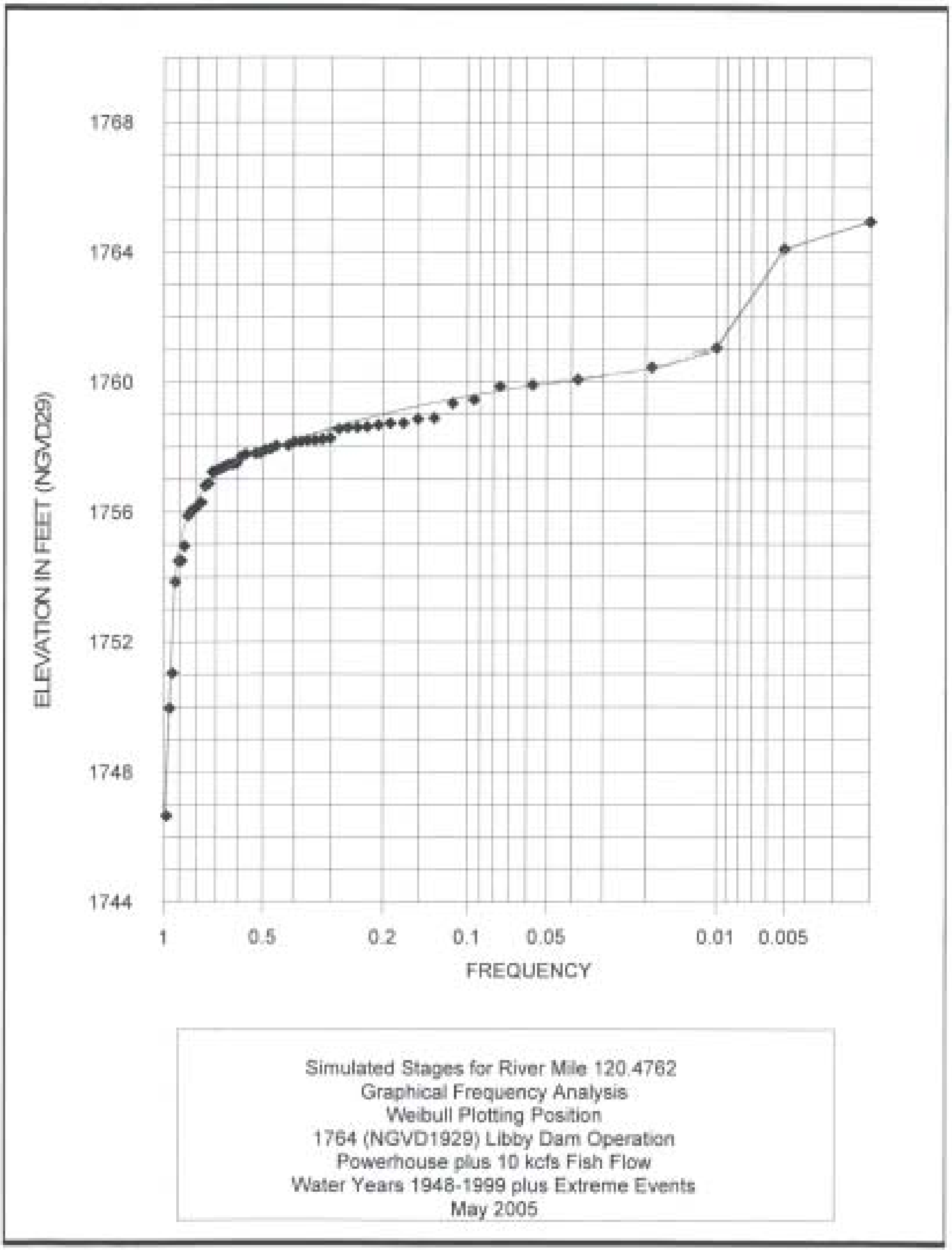

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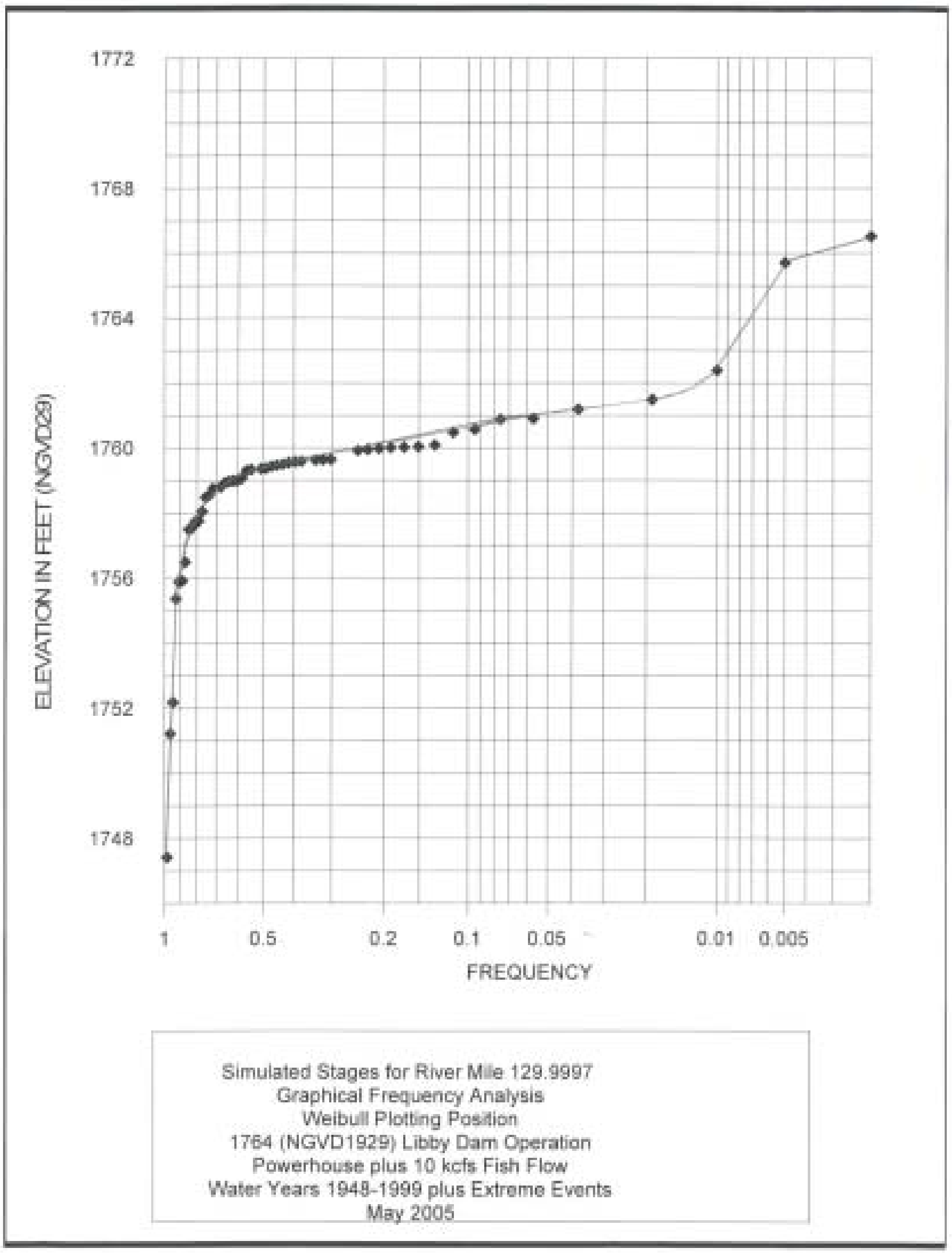

Figure 65. Frequency Curve 28

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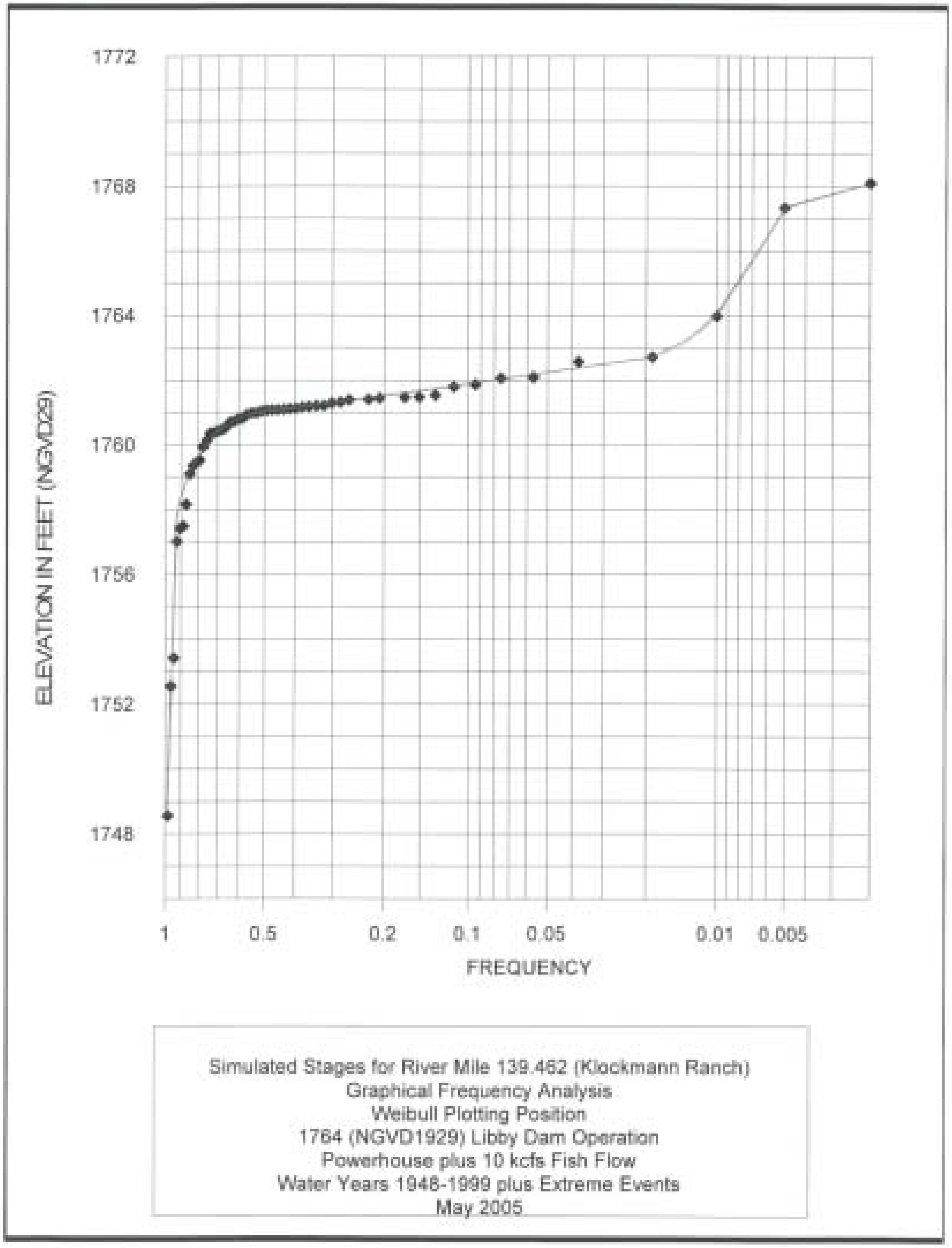

Figure 66. Frequency Curve 29

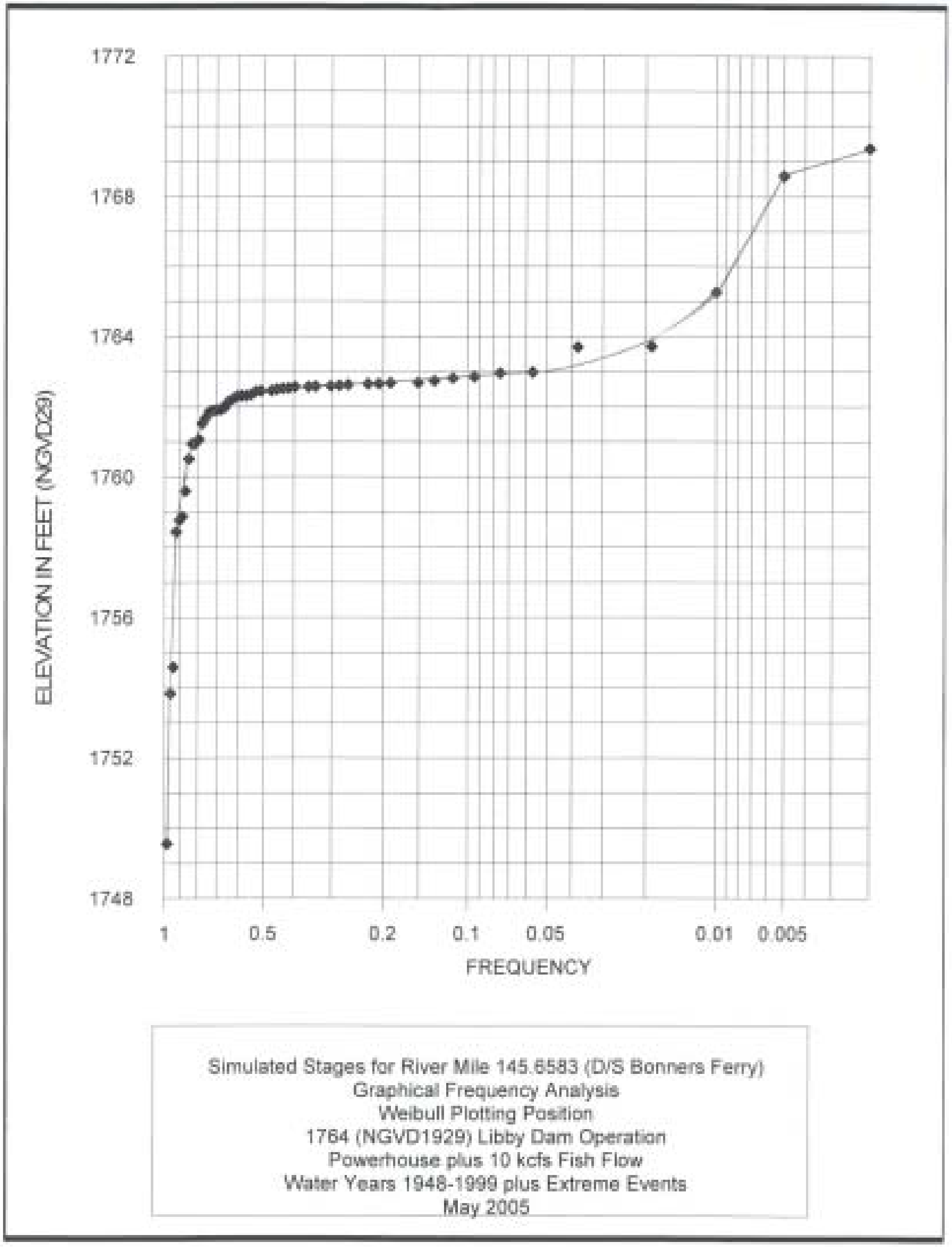

Figure 67. Frequency Curve 30

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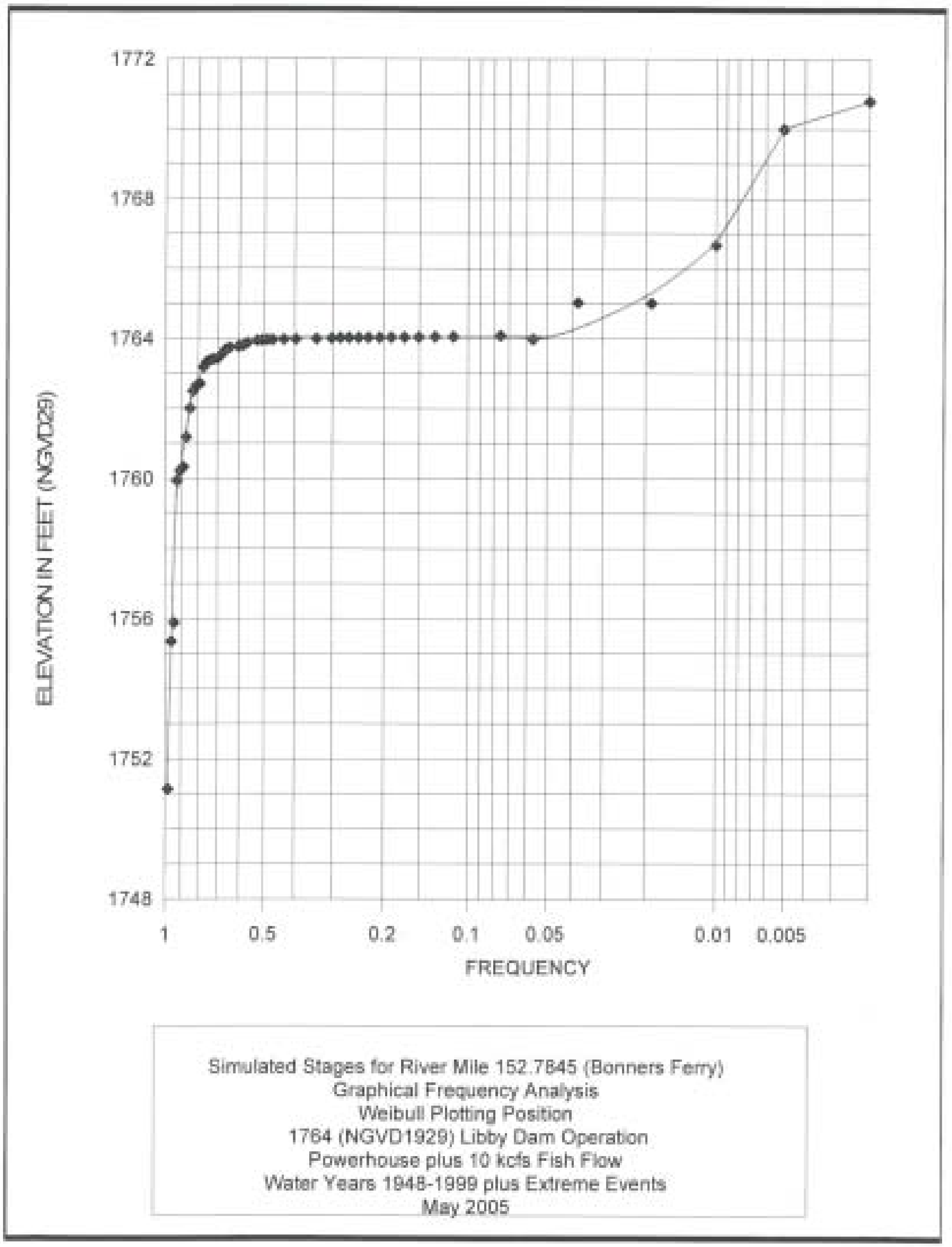

