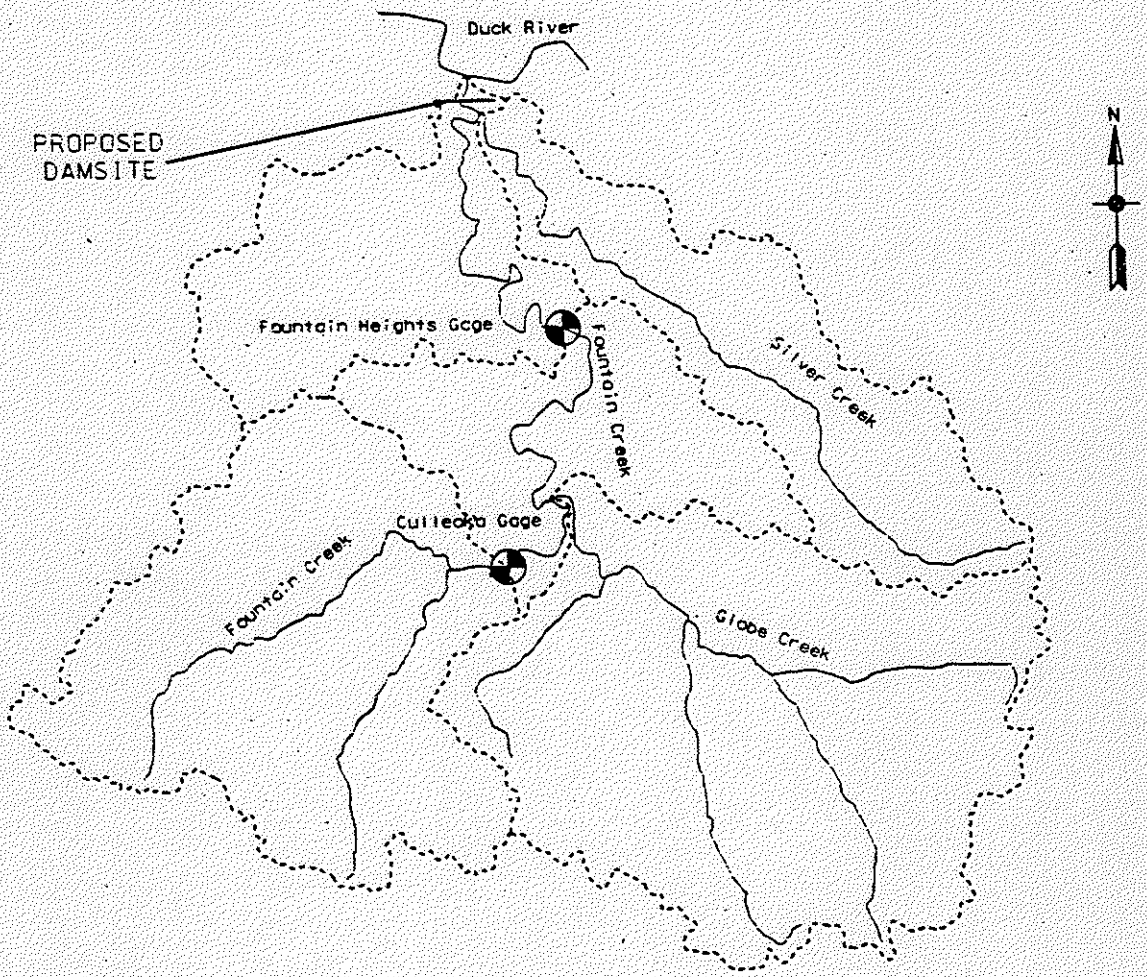
US Army Corps
of Engineers
Nashville District

FINAL REPORT ON CONCEPTUAL DESIGN FOR THE FOUNTAIN CREEK HYDROLOGY AND HYDRAULICS STUDY



OCTOBER 1997

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ON CONCEPTUAL DESIGN
FOR THE FOUNTAIN CREEK
HYDROLOGY AND HYDRAULICS STUDY

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EXECUTIVE SUMMARY

The enclosed report details preliminary hydrologic design assumptions and techniques used to evaluate a water supply dam in the Fountain Creek watershed. The study was performed to determine if the Fountain Creek watershed is capable of yielding a previously estimated water supply need of 43 MGD (Million Gallons per Day) for the Bedford County, Marshall County, Maury County and southern Williamson County areas by the year 2050. The determination of water supply needs in this area is not a part of this study. The intent of this study was to determine if a dam could be located on Fountain Creek within the existing Columbia Dam land acquisition area (up to elevation 630) that could meet future water supply demands in the region. It was not within the scope of this study to consider geotechnical factors, environmental issues, water quality aspects of the reservoir or relocation of existing utilities, roads and structures.

This study looked at two possible dam configurations near the mouth of Fountain Creek. The first was a similar dam configuration as that used in a study performed by TVA in 1987. The minimum expected yield for this dam was determined, in this study, to be 47 MGD with a top of dam elevation of 645 feet MSL and spillway crest elevation of 629. The second dam configuration was designed to meet the estimated water demand of 43 MGD for the year 2050. To meet this criteria, a top of dam elevation of 642 and a service spillway crest of 626 were determined. The top of dam elevations for the two dam configurations were designed to safely permit the discharge of the Probable Maximum Flood (PMF) through the spillway outlet works without overtopping.

Rainfall data from 95 years of continuous record were used to identify severe droughts in the area. This data was then used to determine the volume of water that would be available for storage during critical historic drought periods. Allowances were made in the available storage capacity for sediment deposition in the reservoir, evaporation, leakage and withdrawals for water quality. Design pumping capacity (water supply yield) was determined for each dam configuration by running the critical drought period rainfall through a calibrated hydrologic model and determining the maximum rate of withdrawal possible from the reservoir without depleting the storage capacity set aside for water supply. The computed design pumping rates of 43 and 47 MGD are considered to be low, because of the various assumptions used in this study. Conservative values for sediment storage, evaporation, runoff amounts, leakage and water quality

withdrawals were used in the modeling process. This was done to insure that the actual design pumping rates would be higher.

Subsequent to the "draft" release of this report, a geotechnical investigation was performed at the damsite by the Nashville District USACE. After obtaining geological borings it was discovered that the top of rock elevations in the vicinity of the proposed spillway outlets were much lower than expected. Preliminary analysis defining a new spillway configuration, based on the newly defined elevations, has been added to this report along with the original configuration. Two additional alternatives were formulated in consideration of the newly acquired geological information, one being the 43 MGD and the other a 47 MGD configuration.

Two options are presented in this report for the spillway configurations, each configuration has two alternate normal pool elevations. Option 1 is the original, "draft" study configuration that utilized the existing low topographic area between the main dam and the saddle dam as a spillway location. Option 2 is the "added" configuration to account for newly acquired geotechnical information.

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INVESTIGATION of a WATER SUPPLY DAM
on
FOUNTAIN CREEK

Objective and Scope

The objective of the Hydrologic Analysis is to analyze a potential dam alternative on Fountain Creek near its confluence with the Duck River (approximate rivermile 146). The design of any dams considered in this analysis is to be based solely on water supply. The critical information desired from this analysis is a determination of the yield of water available from a dam on Fountain Creek.

Several factors, not within the scope of this study, could pose significant problems in the overall design of the dam and reservoir. These are issues that would need to be analyzed at a increased level of design detail. As a minimum, there are concerns that may arise from geotechnical, environmental, water quality and relocation issues. Geotechnical concerns are being studied in a ongoing task of the Duck River Water Supply Environmental Impact Statement. Infiltration and leakage from the reservoir should be studied to estimate the volume of water that would be lost due to these factors.

Damsite Selection

The potential damsite on Fountain Creek was determined by TVA in a study performed by them in 1987. In performing this current investigation, the TVA damsite was used. This site is located at the first constricted valley opening, a few tenths of a mile upstream of the confluence of Fountain Creek and the Duck River. This is a good site from the perspective of both storage area and drainage area. It provides for water to be stored in both the Fountain Creek and Silver Creek watersheds, while also gaining an additional drainage area of 15 square miles from Silver Creek. Silver Creek enters Fountain Creek less than a mile upstream of its mouth.

Description of Damsite

The damsite is located at approximate rivermile 0.2 of Fountain Creek. A dam at this site would extend approximately 1500 feet across the narrowest portion of the valley. This dam would control approximately 102 square miles of drainage area. Two low topographic saddles are located on the rim of the watershed divide a few thousand feet from the west abutment of the damsite. The low point in one of the saddles appears to be at approximate elevation 615. Flows over this saddle enter the Duck River less than a thousand feet downstream of its confluence with Fountain Creek. A saddle dam or weir structure would be necessary to close off the saddle. A service type spillway or both a service and emergency spillway could be located in the mound located between the saddle and the natural Fountain Creek. This is the location of a higher level saddle with a low point of approximate elevation 631. This location may provide an opportunity to cut a spillway into the natural rock resulting in additional dam cost savings.

Option 1. This option was presented in the original "draft" report and consists of utilizing existing topographic conditions in the damsite area to provide for outlet works. The alignment would allow for spillway releases to be made away from the dam embankment through an overland cut. Spillway releases would then flow through a natural drain and enter the Duck River approximately 1000 feet downstream of its confluence with Fountain Creek. A general plan view of the option 1 damsite and saddle areas are shown on **Figure 1A**.

Option 2. As a result of the new geotechnical investigation at the damsite it is evident that using the low topographic saddle as a spillway outlet may not be economically feasible. The top of rock elevations in this area are much lower than anticipated and construction of a concrete, or similar, spillway would be required. Therefore, it was necessary to formulate an additional option for the spillway location. Option 2 for the damsite configuration consists of cutting a spillway into the embankment just east of the main dam. Results from the geotechnical investigation show that the top of rock elevations at this location are sufficient to provide a rock spillway. A spillway crest elevation of 629.0 and a width of 400 feet will safely pass the PMF and only increase the top of dam elevation by 2 feet. The 400 foot width was chosen to minimize the amount of rock excavation required for the spillway. Two saddle dams would be required to close the low topographic areas west of the main dam. A general plan view of the option 2 damsite and saddle areas are

shown on **Figure 1B**. Complete details of the geotechnical investigation can be found in the report submitted separately.

Design Alternatives. Two dam configurations at the selected damsite were evaluated for each option in this study. Each dam is assumed to be either earthfill or rockfill and therefore neither is designed to be overtopped. The first configuration has a water supply pool capable of supplying a constant withdrawal of 47 MGD with a normal pool elevation of 629. The second configuration provides 43 MGD with a normal pool elevation of 626. The normal pool elevation is equal to the lowest spillway crest elevation in both cases. **Table 1A** and **Table 1B** contain a summary of the design components used in each of the two dam configurations for option 1 and 2, respectively.