Figure 68. Frequency Curve 31

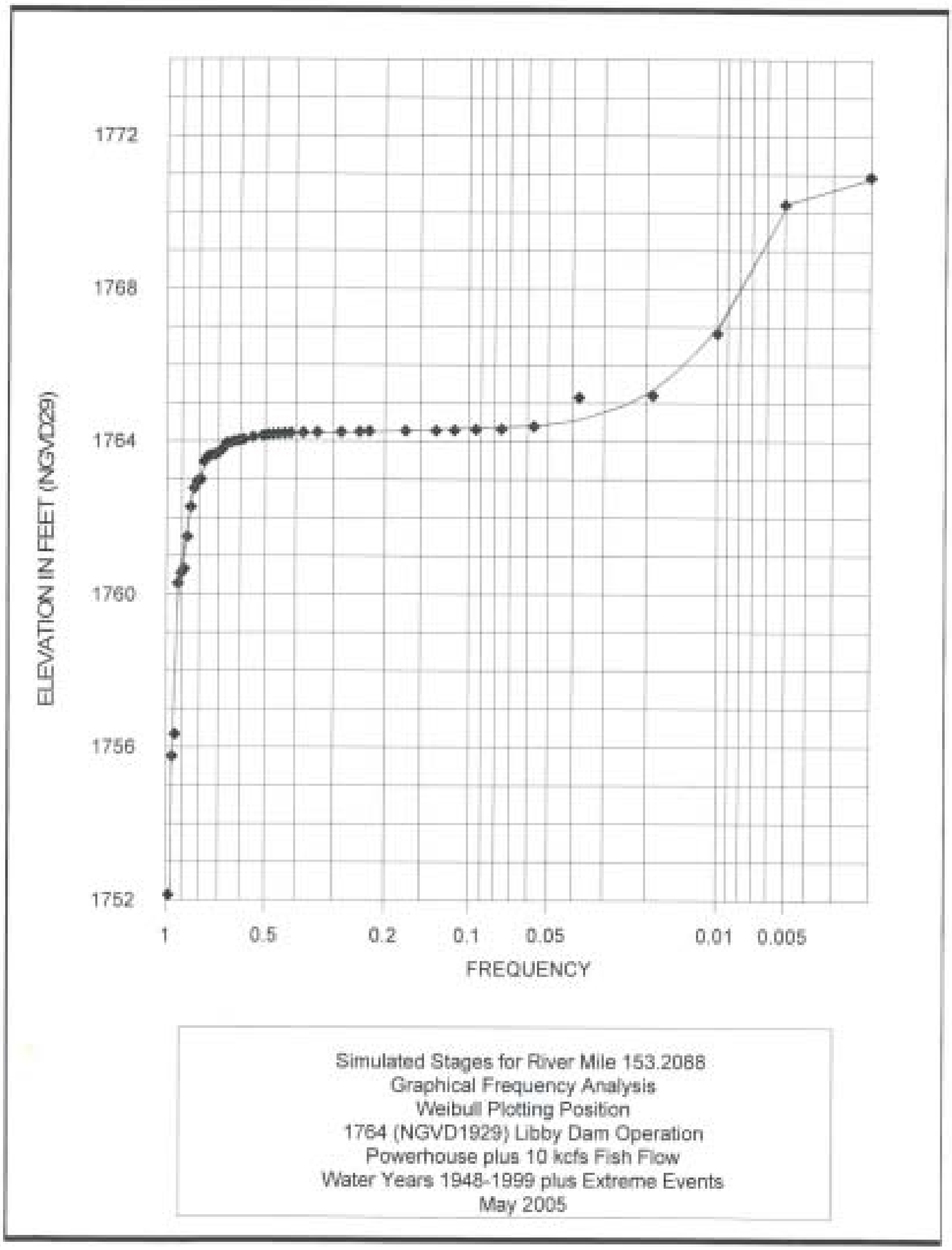

Figure 69. Frequency Curve 32

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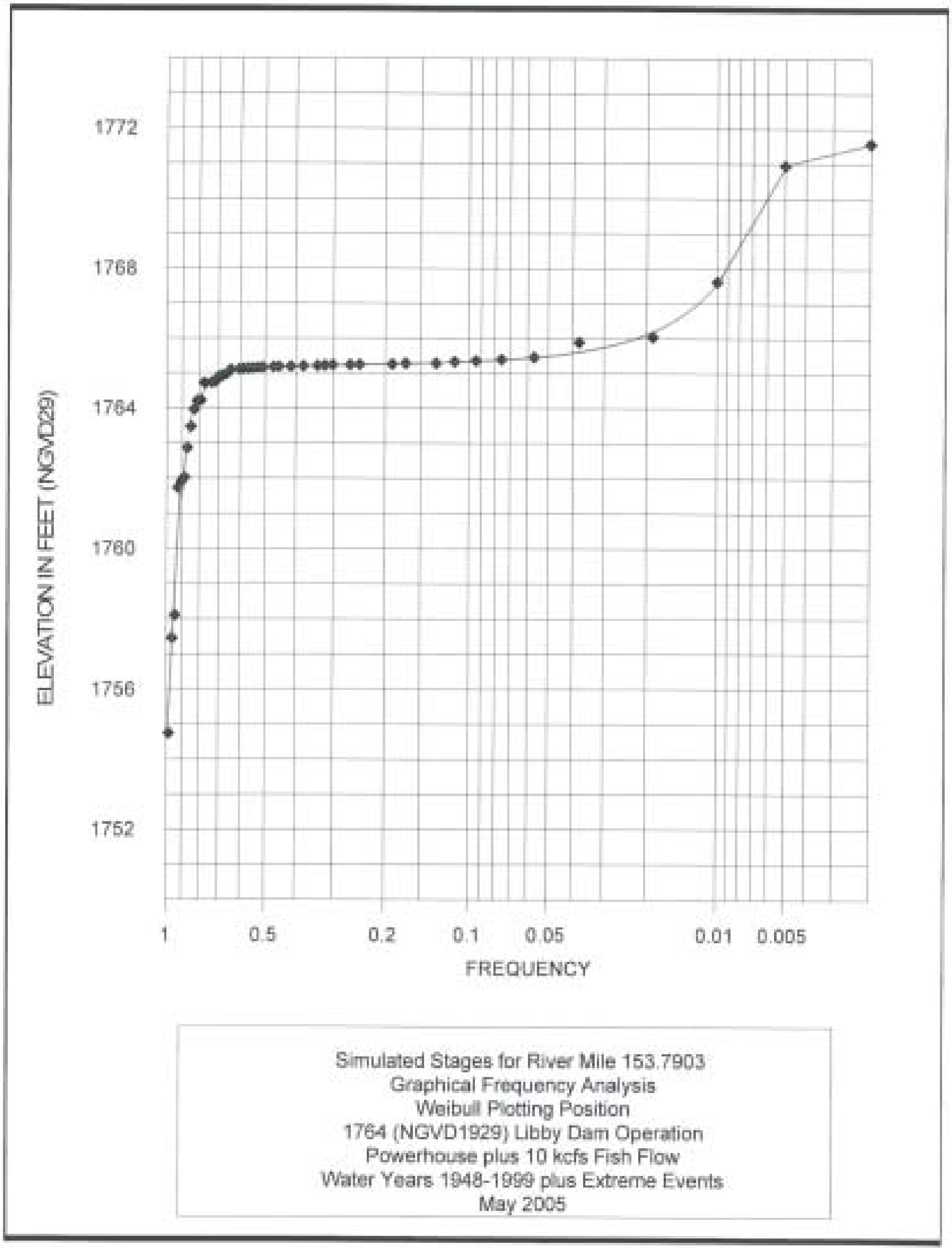


Figure 70. Frequency Curve 33

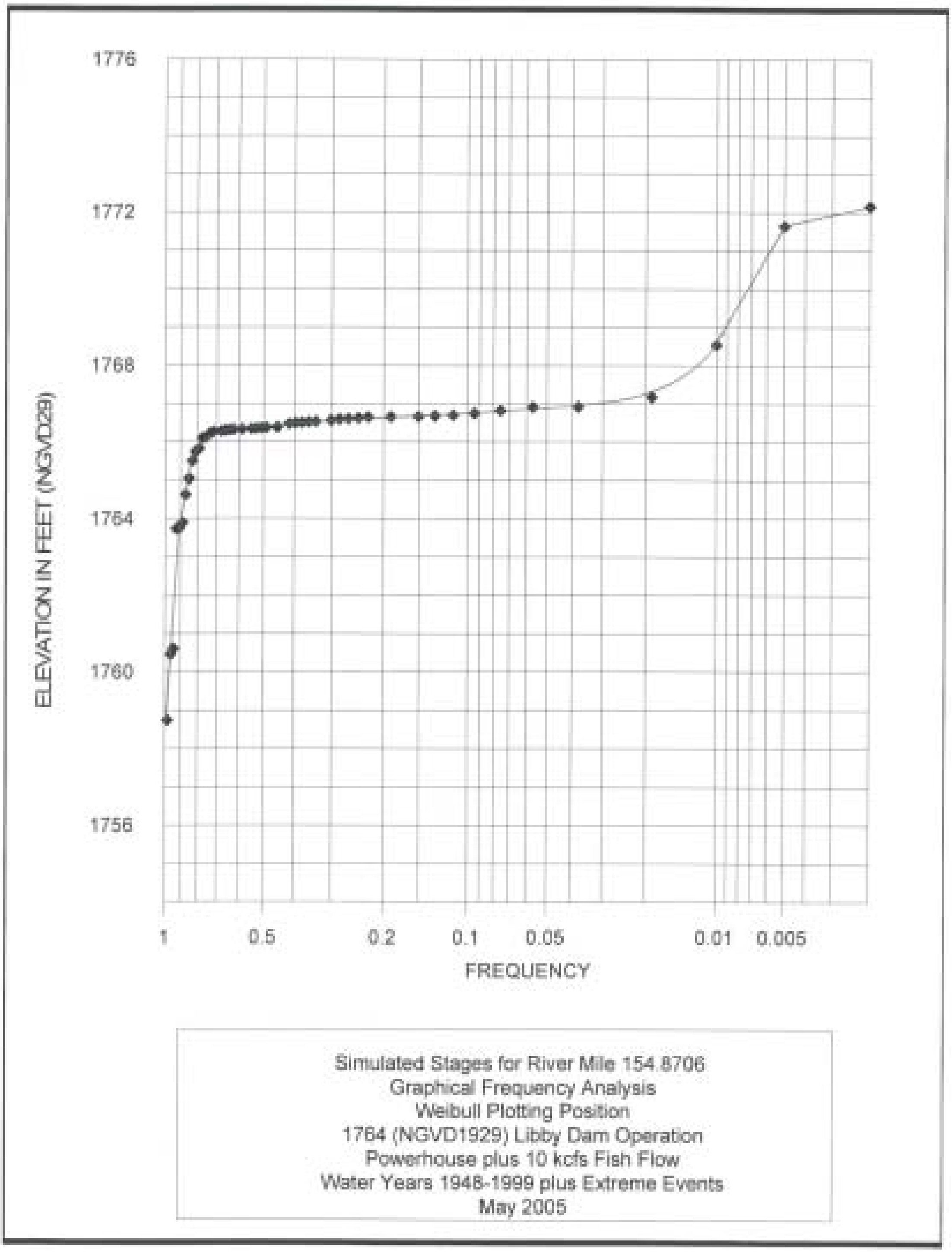


Figure 71. Frequency Curve 34

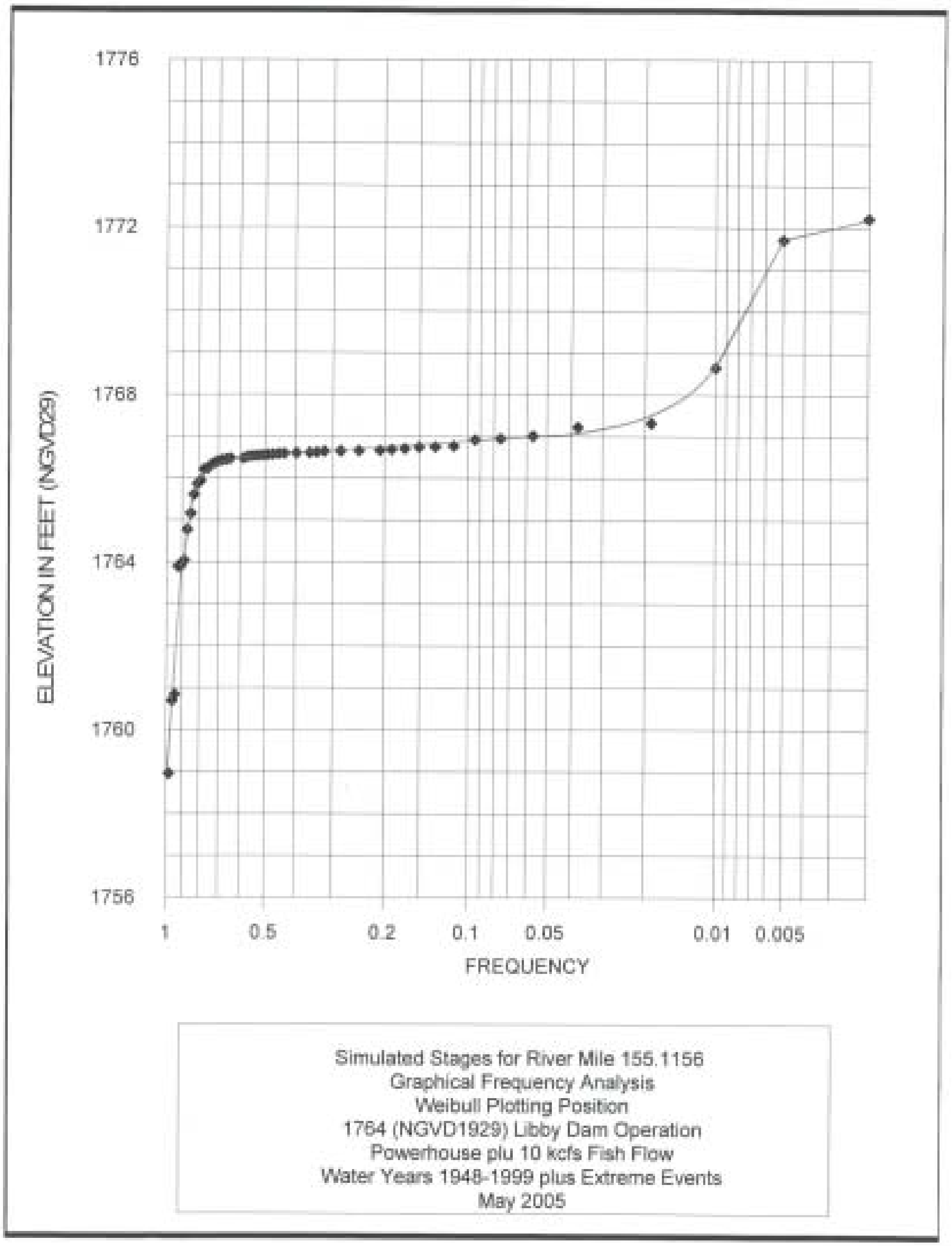


Figure 72. Frequency Curve 35

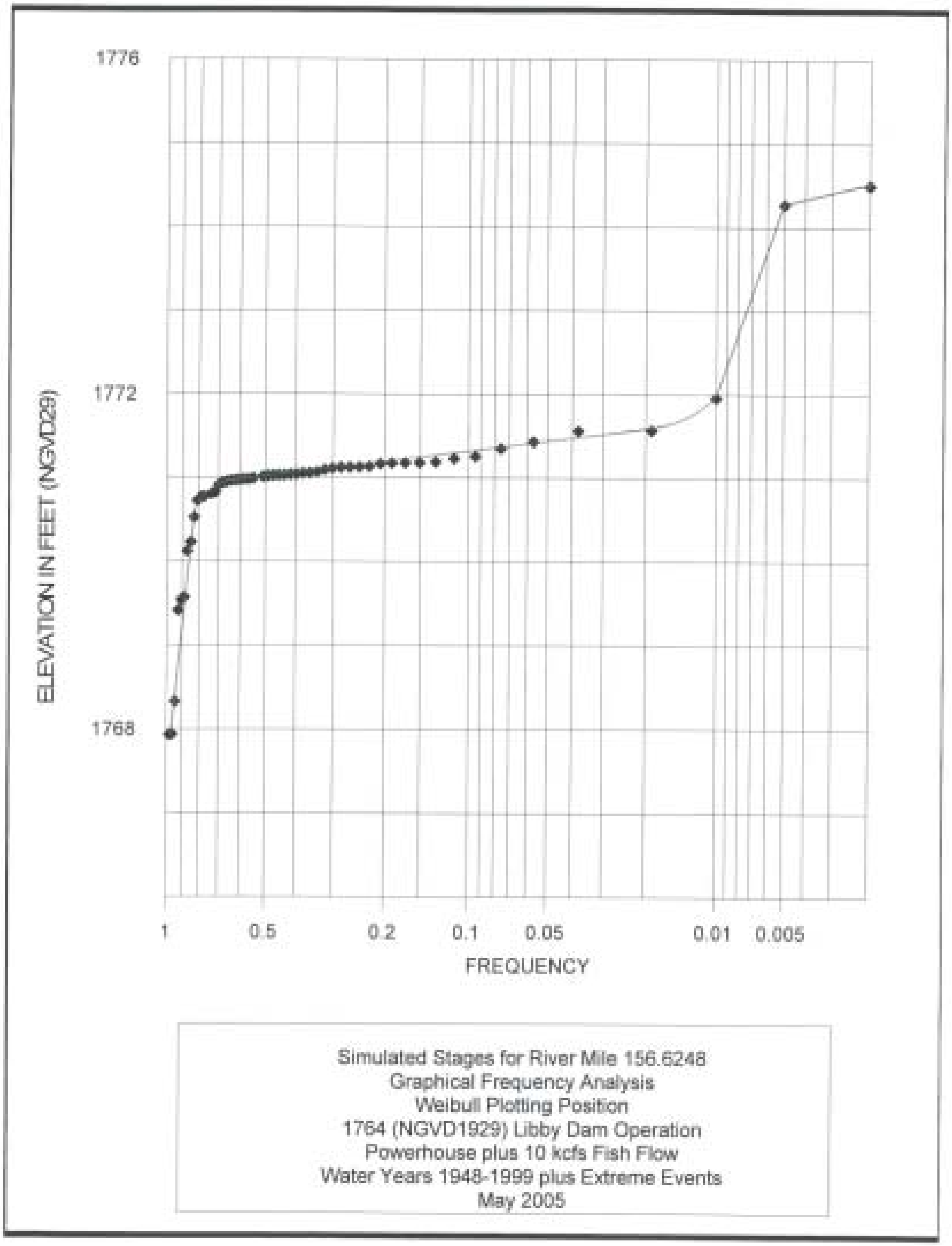


Figure 73. Frequency Curve 36

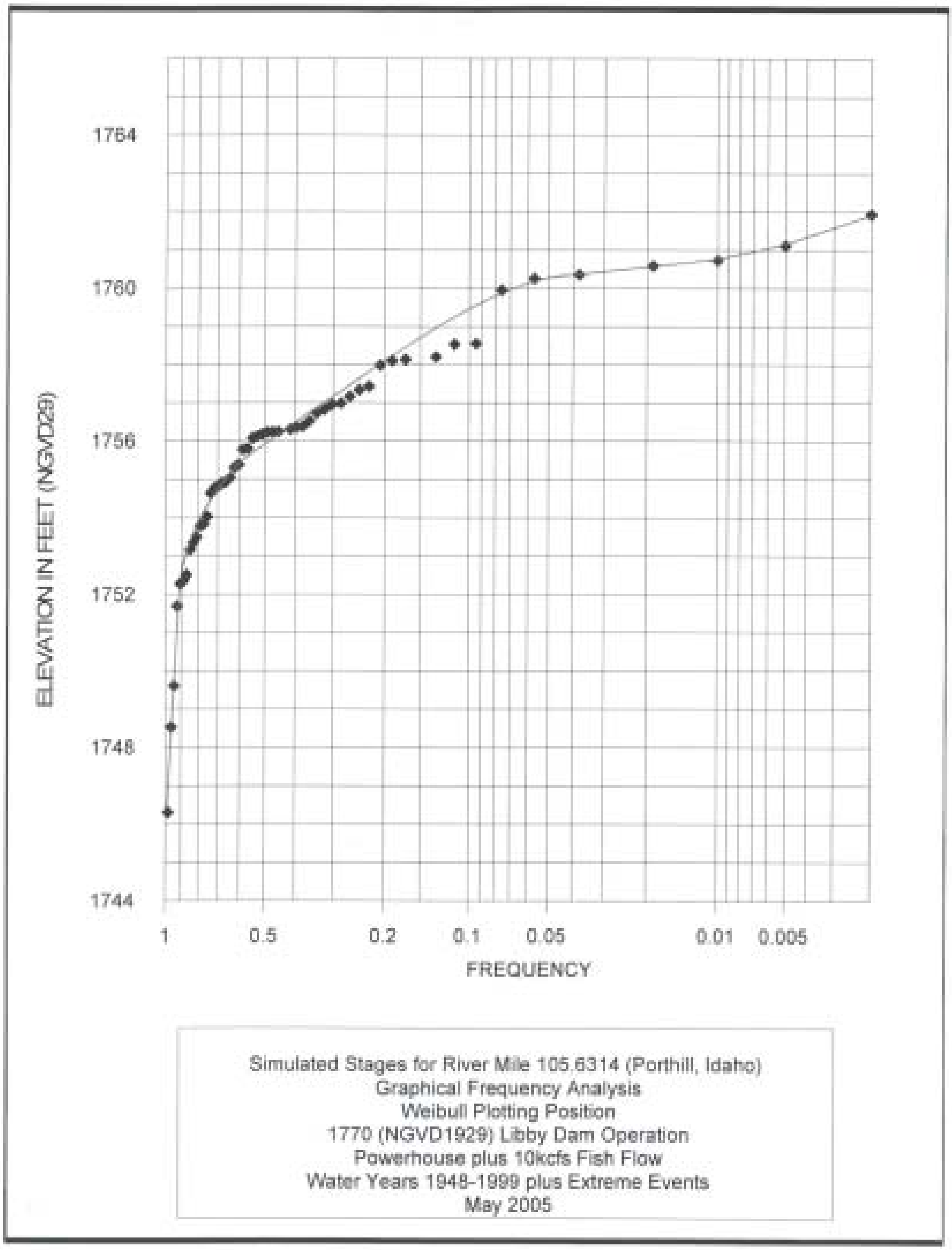


Figure 74. Frequency Curve 37

Frequency curves were developed only to facilitate this study and are not intended to replace any existing frequency curves currently used by local, state, federal, tribal, or private entities.

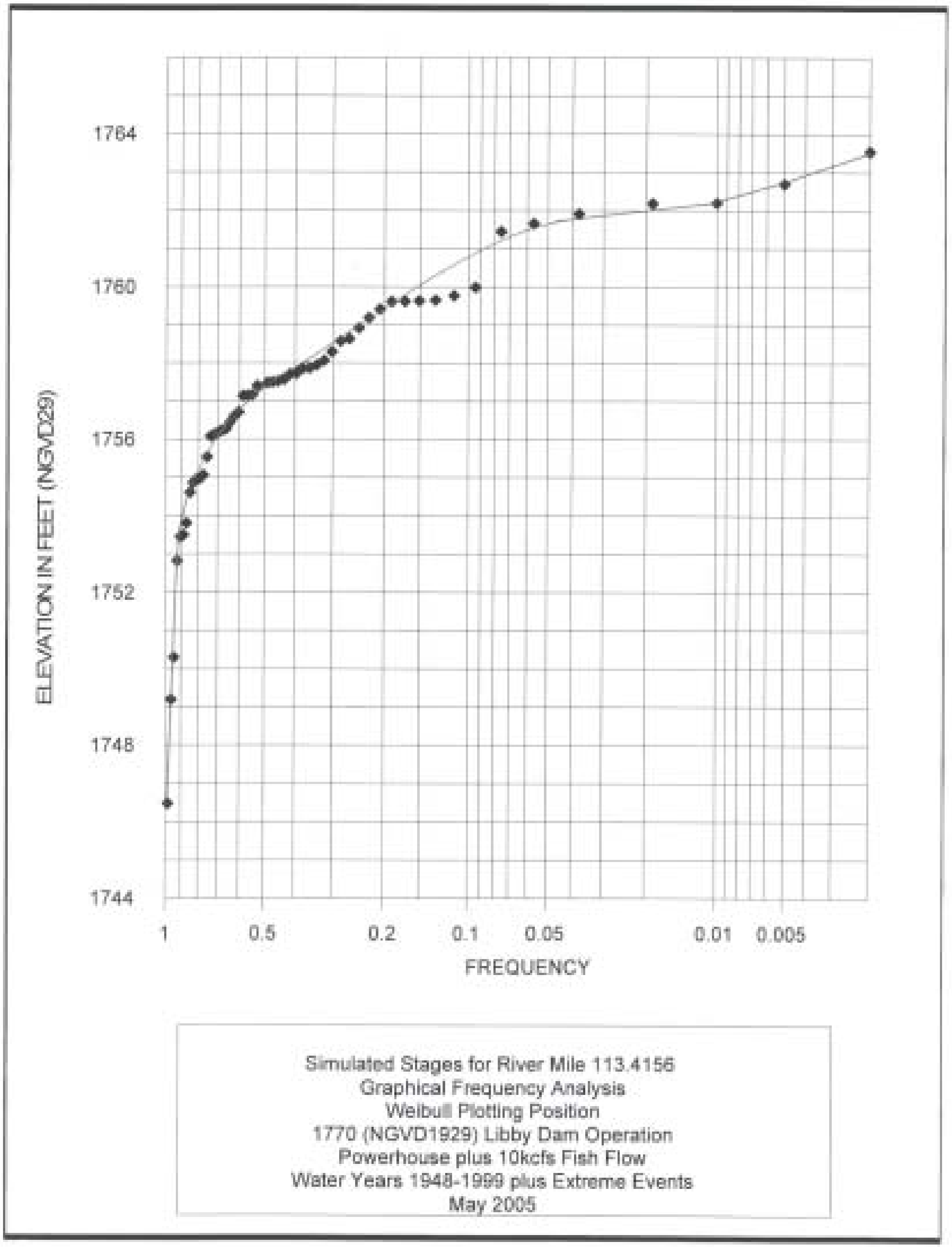
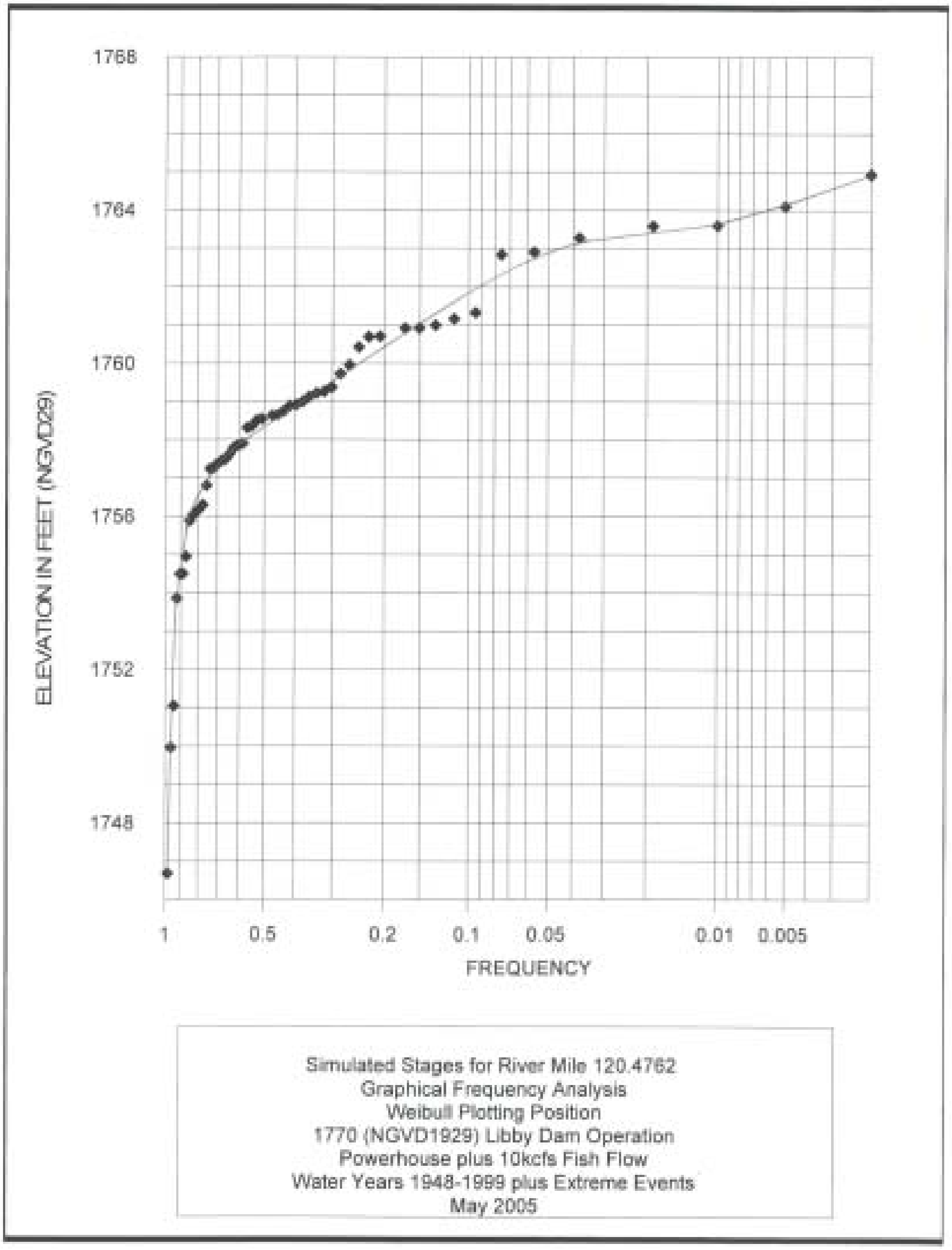


Figure 75. Frequency Curve 38



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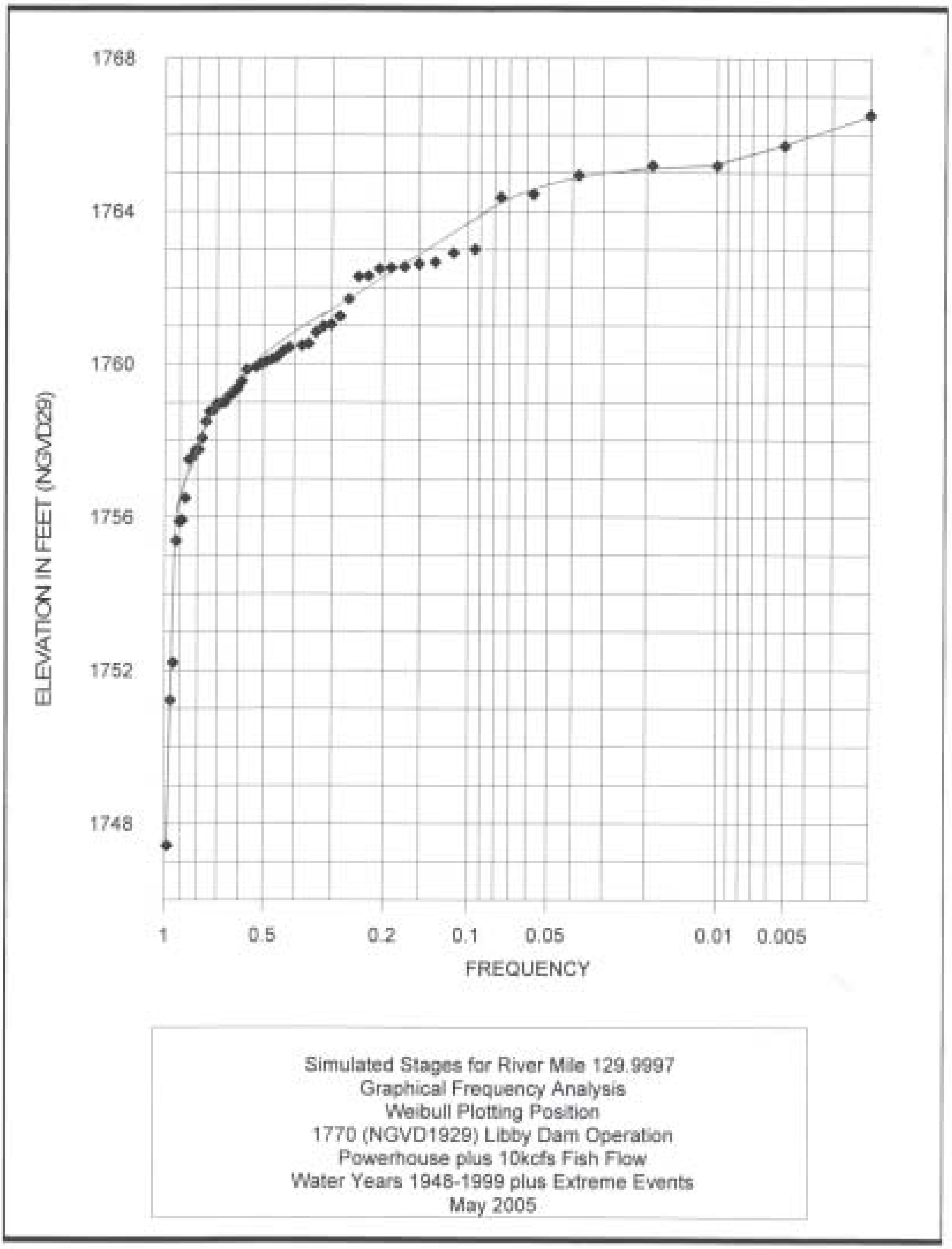

Figure 77. Frequency Curve 40

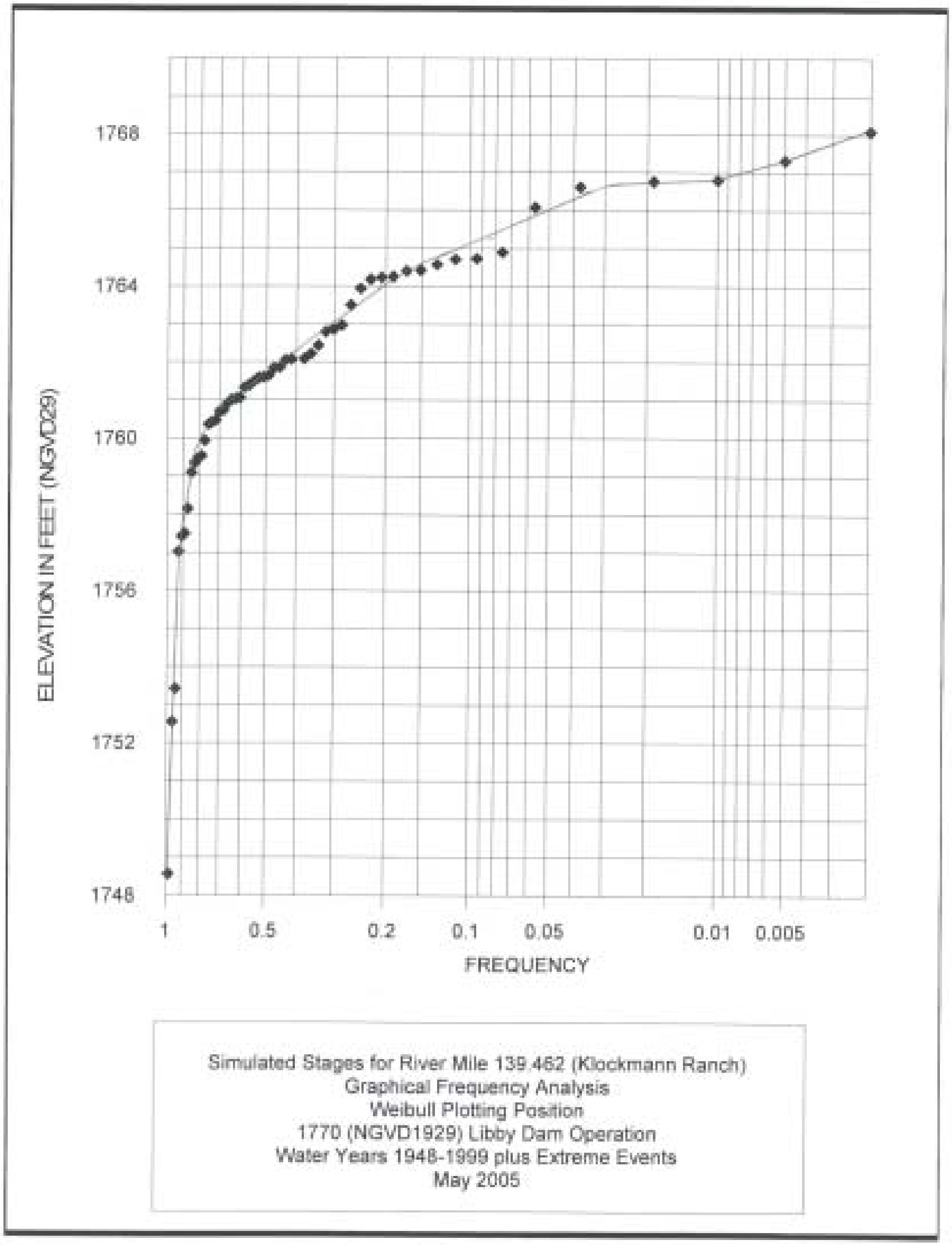

Figure 78. Frequency Curve 41

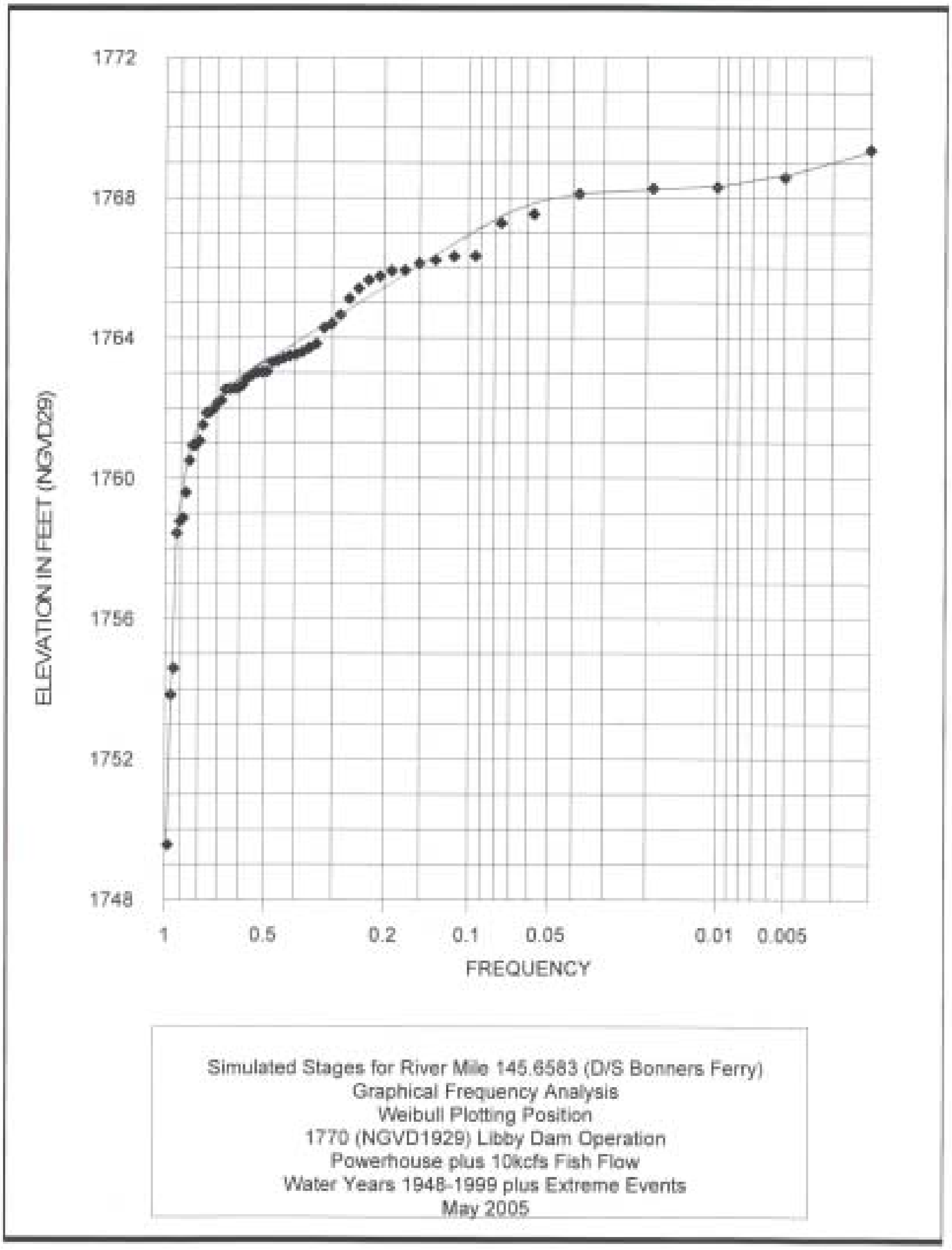

Figure 79. Frequency Curve 42

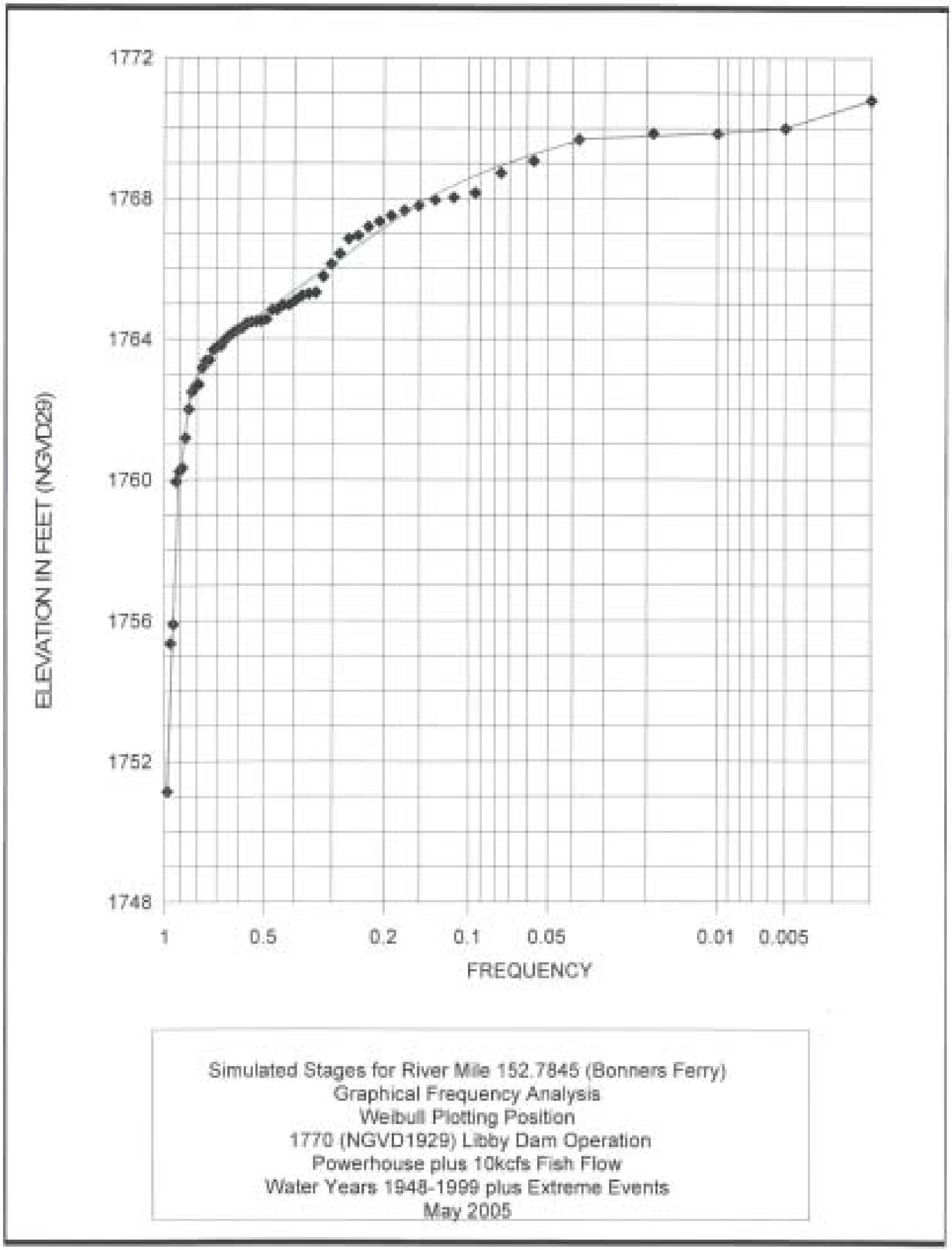

Figure 80. Frequency Curve 43

Frequency curves were developed only to facilitate this study and are not intended to replace any existing frequency curves currently used by local, state, federal, tribal, or private entities.