Hydrologic Engineering Methods

General. The level of study for this investigation is general with respect to dam design with only enough detail to define the range of water supply yield that can be reasonably expected from the watershed. The following sets of design criteria are used to establish both of these objectives.

Water Supply Demand. The water supply yield throughout this report is presented in terms of Million Gallons Per Day or MGD. The first step of this analysis was to determine the yield corresponding to a similar dam design as that considered by TVA in their 1987 evaluations. The dam considered by TVA contained a 50-foot normal flow notch at elevation 629.0 in combination with a 300-foot ogee emergency spillway at elevation 631.5. This was the basis for the option 1 dam configuration. The yield for this dam configuration was determined to be approximately 47 MGD. The yield was found to be dependent on the 629 elevation and not the other spillway widths or top of dam elevation. These other components are designed to safely pass flood events. The yield is dependent on drought type events. The design features such as spillways and top of dam were adjusted within this study based on current model conditions. The basis in establishing both the yield and dam features are contained in the remaining portion of this report.

A second step performed in this investigation was to determine if a different dam spillway configuration at the same location could provide the estimated 43 MGD yield needed in the year 2050. A report completed by the US Geological Survey in 1996 estimates the future water supply demand for Maury,

Table 1A
FOUNTAIN CREEK
WATER SUPPLY DAM
ALTERNATIVES
Option 1

	(629) Normal Pool	(626) Normal Pool
Design pumping capacity (water supply yield)	47 MGD	43 MGD
Contributing drainage area	102 sq mi	102 sq mi
Maximum height of MAIN dam	84 ft	82 ft
Maximum height of SADDLE dam	35 ft	33 ft
Estimated streambed elevation	561	561
Top of dam elevation	645	643
Length of MAIN dam embankment	1500 ft	1500 ft
Length of SADDLE dam embankment	1800 ft	1800 ft
PMF spillway crest elevation	634.0	631.5
PMF spillway width in feet	1100 ft	1100 ft
100 year spillway crest elevation	631.5	628.0
100 year spillway width in feet	400 ft	400 ft
Service spillway crest elevation	629	626
Service spillway width in feet	200 ft	200 ft
Total spillway crest elevation	1700 ft	1700 ft
Reservoir area at service spillway	2000 ac	1900 ac
Total storage in acre feet	43000	37000
Total runoff storage (excluding sediment pool)	7.0 in	6.6 in
Minimum water supply intake elevation	600	600
Maximum depth above intake	29 ft	26 ft
Top of sediment storage pool elevation	600	600
Sediment storage in acre feet	5000	5000
Critical drought design period	1913-1915	1913-1915

Table 1B
FOUNTAIN CREEK
WATER SUPPLY DAM
ALTERNATIVES
Option 2

	(629) Normal Pool	(626) Normal Pool
Design pumping capacity (water supply yield)	47 MGD	43 MGD
Contributing drainage area	102 sq mi	102 sq mi
Maximum height of MAIN dam	86 ft	84 ft
Maximum height of SADDLE dam	37 ft	35 ft
Estimated streambed elevation	561	561
Top of dam elevation	647	645
Length of MAIN dam embankment	1500 ft	1500 ft
Length of SADDLE dam number 1 embankment	1800 ft	1800 ft
Length of SADDLE dam number 2 embankment	600 ft	600 ft
Spillway crest elevation	629.0	626.0
Spillway width in feet	400 ft	400 ft
Reservoir area at spillway crest elevation	2000 ac	1900 ac
Total storage area in acre feet	43000	37000
Total runoff storage (excluding sediment pool)	7.0 in	6.6 in
Minimum water supply intake elevation	600	600
Maximum [illegible] above intake	29 ft	26 ft
Top of sediment storage pool elevation	600	600
Sediment storage in acre feet	5000	5000
Critical drought design period	1913-1915	1913-1915

Marshall, Bedford and Coffee counties to be approximately 43 MGD in the year 2050 (from the Water-Resources Investigation Report 96-4140).

Sediment Allowances. Erosion of the top soil covering the watersheds upstream of dams is a constant source of sediment deposition in reservoirs. The Corps and TVA have set up monitoring programs to measure the amount of sediment buildup in reservoirs. The results of such studies have shown the rates at which sediments are deposited in reservoirs vary from approximately 0.3 to 3.0 acre-feet per square mile per year. An estimate of the sediment deposition rate is important for two reasons. First, the sediment that enters a water supply reservoir occupies storage area that can no longer be used for water supply. This may not be important early in the reservoir life, however as the reservoir gets older, this can become critical, especially if several major floods have occurred. The 0.3 to 3.0 is an average inflow of sediment over several decades, however, most of the sediments enter a reservoir during major flood events. A second reason is that low level outlets such as sluice gates or pump intakes may become silted if located too low in the reservoir. However, neither of these are planned for this project.

For the dams in this study, the depletion of water supply due to sediment deposition was found to be a small percentage of the total required storage. In contrast, the streambed elevation within the reservoir changes drastically with only moderate rates of sediment inflow. This is due to the small incremental change in storage per foot of depth at the lower elevations of the reservoir. This condition is typical of floodplains with very narrow and steep topographic features. The sediment rate used for this study is 1.0 acre-feet per square mile per year. The resulting sediment allowance for each dam is 5100 acre-feet. In terms of elevation the 5100 acre-feet of sediment storage occupies the portion of the reservoir from the streambed at elevation 561 up to elevation 600.

Evaporation. Evaporation is a significant factor to be considered in the design of water supply reservoirs. The evaporation referred to here is the evaporation of water from within the reservoir itself. The evaporation from the rest of the watershed is accounted for in the transformation of rainfall to rainfall excess. Evaporation data in general is very limited. It is usually in the form of pan evaporation values at select National Weather Service gages. Research into pan rates have concluded that seventy percent of the pan values are equivalent

to the evaporation that can be expected in a shallow reservoir. The rate on a deep reservoir would be less. For purposes of this study, evaporation rates used in the design of J. Percy Priest reservoir (Located in Davidson, Wilson and Rutherford counties) were assumed to be reasonable for Fountain Creek.

This is a valid assumption because both of these reservoirs are located in similar climates. The storage losses resulting from evaporation can differ significantly from one reservoir to another. This is because evaporation (being largely a function of wind and surface temperature) effects only surface area. J. Percy Priest's surface area is much larger than that considered for Fountain Creek, which would result in a much larger loss of storage volume, however, the surface depth of losses should be similar. In the analysis of the two dam alternatives in this study, rainfall on the reservoir itself was not considered. Any rainfall that falls directly on the reservoir will be transferred to storage and not lost to evaporation or transpiration as would rainfall on the watershed. This reservoir rainfall would tend to negate some of the losses, due to evaporation, of the storage capacity.

Using the evaporation rates for J. Percy Priest a water supply yield of 48 MGD was computed for the 629 normal pool reservoir and a 43 MGD yield was achieved with a normal pool elevation of 625.

Listed in **Table 2A** are the evaporation rates by month, which are taken from the 1961 Survey Report of Percy Priest Reservoir. The data was developed from 20 to 30 years of recorded evaporation data. The estimated values total approximately 34 inches for a year.

Upon review of the first release of this report it was suggested that land pan evaporation rates for the Neapolis station located in Maury County, TN should be considered. The rates were reduced by seventy percent to equate actual rates that would be experienced in a shallow reservoir. The maximum evaporation rate for each month was used in the water supply yield analysis. The results of this analysis produced a yield of 47 MGD for the 629 normal pool and a 43 MGD yield with a normal pool elevation of 626.

Listed in **Table 2B** are the maximum evaporation rates by month, which are taken from the Neapolis Experiment Station, located in Maury County, TN. The data was developed from 17 years of recorded evaporation data for the site. The estimated values total approximately 38 inches for a year.

Table 2A

Average Monthly Reservoir Evaporation in Inches
Rates Used for Design of J. Percy Priest Reservoir

Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1.05	0.52	1.36	1.92	2.37	4.10	5.07	5.17	4.73	3.18	2.57	1.85

Table 2B

Maximum Monthly Reservoir Evaporation in Inches
Neapolis Experiment Station

Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
na	na	na	4.67	6.08	7.31	6.03	5.38	4.44	3.33	na	na

Leakage and Water Quality Allowances. It was decided at the onset of this study that a constant outflow of 5 cfs (cubic feet per second) should be maintained for environmental and water quality aspects. A flow of this magnitude is easy to achieve, because it usually occurs from leakage at and around the dam. This flow is accomplished, from a modeling standpoint, by assuming that an open pipe is located through the dam with an inlet near the streambed elevation. The diameter of the pipe is sized to approximate a 5 cfs flow over a normal range of pool elevations. The actual design of the outlet works to accomplish water quality goals would be completed at a later stage of design.

Outlet Works. The outlet works for dams typically accumulate into significant costs. However, for option 1, the outlet works cost may be less due to existing topographic conditions at and near the damsite. It is assumed that various notches could be cut at several locations to evacuate excess floodwaters for occurrences up to the PMF (Probable Maximum Flood, described later.) For this study, two lower level spillways were designed.

The lowest notch (designated as the service spillway) was designed to convey the 2-year frequency flood event. The second or higher elevation spillway was designed to convey the 100-year frequency flood event. A third spillway section could be cut that would safely pass the PMF. This open-cut spillway configuration would contain the two smaller, lower elevation spillway notches. The combined spillway section is designated as the emergency spillway. A sketch of this possible scenario for the two normal pool elevations is shown on **Figures 2A and 3A.**

The feasibility of option 1 was dependent on there being sufficient amounts of rock at the low topographic area to provide an adequate spillway foundation. From the geotechnical investigation this was determined not to be true. Therefore option 2 was developed based on the findings of the investigation.

The outlet works for option 2 were designed to minimize the cost of excavation, provide sufficient rock material for the two saddle dams and main dam construction and to keep the dam height to a minimum. The spillway configuration used will convey the 2, 10 and 100-year frequency flood events with acceptable velocities and depths. The spillway will also safely pass the PMF. A sketch of this possible spillway configuration for the two normal pool elevations is shown on **Figures 2B and 3B.**

Spillways and Top of Dam. The top of the dam elevation was approximated as that needed to pass a PMF through the emergency spillway without overtopping. An additional 3 feet of freeboard was added to the top of the dam elevation. The freeboard is included to account for protection from wind and wave runup.