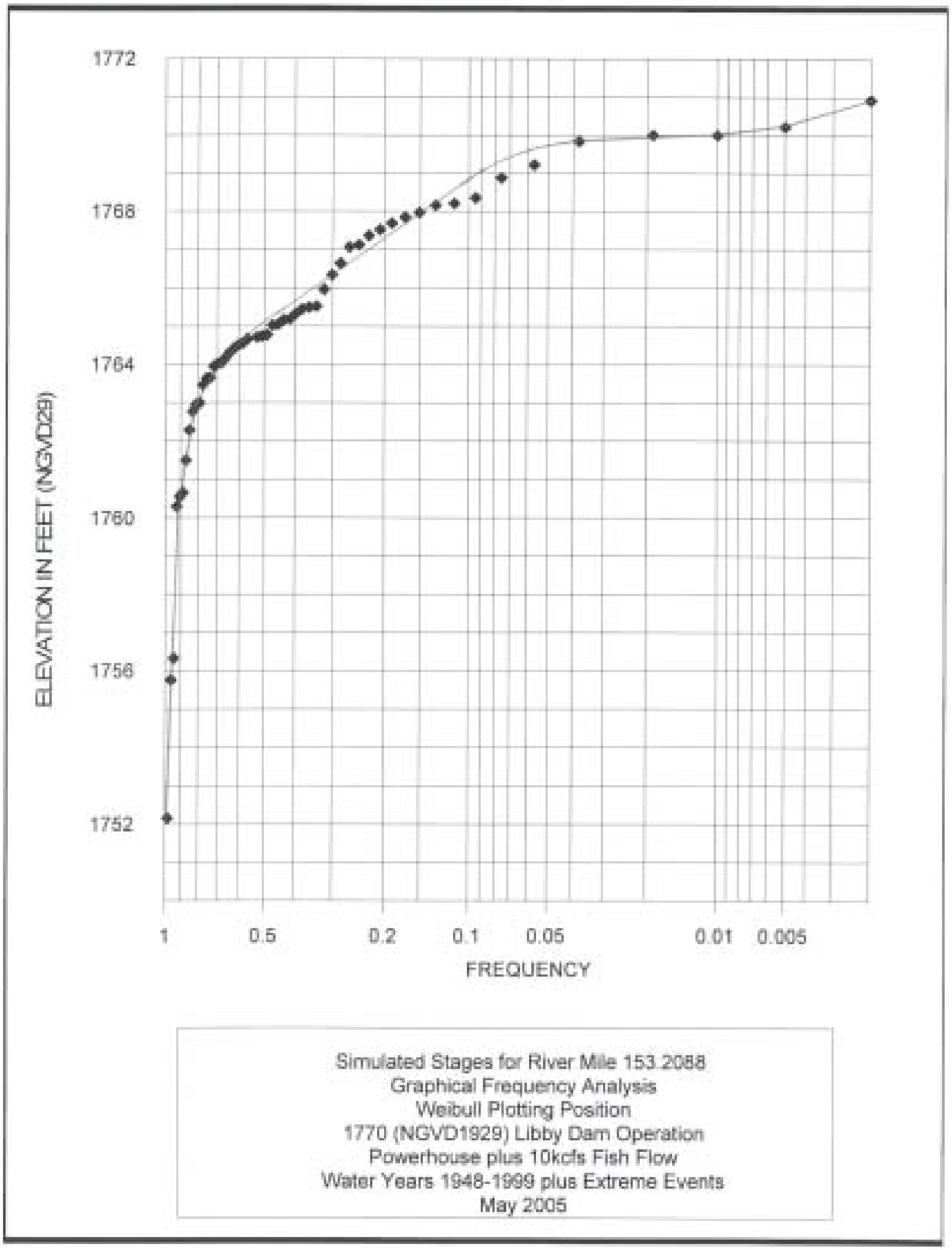

Figure 81. Frequency Curve 44

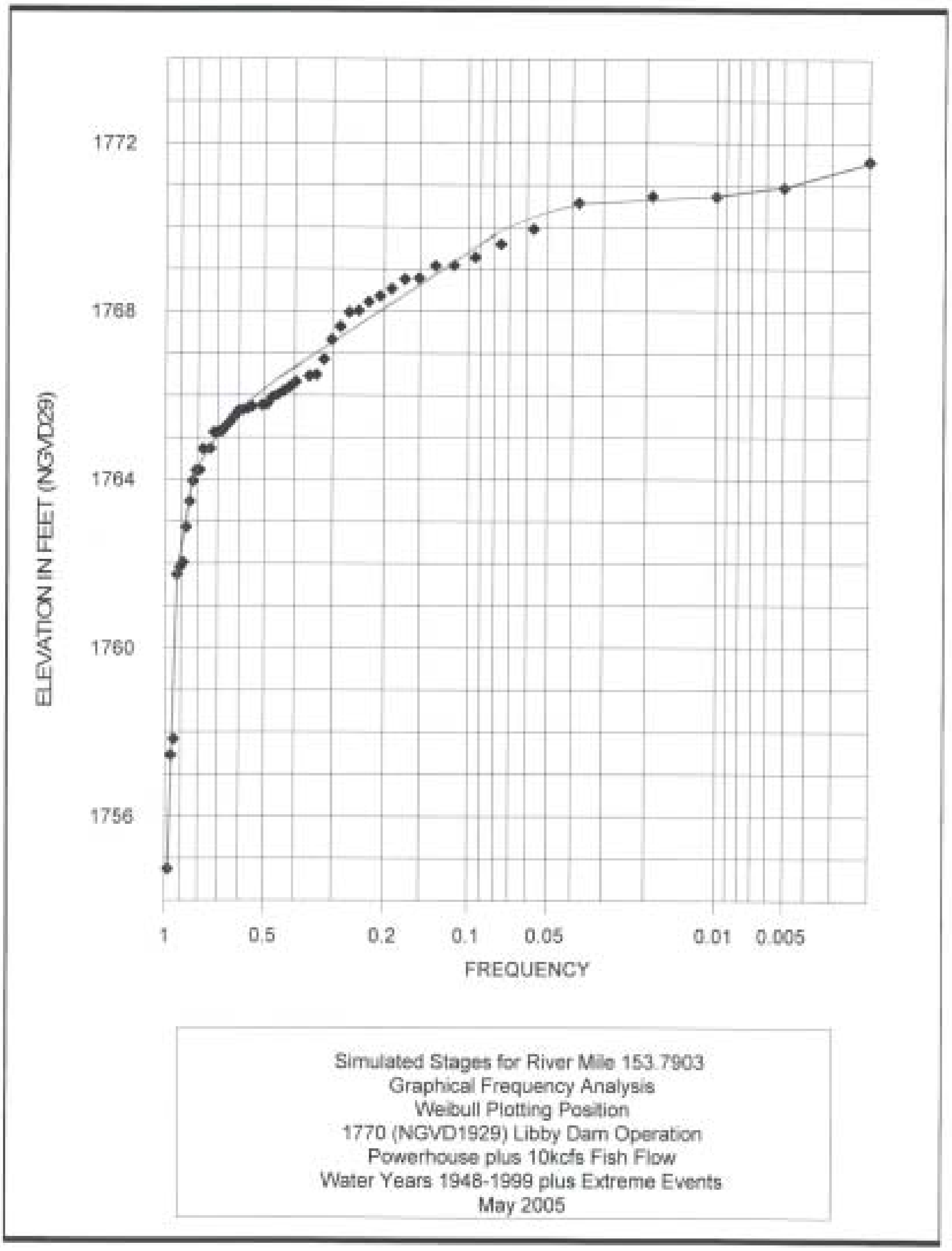

Figure 82. Frequency Curve 45

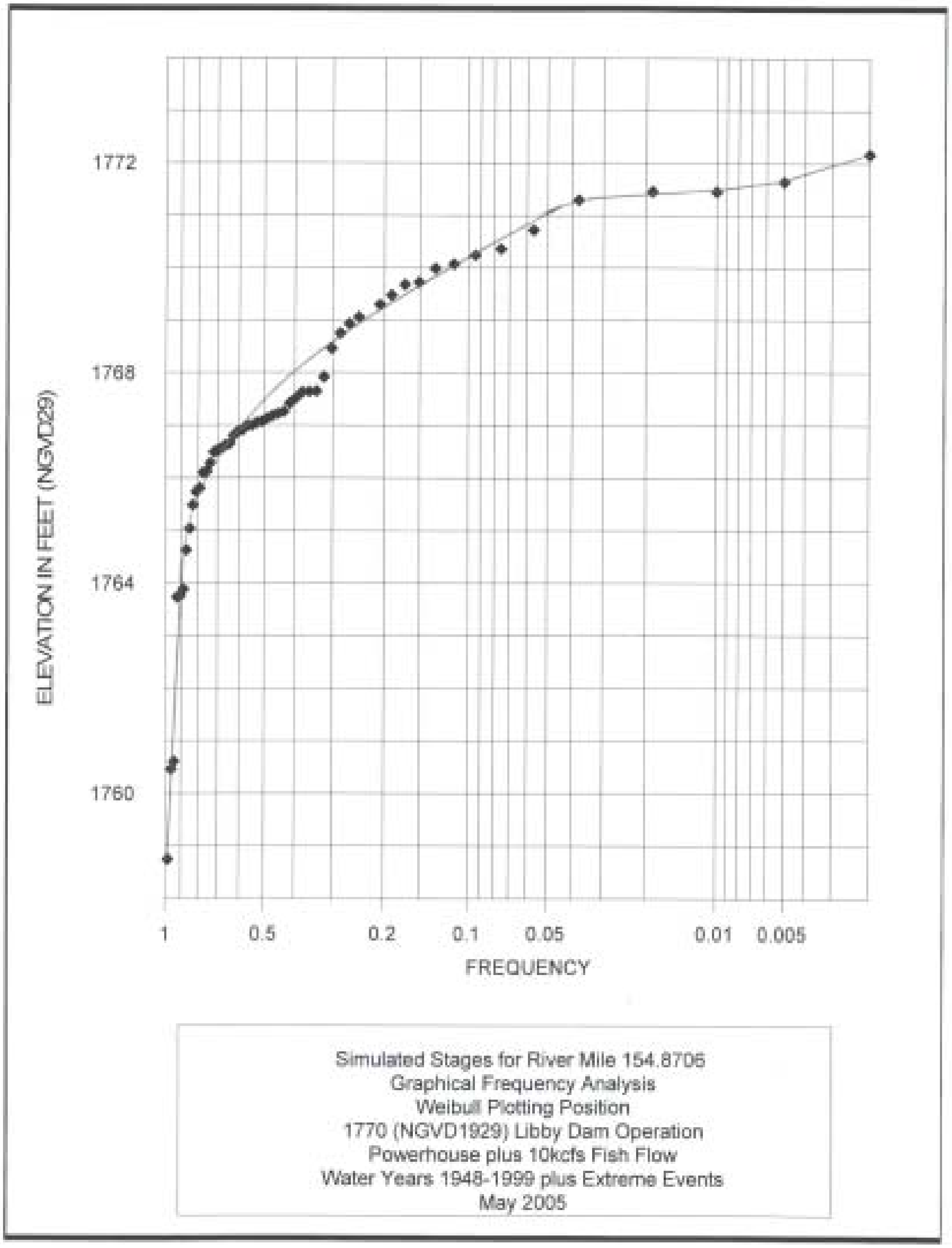

Figure 83. Frequency Curve 46

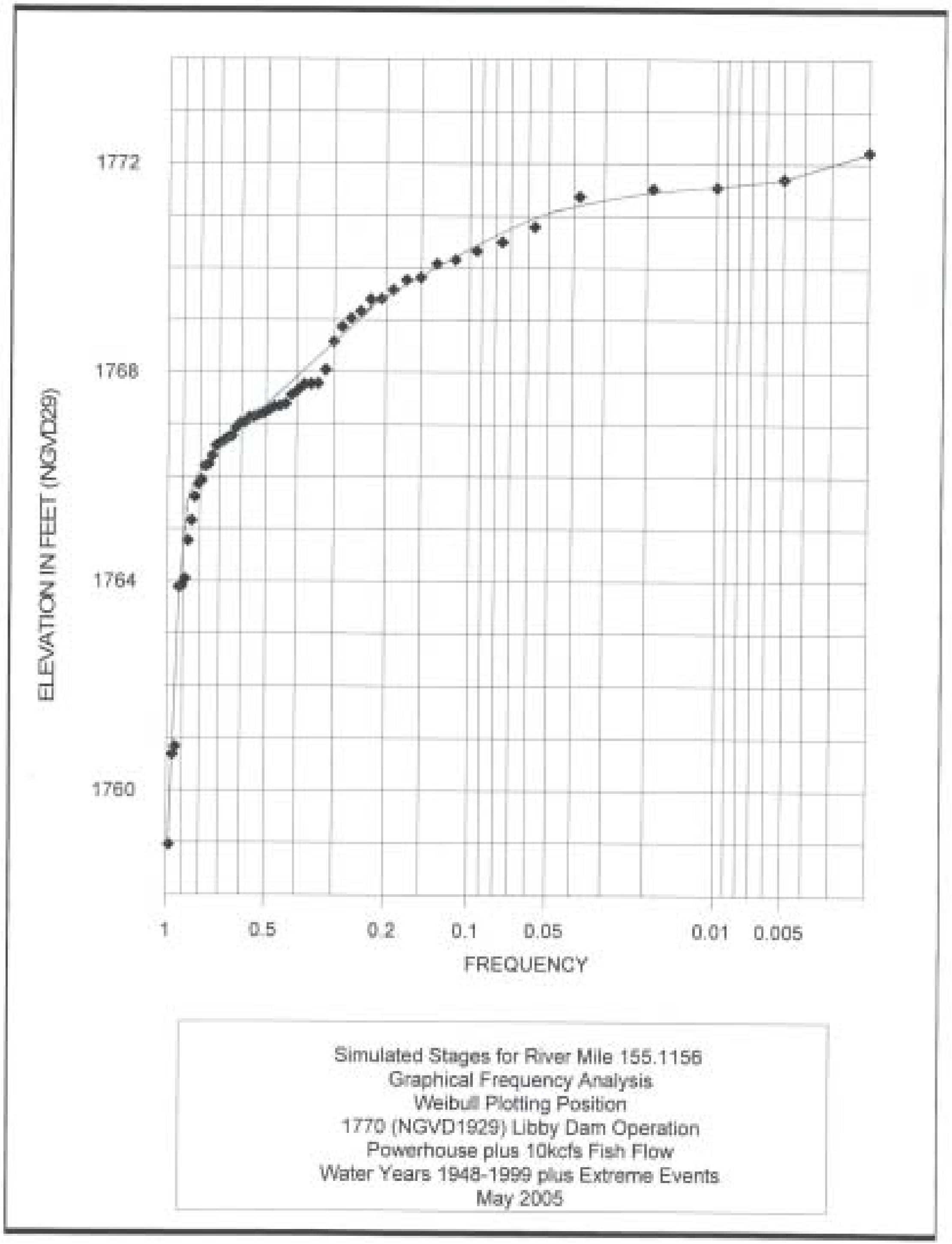

Figure 84. Frequency Curve 47

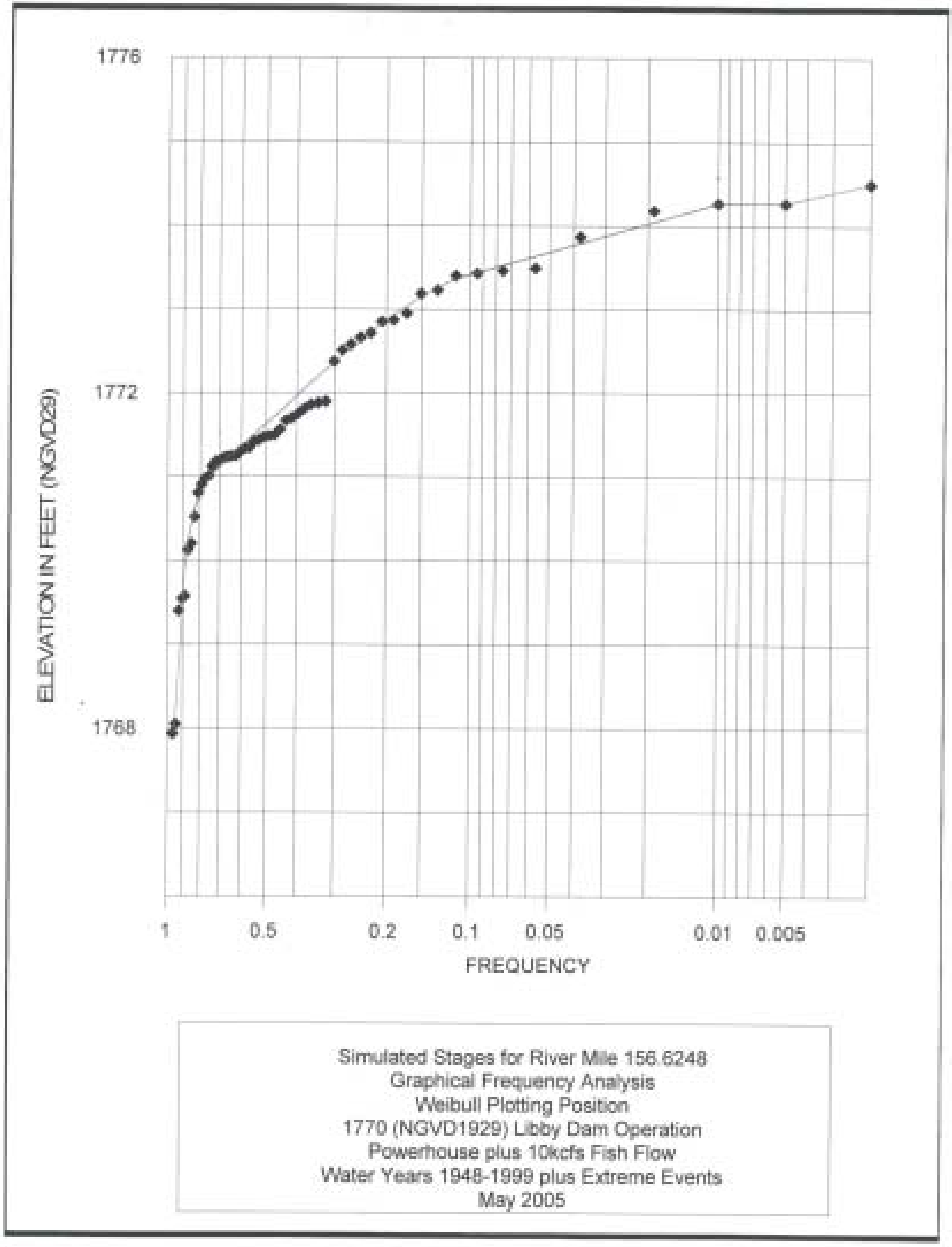

Figure 85. Frequency Curve 48

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7.3 APPENDIX C – STAGE DAMAGE RELATIONSHIPS

[See following pages]

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U.S. Army Corps
of Engineers
Seattle District

Economic Flood Depth-Damage Analysis for the Kootenai River

Bonners Ferry, Idaho to U.S./Canada Border

July 2005

DRAFT

Economic Flood Depth-Damage Analysis for the Kootenai River
Bonners Ferry, Idaho to U.S./Canada Border

for the

Kootenai River, Idaho
Kootenai River Flood Level Assessment Study
Boundary County, Idaho

prepared by:

Tetra Tech Inc.
1925 Post Alley
Seattle, Washington

prepared for:

Seattle District
U.S. Army Corps of Engineers

July 2005

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LIST OF TABLES

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Page No. C-

DRAFT

Economic Flood Depth-Damage Analysis for the Kootenai River Bonners Ferry, Idaho to U.S./Canada Border

for the

Kootenai River, Idaho
Kootenai River Flood Level Assessment Study
Boundary County, Idaho

1.0 INTRODUCTION

The purpose of this report is to document the methodology and procedures followed during the flood damage inventory revision for the Kootenai River Valley area from Bonners Ferry, ID, to the U.S. border with Canada. In addition this report will present findings and results in terms of valuation and potential damages from various magnitude flood events along this reach of the Kootenai River. The goal is to provide stage-damage functions representing varying damage categories and locations for use in the Hydrologic Engineering Center's Flood Damage Assessment program (HEC-FDA.)

This economic analysis is in accordance with standards, procedures, and guidance of the U.S. Army Corps of Engineers. The Planning Guidance Notebook (ER 1105-2-100, April 2000) serves as the primary source for evaluation methods of flood damage reduction studies and was used as reference for this analysis. Additional guidance for risk-based analysis was obtained from EM 1110-2-1619, *Engineering and Design – Risk-Based Analysis for Flood Damage Reduction Studies (August 1996)* and ER 1105-2-101, *Planning Risk-Based Analysis of Hydrology/Hydraulics, Geotechnical Stability, and Economics in Flood Damage Reduction Studies (March 1996).* All values and damages are computed at current (October, 2004) price levels.

2.0 FLOOD PLAIN INVENTORY

2.1 Economic Study Area

For the purposes of determining the flood plain inventory, the study area was separated into 26 storage areas. Each represents an area that can be defined by common hydraulics which allowed for collection and analysis of data prior to the development of final river hydraulics. Land use in the majority of these storage areas is primarily agriculture, with very few structures. Storage areas 2, 2B and 22 have a greater number of structures and include portions of the City of Bonners Ferry (population 2,515.) Locations of each storage area are shown in Figure 86.

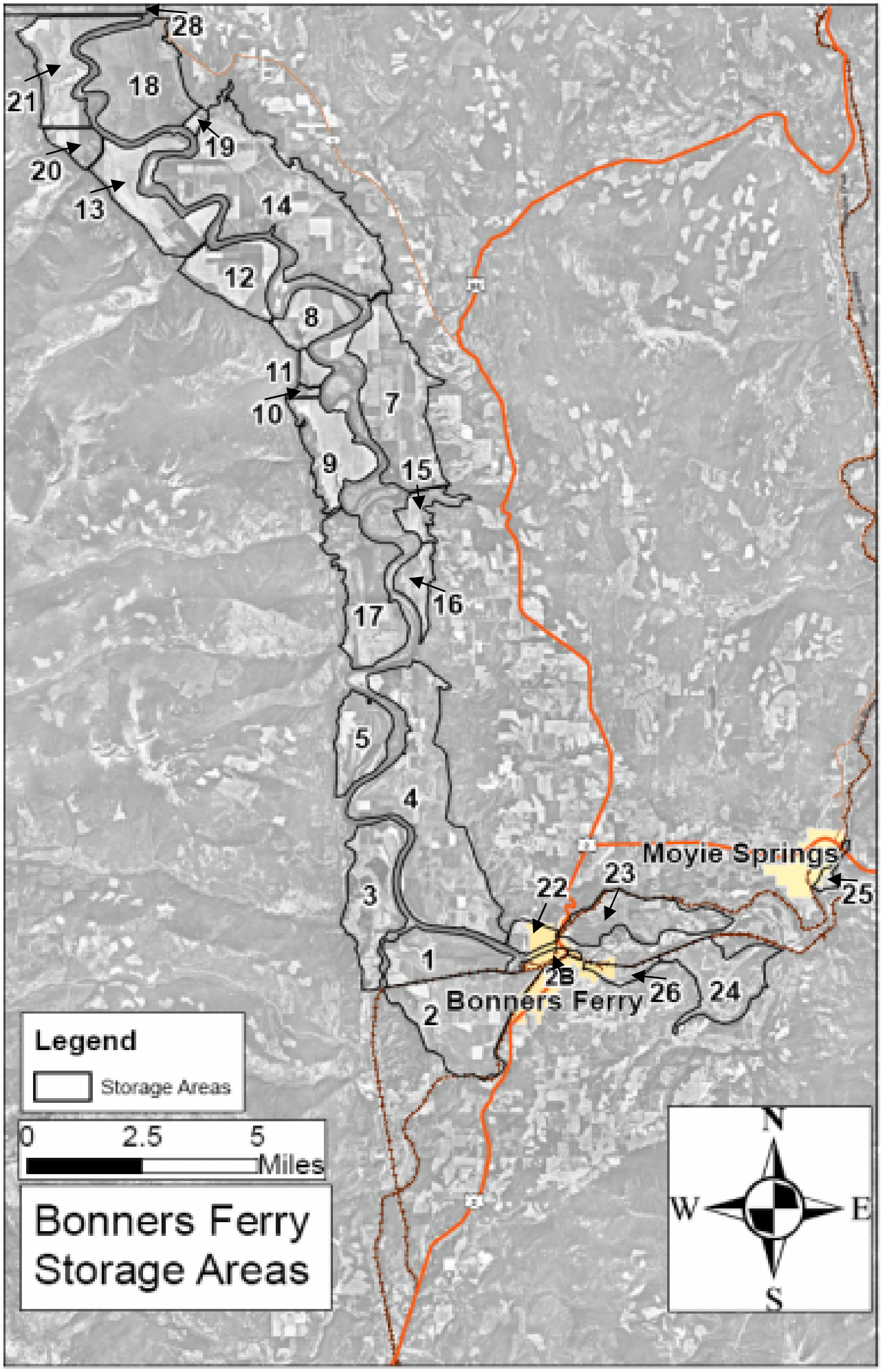

Figure 86. Kootenai River Storage Areas

2.2 Land Use and Structure Value

Land use was inventoried prior to the availability of topographic or hydraulic data. A complete field survey of all commercial and public structures within the outline of each storage area was gathered. Residential, farm structures and outbuildings (such as unattached sheds and garages) were surveyed through a random sample (sample observations are 10 percent of the total population) of the study area. Data collected included structure use, type of construction, structure size, condition, and first-floor elevation. Structure values are based on depreciated replacement value. Structure condition, use, type, and size were used in conjunction with the Marshall & Swift Valuation Service to develop estimates of depreciated replacement costs.

Risk-based errors and standard deviations for depreciated structure replacement values are based on three factors:

a) Uncertainty in price per square foot based on variation in Marshall Valuation Service quality of construction grades.
b) Variation in estimation of building square footage
c) Estimated remaining depreciated value (100% value minus percent depreciated – consistent with Marshall & Swift)

These risk parameters are consistent with methodologies discussed in Chapter 6 of EM 1110-2-1619 and were given either normal or triangular distributions and values of each structure were determined as mean and standard deviation using the function:

Structure Value= $/square foot X square footage X percent remaining value

Content values were set as a percentage of structure value and were taken from both the 1987 Bonners Ferry Data (commercial and public) and similar district studies. The percents used are as follows:

Commercial = 100%
Public= 100%
Farm Buildings = 67%
Residential = 50%

For some of the larger commercial facilities, content values were based on direct information gathered during the current analysis and have content percentages higher than the 100% listed for general commercial uses.

The total number of structures within the inventoried study area (including total structure and content value) are listed in **Table C-1** by land use.

Table C-1. Structural Inventory within All Storage Areas (Combined)

Land Use	Number of Structures	Total Structure Value In $ Millions	Total Content Value In $ Millions
Commercial	94	$ 17.4	$ 18.6
Farm/Out Buildings	378	$ 3.1	$ 2.1
Public	37	$ 7.9	$ 7.9
Residential	521	$ 22.8	$ 11.4
Total	1,030	$51.20	$40.00

2.3 Assignment of Ground and First Floor Elevations

The structures listed above represent the total inventory for the study area. The actual risk of flooding is directly related to the elevation of the individual structures vs. water surface elevation. Many of the structures in the inventory were found to be located at elevations above theoretical flood events. A Geographic Information System (GIS) was used to assign each structure to a physical point location and elevation was taken from the provided Digital Elevation Model (DEM.)* Representative foundation heights were added to the ground elevations at the center point of the structures to derive first floor elevations. Foundation heights were determined during field visitations and their standard errors were selected consistent with EM 1110-2-1619. **Table C-2** displays the total number and value of structures for each storage area. All the structures in storage areas 11 and 15 are located above 1780 feet.

Table C-2. Number of Structures by Storage Area at Selected Elevations

Storage Area	TOTAL NUMBER OF STRUCTURES (At Indicated Elevation)				TOTAL VALUE OF DAMAGEABLE PROPERTY (Structure & Content in $1,000's)			
	1760 feet	1764 feet	1770 feet	1780 feet	1760 feet	1764 feet	1770 feet	1780 feet
1	4	9	24	26	163	263	676	717
2	6	22	77	84	745	7,384	22,702	24,750
2B	1	20	42	50	69	1,060	2,636	3,107
2C	0	0	1	1	0	0	1,536	1,536
3	0	2	2	4	0	32	32	55
4	0	0	5	39	0	0	190	1,959
5	0	0	6	17	0	0	134	335
7	7	11	11	12	135	251	251	265
8	1	8	15	15	13	163	311	311
9	0	11	14	14	0	10,692	10,787	10,787
11	0	0	0	0	0	0	0	0
12	0	1	1	5	0	13	13	218
13	26	26	26	26	509	509	509	509
14	12	26	35	40	214	443	617	708

* DEM provided by Seattle District Hydrology and Hydraulics Section (Pat Wheeler) included the following projection:
File: Prj.adf
Projection: ALBERS
Datum: NAD83
Spheroid GRS80
Units METERS

Storage Area	TOTAL NUMBER OF STRUCTURES (At Indicated Elevation)				TOTAL VALUE OF DAMAGEABLE PROPERTY (Structure & Content in $1,000's)			
	1760 feet	1764 feet	1770 feet	1780 feet	1760 feet	1764 feet	1770 feet	1780 feet
15	0	0	0	0	0	0	0	0
16	3	7	7	7	40	133	133	133
17	3	3	5	13	73	73	100	381
18	9	18	18	18	174	7,003	7,003	7,003
21	0	9	10	11	0	174	187	201
22	2	11	34	45	42	267	914	1,278
23	1	4	4	4	13	108	108	108
24	0	0	0	4	0	0	0	108
26	0	0	6	6	0	0	87	87
Totals	75	188	343	441	2,190	28,568	48,926	54,556

2.4 Agricultural Inventory Crop Data

In addition to structural damages, the Kootenai River Valley is susceptible to agricultural flood losses. Acreage of floodplain lands in agricultural production was inventoried. The inventory is presented below in two sections; general crops and specialized crops (hops).

2.4.1 General Crops

The harvested general crops in the flood plain area are primarily represented by spring wheat, winter wheat, barley, canola and grass seed. Exact locations of each crop and number of acres in production have varied over the years in terms or rotation and harvested crop. Harvested acres for each crop for the years 1999-2003 were taken from the Idaho Agricultural Statistic Service for Boundary County. Based on the annual data, the average distribution for these crops was determined for the study area. These are shown in **Table C-3**.

Table C-3. Representative Crop Distribution in the Study Area

Crop	Percent of Total Field Crops in The Flood plain Area	
	Mean	Standard Deviation
Winter Wheat	40.3 %	4.0 %
Spring Wheat	26.0 %	4.5 %
Barley	19.4 %	3.7 %
Grass seed	12.0 %	0.6 %
Canola	2.3 %	0.1 %

Total acreage in each storage area was determined using GIS. Areas in agricultural production were identified using aerial photos and inventoried as a percent of the total acres in the storage area. Potential acres inundated were measured from one meter contours developed from the DEM. Acres between contours were identified at the mid-point to determine general acres at risk at varying elevations. The number of acres in agricultural production (based on the five crops listed in **Table C-3**) at key elevations are shown in **Table C-4** by storage area.

2.4.2 Specialty Crops (Hops)

Hops are an additional crop grown in the region. Due to their unique value, hops were treated as a separate specialty crop (located in storage areas 9 and 18.) Combined these two storage areas added an additional 1,700 acres (Storage Area 9= 1,200 acres, Storage Area 18= 500 acres) of agricultural inventory in hops.

Table C-4. Agricultural Acres Inundated in Each Storage Area by Elevation

Storage Area	Harvested Acres at Given Flood Plain Elevations (General Crops-Does not Include Hops)				
	1757 feet	1760 feet	1763 feet	1770 feet	1780 feet
1	513	806	1,119	1,303	1,379
2	1,804	2,261	2,342	2,548	2,692
3	1,043	1,553	1,727	1,767	1,784
4	0	104	215	3,809	4,468
5	0	295	870	974	1,023
7	0	654	1,563	2,497	2,550
8	0	43	324	891	897
10	13	23	27	36	43
11	0	0	0	142	167
12	0	386	1,156	1,180	1,192
13	0	657	1,112	1,660	1,691
14	1,446	2,231	3,688	4,914	4,965
15	0	0	7	206	242
16	34	45	82	161	184
17	0	0	49	1,256	1,358
18	1,742	1,813	1,832	1,861	1,871
19	45	59	64	64	64
20	182	266	294	298	301
21	520	905	1,124	1,358	1,367
22	0	0	1	7	19
23	0	0	0	112	771
24	0	0	0	0	245
25	0	0	0	0	8
26	0	0	0	3	8
Total	7,342	12,101	17,596	27,047	29,289

3.0 CALCULATION OF SINGLE EVENT DAMAGES

3.1 Structural Damage Estimates

Magnitude of loss to structures and their contents are directly related to the depth of flooding relative to the first floor. As depths increase, damages increase. In the model, flood elevation is compared with first floor elevation to determine depth. Damages are then a function of value times the percent loss at the indicated depth.

3.2 Depth Percentage Damage Curves

Residential depth-damage functions were taken from Economic Guidance Memorandum (EGM) 04-01, Generic Depth-Damage Relationships. Percent depth-damage curves were used for both structure and content and for residences determined with and without basements. Structure depth-damage functions for commercial, public, farm buildings and mobile homes were based on adjusted 1998 Federal Emergency Management Agency (FEMA) National Flood Insurance Program data. Commercial and Public content curves were derived from linear regression functions using the site specific survey data from the 1987 Bonners Ferry study. Farm building and Mobile Home content functions were based on findings from similar district studies. Depth-damage curves, by land use can be found in **Table C-**5.Uncertainties in percent loss were based on estimates found in EGM 04-01.

Table C-5. Depth-Damage Functions

Depth	Residential W/O Basement	Residential With Basement	Mobile Home	Commercial	Public	Farm Building
	Structure					
1	23 %	32 %	43 %	16 %	16 %	16 %
2	32 %	39 %	58 %	25 %	25 %	25 %
5	53 %	59 %	78 %	31 %	31 %	31 %
8	67 %	74 %	80 %	43 %	43 %	43 %
12	77 %	81 %	80 %	47 %	47 %	47 %
15	80 %	81 %	80 %	50 %	50 %	50 %
	Content[1]					
1	26 %	38 %	27 %	18 %	11 %	17 %
2	36 %	44 %	45 %	31 %	18 %	28 %
5	58 %	60 %	74 %	69 %	40 %	39 %
8	71 %	73 %	77 %	107 %	63 %	52 %
12	79 %	78 %	77 %	107 %	63 %	60 %
15	80 %	78 %	77 %	107 %	63 %	60 %

[1] Commercial and public content percent was determined as a percentage of structure value. Commercial content damages above 100% represent that total content value can be greater than structure value. Residential content percents were modified to represent percent loss of content value not structure value (assuming content 50% of structure value.)

3.3 Crop Losses - General Crops

Agricultural losses for general crops were calculated based on estimated loss per acre for each crop times the number of acres inundated. Losses per acre were determined using data gathered from the University of Idaho Northern Idaho Crop Costs and Return Estimates (College of Agricultural and Life Sciences.) Flood losses for a typical flood event were estimated for Winter Wheat, Spring Wheat, Barley, Canola, and Grass Seed.

In contrast to structural damages, crop losses are affected more by time year (month of flood event relative to plantings and harvest) and duration of inundation. Based on average depths and durations expected in the floodplain, floods prior to harvest would cause total loss in yield. Provided monthly probabilities indicated that floods were most likely to occur in May (30%) June (63%) and July (7 %.)

An example of the methodology used to estimate losses per acre are shown for Winter Wheat as a representative crop and are shown in **Table C-6**. Flood losses for the remaining general use crops were determined using a similar methodology.

Table C-6. Flood Losses for Winter Wheat

		Mean	***S.D.***		***1996***	***1997***	***1998***	***1999***	***2000***	***2001***	***2002***	***2003***
Yield		*75*	*8.45*		*78.7*	*61*	*68.5*	*65.1*	*94.3*	*75.5*	*82.8*	*87.0*
Price		*3.5*	*0.27*									
Gross Income		*$262.54*	*36.07*									
Total Production Cost		*$189.32*	*22.42*									
Net Income		*$73.22*	*42.38*									
Flood Weights			*0*	*0*	*0*	*0*	*0*	*0*	*0.3*	*0.63*	*0.07*	
Month:	**Sep**	**Oct**	**Nov**	**Dec**	**Jan**	**Feb**	**Mar**	**Apr**	**May**	**Jun**	**Jul**	**Aug**
Variable Cost Expended	80.35	0.78	0.78	0.78	0.78	0.78	0.78	17.79	0.78	0.78	0.78	15.61
Cumulative Cost		81.13	81.91	82.69	83.47	84.25	85.03	102.82	103.60	104.38	105.16	120.77
Losses (Cost + NI)		154.31	155.09	155.87	156.65	157.43	158.21	176.00	176.78	177.56	178.34	193.95
Weighted Loss									53.03	111.86	12.48	
Total Weighted Loss		Calculated =		$177.37		Monte Carlo Results Mean = $177.90, SD = $41.42						

The same process was performed for each crop. Monte Carlo Simulations were performed to determine mean and standard deviation of crop loss based on uncertainty in yield, price paid and production costs. **Table C-7** lists the results for each crop and the weighted average based on crop distribution from **Table C-3**. Additional losses in clean up costs bring the total average loss per inundated acre at $342.

Table C-7. Damage per Acre by Crop Type

Category	Mean Damage per acre	Standard Deviation
Winter Wheat	178	41
Spring Wheat	148	18
Barley	102	9
Seed Grass	88	39
Canola	101	10
Weighted Average	142	21
Clean Up Costs	200	16
Total Agricultural Loss	342	26

3.4 Crop Losses - Hops

Potential losses to hops are significantly greater than any of the general crops listed in **Table C-7**. Higher production costs plus greater losses in net income lead to larger damages from lost yield following flood inundation. In addition to losses during the flood year, accelerated reestablishment costs will be required as plants would die from extended duration of inundation. These plants normally have an average life expectancy of around twenty years and are replanted on a rotating basis. The flooding would accelerate this replanting schedule and add to the cost of production. Yields in immediate years following replanting would be reduced leading to further losses. Due to the competitive nature and limited growers in the area, exact costs, prices and net income are not explicitly reported in this document as per agreement with the hop producer.

Losses were estimated for two flooding conditions. One for shallow short duration events that would not cause permanent damage but would reduce yield for the year of the event, and the second for deeper longer duration events requiring reestablishment. Losses for the small event were estimated to have a mean value of $1,594 and a standard deviation of $71 per acre. Losses for the larger event, to include flood year loss of production, advanced reestablishment, and year one and two (after replanting) reductions in yield, were estimated to have a mean value of $9,400 and a standard deviation of $343 per acre.

3.5 Emergency Costs

Emergency costs were estimated to account for two types of flood losses beyond structural damages, Temporary Rental Assistance (TRA) and Public Assistance (PA). Losses for these categories were taken from other Seattle District studies (such as Skagit River and Chehalis River flood damage reduction studies) and were based on averages from national FEMA disaster reports. TRA was estimated to have a mean value of $1,500 and a standard deviation of $400 per household inundated. PA was estimated as a function of 3 times TRA per household. Total emergency costs were then estimated based on the number of residential structures inundated, at each stage event, multiplied by the TRA and PA costs per household.

3.6 Transportation Delays

There are two transportation lines in Bonners Ferry that could experience losses due to delays caused by flooding. The first is the BNSF rail line that runs through Bonners Ferry with travel through Washington and Montana. On average, thirty trains per day run through Bonners Ferry. With the line interrupted due to flooding, the re-routing could range from 300 to 500 miles depending on various final destinations (not all of the thirty re-routed trains would follow the same detour route.) Losses would include increased variable operating costs for the additional miles and time loss in terms of extra labor hours expended. Uncertainties in operating costs per mile, wage rates, miles traveled, duration, and trips per day were considered in determining losses per day. Losses for single flood events would then be a function of duration of flooding times total losses per day. Mean and standard deviation of railroad traffic delay costs are shown in **Table C-8**.

Note: These losses are determined based on duration. Assignment of the corresponding stages will require hydraulic information in terms of exterior (in channel) stages. When this hydraulic data is available, development of transportation delay stage-damage functions can be easily completed. To complete just link the damages from the duration damage curves provided to the stage-duration function.

Table C-8. Railroad Delay Losses in $1,000's

	Mean	**Standard Deviation**
Additional Travel Operating Costs Per Day	189	51
Additional Time Cost Per Day	53	24
Total Delay Losses Per Day	242	62
Losses – One Day Duration	242	72
Losses – Three Day Duration	726	216
Losses – Five Day Duration	1,210	361
Losses – Ten Day Duration	2,420	723
Losses – Fifteen Day Duration	3,631	1,085
Losses – Twenty Day Duration	4,842	1,450

The second transportation line subject to delays is U.S. Route 95. Closure of U.S. Route 95 at Bonners Ferry would require trucks and autos to be re-routed east on U.S. Route 200 into Montana, then north on State Route 56 to U.S. Route 2 back into Idaho. The total additional miles per trip would be around 72 miles. And with an average of 1,900 vehicles per day (source- Idaho Transportation Department), total additional miles traveled per day would be over 137,000 miles. Uncertainties included number of miles re-routed, number of vehicles, variable costs per mile (0.204 for autos and 0.48 for trucks taken from the Centralia Flood Damage Reduction Project Chehalis River, Washington) hours lost and rate per hour ($12.5 per hour taken from the Chehalis River study). Monte Carlo simulations were run and mean and standard deviations for transportation losses associated with closure of U.S. Route 95 at Bonners Ferry are found in **Table C-9**.

Table C-9. Travel Delay from U.S. Route 95 Closure at Bonners Ferry in $1,000's

	Mean	Standard Deviation
Additional Travel Operating Costs Per Day	32	5
Additional Time Cost Per Day	39	6
Total Delay Losses Per Day	71	8
Losses – One Day Duration	71	10
Losses – Three Day Duration	212	29
Losses – Five Day Duration	353	48
Losses – Ten Day Duration	706	97
Losses – Fifteen Day Duration	1,060	145
Losses – Twenty Day Duration	1,413	193

4.0 DRAFT STAGE/DAMAGE FUNCTIONS

Expected damages by category and by storage area were calculated for single events according to varying water surface elevations. Damage calculations for each storage area assume that the interior stage is uniform throughout the entire storage area. Different storage areas may have different interior stages for the same frequency or discharge event. For use in HEC-FDA, these interior stages in the individual flood plains need to be linked at some index points (to be determined) to exterior stages in the river channel. The interior and exterior stages may not be equal. These damages correspond to theoretical depths in the flood plain. Damage estimates at the lowest stages may not actually occur. Levees or top of bank elevations may be greater than the interior stages listed. Damages were calculated independent of any hydraulic or geotechnical restriction. For each category, Monte Carlo simulations were run with probability distributions for the variables used in the calculations to account for uncertainty. Damages are reported here in terms of mean and standard deviation assuming a normal distribution. All of the interior stage-damage functions by storage area are displayed in **Tables C-10 to C-35**.

5.0 FINALIZATION OF STAGE/DAMAGE FUNCTIONS

As mentioned in Section 2.1 above, stage-damage functions were developed prior to the final hydraulic data. With the economic damage assessment complete the only task left to finalize these damage functions is to link the interior stage to exterior stage in the channel at selected index points for use in the HEC-FDA model. The stage damage functions listed here are complete from an economic perspective. Addition of hydrology, hydraulics and geo-technical levee data are all that is needed to complete the HEC-FDA model. The stage damage functions listed below in **Tables C-10 to C-35** can be directly entered in the HEC-FDA model and later linked to exterior stage using the "Levee Features – Exterior/Interior Relationship" menu in the model.