Once the general height of the dam was set for water supply criteria, the spillway was designed to fit the contours of the site as well as continue to safely pass extreme flood events. The option 1 spillway configuration uses a combination of several notches. A 200 foot notch was needed to safely evacuate the 2-year frequency flood event. Another spillway notch was designed so that the two notches combined would safely pass the 100-year frequency flood event. A third, larger notch was designed such that a combination of the three spillways would safely pass the full PMF. The option 2 spillway consists of one notch that will safely pass the 2, 10, 100-year and the PMF.

Construction of the spillway will most likely require excavation of rock material that could provide fill material necessary for the construction of the dam embankments. Top of Dam elevations and spillway configurations are shown for the two

normal pool configurations on **Figures 2A and 3A** and **Figures 2B and 3B** for option 1 and 2, respectively.

Hydrologic Modeling Techniques

General. The first step in sizing and evaluating the Fountain Creek water supply dam was to develop and calibrate a rainfall-runoff model. The model is used to convert any design flood or historical rainfall events to runoff or inflow into the proposed reservoir. A storage pool is created in the model that meets the necessary storage requirements listed previously for sediment, water quality, evaporation, and water supply demand. Rainfall is then applied, using the model, over the watershed that drains to the reservoir. A pump is simulated within the model to pull the daily water supply demand from the reservoir. A pipe set at streambed elevation is used to simulate the discharge estimated for leakage and water quality releases from the reservoir. A starting reservoir water surface elevation equal to the service spillway crest elevation is used to start the model. The dams are then sized to safely pass the design flood through the outlet works.

Next, the model is changed to simulate rainfall-runoff over a continuous period of time. For this, daily rainfall is applied to the model. Based on the same conditions used in the design flood analysis, the pool elevation is monitored on a daily basis over the simulation periods. Historic drought years are used for this simulation. If the reservoir water surface drops to the top of the sediment pool elevation, from pumping, leakage and evaporation, the daily water supply demand is considered too great and reduced. This process is continued until the maximum water supply volume that can be pumped for each dam configuration is determined for all historical drought conditions on record. The resulting water supply yields from this analysis are shown in **Table 1A and 2A** for option 1 and 2, respectively. The following paragraphs describe this modeling process in more detail.

Rainfall-Runoff Modeling. The Corps' computer program, HEC-1 "Flood Hydrograph Package," was used to size and analyze the effectiveness of this water supply dam. HEC-1 produces discharge hydrographs which represent inflow into the reservoir. These discharge hydrographs are a result of applying rainfall excess, or runoff, to a unit hydrograph. The unit hydrographs are developed by synthetic methods and calibrated to historic events by simulating observed events. Rainfall excess is essentially that portion of rainfall that can be observed in a stream as

either baseflow or surface runoff. A significant portion of the volume of rainfall is either absorbed into the ground or vegetation. A modified version of HEC-1 was used to predict these transformations and provide the tools necessary to complete basic hydrologic designs for each of the potential water supply reservoirs. This section outlines the procedures and assumptions used to develop the HEC-1 model and the results of analyzing different dam alternatives.

Subbasin Delineation. The first step in a rainfall-runoff modeling process is to delineate and measure the drainage area of the subbasins. The portion of Fountain Creek watershed upstream of the proposed damsite was divided into five subbasins. The subbasins were defined to provide computed flows at two previously gaged locations on Fountain Creek and to combine major tributary inflows such as Globe Creek and Silver Creek. The subbasins range in size from 12 to 35 square miles of drainage area, with a total drainage basin area of 102 square miles above the damsite. The subbasin boundaries were determined from USGS 7.5 minute quadrangle topographic maps at a scale of: 1" = 2000' and a contour interval of 20 feet. These boundaries were digitized into a Computer Aided Design and Drafting (CADD) format. The CADD format provides an expedient method to change subbasin configurations and resulting drainage areas. The drainage basin configuration and stream gage locations are shown on **Figure 4**.

Estimation of Modeling Parameters. A unit hydrograph is used to distribute the runoff for a subbasin with respect to time. Each of the subbasins shown in **Figure 4** are represented by a Clark synthetic unit hydrograph. This type unit hydrograph is based on average basin time of concentration, a storage or basin shape coefficient (R), and a time-area curve. The Clark method produces better control than other methods in defining the shape and the volume expected from a subbasin.

The definition of shape and volume for any hydrograph is usually very important, however, for reservoir water supply type analyses only volume is critical. This is mainly due to two factors. First, all the flows from these basins are expected to be either captured in the reservoirs or delayed by the reservoir a sufficient amount of time to negate the need to accurately define the shape. Second, the available rainfall data for the historical drought periods found for this study are recorded as daily values. The time of concentrations for the basins delineated are much less than 24 hours. Therefore, the peaking of hydrographs due to intense rainfall bursts can not be accurately simulated with this type data.

Time of Concentration. The time of concentration (T_c) for each subbasin was determined by reproducing flow data at the Fountain Creek gage located near Fountain Heights. Using this method it was determined that the time of concentrations range from 4 to 12 hours for selected subbasins in the watershed.

Unit Hydrograph. The Clark R coefficient is usually referred to as a storage coefficient. This is because R affects the amount the hydrograph is lengthened with respect to time. Because this coefficient directly affects the shape of the outflow hydrograph, it can also be used to account for the effects of the basin shape. The basin shape is typically accounted for by a time area curve, however, such a curve is very cumbersome to develop. Therefore, the use of the R coefficient is faster and more direct. The coefficient R can be estimated using the relationship $R/(T_c + R)$. Then by setting this expression equal to a value between 0.1 and 0.7 a sharply peaked to an attenuated unit hydrograph can be obtained. Selection of this value is based on engineering judgment of the expected shape of each subbasin's outflow hydrograph and calibration to the average daily flow data at the Fountain Creek near Fountain Heights gage. A Clark R coefficient of 0.3 was selected for all subbasins in the watershed. In using R to account for basin shape, the default time area curve option was selected within the HEC-1 program.

API Continuous Losses. The Antecedent Precipitation Index (API) method was used to determine rainfall loss rates for the continuous simulations. The model used was developed by the Nashville District to model continuous events on the Cumberland River and its tributaries. Due to the fact that the general range of temperatures, seasonal variation in temperatures and vegetation types are similar for the Cumberland River Basin and the Fountain Creek basin, it is reasonable to assume this method is also valid for the Fountain Creek watershed. The API method of transforming rainfall to runoff is empirically based. Therefore, matching of observed occurrences is accomplished by using numerical techniques. The actual rainfall loss rate is based on the day of the year and the antecedent rainfall. The day of the year accounts for several values such as average temperature, evaporation rates, vegetation and hours of sunlight. The calibration of this model and its use in this study is discussed at the end of this section.

Reservoir Routing. The Modified Puls routing technique within HEC-1 was used to perform all reservoir routings. This is a simple level pool storage routing technique based on the

principle of conservation of mass and the relationship that change in reservoir storage, for a given time period, is equal to average inflow minus average outflow. The reservoir storage data used in HEC-1 were provided by TVA at 10-foot intervals. These values and their corresponding surface areas are shown in **Table 3**. The reservoir surface area at elevation 645 was digitized from 7.5 minute USGS topographic maps and is shown on **Figure 5**.

Storm Development. Several different types of storm events were used in the design of the outlet works for this water supply dam. Each of the three storm types described in the following paragraphs were used to size some portion of the dam. As described previously, the top of dam and emergency spillway are determined based on the PMF. The service spillway is based on a 2-year event and the water supply production capacity of the reservoir is based on a drought study. A spillway was also included to contain the 100-year flood event. In the following paragraphs there is a general discussion of each of the storm events.

Table 3
Fountain Creek
Elevation Area Storage Capacity Data
(from TVA)

Elevation	Area in acres	Storage in acre-feet
650	3770	103,100
640	2920	69,600
630	2090	44,600
620	1550	26,300
610	980	13,700
600	580	5,950
590	240	1,850
580	45	420
570	20	90
561	0	0

Probable Maximum Flood (PMF). The Probable Maximum Storm (PMS) is the precipitation event that produces the Probable Maximum Flood (PMF). This storm represents the most severe combination

of flood-producing rainfall patterns possible for a given area. The PMF is selected as the design flood for projects that allow no degree of risk and an unusually high degree of protection. This condition usually exists for high head dams or where a threat to life or extreme damage could occur downstream should the dam ever fail. The selection of the design storm is a key link between a rainfall-runoff model and the determination of the size of the actual dam. For this study, the PMF is used to insure the dam is safe from a flood of any possible magnitude. Even though this is a conservative assumption, it results in only a few more feet of dam height above the one half PMF and provides the ultimate in hydrologic safety. The PMS used in this study to produce the PMF was supplied by TVA. It was determined using methods provided in National Weather Service publication "HMR-56." The 6-hour rainfall values provided were arranged in a triangular distribution as shown in **Table 4** and then input into the HEC-1 model described previously to determine the final PMF level pool storage routing.

Table 4
Probable Maximum Storm Precipitation
6 Hour Values
(from TVA using HMR-56)

Depth in inches for 6-hour increments of PMS											
0.39	0.47	0.61	0.85	1.40	4.21	23.64	2.09	1.05	0.71	0.53	0.43

2-Year and 100-Year Flood Events. To develop the 2-year and 100-year flood events, the hypothetical storm method available in HEC-1 was utilized. This method used frequency rainfall data obtained from the US Weather Bureau publication "Technical Paper No. 40" to generate a triangular precipitation distribution. The storms were simulated in HEC-1 and the resulting flood flows into the reservoir were then determined. This hypothetical storm approach was used to determine the configuration of the service spillway notches. **Table 5** lists the 2-year and 100-year rainfall amounts for the study area.

Table 5
2-Year and 100-Year Frequency Rainfall (from TP-40 and TP-49)

Frequency	Duration in hours							
	1	2	3	6	12	24	48	96
2-Year	1.6	1.9	2.2	2.6	3.1	3.6	4.2	5.0
100-Year	3.0	3.9	4.3	5.1	6.0	7.0	8.5	10.0

Historical Rainfall Information. Historical precipitation data for the Fountain Creek watershed and surrounding areas were collected from various sources. Only one precipitation gage was available with a substantial period of record within the Fountain Creek watershed. The other precipitation gages used in this study were in the vicinity of the basin not more than thirty miles away. A total of ten gages were selected and collectively used to determine total continuous rainfall amounts and patterns for the years 1900 to 1995. The gages and their respective years of record available for use in this study are listed in **Table 6**.

Rainfall records for all of the gages listed in **Table 6** were retrieved from CD-ROM's containing NWS records, with the exception of the Culleoka gage. The Culleoka gage is an observer gage that was managed by TVA and not available in digital format. The records for this gage were obtained from the monthly "Precipitation in the Tennessee River Basin" books published by TVA. All of the precipitation gages used in this study had daily precipitation value totals as the minimum time period available.