Table C-10. Storage Area 1 Stage-Damage ($1,000's)

Interior Stage (in flood plain)	BASIC CROPS		FARMBUILDING		RESIDENTIAL		EMERGENCY	
	Mean	Standard Deviation	Mean	Standard Deviation	Mean	Standard Deviation	Mean	Standard Deviation
1752	95.6	9.1	0	0	0	0	0	0
1753	109.1	10.4	0.0	0.0	1.7	2.8	0.0	0.0
1754	122.6	11.6	0.0	0.0	3.5	5.6	0.0	0.0
1755	136.0	12.9	1.6	0.5	8.6	6.4	1.6	1.6
1756	149.5	14.2	3.2	1.0	13.8	7.2	3.3	3.2
1757	175.8	16.7	4.6	1.3	19.2	7.1	4.6	2.4
1758	202.1	19.2	6.1	1.5	24.6	7.0	6.0	1.6
1759	228.4	21.8	7.5	1.8	32.5	8.6	6.1	1.8
1760	276.4	26.2	8.8	2.1	40.5	10.2	6.3	2.0
1761	324.3	30.6	17.8	2.8	49.7	10.6	8.5	2.7
1762	372.3	35.0	26.7	3.4	58.9	11.1	10.7	3.3
1763	383.7	36.2	34.4	4.2	67.0	11.4	11.4	2.8
1764	395.2	37.3	42.0	5.0	75.2	11.8	12.0	2.3
1765	406.6	38.5	52.3	5.5	81.7	12.2	12.1	2.3
1766	414.8	39.3	62.6	6.1	88.1	12.7	12.1	2.4
1767	423.1	40.1	80.1	7.1	96.1	13.2	15.1	2.6
1768	431.3	40.9	97.6	8.2	104.0	13.8	18.0	2.8
1769	439.5	41.6	115.1	9.0	117.0	15.7	18.6	3.3
1770	446.8	42.4	132.7	9.8	130.0	17.6	19.1	3.7
1771	454.1	43.1	147.9	10.5	149.6	17.5	26.6	4.3
1772	461.4	43.9	163.0	11.2	169.2	17.5	34.0	4.9
1773	463.6	44.0	172.5	11.8	184.2	17.7	35.0	4.4
1774	465.8	44.2	182.0	12.4	199.1	17.8	36.0	3.9
1775	467.9	44.4	190.5	13.0	208.8	18.2	36.0	3.9
1776	468.9	44.5	199.0	13.5	218.5	18.5	36.0	3.9
1777	469.8	44.6	203.5	13.7	225.6	19.0	36.0	3.9
1778	470.7	44.7	208.1	14.0	232.6	19.4	36.0	3.9
1779	471.7	44.8	210.5	14.1	237.6	19.8	36.0	3.9
1780	472.9	44.8	212.9	14.2	242.5	20.1	36.0	3.9
1781	474.1	44.8	213.9	14.3	245.7	20.3	36.0	3.9
1782	475.3	44.8	214.9	14.4	249.0	20.5	36.0	3.9

Table C-11. Storage Area 2 Stage-Damage ($1,000's)

Interior Stage	BASIC CROPS		COMMERCIAL		FARMBUILDING		PUBLIC		RESIDENTIAL		EMERGENCY	
	Mean	Standard Deviation	Mean	Standard Deviation	Mean	Standard Deviation	Mean	Standard Deviation	Mean	Standard Deviation	Mean	Standard Deviation
1752	388.1	36.7	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
1753	413.1	39.0	6.5	3.8	1.0	0.3	0.0	0.0	2.0	2.9	0.0	0.0
1754	457.2	43.1	12.9	7.6	2.0	0.7	0.0	0.0	4.0	5.9	0.0	0.0
1755	501.2	47.1	32.7	10.4	4.6	1.2	0.0	0.0	9.2	6.5	1.8	1.6
1756	545.3	51.2	52.6	13.2	7.2	1.7	0.0	0.0	14.3	7.2	3.5	3.2
1757	618.7	58.4	66.9	16.1	8.0	1.8	0.0	0.0	19.7	7.0	4.8	2.4
1758	692.2	65.7	81.3	19.0	8.7	2.0	0.0	0.0	25.1	6.9	6.0	1.6
1759	765.7	72.9	133.1	26.0	9.7	2.2	0.0	0.0	29.8	7.3	6.0	1.6
1760	775.4	73.9	184.8	33.0	10.7	2.5	0.0	0.0	34.5	7.6	6.0	1.6
1761	785.1	74.8	278.1	42.7	11.5	2.7	5.8	5.6	38.1	8.1	6.0	1.6
1762	794.8	75.7	371.4	52.3	12.3	2.8	11.7	11.3	41.8	8.5	6.0	1.6
1763	803.0	76.4	879.0	81.0	14.4	3.0	140.3	31.3	44.6	8.9	6.0	1.6
1764	811.1	77.1	1386.6	109.7	16.5	3.2	268.9	51.4	47.5	9.4	6.0	1.6
1765	819.3	77.8	2338.4	146.6	20.0	3.5	414.9	62.5	49.4	9.7	6.0	1.6
1766	827.5	78.5	3290.1	183.5	23.5	3.7	560.9	73.6	51.3	10.0	6.0	1.6
1767	840.9	79.8	4232.9	216.5	33.0	4.1	675.5	85.3	55.1	10.9	6.0	1.6
1768	854.4	81.1	5175.8	249.5	42.4	4.5	790.2	97.0	58.8	11.7	6.0	1.6
1769	867.8	82.4	6239.2	289.6	56.2	5.2	1012.4	125.9	64.2	11.6	8.8	2.2
1770	873.9	83.0	7302.7	329.8	69.9	5.9	1234.5	154.8	69.5	11.5	11.5	2.8
1771	880.0	83.7	8369.0	358.5	77.7	6.4	1572.7	169.8	74.5	11.5	11.8	2.5
1772	886.1	84.4	9435.3	387.2	85.4	6.9	1910.9	184.8	79.5	11.5	12.0	2.3
1773	890.2	84.7	10145.7	403.8	93.4	7.4	2237.6	198.7	83.7	11.8	12.0	2.3
1774	894.4	85.0	10856.1	420.4	101.5	8.0	2564.3	212.6	88.0	12.0	12.0	2.3
1775	898.6	85.3	11357.4	432.4	107.6	8.4	2890.5	235.5	91.1	12.3	12.0	2.3
1776	902.7	85.6	11858.7	444.3	113.8	8.8	3216.8	258.5	94.2	12.6	12.0	2.3
1777	908.0	85.9	12216.6	455.2	117.4	9.1	3498.8	273.2	96.7	12.8	12.0	2.3
1778	913.2	86.3	12574.6	466.0	121.0	9.3	3780.9	287.9	99.2	13.1	12.0	2.3
1779	918.5	86.6	12714.6	469.9	123.4	9.4	3949.8	294.8	100.9	13.3	12.0	2.3
1780	922.9	87.4	12854.6	473.9	125.9	9.6	4118.6	301.7	102.6	13.5	12.0	2.3
1781	927.4	88.1	12897.4	474.8	128.4	9.7	4198.7	305.1	103.3	13.5	12.0	2.3
1782	931.9	88.9	12940.2	475.8	131.0	9.9	4278.8	308.5	104.1	13.5	12.0	2.3

Table C-12. Storage Area 2B Stage-Damage ($1,000's)

Interior Stage	FARMBUILDING		PUBLIC		RESIDENTIAL		EMERGENCY	
	Mean	Standard Deviation	Mean	Standard Deviation	Mean	Standard Deviation	Mean	Standard Deviation
1758	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
1759	0.0	0.0	0.0	0.0	7.8	2.8	2.3	1.5
1760	0.0	0.0	0.0	0.0	15.6	5.7	4.5	3.1
1761	2.5	0.5	0.0	0.0	58.4	10.5	13.5	4.4
1762	5.0	0.9	0.0	0.0	101.3	15.4	22.5	5.8
1763	10.8	1.6	0.0	0.0	188.8	18.9	44.0	6.7
1764	16.7	2.2	0.0	0.0	276.4	22.4	65.5	7.7
1765	20.6	2.6	0.0	0.0	384.2	25.7	82.6	7.7
1766	24.5	2.9	0.0	0.0	492.0	28.9	99.8	7.7
1767	30.4	3.4	0.0	0.0	636.0	32.2	123.0	9.0
1768	36.3	3.8	0.0	0.0	780.0	35.4	146.2	10.3
1769	41.5	4.3	27.3	8.8	909.6	38.4	157.5	9.6
1770	46.7	4.7	54.5	17.5	1039.1	41.3	168.9	8.8
1771	51.5	5.0	86.7	19.3	1149.6	43.6	176.4	9.1
1772	56.3	5.4	118.9	21.0	1260.0	45.9	183.8	9.4
1773	59.2	5.6	152.8	24.2	1356.4	48.5	186.7	9.5
1774	62.1	5.8	186.6	27.5	1452.8	51.2	189.6	9.6
1775	64.3	6.0	219.8	31.0	1533.8	53.1	194.2	9.5
1776	66.4	6.2	253.0	34.6	1614.8	55.1	198.8	9.3
1777	67.9	6.3	274.9	36.2	1672.7	56.0	204.4	9.4
1778	69.3	6.5	296.9	37.8	1730.6	56.9	210.0	9.4
1779	70.0	6.5	308.6	38.3	1768.1	57.7	210.0	9.4
1780	70.7	6.6	320.3	38.8	1805.5	58.5	210.0	9.4
1781	70.9	6.6	324.0	39.2	1829.3	59.0	210.0	9.4
1782	71.2	6.6	327.7	39.5	1853.0	59.5	210.0	9.4

Table C-13. Storage Area 2C Stage-Damage ($1,000's)

Interior Stage	COMMERCIAL	
	Mean	Standard Deviation
1766	0.0	0.0
1767	5.3	10.5
1768	10.6	21.0
1769	130.9	59.2
1770	251.1	97.4
1771	394.3	107.8
1772	537.4	118.1
1773	647.1	135.8
1774	756.8	153.5
1775	888.6	177.5
1776	1020.4	201.5
1777	1090.5	212.0
1778	1160.7	222.4
1779	1170.1	223.8
1780	1179.4	225.2
1781	1186.1	226.2
1782	1192.9	227.2
1783	1199.3	229.0
1784	1205.8	230.8
1785	1206.3	230.9
1786	1206.9	231.0

Table C-14. Storage Area 3 Stage-Damage ($1,000's)

Interior Stage	BASIC CROPS		PUBLIC		RESIDENTIAL		EMERGENCY	
	Mean	Standard Deviation	Mean	Standard Deviation	Mean	Standard Deviation	Mean	Standard Deviation
1755	210.8	20.0	0.0	0.0	0.0	0.0	0.0	0.0
1756	284.3	27.0	0.0	0.0	0.0	0.0	0.0	0.0
1757	357.8	33.9	0.0	0.0	0.0	0.0	0.0	0.0
1758	431.3	40.9	0.0	0.0	0.0	0.0	0.0	0.0
1759	481.9	45.8	0.0	0.0	0.0	0.0	0.0	0.0
1760	532.5	50.7	0.0	0.0	0.0	0.0	0.0	0.0
1761	583.2	55.6	1.6	0.3	0.0	0.0	0.0	0.0
1762	587.6	55.8	3.2	0.7	0.0	0.0	0.0	0.0
1763	592.1	55.9	5.5	1.2	0.0	0.0	0.0	0.0
1764	596.6	56.1	7.8	1.6	0.0	0.0	0.0	0.0
1765	598.1	56.4	9.3	1.9	0.0	0.0	0.0	0.0
1766	599.5	56.6	10.7	2.2	0.0	0.0	0.0	0.0
1767	601.0	56.8	12.5	2.6	0.0	0.0	0.0	0.0
1768	602.5	57.0	14.3	3.0	0.0	0.0	0.0	0.0
1769	604.2	57.2	15.5	3.2	0.0	0.0	0.0	0.0
1770	605.8	57.4	16.8	3.5	0.0	0.0	0.0	0.0
1771	607.5	57.6	17.8	3.6	0.0	0.0	0.0	0.0
1772	608.1	57.7	18.9	3.6	0.0	0.0	0.0	0.0
1773	608.7	57.8	20.2	3.7	0.0	0.0	0.0	0.0
1774	609.3	57.9	21.4	3.8	0.0	0.0	0.0	0.0
1775	609.6	57.9	22.3	3.9	0.0	0.0	0.0	0.0
1776	610.0	57.9	23.2	4.0	0.0	0.0	0.0	0.0
1777	610.3	57.9	24.2	4.2	0.0	0.0	0.0	0.0
1778	610.7	57.9	25.2	4.3	0.0	0.0	0.0	0.0
1780	611.3	58.0	27.5	4.4	0.0	0.0	0.0	0.0
1782	611.9	58.1	29.8	4.5	0.0	0.0	0.0	0.0
1783	612.5	58.2	31.3	4.6	8.0	2.7	1.3	1.6
1784	613.1	58.1	32.9	4.6	15.9	5.3	2.6	3.1
1785	613.6	58.0	34.2	4.7	27.6	6.8	4.3	2.4
1786	614.2	58.0	35.6	4.8	39.3	8.2	6.0	1.6

Table C-15. Storage Area 4 Stage-Damage ($1,000's)

Interior Stage	BASIC CROPS		FARMBUILDING		PUBLIC		RESIDENTIAL		EMERGENCY	
	Mean	Standard Deviation	Mean	Standard Deviation	Mean	Standard Deviation	Mean	Standard Deviation	Mean	Standard Deviation
1762	61.0	5.8								
1763	73.7	7.0								
1764	167.0	15.9	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
1765	260.4	24.8	0.0	0.0	0.0	0.0	3.1	1.6	0.1	0.6
1766	353.7	33.7	0.0	0.0	0.0	0.0	6.2	3.3	0.2	1.2
1767	659.3	62.3	0.6	0.3	0.0	0.0	14.8	4.2	3.1	1.4
1768	964.8	90.9	1.3	0.7	0.0	0.0	23.3	5.1	6.0	1.6
1769	1270.3	119.5	3.5	1.4	0.0	0.0	35.3	7.3	6.0	1.6
1770	1306.1	123.1	5.7	2.1	0.0	0.0	47.2	9.5	6.0	1.6
1771	1342.0	126.8	10.7	2.9	0.0	0.0	76.7	11.1	13.9	3.2
1772	1377.8	130.4	15.7	3.7	0.0	0.0	106.2	12.7	21.9	4.8
1773	1397.8	132.3	19.6	4.3	3.2	0.7	172.9	17.7	29.7	5.4
1774	1417.9	134.3	23.6	5.0	6.4	1.4	239.5	22.8	37.5	5.9
1775	1437.9	136.2	30.4	5.7	12.9	2.0	331.7	24.2	60.1	6.5
1776	1457.9	138.2	37.2	6.3	19.3	2.5	423.9	25.7	82.6	7.0
1777	1479.1	140.3	45.4	7.1	24.8	2.8	527.6	29.3	89.8	7.2
1778	1500.3	142.5	53.7	7.8	30.2	3.2	631.2	33.0	97.0	7.3
1779	1521.4	144.7	61.6	8.5	35.7	3.7	731.9	35.9	104.1	7.4
1780	1532.0	145.6	69.5	9.2	41.2	4.3	832.6	38.8	111.1	7.5
1781	1542.6	146.4	75.1	9.7	45.6	4.7	941.1	41.9	120.1	7.9
1782	1553.2	147.3	80.8	10.2	49.9	5.1	1049.6	44.9	129.1	8.3
1783	1560.8	148.1	86.7	10.7	62.8	7.3	1146.5	46.9	140.9	8.5
1784	1568.3	148.8	92.7	11.2	75.7	9.5	1243.5	48.8	152.8	8.7
1785	1575.8	149.6	96.1	11.6	86.2	10.1	1329.8	50.9	156.7	8.7
1786	1583.3	150.4	99.6	11.9	96.6	10.7	1416.2	53.1	160.5	8.7
1787	1591.1	150.8	102.8	12.1	104.2	12.1	1485.9	54.6	164.2	8.5
1788	1598.9	151.2	105.9	12.3	111.9	13.4	1555.7	56.1	168.0	8.4
1789	1606.7	151.6	108.2	12.5	121.6	15.0	1613.7	57.4	170.1	8.7
1790	1612.4	152.1	110.4	12.6	131.2	16.5	1671.7	58.8	172.2	9.0
1791	1618.1	152.7	111.3	12.7	135.1	17.0	1724.3	60.1	176.5	9.0
1792	1623.8	153.2	112.3	12.8	139.1	17.6	1776.9	61.4	180.8	8.9

Table C-16. Storage Area 5 Stage-Damage ($1,000's)

Interior Stage	BASIC CROPS		FARMBUILDING		RESIDENTIAL		EMERGENCY	
	Mean	Standard Deviation	Mean	Standard Deviation	Mean	Standard Deviation	Mean	Standard Deviation
1759	7.1	0.7	0.0	0.0	0.0	0.0	0.0	0.0
1760	101.0	9.6	0.0	0.0	0.0	0.0	0.0	0.0
1761	194.9	18.6	0.0	0.0	0.0	0.0	0.0	0.0
1762	288.8	27.5	0.0	0.0	0.0	0.0	0.0	0.0
1763	298.2	28.3	0.0	0.0	0.0	0.0	0.0	0.0
1764	307.7	29.1	0.0	0.0	0.0	0.0	0.0	0.0
1765	317.2	29.9	0.0	0.0	0.0	0.0	0.0	0.0
1766	320.7	30.2	0.0	0.0	0.0	0.0	0.0	0.0
1767	324.2	30.5	0.3	0.1	0.0	0.0	0.0	0.0
1768	327.6	30.8	0.6	0.2	0.0	0.0	0.0	0.0
1769	331.1	31.1	5.6	0.8	11.9	2.6	3.0	0.8
1770	334.1	31.5	10.6	1.5	23.8	5.2	6.0	1.6
1771	337.1	31.8	17.8	2.2	28.6	5.9	6.0	1.6
1772	340.1	32.1	25.0	2.8	33.5	6.5	6.0	1.6
1773	341.9	32.4	29.8	3.2	37.0	7.2	6.0	1.6
1774	343.6	32.6	34.5	3.6	40.4	7.8	6.0	1.6
1775	345.4	32.9	41.7	4.1	43.6	8.4	6.0	1.6
1776	346.4	32.9	48.9	4.6	46.9	9.0	6.0	1.6
1777	347.4	32.9	55.6	5.0	57.0	10.0	9.0	1.9
1778	348.4	33.0	62.2	5.5	67.1	10.9	12.0	2.3
1779	349.4	33.0	71.2	5.9	73.8	11.2	12.0	2.3
1780	350.7	33.2	80.3	6.3	80.5	11.5	12.0	2.3
1781	352.1	33.4	85.2	6.6	85.5	11.9	12.0	2.3
1782	353.4	33.6	90.2	6.9	90.6	12.3	12.0	2.3
1783	354.7	33.6	96.8	7.3	94.5	12.8	12.0	2.3
1784	355.9	33.7	103.4	7.7	98.4	13.3	12.0	2.2
1785	357.1	33.7	114.9	8.2	100.7	13.6	12.0	2.3
1786	358.3	33.8	126.4	8.8	102.9	13.9	12.0	2.3
1787	359.4	33.9	132.7	9.1	130.0	15.9	17.2	3.2
1788	360.5	34.0	139.1	9.4	157.0	17.9	22.4	4.1

Table C-17. Storage Area 7 Stage-Damage ($1,000's)

Interior Stage	BASIC CROPS		COMMERCIAL		FARMBUILDING		RESIDENTIAL		EMERGENCY	
	Mean	Standard Deviation	Mean	Standard Deviation	Mean	Standard Deviation	Mean	Standard Deviation	Mean	Standard Deviation
1759	144.8	13.8	1.0	0.2	3.3	0.7	0.0	0.0	0.0	0.0
1760	224.4	21.3	2.1	0.4	6.5	1.4	0.0	0.0	0.0	0.0
1761	303.9	28.8	6.3	1.1	13.8	2.7	0.0	0.0	0.0	0.0
1762	383.4	36.4	10.5	1.8	21.1	4.1	0.0	0.0	0.0	0.0
1763	536.0	51.0	14.8	2.1	24.3	4.6	4.2	3.6	0.0	0.0
1764	688.6	65.6	19.1	2.5	27.5	5.1	8.5	7.1	0.0	0.0
1765	841.2	80.2	22.6	3.0	31.1	5.8	15.7	7.9	0.0	0.0
1766	844.9	80.2	26.1	3.4	34.7	6.4	22.9	8.6	0.0	0.1
1767	848.6	80.2	30.0	3.9	37.1	6.9	33.0	8.9	1.9	1.6
1768	852.2	80.2	33.8	4.4	39.4	7.3	43.2	9.2	3.8	3.2
1769	854.2	80.5	35.7	4.7	41.4	7.6	52.4	9.5	4.9	2.4
1770	856.1	80.8	37.5	4.9	43.5	8.0	61.5	9.8	6.0	1.6
1771	858.1	81.1	37.8	5.0	44.1	8.1	69.2	10.5	6.0	1.6
1772	860.0	81.4	38.0	5.0	44.7	8.2	76.8	11.2	6.0	1.6
1773	862.3	81.6	38.3	5.0	46.8	8.3	82.4	11.8	6.0	1.6
1774	864.6	81.9	38.5	5.0	48.9	8.4	88.0	12.4	6.0	1.6
1775	866.8	82.2	38.7	5.0	49.6	8.5	92.2	12.8	6.0	1.6
1776	868.7	82.4	38.8	5.1	50.3	8.5	96.4	13.3	6.0	1.6
1777	870.6	82.7	38.8	5.0	50.8	8.6	98.9	13.5	9.0	1.9
1778	872.4	83.0	38.8	5.0	51.2	8.7	101.3	13.6	12.0	2.3
1779	873.5	83.0	38.8	5.0	51.8	8.8	102.5	13.7	12.0	2.3
1780	874.6	83.1	38.8	5.0	52.3	8.8	103.7	13.9	12.0	2.2
1781	875.7	83.1	38.8	5.0	52.7	8.8	104.1	13.9	12.0	2.3
1782	876.7	83.1	38.8	5.0	53.0	8.8	104.6	14.0	12.0	2.3
1783	877.9	83.3	38.8	5.0	53.2	8.9	104.8	14.0	12.0	2.3
1784	879.0	83.4	38.8	5.0	53.3	8.9	104.9	14.0	12.0	2.3
1785	880.2	83.6	38.8	5.0	53.3	8.9	104.9	14.0	12.0	2.3

Table C-18. Storage Area 8 Stage-Damage ($1,000's)

Interior Stage	BASIC CROPS		FARMBUILDING		RESIDENTIAL		EMERGENCY	
	Mean	Standard Deviation	Mean	Standard Deviation	Mean	Standard Deviation	Mean	Standard Deviation
1758	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
1759	13.9	1.3	1.2	0.3	0.0	0.0	0.0	0.0
1760	14.6	1.4	2.4	0.6	0.0	0.0	0.0	0.0
1761	15.4	1.5	12.5	2.0	3.0	3.4	0.0	0.1
1762	16.1	1.5	22.5	3.4	6.0	6.9	0.0	0.3
1763	111.1	10.6	26.8	4.1	12.2	6.7	2.5	1.4
1764	206.1	19.6	31.0	4.8	18.3	6.5	5.1	2.6
1765	301.1	28.7	35.1	5.4	25.1	7.2	5.5	2.1
1766	302.4	28.7	39.1	5.9	31.9	7.9	6.0	1.6
1767	303.6	28.7	47.4	6.6	41.7	9.3	7.0	2.5
1768	304.8	28.7	55.7	7.4	51.5	10.7	8.0	3.4
1769	305.1	28.8	64.0	7.9	60.8	11.1	9.9	2.9
1770	305.4	28.8	72.3	8.5	70.1	11.5	11.8	2.5
1771	305.8	28.9	75.6	8.7	78.0	12.1	11.9	2.4
1772	306.1	29.0	78.9	8.9	85.9	12.6	12.0	2.2
1773	306.3	29.0	82.5	9.2	91.8	13.2	12.0	2.3
1774	306.5	29.0	86.1	9.5	97.7	13.7	12.0	2.3
1775	306.7	29.1	88.9	9.7	101.8	14.0	12.0	2.3
1776	306.9	29.1	91.6	9.9	105.9	14.4	12.0	2.3
1777	307.1	29.2	92.9	9.9	108.4	14.6	12.0	2.3
1778	307.3	29.2	94.2	10.0	110.9	14.9	12.0	2.3
1779	307.5	29.2	94.6	10.0	112.4	15.0	12.0	2.3
1780	307.6	29.2	95.1	10.0	113.9	15.1	12.0	2.2
1781	307.8	29.2	95.4	10.1	114.4	15.1	12.0	2.3
1782	307.9	29.2	95.8	10.1	115.0	15.2	12.0	2.3
1783	308.0	29.2	95.8	10.1	115.2	15.3	12.0	2.3
1784	308.1	29.3	95.9	10.1	115.4	15.3	12.0	2.3