DSS Database. All of the historical precipitation gage data records were entered into the Corps' DSS database system. The data were entered in regular time series format. The use of this database system allows direct input and output from many Corps' models such as HEC-1 and PRECIP.

Basin Average Rainfall. The Corps' computer program, PRECIP was used to develop a basin average rainfall for the Fountain Creek

watershed. PRECIP computes area-average hyetographs from observed precipitation gage data. Like HEC-1, the program is

Table 6
Available Precipitation Gage Information Within 30 Miles of Fountain Creek Watershed

Gage Name	Type	Dates in Operation
Ashwood	Precipitation	January 1900 - March 1977
Columbia	Precipitation	August 1948 - Present
Monsanto	Precipitation	August 1948 - July 1976
Mt. Pleasant	Precipitation	February 1953 - Present
Palmetto	Precipitation	January 1927 - March 1977
Lynville	Precipitation	January 1928 - December 1971
Lewisburg Experiment Station	Precipitation	January 1928 - Present
Culleoka	Precipitation	February 1967 - April 1980
Neapolis	Precipitation	January 1952 - March 1972
Neapolis Experiment Station	Precipitation	November 1975 - Present

designed for use with a DSS database. Rainfall at the centroid of the watershed is computed based on a weighted average of nearby rain gages. The gages are weighted based on the least distance squared from the watershed centroid. The daily basin average precipitation values for critical drought periods (discussed in later paragraphs) were computed by PRECIP and written into the DSS database. The computed values were then read directly into HEC-1 and transformed to inflow to the water supply reservoir system.

Drought Investigations. Drought data is the obvious means by which most water supply alternatives are designed. The idea behind the design of water supply reservoirs is to capture and store enough water during flood events within the reservoir to make it through the worst of drought conditions. Some water supply systems are sensitive to short droughts with no rain while others are sensitive to longer duration droughts with only moderate rain. Therefore, the development of a good historical database of rainfall is essential to the evaluation of water

supply alternatives. For this study, drought investigations were conducted by first identifying continuous periods that were particularly dry for 3, 4, 6, 12 and 24 months. This information is then applied to a continuous moisture accounting rainfall runoff model to determine the inflow that could be expected into a reservoir on Fountain Creek during critical drought periods. A review of the historical record was made and all periods indicating possible drought conditions were identified. From this review, the driest years on record were identified. They were: 1903-1904, 1913-1915, 1924, 1930-1931, 1939-1942, 1953, 1965-1967, 1980-1981, and 1987-1988. Of these droughts, the most critical to the reservoir evaluated on Fountain Creek was the 1913-1915 drought. A composite tabulation of basin average monthly rainfall data, for these periods, was computed using the PRECIP program for gages near the Fountain Creek watershed (from **Table 6**) and are listed in **Table 7**.

A review of rainfall data only provides an indication of when drought conditions might have occurred and their severity. Since runoff and not rainfall is the largest contributor of inflow to a reservoir, a rainfall-runoff model must be used. Small amounts of rainfall during one season of the year may contribute greatly to runoff while not at all during other seasons. The API methodology, described previously, was used to convert rainfall to runoff to account for these differences. This API continuous loss rate methodology was added by Nashville District to the HEC-1 computer model. To use such a modeling technique, the API parameters were calibrated to this region.

Rainfall-Runoff Model Calibration. Observed stream gage data must be available to perform any accurate type calibration on the HEC-1/API model. In the Fountain Creek watershed two stream gages were in operation from 1966 to 1968. Information from these gages is shown in **Table 8.** The location of these two gages are shown in **Figure 4.** Since the gages were in operation for essentially the same time period and all flows from the Culleoka gage were included in flows at the Fountain Heights gage, only the Fountain Heights gage was used for the calibration. The information recorded for these gages was published by the USGS in the annual water year publications for the years 1967 and 1968 and include the volume of water passing these gages daily, monthly and yearly. This number, when factored for the size of the contributing drainage area, provides the amount of total runoff from the basin upstream of the gage. The gaged area

Table 7
Fountain Creek Watershed Area
Droughts Over a 95 Year Period of Record
1900-1995
Monthly Rainfall Totals
(in inches)

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1903	3.22	5.27	5.66	2.23	4.33	5.02	3.90	1.40	0.50	0.30	4.63	3.47
1904	2.95	1.00	5.85	3.38	3.51	4.61	3.31	2.67	2.30	1.79	2.33	5.63
1913	8.82	4.80	4.89	1.67	2.72	0.60	2.75	4.44	1.77	2.90	0.91	2.45
1914	1.62	3.45	4.50	2.14	1.62	1.87	4.85	3.23	2.10	2.82	2.49	5.81
1915	5.42	2.43	3.09	0.30	4.54	3.10	2.00	9.54	4.25	2.34	6.45	6.40
1924	5.35	5.15	2.50	3.63	6.85	4.72	1.20	1.05	3.75	0.00	0.40	7.85
1930	3.38	5.19	4.65	1.09	7.38	0.91	1.81	2.29	2.52	3.49	4.34	2.87
1931	2.32	5.41	4.99	2.13	2.45	1.86	3.15	3.46	0.97	1.66	3.43	10.12
1939	7.66	12.54	6.58	5.01	3.97	10.93	2.77	2.15	1.28	1.08	1.37	3.54
1940	1.85	5.98	6.49	5.62	4.17	2.39	4.33	4.01	0.87	2.27	4.20	2.98
1941	2.62	1.18	2.70	4.20	1.16	2.14	9.20	4.00	0.40	3.01	3.36	3.69
1942	3.61	4.58	4.40	2.27	1.91	4.64	3.76	7.45	2.82	2.78	1.63	6.24
1953	5.96	8.15	6.49	7.47	3.95	2.98	5.11	1.54	1.74	0.27	0.57	4.45
1965	3.34	3.88	8.67	7.27	3.43	6.35	5.83	1.90	3.69	2.26	3.10	1.02
1966	3.68	3.91	1.79	3.51	5.86	2.16	5.17	6.18	2.62	3.38	3.06	7.57
1967	1.90	2.42	3.74	3.49	14.42	4.47	8.36	5.31	3.44	2.50	3.11	5.81
1980	4.56	1.66	12.89	4.52	8.12	3.29	2.87	1.15	2.37	1.98	2.72	1.37
1981	1.69	3.83	4.21	3.91	2.96	5.09	5.91	3.93	3.23	3.76	2.27	3.43
1987	3.18	6.21	2.45	1.85	2.56	3.00	4.47	1.95	3.09	0.47	5.10	5.26
1988	3.99	2.96	3.18	5.69	1.83	0.17	4.48	1.86	4.87	2.40	6.56	4.56

represents approximately 70 percent of the total inflow into the water supply reservoir. A schematic of the routing technique and subbasin combinations is shown in **Figure 6**.

Table 8
Available Stream Gage Information
Fountain Creek Watershed

Gage Name	Type	Drainage Area	Dates of Operation
Fountain Creek Near Culleoka	stage -rated flow	26.9 sq. mi.	May 1966 - September 1968
Fountain Creek Near Fountain Heights	stage -rated flow	70.4 sq. mi.	March 1966 - September 1968

To calibrate the Fountain Creek watershed models, the resulting inches of total annual runoff at the Fountain Heights gage were matched as close as possible to the observed data by varying the API parameters. Plots of the final model calibration runs are shown in **Figure 7**.

The calibrated HEC-1/API model was used to determine the maximum safe yield, for the Fountain Creek reservoir, based on historic data from 1900 to 1995. This was obtained by simulating, for continuous periods the actual rainfall leading up to and proceeding the drought years defined previously. The daily pumping rate from the reservoir was varied until the necessary storage remained available in the reservoir to maintain that pumping rate over a continuous period. These values are reported as the design capacities in Table 1.

Dam Alternatives

The two dam configurations, for option 1 and 2, used in the scope of this study, assumed earthfill or rockfill embankments that are not overtoppable. However, a concrete or rollfill dam embankment that would be overtoppable could also be used at the damsite, this would allow the top of dam elevation to be lowered approximately 10 to 15 feet. The savings in fill materials and

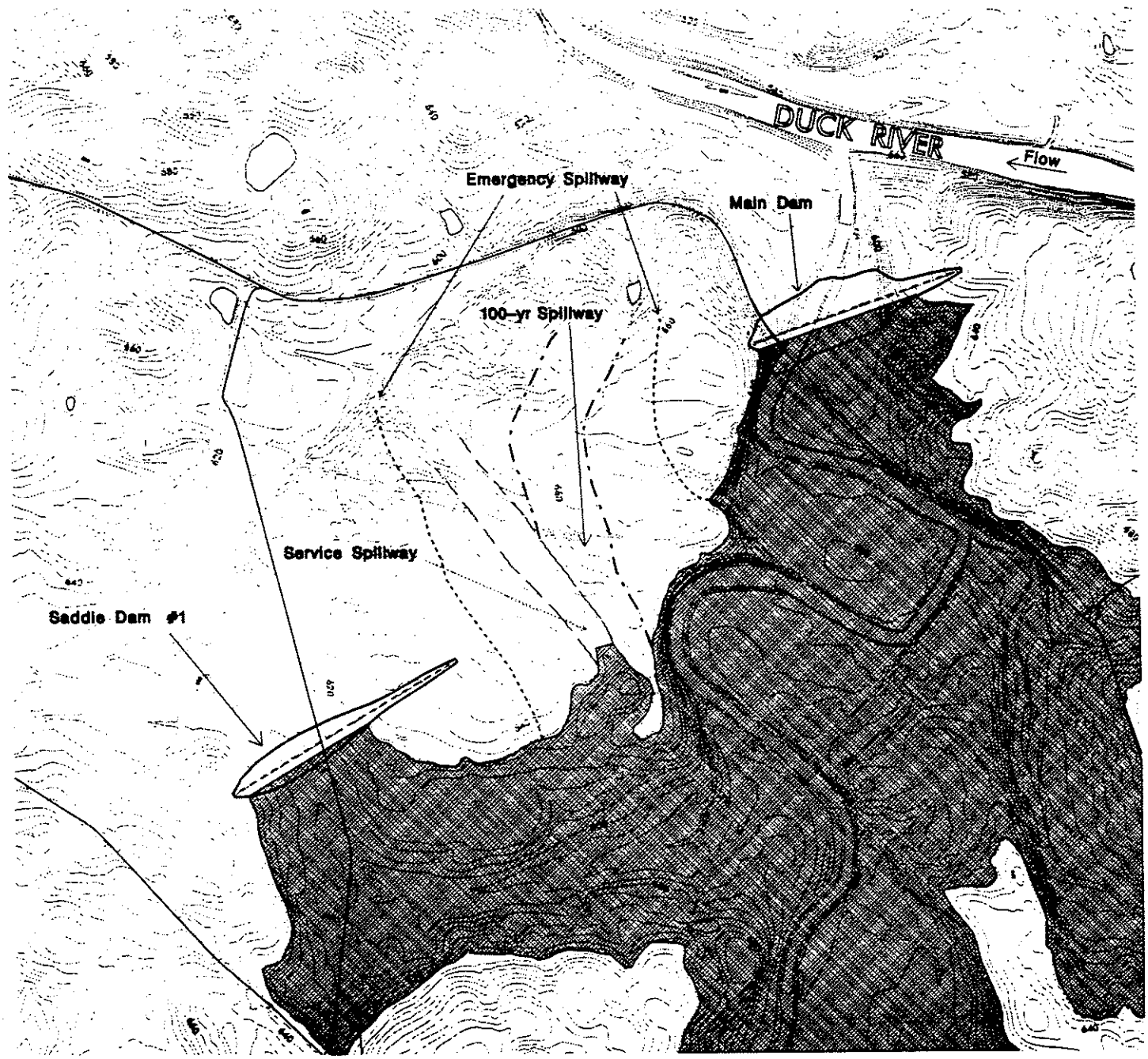
real estate acquisition may offset the increased cost of using a concrete or rollfill dam.

Another alternative would be to locate the emergency spillway in the lower saddle area to the west of the Fountain Creek main dam rather than filling this area with a saddle dam embankment (see **Figure 1A and 1B**). This alternative could possibly be a cost efficient means of filling the saddle area and preventing excavation in the mound between the lower saddle area and the Fountain Creek main dam.

A cost comparison between the option 1 and option 2 spillway configurations (see **Figure 1A and 1B**), should also be made to compare the cost of installing a spillway in the low topographic area versus cutting a spillway in the bank just east of the main dam. These measures should be done at an increased level of design.

Downstream Flooding

The purpose of performing this study was to determine the applicability of the Fountain Creek damsite for water supply purposes only. The associated benefit of flood control was not considered as part of the analysis of the reservoir. The frequency floods (2-year and 100-year) and the PMF were considered in designing the outlet works to prevent overtopping only, and not to prevent any downstream flooding. However, the dam is intended to store as much runoff as possible and only release excess amounts. This is runoff that would be discharged from the watershed under natural conditions anyway. Therefore, flooding downstream of the dam should not be appreciably increased due to the dam. However, a detailed study of downstream flooding impacts would be required to determine the exact impacts, if any.



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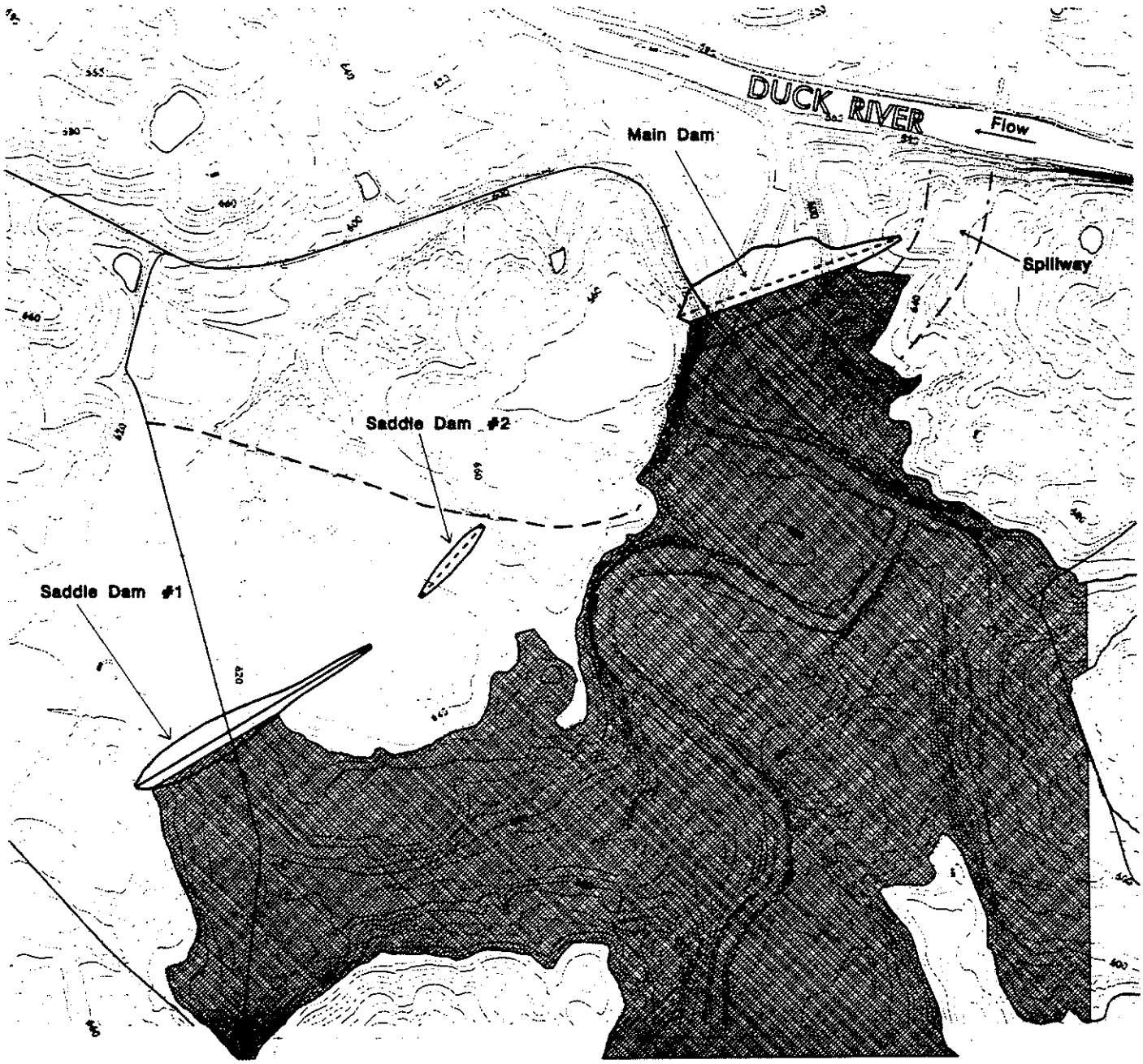
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Fountain Creek Damsite
General Plan
Option 1

U.S. Army Corps of Engineers
Nashville District

May 1997

Figure 1A



Scale: 1" = 1000'
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Fountain Creek Damsite
General Plan
Option 2