Table C-19. Storage Area 9 Stage-Damage ($1,000's)

Interior Stage	HOPS		COMMERCIAL		FARMBUILDING		RESIDENTIAL		EMERGENCY	
	Mean	Standard Deviation	Mean	Standard Deviation	Mean	Standard Deviation	Mean	Standard Deviation	Mean	Standard Deviation
1755	91.3	3.3	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
1756	159.8	5.7	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
1757	228.3	8.1	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
1758	296.9	10.5	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
1759	365.4	12.9	57.7	7.4	0.0	0.0	0.0	0.0	0.0	0.0
1760	686.9	24.8	115.5	14.8	0.0	0.0	0.0	0.0	0.0	0.0
1761	1008.4	36.7	1277.9	74.7	0.0	0.0	0.0	0.0	0.0	0.0
1762	1329.9	48.6	2440.2	134.5	0.0	0.0	0.0	0.0	0.0	0.0
1763	2233.8	81.4	3390.1	186.6	0.0	0.0	0.0	0.0	0.0	0.0
1764	3137.7	114.3	4339.9	238.7	0.0	0.0	0.0	0.0	0.0	0.0
1765	4041.6	147.1	5165.0	283.4	0.0	0.0	0.0	0.0	0.0	0.0
1766	6007.0	237.7	5990.1	328.2	0.0	0.0	0.0	0.0	0.0	0.0
1767	7972.4	328.3	6943.6	380.0	0.0	0.0	0.0	0.0	0.0	0.0
1768	9937.8	418.9	7897.1	431.8	0.0	0.0	0.0	0.0	0.0	0.0
1769	10065.6	425.0	8198.9	447.6	0.0	0.0	0.0	0.0	0.0	0.0
1770	10193.4	431.0	8500.7	463.4	0.0	0.0	0.0	0.0	0.0	0.0
1771	10321.2	437.1	8549.8	467.0	0.6	0.2	0.0	0.0	0.0	0.0
1772	10449.1	443.2	8599.0	470.5	1.2	0.4	0.0	0.0	0.0	0.0
1773	10538.1	447.2	8646.1	473.3	2.4	0.7	0.0	0.0	0.0	0.0
1774	10627.1	451.3	8693.2	476.1	3.6	1.1	0.0	0.0	0.0	0.0
1775	10716.0	455.3	8716.1	477.9	5.2	1.3	0.0	0.0	0.0	0.0
1776	10716.1	456.0	8739.0	479.7	6.8	1.5	0.0	0.0	0.0	0.0
1777	10716.1	456.6	8739.7	479.6	8.2	1.7	6.9	2.7	3.0	0.8
1778	10716.1	457.3	8740.4	479.5	9.5	2.0	13.8	5.3	6.0	1.6
1779	10716.1	456.7	8740.2	479.9	10.1	2.1	19.3	5.3	6.0	1.6
1780	10716.1	456.1	8740.0	480.2	10.7	2.2	24.8	5.3	6.0	1.6
1781	10716.0	455.6	8740.2	479.6	11.7	2.4	29.6	6.0	6.0	1.6
1782	10716.0	455.0	8740.5	478.9	12.7	2.7	34.4	6.6	6.0	1.6
1783	10716.0	454.6	8740.1	477.5	15.3	2.8	38.3	7.4	6.0	1.6
1784	10716.0	454.1	8739.7	476.0	17.9	3.0	42.3	8.1	6.0	1.6
1785	10716.0	453.7	8740.1	478.2	19.9	3.1	45.2	8.7	6.0	1.6

Table C-20. Storage Area 10 Stage-Damage ($1,000's)

Interior Stage	BASIC CROPS	
	Mean	Standard Deviation
1765	0.2	0.0
1766	2.3	0.2
1767	4.3	0.4
1768	6.4	0.6
1769	7.1	0.7
1770	7.8	0.7
1771	8.4	0.8
1772	8.8	0.8
1773	9.1	0.9
1774	9.5	0.9
1775	9.8	0.9
1776	10.3	1.0
1777	10.9	1.0
1778	11.4	1.1
1779	11.9	1.1
1780	12.5	1.2
1781	13.0	1.2
1782	13.2	1.3
1783	13.5	1.3
1784	13.7	1.3
1785	13.9	1.3
1786	14.1	1.3
1787	14.4	1.4
1788	14.6	1.4
1789	14.7	1.4
1790	14.9	1.4
1791	15.1	1.4
1792	15.1	1.4
1793	15.2	1.4

Table C-21. Storage Area 11 Stage-Damage ($1,000's)

Interior Stage	BASIC CROPS		FARMBUILDING		RESIDENTIAL		EMERGENCY	
	Mean	Standard Deviation	Mean	Standard Deviation	Mean	Standard Deviation	Mean	Standard Deviation
1764	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
1765	38.1	3.6	0.0	0.0	0.0	0.0	0.0	0.0
1766	40.7	3.9	0.0	0.0	0.0	0.0	0.0	0.0
1767	43.2	4.1	0.0	0.0	0.0	0.0	0.0	0.0
1768	45.7	4.3	0.0	0.0	0.0	0.0	0.0	0.0
1769	47.1	4.5	0.0	0.0	0.0	0.0	0.0	0.0
1770	48.5	4.6	0.0	0.0	0.0	0.0	0.0	0.0
1771	49.9	4.8	0.0	0.0	0.0	0.0	0.0	0.0
1772	51.3	4.9	0.0	0.0	0.0	0.0	0.0	0.0
1773	52.4	5.0	0.0	0.0	0.0	0.0	0.0	0.0
1774	53.5	5.1	0.0	0.0	0.0	0.0	0.0	0.0
1775	54.5	5.2	0.0	0.0	0.0	0.0	0.0	0.0
1776	55.2	5.2	0.0	0.0	0.0	0.0	0.0	0.0
1777	55.9	5.3	0.0	0.0	0.0	0.0	0.0	0.0
1778	56.6	5.4	0.0	0.0	0.0	0.0	0.0	0.0
1779	57.0	5.4	0.0	0.0	0.0	0.0	0.0	0.0
1780	57.3	5.4	0.0	0.0	0.0	0.0	0.0	0.0
1781	57.7	5.5	0.0	0.0	0.0	0.0	0.0	0.0
1782	58.1	5.5	0.0	0.0	0.0	0.0	0.0	0.0
1783	58.4	5.5	0.0	0.0	0.0	0.0	0.0	0.0
1784	58.8	5.6	0.0	0.0	0.0	0.0	0.0	0.0
1785	59.2	5.6	0.0	0.0	0.0	0.0	0.0	0.0
1786	59.5	5.6	0.0	0.0	0.0	0.0	0.0	0.0
1787	59.8	5.7	0.0	0.0	0.6	0.4	0.0	0.0
1788	60.1	5.7	0.0	0.0	1.3	0.9	0.0	0.1
1789	60.4	5.7	2.1	0.5	5.5	1.6	3.0	0.9
1790	60.7	5.7	4.1	1.1	9.7	2.4	6.0	1.7
1791	61.0	5.8	7.2	1.6	13.5	2.9	6.0	1.6
1792	61.2	5.8	10.2	2.0	17.3	3.5	6.0	1.6
1793	61.4	5.8	15.7	2.5	20.3	4.0	6.0	1.6
1794	61.6	5.8	21.1	3.0	23.3	4.5	6.0	1.6
1795	61.9	5.9	26.0	3.5	25.4	4.9	6.0	1.6

Table C-22. Storage Area 12 Stage-Damage ($1,000's)

Interior Stage	BASIC CROPS		FARMBUILDING		RESIDENTIAL		EMERGENCY	
	Mean	Standard Deviation	Mean	Standard Deviation	Mean	Standard Deviation	Mean	Standard Deviation
1759	1.8	0.2	0.0	0.0	0.0	0.0	0.0	0.0
1760	132.5	12.6	0.0	0.0	0.0	0.0	0.0	0.0
1761	263.3	25.1	0.0	0.0	0.0	0.0	0.0	0.0
1762	394.1	37.5	0.0	0.0	0.0	0.0	0.0	0.0
1764	396.3	37.6	1.5	0.5	0.0	0.0	0.0	0.0
1765	398.5	37.7	2.6	0.9	0.0	0.0	0.0	0.0
1766	400.8	37.8	3.7	1.2	0.0	0.0	0.0	0.0
1767	401.6	37.9	4.1	1.3	0.0	0.0	0.0	0.0
1768	402.4	37.9	4.4	1.4	0.0	0.0	0.0	0.0
1769	403.2	37.9	5.0	1.6	0.0	0.0	0.0	0.0
1770	404.0	38.0	5.7	1.8	0.0	0.0	0.0	0.0
1771	404.6	38.1	6.0	1.9	19.0	5.8	3.9	1.8
1772	405.2	38.2	6.2	2.0	37.9	11.6	7.8	3.7
1773	405.8	38.3	6.5	2.1	52.8	11.5	9.9	3.0
1774	406.3	38.5	6.8	2.2	67.6	11.3	12.0	2.3
1775	406.8	38.6	8.1	2.4	80.4	12.6	12.0	2.3
1776	407.2	38.7	9.3	2.6	93.3	13.8	12.0	2.3
1777	407.5	38.7	11.5	3.0	102.9	15.1	12.0	2.3
1778	407.8	38.7	13.7	3.4	112.6	16.4	12.0	2.3
1779	408.1	38.6	14.9	3.8	120.6	17.5	12.0	2.3
1780	408.4	38.6	16.2	4.1	128.6	18.5	12.0	2.2
1781	408.8	38.7	18.6	4.5	133.7	19.0	12.0	2.2
1782	409.2	38.8	21.0	5.0	138.8	19.5	12.0	2.2
1783	409.6	38.9	22.9	5.5	141.7	19.9	12.0	2.2
1784	410.0	38.9	24.9	6.0	144.5	20.3	12.0	2.2
1785	410.4	38.8	27.4	6.3	145.6	20.3	12.0	2.2
1786	410.7	38.8	29.9	6.7	146.6	20.3	12.0	2.2
1787	411.0	38.8	32.8	7.0	147.1	20.4	12.0	2.2
1788	411.3	38.8	35.7	7.4	147.6	20.5	12.0	2.3
1789	411.6	38.8	37.4	7.7	147.7	20.5	12.0	2.1
1790	411.9	38.8	39.1	8.0	147.7	20.5	12.0	2.0

Table C-23. Storage Area 13 Stage-Damage ($1,000's)

Interior Stage	BASIC CROPS		FARMBUILDING		RESIDENTIAL		EMERGENCY	
	Mean	Standard Deviation	Mean	Standard Deviation	Mean	Standard Deviation	Mean	Standard Deviation
1754	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
1755	106.5	10.1	0.0	0.0	0.0	0.0	0.0	0.0
1756	127.0	12.1	0.0	0.0	0.0	0.0	0.0	0.0
1757	147.6	14.0	0.0	0.0	0.0	0.0	0.0	0.0
1758	168.2	16.0	0.0	0.0	0.0	0.0	0.0	0.0
1760	188.8	17.9	3.4	0.6	0.0	0.0	0.0	0.0
1761	225.2	21.4	34.5	2.6	10.5	5.9	0.0	0.0
1762	261.6	24.9	65.7	4.6	21.0	11.8	0.0	0.0
1763	298.0	28.4	78.7	5.5	37.4	11.9	0.0	0.0
1764	381.2	36.1	91.6	6.3	53.8	11.9	0.0	0.0
1765	464.3	43.8	100.6	7.0	70.1	12.2	0.0	0.0
1766	547.4	51.5	109.6	7.6	86.3	12.5	0.0	0.0
1767	553.0	52.1	122.1	8.4	103.6	13.9	0.0	0.0
1768	558.6	52.8	134.5	9.2	120.9	15.2	0.0	0.0
1769	564.2	53.4	143.1	9.7	136.7	15.9	0.0	0.0
1770	566.7	53.6	151.8	10.3	152.5	16.6	0.0	0.0
1771	569.3	53.9	155.5	10.4	165.0	17.3	0.0	0.0
1772	571.8	54.2	159.3	10.6	177.6	17.9	0.0	0.0
1773	574.4	54.4	162.3	10.8	186.6	18.5	0.0	0.0
1774	575.4	54.6	165.2	10.9	195.6	19.0	0.0	0.0
1775	576.5	54.8	166.9	11.0	201.2	19.4	0.0	0.0
1776	577.5	54.9	168.5	11.1	206.7	19.8	0.0	0.0
1777	578.1	54.9	169.4	11.3	209.9	19.9	0.6	1.4
1778	578.7	54.9	170.2	11.5	213.0	20.0	1.2	2.7
1779	579.3	54.9	170.6	11.5	214.8	20.2	7.9	3.7
1780	579.6	55.0	171.0	11.6	216.6	20.4	14.6	4.7
1781	580.0	55.0	171.2	11.6	217.3	20.3	16.3	3.8
1782	580.3	55.1	171.4	11.6	217.9	20.3	18.0	2.8
1783	580.6	55.1	171.5	11.6	218.2	20.4	18.7	3.3
1784	580.9	55.1	171.6	11.6	218.4	20.4	19.3	3.8
1785	581.2	55.0	171.6	11.6	218.5	20.4	21.5	3.7
1786	581.5	54.9	171.6	11.6	218.6	20.3	23.6	3.5

Table C-24. Storage Area 14 Stage-Damage ($1,000's)

Interior Stage	BASIC CROPS		COMMERCIAL		FARMBUILDING		RESIDENTIAL		EMERGENCY	
	Mean	Standard Deviation	Mean	Standard Deviation	Mean	Standard Deviation	Mean	Standard Deviation	Mean	Standard Deviation
1752	29.1	2.8	0.0	0.0	12.3	1.7	0.0	0.0	0.0	0.0
1753	144.7	13.7	0.0	0.0	16.2	2.2	0.0	0.0	0.0	0.0
1754	260.4	24.6	0.0	0.0	20.1	2.7	0.0	0.0	0.0	0.0
1755	376.0	35.5	0.0	0.0	21.3	2.9	0.0	0.0	0.0	0.0
1756	435.9	41.2	0.0	0.0	22.6	3.1	0.0	0.0	0.0	0.0
1757	495.8	46.9	0.0	0.0	26.7	3.6	0.0	0.0	0.0	0.0
1758	555.7	52.7	0.0	0.0	30.9	4.0	0.0	0.0	0.0	0.0
1759	615.5	58.4	0.0	0.0	37.0	4.4	0.5	1.7	0.0	0.0
1760	765.0	72.6	0.0	0.0	43.2	4.8	1.1	3.4	0.0	0.0
1761	914.5	86.8	0.0	0.0	64.0	5.6	9.5	6.7	1.0	1.7
1762	1,064.0	101.1	0.0	0.0	84.9	6.4	18.0	10.0	2.1	3.3
1763	1,264.5	119.7	0.0	0.0	98.2	7.1	29.1	9.8	6.5	3.3
1764	1,465.0	138.3	0.0	0.0	111.5	7.8	40.2	9.6	10.8	3.3
1765	1,665.5	157.0	0.0	0.0	124.7	8.7	50.4	9.9	11.4	2.8
1766	1,670.7	157.9	0.0	0.0	137.8	9.5	60.6	10.1	12.0	2.3
1767	1,675.9	158.7	0.0	0.0	150.5	10.3	68.7	10.7	12.0	2.3
1768	1,681.1	159.6	0.0	0.0	163.1	11.0	76.9	11.2	12.0	2.3
1769	1,683.1	159.8	0.0	0.0	177.1	12.1	84.0	12.2	12.0	2.3
1770	1,685.1	160.0	0.0	0.0	191.1	13.2	91.0	13.2	12.0	2.3
1771	1,687.1	160.2	1.2	0.5	197.5	13.8	99.9	14.4	12.6	2.9
1772	1,689.2	160.4	2.5	1.0	203.9	14.3	108.7	15.6	13.2	3.4
1773	1,691.1	160.5	6.9	2.7	210.8	14.8	118.4	15.7	15.4	3.3
1774	1,693.1	160.6	11.4	4.5	217.6	15.3	128.1	15.8	17.5	3.2
1775	1,695.0	160.7	14.5	5.7	226.1	15.8	136.9	16.0	20.3	3.5
1776	1,696.7	160.6	17.7	7.0	234.6	16.2	145.6	16.1	23.1	3.8
1777	1,698.5	160.5	21.4	8.4	238.2	16.5	152.6	16.4	23.5	3.5
1778	1,700.2	160.3	25.0	9.9	241.8	16.7	159.5	16.6	24.0	3.2
1779	1,701.4	160.6	28.0	11.1	244.5	16.9	164.9	16.9	24.0	3.2
1780	1,702.5	160.8	31.0	12.2	247.1	17.1	170.3	17.2	24.0	3.2
1781	1,703.7	161.0	31.3	12.4	249.7	17.2	177.5	18.1	24.2	3.3
1782	1,704.9	161.2	31.6	12.5	252.2	17.4	184.6	19.0	24.3	3.5
1783	1,706.1	161.3	31.7	12.6	254.8	17.4	192.7	19.2	26.5	3.8
1784	1,707.3	161.4	31.9	12.6	257.3	17.5	200.8	19.3	28.8	4.2

Table C-25. Storage Area 15 Stage-Damage ($1,000's)

Interior Stage	BASIC CROPS	
	Mean	Standard Deviation
1762	0.0	0.0
1763	2.5	0.2
1764	5.0	0.5
1765	7.6	0.7
1766	26.8	2.5
1767	46.0	4.4
1768	65.2	6.2
1769	67.8	6.4
1770	70.5	6.7
1771	73.2	6.9
1772	73.5	6.9
1773	73.8	7.0
1774	74.0	7.0
1775	74.3	7.0
1776	76.9	7.3
1777	79.5	7.5
1778	82.1	7.8
1779	82.5	7.8
1780	83.0	7.9
1781	83.4	7.9
1782	83.7	7.9
1783	83.9	8.0
1784	84.2	8.0
1785	84.4	8.0
1786	84.9	8.1
1787	85.3	8.1
1788	85.7	8.1

Table C-26. Storage Area 16 Stage-Damage ($1,000's)

Interior Stage	BASIC CROPS		FARMBUILDING	
	Mean	Standard Deviation	Mean	Standard Deviation
1755	9.64	0.92	0.0	0.0
1756	10.69	1.02	0.0	0.0
1757	11.73	1.11	0.0	0.0
1758	12.77	1.21	0.0	0.0
1759	13.81	1.30	0.9	0.2
1760	15.38	1.46	1.8	0.3
1761	16.94	1.61	12.3	1.5
1762	18.50	1.76	22.8	2.6
1763	28.28	2.69	26.2	3.0
1764	38.05	3.61	29.6	3.4
1765	47.82	4.53	33.0	3.8
1766	49.78	4.72	36.3	4.2
1767	51.73	4.90	39.6	4.5
1768	53.68	5.08	43.0	4.9
1769	54.47	5.16	45.2	5.2
1770	55.25	5.23	47.4	5.5
1771	56.03	5.30	48.0	5.5
1772	56.81	5.37	48.5	5.6
1773	57.55	5.45	49.1	5.6
1774	58.29	5.52	49.7	5.7
1775	59.03	5.59	49.9	5.7
1776	59.81	5.67	50.1	5.8
1777	60.59	5.74	50.1	5.8
1778	61.37	5.81	50.1	5.8
1779	62.19	5.88	50.1	5.8
1780	63.00	5.95	50.1	5.8
1781	63.81	6.02	50.1	5.8
1782	64.63	6.09	50.1	5.8
1783	65.33	6.18	50.1	5.8
1784	66.02	6.27	50.1	5.8
1785	66.72	6.36	50.1	5.8

Table C-27. Storage Area 17 Stage-Damage ($1,000's)

Interior Stage	BASIC CROPS		FARMBUILDING		RESIDENTIAL		EMERGENCY	
	Mean	Standard Deviation	Mean	Standard Deviation	Mean	Standard Deviation	Mean	Standard Deviation
1758	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
1759	0.0	0.0	1.0	0.2	2.6	2.2	0.0	0.1
1760	0.0	0.0	1.9	0.4	5.2	4.4	0.0	0.1
1761	2.5	0.2	4.5	1.0	9.0	4.1	3.0	0.9
1762	5.0	0.5	7.1	1.5	12.8	3.7	6.0	1.6
1763	16.8	1.6	7.9	1.6	16.4	4.0	6.0	1.6
1764	28.6	2.7	8.7	1.8	20.0	4.2	6.0	1.6
1765	40.4	3.8	9.9	2.1	23.0	4.6	6.0	1.6
1766	166.6	15.9	11.2	2.3	26.0	5.1	6.0	1.6
1767	292.9	27.9	11.8	2.4	28.2	5.5	6.0	1.6
1768	419.1	39.9	12.5	2.6	30.5	5.9	6.0	1.6
1769	424.8	40.3	14.4	2.7	32.2	6.2	6.0	1.6
1770	430.5	40.7	16.3	2.9	33.8	6.6	6.0	1.6
1771	436.2	41.0	19.4	3.1	35.0	6.7	6.0	1.6
1772	440.0	41.5	22.5	3.3	36.2	6.9	6.0	1.6
1773	443.7	41.9	24.6	3.5	39.2	8.1	6.0	1.6
1774	447.5	42.3	26.7	3.6	42.1	9.4	6.0	1.7
1775	451.3	42.7	28.6	3.8	50.0	10.5	8.1	2.6
1776	454.8	43.1	30.6	4.0	57.8	11.7	10.1	3.5
1777	458.3	43.4	34.6	4.4	68.6	11.9	13.0	3.7
1778	461.8	43.8	38.6	4.8	79.3	12.2	15.9	3.9
1779	463.7	44.0	43.6	5.1	93.7	13.3	16.9	3.3
1780	465.6	44.2	48.5	5.4	108.2	14.5	18.0	2.8
1781	467.5	44.5	52.4	5.9	122.8	14.7	21.0	3.0
1782	468.7	44.5	56.2	6.3	137.3	14.8	24.0	3.2
1783	469.9	44.6	61.5	6.7	149.6	15.5	24.0	3.2
1784	471.1	44.7	66.9	7.1	161.8	16.1	24.0	3.2
1785	472.3	44.8	72.3	7.6	171.3	16.8	24.0	3.2
1786	473.7	44.9	77.7	8.0	180.8	17.6	24.0	3.1
1787	475.2	45.1	81.8	8.5	187.4	18.1	24.0	3.1
1788	476.6	45.3	85.8	9.0	194.1	18.7	24.0	3.1
1789	477.7	45.3	90.0	9.3	198.1	19.1	24.0	3.2
1790	478.7	45.3	94.1	9.7	202.1	19.5	24.0	3.2