U.S. Army Corps of Engineers
Nashville District

May 1997

Figure 1B

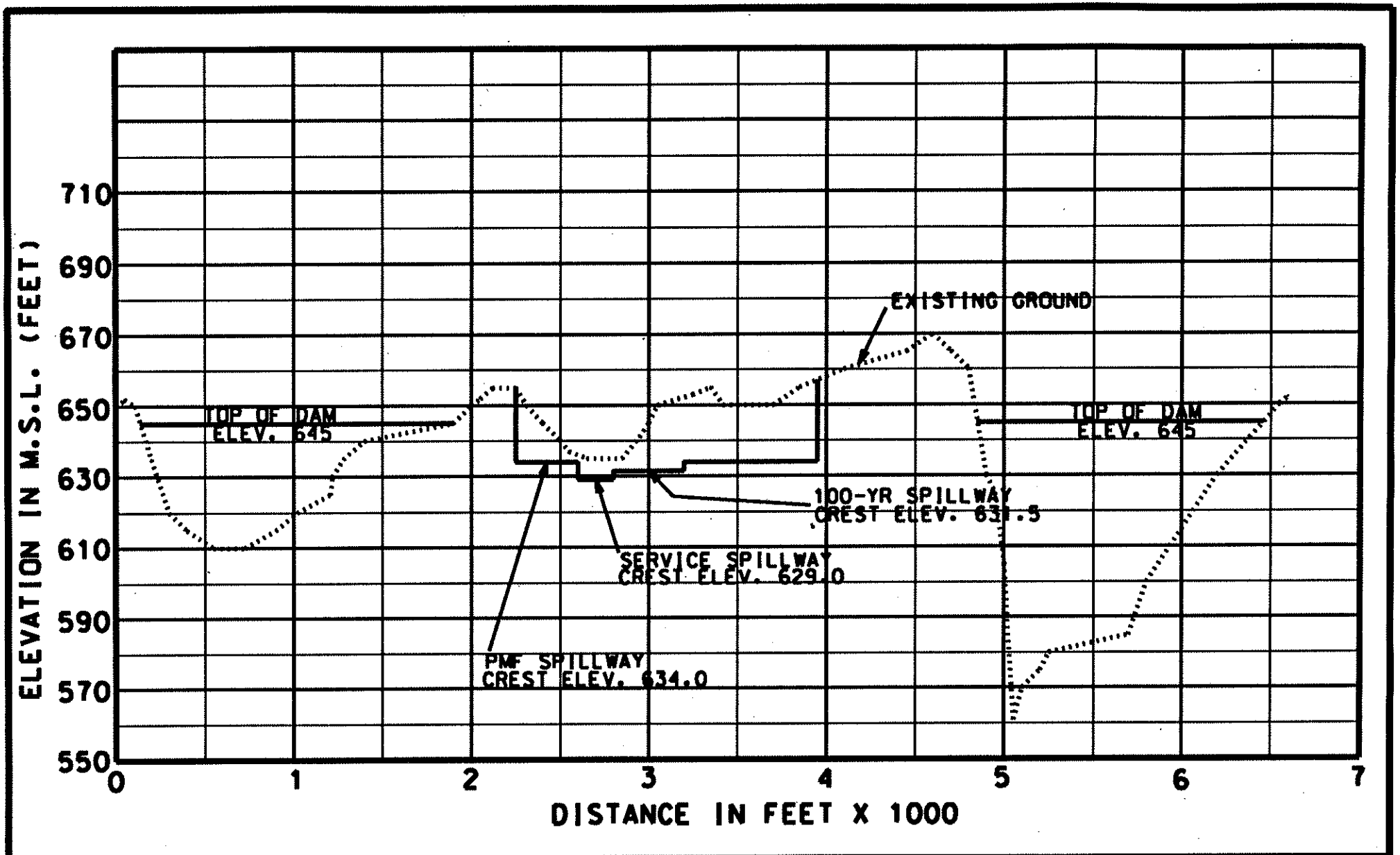


Figure 2A

Fountain Creek (629) Normal Pool and Spillway Section Option 1

U.S. Army Corps of Engineers
Nashville District

October 1997

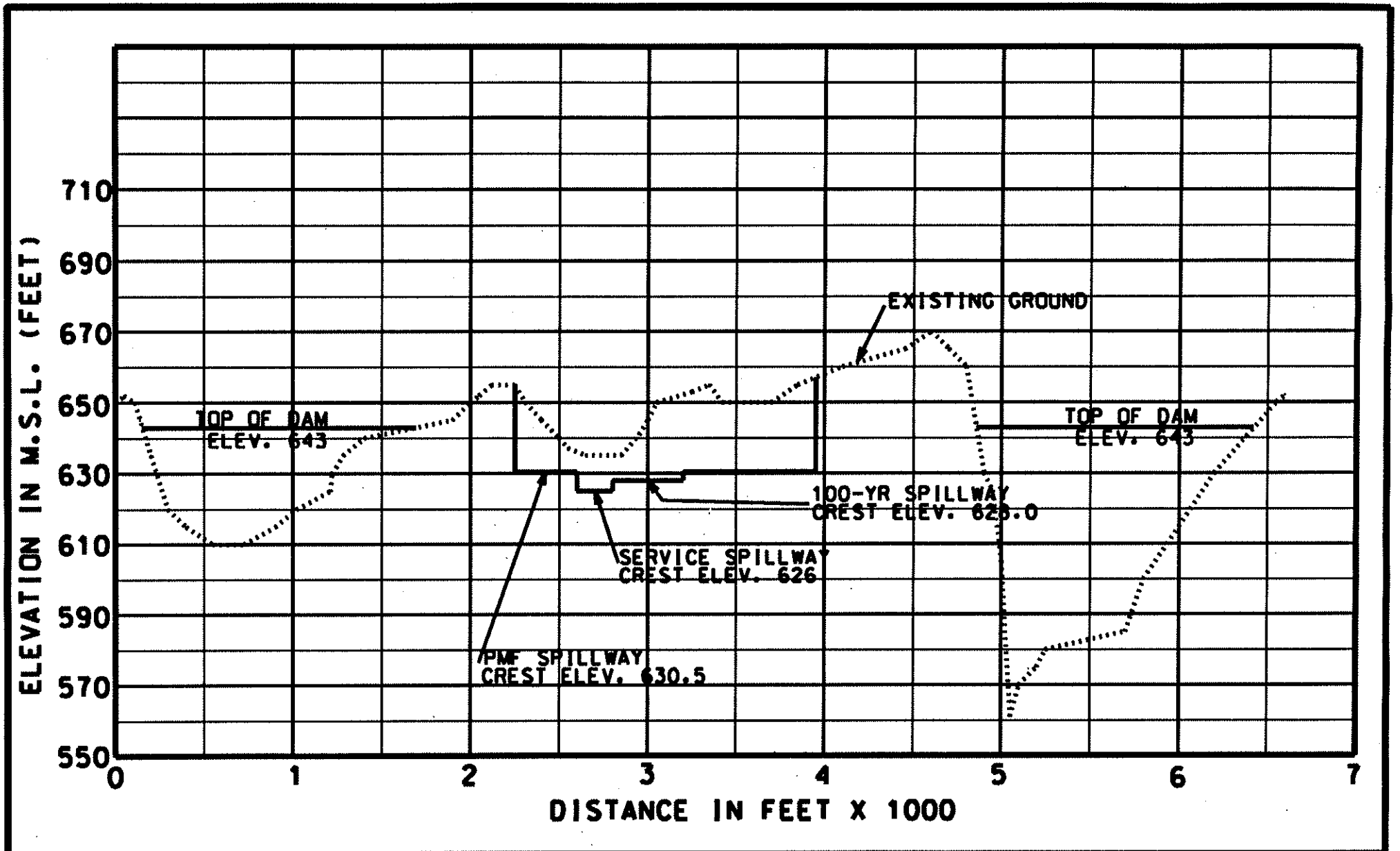


Figure 28

Fountain Creek (626) Normal Pool and Spillway Section Option 1

U.S. Army Corps of Engineers
Nashville District

October 1997

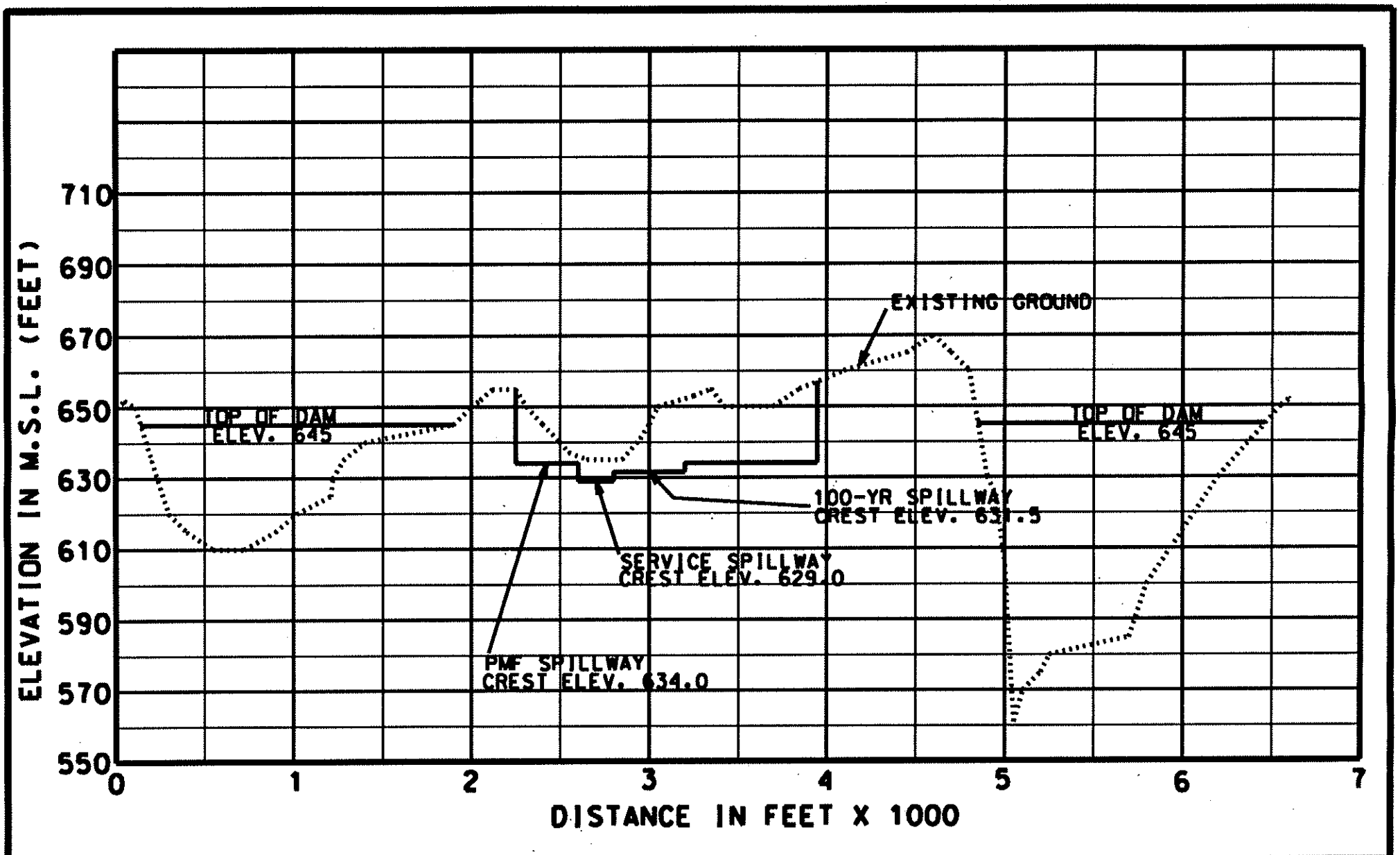


Figure 3A

Fountain Creek (629) Normal Pool and Spillway Section Option 2

U.S. Army Corps of Engineers
Nashville District

October 1997

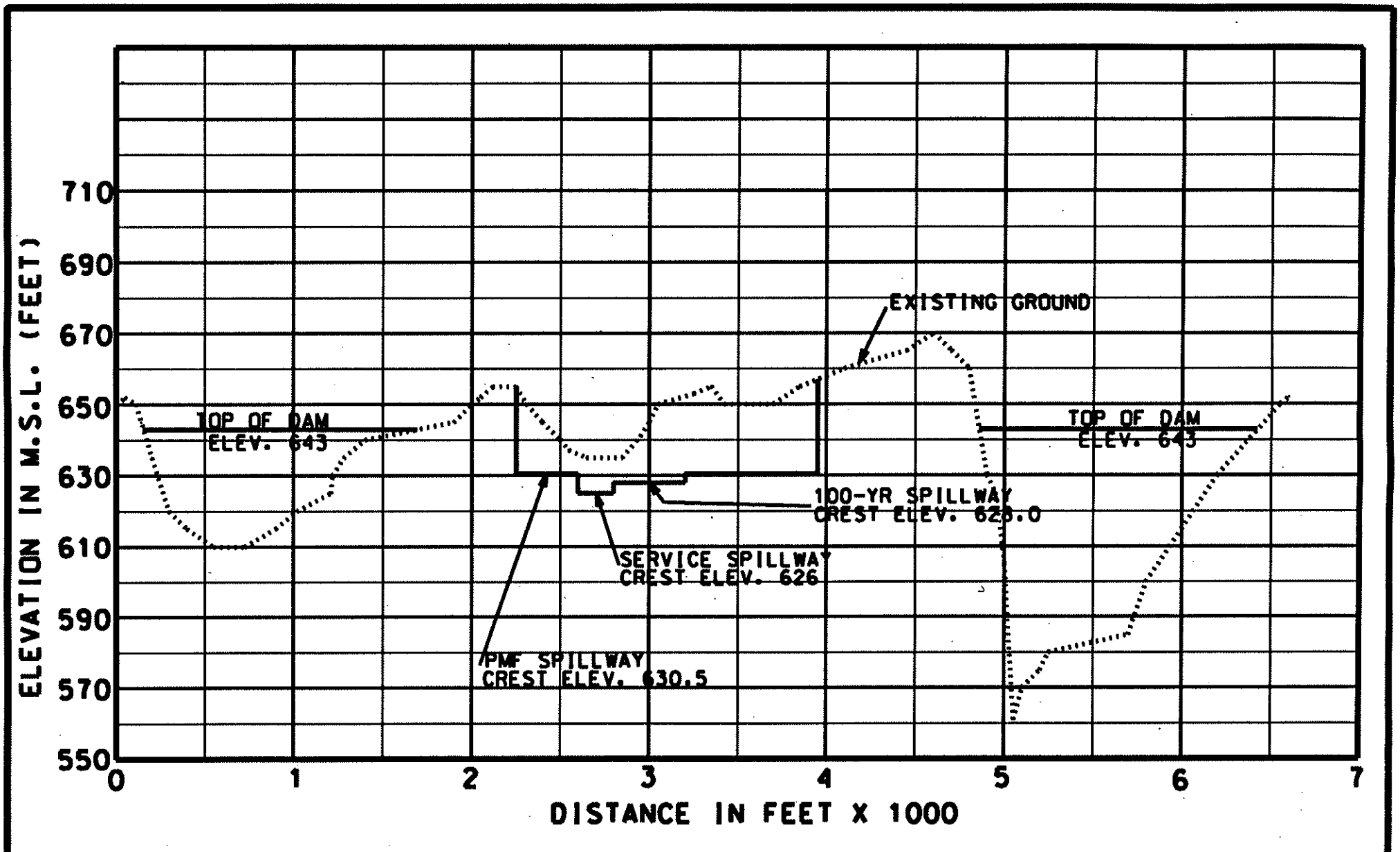
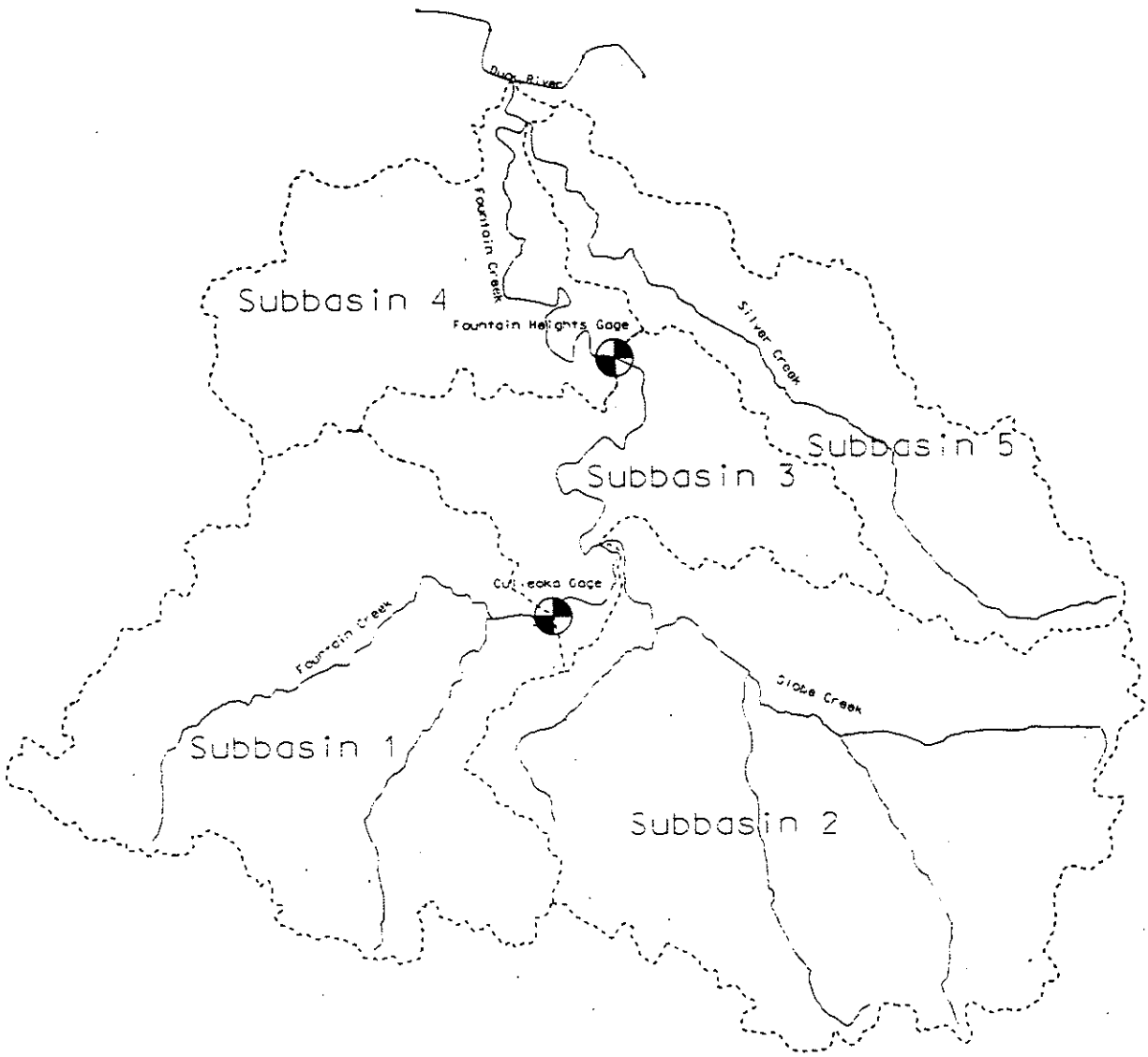