Table C-28. Storage Area 18 Stage-Damage ($1,000's)

Interior Stage	BASIC CROPS		HOPS		COMMERCIAL		FARMBUILDING		RESIDENTIAL		EMERGENCY	
	Mean	Standard Deviation	Mean	Standard Deviation	Mean	Standard Deviation	Mean	Standard Deviation	Mean	Standard Deviation	Mean	Standard Deviation
1750	377.0	38.7	1642.3	68.9	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
1751	416.2	42.6	2236.3	92.1	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
1752	455.3	46.6	2830.4	115.3	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
1753	491.1	50.2	3102.8	127.5	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
1754	526.9	53.7	3375.3	139.6	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
1755	562.6	57.3	3647.7	151.7	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
1756	580.0	58.9	3796.5	158.4	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
1757	597.4	60.5	3945.4	165.1	0.0	0.0	2.4	0.5	0.0	0.0	0.0	0.0
1758	614.8	62.1	4094.2	171.9	0.0	0.0	4.8	1.0	0.0	0.0	0.0	0.0
1759	618.3	62.6	4217.8	177.7	0.0	0.0	12.6	1.8	1.1	2.4	0.0	0.0
1760	621.8	63.1	4341.4	183.6	0.0	0.0	20.3	2.6	2.2	4.7	0.0	0.0
1761	625.4	63.6	4465.0	189.5	295.2	40.9	26.3	3.1	9.2	7.1	1.2	1.5
1762	626.8	63.7	4465.0	189.7	590.4	81.8	32.4	3.7	16.2	9.4	2.3	3.1
1763	628.3	63.9	4465.0	189.8	1305.8	108.1	35.9	4.1	26.9	9.6	6.0	3.4
1764	629.8	64.1	4465.0	190.0	2021.1	134.4	39.4	4.5	37.7	9.8	9.6	3.6
1765	631.3	64.2	4465.0	190.1	2570.8	162.7	43.4	5.0	48.0	10.0	10.8	3.0
1766	633.1	64.3	4465.0	190.0	3120.6	191.0	47.3	5.4	58.3	10.1	12.0	2.3
1767	635.0	64.4	4465.0	189.9	3719.3	226.3	50.0	5.7	66.7	10.6	12.0	2.3
1768	636.8	64.5	4465.0	189.7	4318.0	261.7	52.6	6.0	75.2	11.1	12.0	2.3
1769	637.5	64.5	4465.0	190.0	4804.7	287.4	53.9	6.2	81.8	11.8	12.0	2.3
1770	638.2	64.5	4465.0	190.3	5291.5	313.2	55.2	6.3	88.5	12.4	12.0	2.3
1771	638.8	64.5	4465.0	190.5	5341.7	315.8	55.8	6.4	93.4	12.9	12.0	2.3
1772	639.1	64.5	4465.0	190.3	5391.8	318.4	56.4	6.4	98.3	13.4	12.0	2.3
1773	639.4	64.6	4465.0	190.1	5416.2	318.4	56.8	6.5	101.4	13.7	12.0	2.3
1774	639.7	64.7	4465.0	189.8	5440.5	318.4	57.2	6.5	104.6	14.0	12.0	2.3
1775	640.0	64.8	4465.0	189.6	5556.5	356.7	57.2	6.6	106.1	14.2	12.0	2.3
1776	640.3	64.8	4465.0	189.4	5672.4	395.1	57.2	6.7	107.6	14.4	12.0	2.3
1777	640.6	64.8	4465.0	189.2	5677.5	394.9	57.2	6.7	108.2	14.5	12.0	2.3
1778	640.9	64.7	4465.0	189.1	5682.6	394.8	57.2	6.7	108.8	14.5	12.0	2.3
1779	641.2	64.8	4465.0	189.5	5682.9	397.0	57.2	6.7	109.0	14.6	12.0	2.3
1780	641.5	64.9	4465.0	190.0	5683.2	399.1	57.2	6.7	109.2	14.6	12.0	2.3
1781	641.8	64.9	4465.0	190.4	5683.2	397.6	57.2	6.7	109.3	14.6	12.0	2.3
1782	642.2	65.1	4465.0	189.8	5683.1	396.1	57.2	6.7	109.3	14.6	12.0	2.3

Table C-29. Storage Area 19 Stage-Damage ($1,000's)

Interior Stage	BASIC CROPS	
	Mean	Standard Deviation
1754	0.00	0.00
1755	10.7	1.0
1756	13.0	1.2
1757	15.3	1.4
1758	17.6	1.7
1759	18.9	1.8
1760	20.3	1.9
1761	21.7	2.1
1762	21.7	2.1
1763	21.8	2.1
1764	21.9	2.1
1765	21.9	2.1
1766	21.9	2.1
1767	21.9	2.1
1768	21.9	2.1
1769	21.9	2.1
1770	21.9	2.1
1771	21.9	2.1
1772	21.9	2.1
1773	21.9	2.1
1774	21.9	2.1
1775	21.9	2.1
1776	21.9	2.1
1777	21.9	2.1
1778	21.9	2.1
1779	21.9	2.1
1780	21.9	2.1
1781	21.9	2.1
1782	21.9	2.1

Table C-30. Storage Area 20 Stage-Damage ($1,000's)

Interior Stage	BASIC CROPS	
	Mean	Standard Deviation
1754	0.0	0.0
1755	40.1	3.8
1756	51.2	4.9
1757	62.3	5.9
1758	73.4	7.0
1759	82.3	7.8
1760	91.2	8.7
1761	100.1	9.5
1762	100.5	9.5
1763	100.9	9.5
1764	101.2	9.5
1765	101.4	9.6
1766	101.6	9.6
1767	101.8	9.6
1768	101.9	9.6
1769	102.1	9.7
1770	102.2	9.7
1771	102.4	9.7
1772	102.5	9.7
1773	102.7	9.8
1774	102.8	9.8
1775	102.9	9.8
1776	103.0	9.8
1777	103.0	9.8
1778	103.1	9.8
1779	103.1	9.8
1780	103.2	9.8
1781	103.2	9.8
1782	103.2	9.8

Table C-31. Storage Area 21 Stage-Damage ($1,000's)

Interior Stage	BASIC CROPS		FARMBUILDING		RESIDENTIAL		EMERGENCY	
	Mean	Standard Deviation	Mean	Standard Deviation	Mean	Standard Deviation	Mean	Standard Deviation
1755	119.3	11.3	0.0	0.0	0.0	0.0	0.0	0.0
1756	148.9	14.1	0.0	0.0	0.0	0.0	0.0	0.0
1757	178.4	16.9	0.0	0.0	0.0	0.0	0.0	0.0
1758	207.9	19.7	0.0	0.0	0.0	0.0	0.0	0.0
1759	237.5	22.4	0.0	0.0	0.0	0.0	0.0	0.0
1760	273.9	25.8	0.0	0.0	0.0	0.0	0.0	0.0
1761	310.4	29.2	0.0	0.0	0.6	1.7	0.0	0.0
1762	346.8	32.6	0.0	0.0	1.1	3.5	0.0	0.0
1763	385.3	36.4	7.4	0.9	5.4	5.3	0.7	1.3
1764	423.9	40.2	14.8	1.9	9.7	7.2	1.3	2.6
1765	462.4	44.0	24.1	2.8	15.3	7.0	3.4	2.4
1766	463.3	44.1	33.5	3.7	20.8	6.8	5.6	2.2
1767	464.1	44.2	36.8	4.0	25.9	7.0	5.8	1.9
1768	465.0	44.3	40.0	4.3	30.9	7.2	6.0	1.6
1769	465.3	44.3	45.2	4.9	34.9	7.6	6.0	1.6
1770	465.6	44.3	50.4	5.5	38.9	8.0	6.0	1.6
1771	465.9	44.2	53.7	5.8	42.2	8.5	6.0	1.6
1772	466.2	44.2	56. 9	6.2	45.4	8.9	6.0	1.6
1773	466.6	44.3	59.2	6.4	47.6	9.3	6.0	1.6
1774	467.0	44.3	61.5	6.7	49.9	9.6	6.0	1.6
1775	467.3	44.4	62.6	6.8	51.3	9.7	6.0	1.6
1776	467.6	44.4	63.8	6.8	52.8	9.9	6.0	1.6
1777	467.9	44.5	65.8	7.0	53.4	10.0	6.0	1.6
1778	468.2	44.6	67.7	7.1	54.0	10.1	6.0	1.6
1779	468.5	44.6	68.2	7.1	54.2	10.2	6.0	1.6
1780	468.7	44.5	68.7	7.2	54.5	10.3	6.0	1.6
1781	468.9	44.5	69.2	7.2	54.6	10.3	6.0	1.6
1782	469.1	44.5	69.8	7.2	54.6	10.3	6.0	1.6
1783	469.3	44.4	70.2	7.3	54.6	10.3	6.0	1.6

Table C-32. Storage Area 22 Stage-Damage ($1,000's)

Interior Stage	BASIC CROPS		PUBLIC		RESIDENTIAL		EMERGENCY	
	Mean	Standard Deviation	Mean	Standard Deviation	Mean	Standard Deviation	Mean	Standard Deviation
1758	0.0	0.0	0.0	0.0	0.0	0.0	4.6	2.9
1759	0.0	0.0	0.0	0.0	0.3	0.3	5.6	2.6
1760	0.0	0.0	0.0	0.0	0.6	0.6	6.6	2.4
1761	0.0	0.0	0.0	0.0	6.5	1.8	11.4	3.6
1762	0.1	0.0	0.0	0.0	12.3	3.0	16.2	4.7
1763	0.4	0.0	0.0	0.0	27.2	4.8	28.2	5.3
1764	0.7	0.1	0.0	0.0	42.2	6.7	40.2	5.8
1765	0.9	0.1	0.0	0.0	69.0	7.9	50.5	5.7
1766	1.2	0.1	0.0	0.0	95.7	9.1	60.7	5.7
1767	1.5	0.1	0.0	0.0	144.2	11.3	103.5	8.2
1768	1.8	0.2	0.0	0.0	192.6	13.6	146.2	10.6
1769	2.1	0.2	0.0	0.0	285.1	15.6	153.5	9.6
1770	2.5	0.2	0.0	0.0	377.5	17.5	160.7	8.5
1771	2.8	0.3	0.0	0.0	443.0	19.1	161.7	8.5
1772	3.1	0.3	0.0	0.0	508.5	20.6	162.6	8.5
1773	3.9	0.4	0.0	0.0	568.5	22.0	168.4	8.6
1774	4.6	0.4	0.0	0.0	628.5	23.3	174.2	8.8
1775	5.3	0.5	0.0	0.0	674.1	24.4	180.5	9.0
1776	5.6	0.5	0.0	0.0	719.7	25.5	186.9	9.2
1777	5.8	0.6	0.0	0.0	747.7	26.3	198.4	9.3
1778	6.1	0.6	0.0	0.0	775.7	27.1	210.0	9.5
1779	6.2	0.6	0.0	0.0	806.4	27.5	215.5	9.7
1780	6.4	0.6	0.0	0.0	837.2	27.9	221.1	9.9
1781	6.5	0.6	0.0	0.0	859.6	28.4	224.5	9.9
1782	6.6	0.6	0.0	0.0	882.1	28.9	228.0	9.8
1783	6.8	0.6	47.5	9.0	893.3	29.3	228.0	9.9
1784	7.0	0.7	95.0	17.9	904.5	29.7	228.0	10.0
1785	7.2	0.7	145.6	27.4	925.9	30.7	234.1	10.6

Table C-33. Storage Area 23 Stage-Damage ($1,000's)

Interior Stage	BASIC CROPS		FARMBUILDING		RESIDENTIAL		EMERGENCY	
	Mean	Standard Deviation	Mean	Standard Deviation	Mean	Standard Deviation	Mean	Standard Deviation
1760	0.0	0.0	0.7	0.2	0.0	0.0	0.0	0.0
1761	0.0	0.0	3.8	0.7	1.5	2.6	0.0	0.0
1762	0.0	0.0	6.9	1.2	3.0	5.3	0.0	0.0
1763	0.0	0.0	9.4	1.6	8.0	6.2	1.5	1.6
1764	0.0	0.0	11.9	2.0	13.0	7.1	2.9	3.2
1765	1.3	0.1	13.1	2.2	18.5	7.0	4.5	2.4
1766	2.0	0.2	14.3	2.4	23.9	6.8	6.0	1.7
1767	2.7	0.3	16.0	2.7	28.7	7.1	6.0	1.6
1768	3.4	0.3	17.6	3.0	33.5	7.4	6.0	1.6
1769	20.9	2.0	18.4	3.1	37.3	7.9	6.0	1.6
1770	38.5	3.7	19.3	3.3	41.0	8.3	6.0	1.6
1771	56.0	5.3	19.9	3.4	44.0	8.7	6.0	1.6
1772	78.9	7.5	20.6	3.5	46.9	9.1	6.0	1.6
1773	101.7	9.6	20.8	3.5	48.9	9.4	6.0	1.6
1774	124.6	11.7	21.0	3.5	50.9	9.8	6.0	1.6
1775	137.1	12.9	21.2	3.6	52.1	9.9	6.0	1.6
1776	149.6	14.1	21.4	3.6	53.2	10.0	6.0	1.6
1777	162.1	15.3	21.4	3.6	53.7	10.0	6.0	1.6
1778	174.6	16.5	21.4	3.6	54.2	10.1	6.0	1.6
1779	219.4	20.8	21.4	3.6	54.3	10.1	6.0	1.6
1780	264.3	25.0	21.4	3.6	54.5	10.1	6.0	1.6
1781	309.1	29.3	21.4	3.6	54.6	10.1	6.0	1.6
1782	314.2	29.8	21.4	3.6	54.6	10.1	6.0	1.6
1783	319.3	30.3	22.7	3.6	56.6	10.9	6.0	1.6
1784	324.5	30.9	24.0	3.6	58.6	11.7	6.0	1.6
1785	327.6	31.1	27.2	3.8	63.8	12.0	7.8	2.6
1786	330.7	31.4	30.4	3.9	69.0	12.3	9.5	3.6
1787	333.8	31.7	32.2	4.0	75.8	12.7	10.8	2.9

Table C-34. Storage Area 24 Stage-Damage ($1,000's)

Interior Stage	BASICCROPS		FARMBUILDING		RESIDENTIAL		EMERGENCY	
	Mean	Standard Deviation	Mean	Standard Deviation	Mean	Standard Deviation	Mean	Standard Deviation
1771	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
1772	0.2	0.0	0.0	0.0	0.0	0.0	0.0	0.0
1773	2.1	0.2	0.0	0.0	0.0	0.0	0.0	0.0
1774	4.0	0.4	0.0	0.0	0.0	0.0	0.0	0.0
1775	5.9	0.6	0.0	0.0	0.0	0.0	0.0	0.0
1776	8.0	0.8	0.0	0.0	0.0	0.0	0.0	0.0
1777	10.0	1.0	1.0	0.2	0.0	0.0	0.0	0.0
1778	12.1	1.2	1.9	0.4	0.0	0.0	0.0	0.0
1779	15.7	1.5	6.1	1.0	2.2	3.0	0.0	0.1
1780	19.3	1.8	10.2	1.7	4.4	6.1	0.0	0.2
1781	22.9	2.2	19.2	2.3	13.6	8.1	2.2	1.9
1782	26.5	2.5	28.3	3.0	22.7	10.1	4.4	3.7
1783	35.2	3.3	33.7	3.4	33.6	10.0	7.8	3.4
1784	43.9	4.2	39.1	3.9	44.5	9.9	11.1	3.1
1785	52.7	5.0	48.1	4.4	54.3	10.2	11.6	2.7
1786	59.1	5.6	57.1	4.9	64.1	10.6	12.0	2.3
1787	65.6	6.2	62.5	5.4	72.0	11.1	12.0	2.3
1788	72.1	6.8	67.9	5.8	79.8	11.7	12.0	2.3
1789	78.1	7.4	71.7	6.1	85.9	12.4	12.0	2.3
1790	84.1	8.0	75.4	6.4	92.1	13.0	12.0	2.2
1791	90.2	8.5	80.5	6.6	96.4	13.4	12.0	2.3
1792	96.2	9.1	85.5	6.8	100.7	13.8	12.0	2.3
1793	125.9	11.9	88.7	7.0	103.3	14.1	12.0	2.3
1794	155.5	14.7	92.0	7.2	105.9	14.4	12.0	2.3

Table C-35. Storage Area 25 Stage-Damage ($1,000's)

Interior Stage	BASIC CROPS	
	Mean	Standard Deviation
1777	0.0	0.0
1778	1.9	0.2
1779	2.3	0.2
1780	2.8	0.3
1781	3.3	0.3
1782	3.8	0.4
1783	4.3	0.4
1784	4.9	0.5
1785	5.4	0.5
1786	5.9	0.6
1787	6.4	0.6
1788	6.9	0.7
1789	7.1	0.7
1790	7.3	0.7
1791	7.4	0.7
1792	7.5	0.7
1793	7.6	0.7
1794	7.7	0.7
1795	7.8	0.7

Table C-36. Storage Area 26 Stage-Damage ($1,000's)

Interior Stage	BASIC CROPS		COMMERCIAL	
	Mean	Standard Deviation	Mean	Standard Deviation
1764	0.0	0.0	0.0	0.0
1765	0.6	0.1	0.0	0.0
1766	0.7	0.1	0.0	0.0
1767	0.8	0.1	0.0	0.0
1768	0.9	0.1	0.0	0.0
1769	1.0	0.1	0.0	0.0
1770	1.1	0.1	0.0	0.0
1771	1.3	0.1	4.5	0.6
1772	1.4	0.1	9.0	1.1
1773	1.6	0.1	12.0	1.5
1774	1.8	0.2	15.1	1.9
1775	2.0	0.2	18.0	2.3
1776	2.1	0.2	21.0	2.6
1777	2.3	0.2	29.3	3.3
1778	2.4	0.2	37.7	3.9
1779	2.6	0.2	42.8	4.4
1780	2.7	0.3	47.8	4.9
1781	2.9	0.3	50.9	5.3
1782	3.0	0.3	54.0	5.7
1783	5.4	0.5	58.7	6.4
1784	7.9	0.7	63.4	7.1
1785	10.3	1.0	65.2	7.4
1786	10.6	1.0	66.9	7.7
1787	10.9	1.0	67.2	7.8
1788	11.2	1.1	67.5	7.8

7.4 APPENDIX D – KOOTENAI RIVER CHANNEL CAPACITY STUDY REPORT

[See following pages]

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7.5 APPENDIX E – ESTIMATED COSTS OF DAMAGED LEVEE REPAIR, MEMORANDUM FOR RECORD

[See following pages]

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CENWS-EC-DB-CS (1110-2-1150a) 23 September 2005

MEMORANDUM FOR RECORD

SUBJECT: Bonners Ferry Flood Damage Study, Bonners Ferry, Idaho

1. The levees along the right and left banks of the Kootenai river from just upstream of the town of Bonners Ferry, Idaho to the Canadian border have suffered various degrees of damage since Libby Dam was built. Many areas along this fifty plus mile stretch of river have experienced erosion damage that has encroached on the existing levee prisms thus rendering them susceptible to failure in the event of a catastrophic flood. The current condition of the levees has been compiled over the last 10 plus years by a series of boat trip evaluations and a series of cross sections taken from a 1995 aerial survey.

2. This memo does not address the possible causes of levee damage or responsibility for repairs. The estimates below are based on observations and inspection reports taken during eleven boat trips and limited field inspections by the signatory of this memorandum for record. The proposed repairs are limited to sites that currently are considered susceptible to imminent failure during a very major flood event and it should be noted that there are other areas that may become weakened in the future that are not addressed in this analysis.

3. The river was segmented into 26 distinct storage areas (SA) for this analysis (see encl. 1 [included as Figure 5 in main report]) and 14 of these were found to be susceptible to failure for at least the worst of three different scenarios; a flood at elevation 1764, a flood at elevation 1770, and a flood at elevation 1770 with an additional inflow of 10,000 cubic feet per second (cfs) of local runoff (all elevations are in NGVD 29 datum.)

4. Numerous assumptions were made to come up with the estimated costs to repair these 14 vulnerable sites. The average height throughout the river was assumed to be 45 feet from river bed to the top of the riprap placement. The average cost for riprap placed was assumed to be $60 per cubic yard (CY). The average cost for incidental construction measures such as slope grading and access road building/repairing is estimated to add 40% to the cost to place riprap. The length of damaged areas was based on a percentage of damage in each SA based on results of the above mentioned boat inspections. It must be understood that these estimates can change by orders of magnitude once the actual damage areas are physically surveyed and only repairing a few sites will also raise the costs per lineal foot significantly.

SA21: Repair/build access road. Regrade slope to 2 horizontal (H) on 1 vertical (V). Place riprap from bed of river to bank height. Regrade levee and hydroseed. Estimated cost = $3,992,000.

SA18: Same as SA21. Estimated cost = $4,865,000.

SA20: Same as SA21. Estimated cost = $500,000.

SA19: Same as SA21. Estimated cost = $749,000.

SA13: Same as SA21. Estimated cost = $11,227,000.

SA14: Same as SA21. Estimated cost = $10,977,000.

SA12: Same as SA21. Estimated cost = $874,000.

SA8: Same as SA21. Estimated cost = $3,992,000.

SA9: Same as SA21. Estimated cost = $749,000.

SA17: Same as SA21. Estimated cost = $7,111,000.

SA5: Same as SA21. Estimated cost = $1,996,000.

SA2C: Construct a concrete flood wall four feet above ground height for 1000 lineal feet. Estimated cost = $300,000.

SA26: Construct a one foot tall levee. Estimated cost = $20,000.

SA24: Construct a three foot tall levee. Estimated cost = $3,792,500.

5. If there are any questions, please contact Monte Kaiser @ (206) 764-6194.

MONTE KAISER
Civil/Soils Section
Design Branch

1 Encl. [included as Figure 5 in main report]