Figure 38

Fountain Creek (626) Normal Pool and Spillway Section Option 2

U.S. Army Corps of Engineers
Nashville District

October 1997

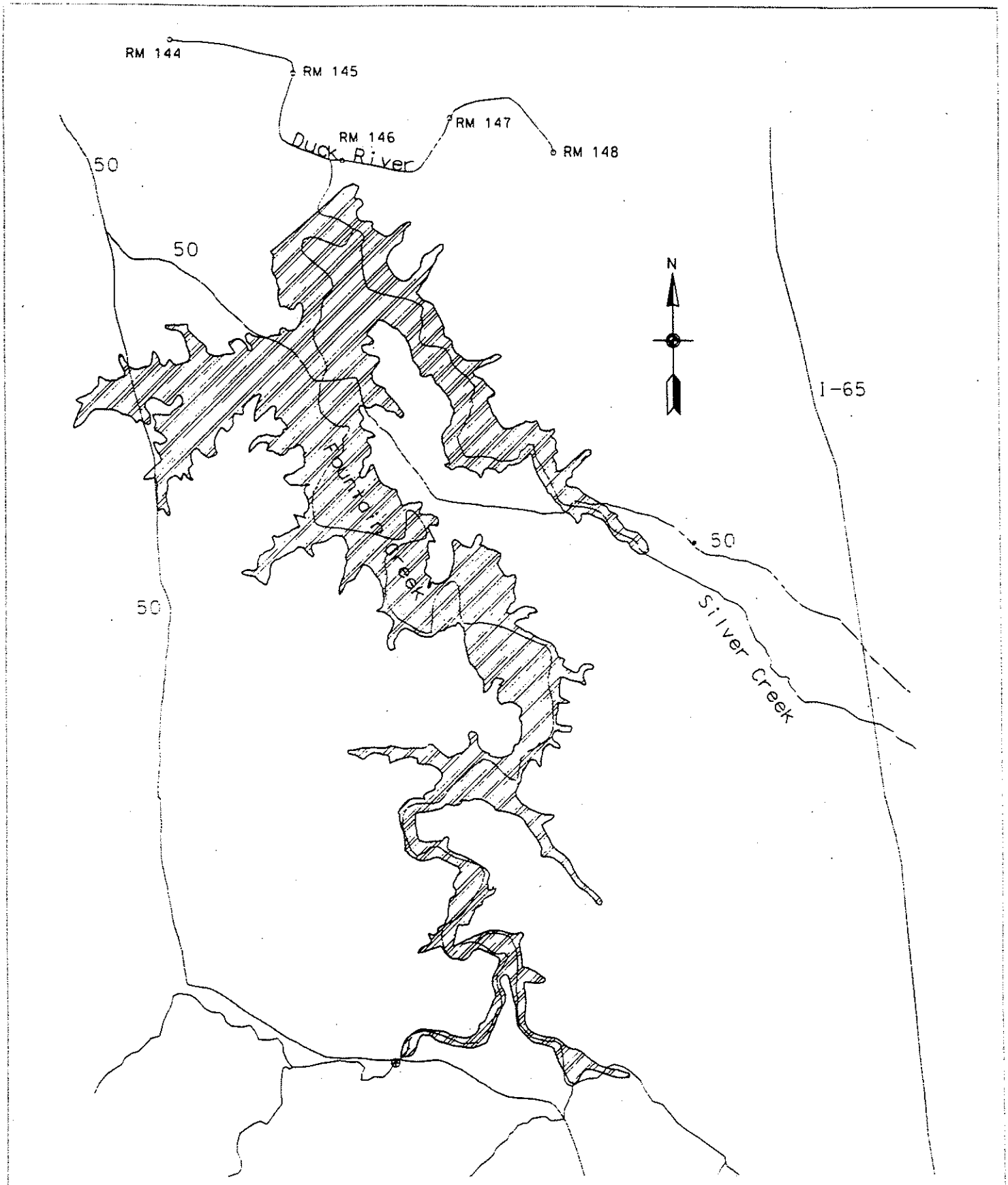


Fountain Creek Watershed
Subbasin Delineation

U.S. Army Corps of Engineers
Nashville District

October 1997

Figure 4

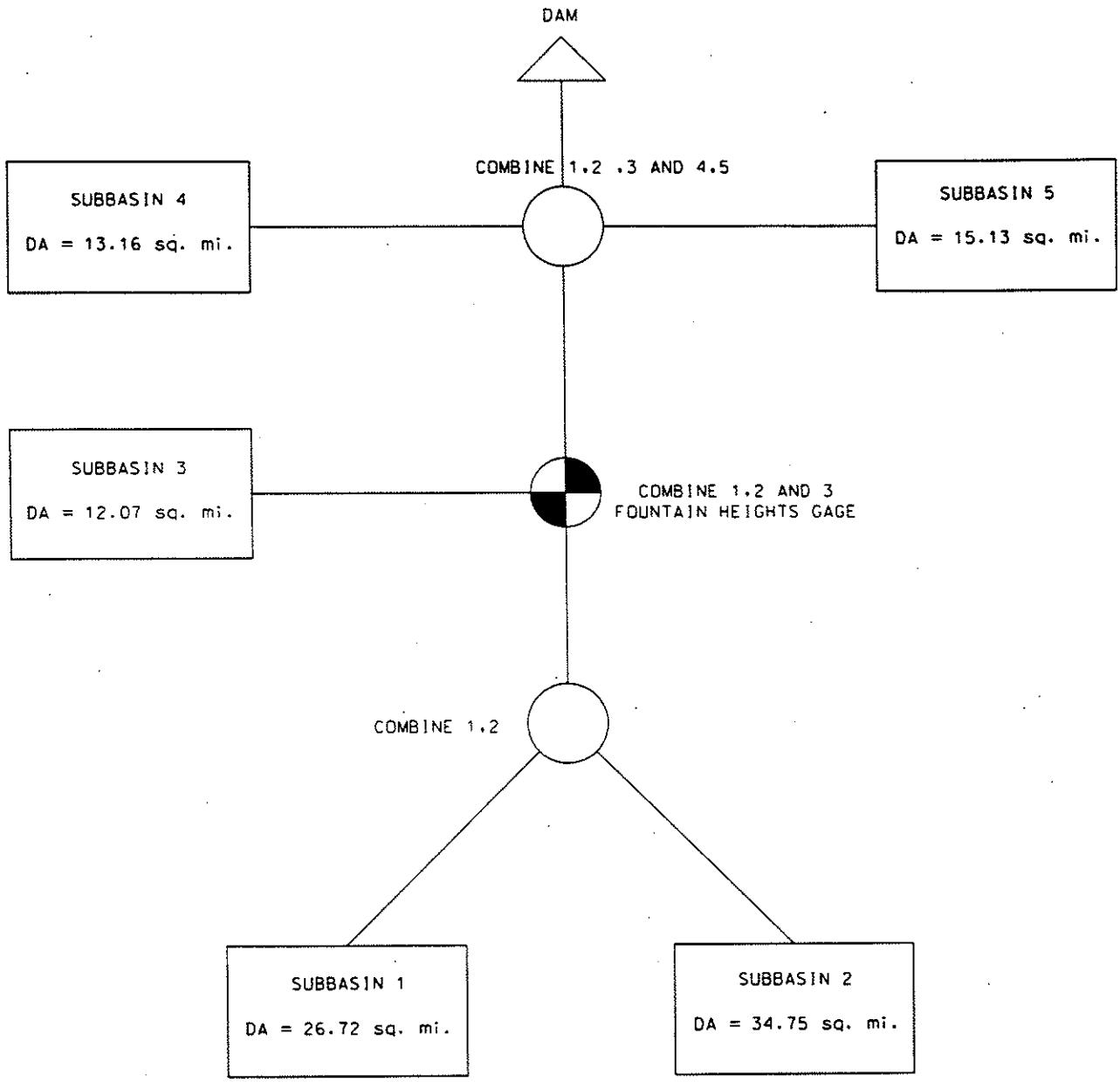


Fountain Creek Dam
Reservoir Surface Area
At Elevation 645

U.S. Army Corps of Engineers
Nashville District

October 1997

Figure 5



Fountain Creek Dam
HEC-1 Schematic

U.S. Army Corps of Engineers
Nashville District

October 1997

Figure 6

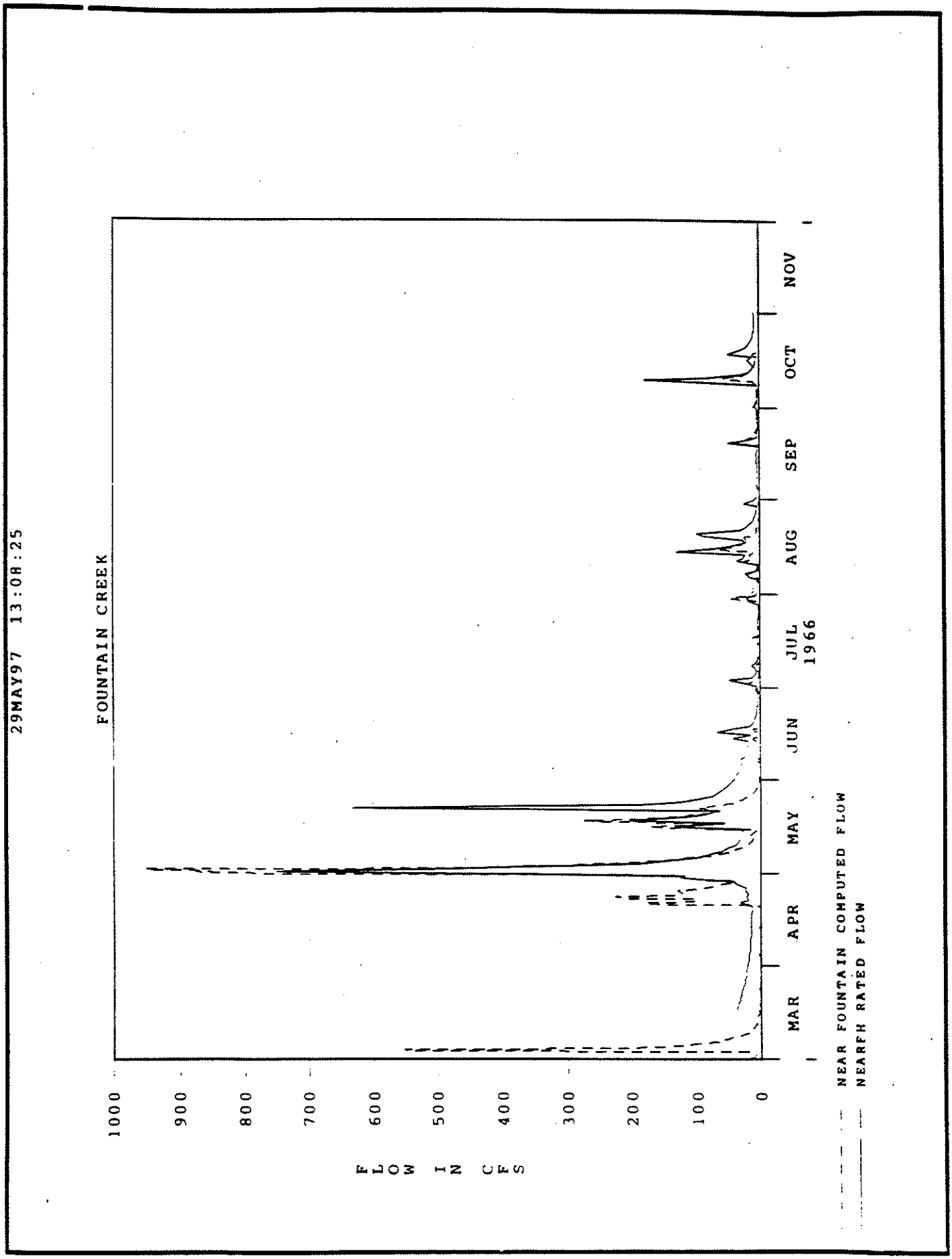

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NEAR FOUNTAIN COMPUTED FLOW
NEARFH RATED FLOW

Figure 7

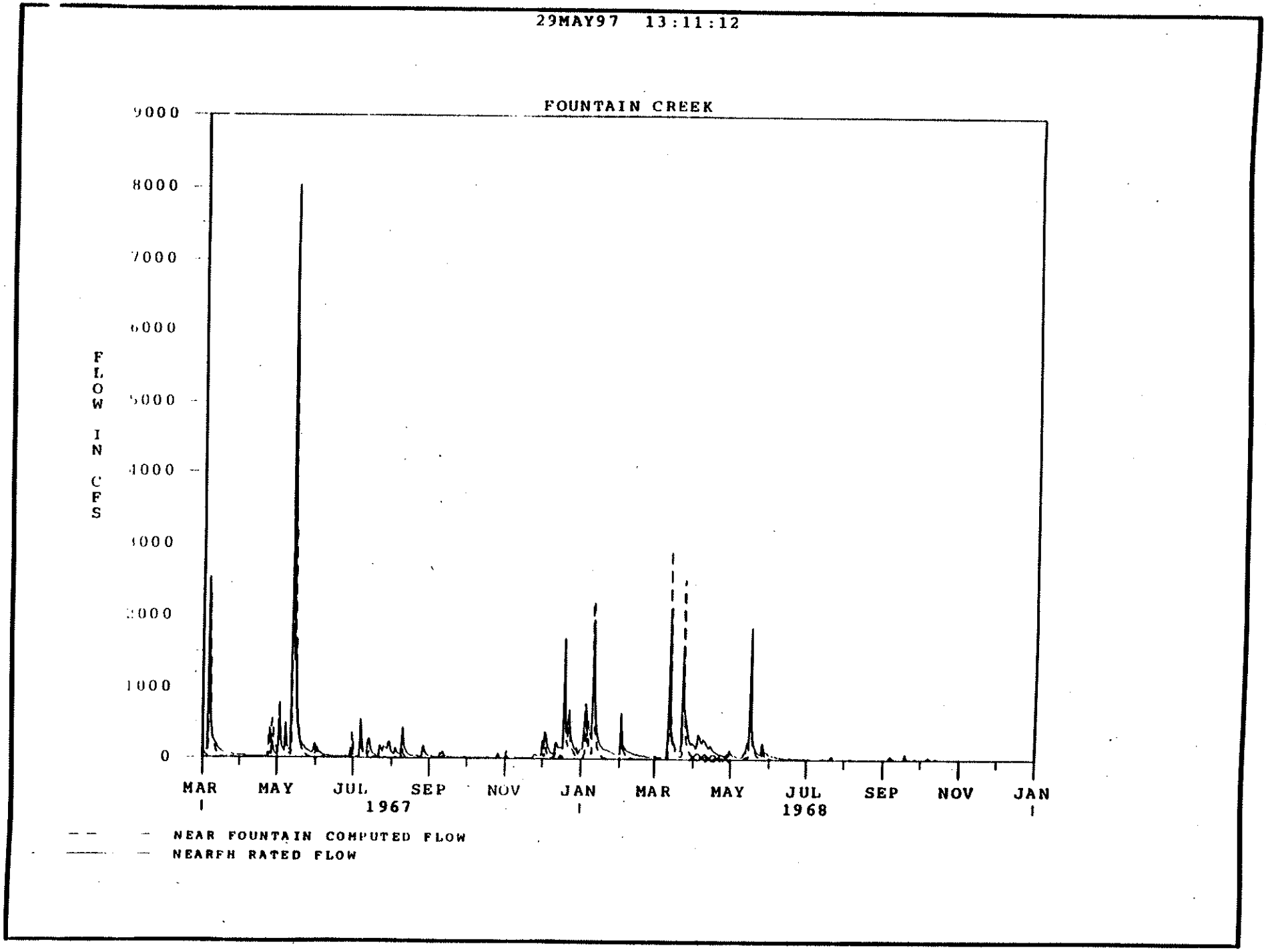

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JAN
1967
1968
NEAR FOUNTAIN COMPUTED FLOW
NEARFH RATED FLOW

Figure 7