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December 30, 1983
RBG- 16,676
File No. G9.5, G9.8.6.2

Mr. Harold R. Denton, Director
Office of Nuclear Reactor Regulation
U. S. Nuclear Regulatory Commission
Washington, D. C. 20555

Dear Mr. Denton:

River Bend Station Units 1 & 2
Docket Nos. 50-458/50-459

Enclosed are Gulf States Utilities Company's (GSU) responses to the items identified by reviewers from the Environmental and Hydrological Engineering Branch (EHEB) in the Draft Safety Evaluation Report (DSER). The Nuclear Regulatory Commission (NRC) Staff indicated on DSER Pages 2-40 and 2-42 (Sections 2.4.8.2 and 2.4.10 respectively) that groundwater levels which were used in combination with other environmental loads must be identified. The Final Safety Analysis Report (FSAR) establishes normal and maximum groundwater evaluations in Section 2.4.13.5, while loads and loading combinations are described in Section 3.8.4.3. Also, the DSER identified in Section 2.4.10, Page 2-42, disagreements on the appropriate Design Basis Flood Level (DBFL) for flooding from local intense precipitation on West Creek and the plant site drainage system. GSU has reviewed Hydrometeorological Report No. 52 and River Bend Station (RBS) assumptions and conservatisms used to determine the DBFL.

The enclosure to this letter provides the changes to FSAR Section 2.4 (replacing Pages 2.4-2 thru 2.4-30, i.e. thru 2.4.4.2) and all associated graphs and figures, and represents the results of the DBFL reanalysis to be included in the next FSAR amendment. These revisions include discussion of the West Creek sediment deposition (DSER 2.4.3.2, Page 2-26) supplementing Question 240.8, West Creek overflow

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into Unit 2 excavation area (DSER 2.4.3.2 Pages 2-24/28) supplementing Question 240.10, and West Creek unit graph discrepancies between the construction permit and operating license stage (DSER 2.4.3.2 Page 2-25) supplementing Questions 240.7. This information provides the responses requested by EHEB reviewers to complete their analysis of local flooding for the Safety Evaluation Report (SER).

Sincerely,

J. E. Booker

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Manager-Engineering
Nuclear Fuels & Licensing
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Enclosure

ENCLOSURE

2.4 HYDROLOGIC ENGINEERING

2.4.1 Hydrologic Description

2.4.1.1 Site and Facilities

A description of the site is presented in Section 2.1.

Fig. 2.1-2 is a topographic map of the site as it is to look after construction.

The site includes two general levels of terrace. The alluvial floodplain on the east side of the river varies from 3,000 to 4,000 ft wide, and is at about 35 ft msl. The upper terrace has an average elevation of over 100 ft msl. The station buildings and all safety-related equipment are located on the upper terrace. The original ground grade in this area was about el 110 ft msl. The finished ground grade is el 95 ft msl.

The site is drained by Grants Bayou on the east and Alligator Bayou on the west. Numerous unnamed intermittent streams cross the site and drain to either Grants or Alligator Bayou. Grants Bayou enters Alligator Bayou to the south of the site. It then flows south into Thompson Creek, which enters the Mississippi River approximately 7 mi downstream of the River Bend Station embayment.

The maximum postulated floods that can occur at the site are identified in Section 2.4.3. Section 3.4 describes the design considerations in regard to these floods. All safety-related equipment is contained in Seismic Category I buildings. Equipment in buildings not sealed from floodwater entry is at a minimum elevation of 98 ft msl.

The plant drainage system and the ability of the site to withstand a local intense Probable Maximum Precipitation event are discussed in Section 2.4.2.3.

2.4.1.2 Hydrosphere

The hydrologic behavior of nearby rivers, streams, and ponds has a strong influence on plant siting and elevations. Other hydrologic features considered in siting are dams, levees, and floodways as well as the present users of surface and ground water. 1

2.4.1.2.1 Mississippi River

The River Bend Station site is located adjacent to the Mississippi River at about River Mile 262. The river at St. Francisville (River Mile 266.0) has a contributing drainage area of about 1,129,400 sq mi. This area includes 41 percent of the conterminous United States(ref 1)

The major tributary rivers above Red River Landing (River Mile 302) with their respective contributing and noncontributing drainage areas and major subdivisions are listed in Table 2.4-1 and shown in Fig. 2.4-1.

The western limit of the drainage area is the Rocky Mountains. The eastern limit is the Appalachian Mountain chain. About 13,000 sq mi of the drainage area are in Canada(ref 1).

The Mississippi River rises in northern Minnesota and flows in a southerly direction to discharge into the Gulf of Mexico at the Head of Passes. Among the principal influents are the Missouri River at River Mile 1,159, the Ohio River at River Mile 964, the White River at River Mile 583, and the Arkansas River at River Mile 575(ref 1).

The Red and Ouachita Rivers do not physically join the main stem of the Mississippi River, but discharge directly to the Gulf through the Atchafalaya River. At River Mile 315, part of the discharge leaves the main stem of the Mississippi River and flows through the Old River control structures to the Atchafalaya River.

The valley walls on both sides of the floodplain converge at the latitude of Red River Landing near Torras, LA (River Mile 302). This section marks the beginning of the deltaic plain and of the Atchafalaya River.

The average annual precipitation over the entire Mississippi River basin is about 30.8 in and varies from 21.8 in over the Missouri River Basin to 48.5 in over the Lower Mississippi River Basin(ref 1).

River discharge and stage measurement stations are maintained at numerous locations by the U.S. Army Corps of Engineers and the U.S. Geological Survey. A few have records beginning in the 1870s, but most of the records started in the 1920s. The runoff volume for the entire basin averages 480 million acre-feet annually. This runoff is equivalent to a mean annual discharge of 660,000 cfs for the entire basin. Based on Corps of Engineer flow records at Tarbert Landing, MI, and Red River Landing, LA, the estimated mean annual discharge at the site is about 447,000 cfs. Table 2.4-2 lists monthly and annual runoff for the drainage areas shown in Fig. 2.4-1.

The Mississippi River and its tributaries have many flow control structures, such as levees, floodways, and dams The following discussion provides a description of these structures.

Levees

The alluvial valley of the Mississippi River extends from Cape Girardeau, MO, about 50 mi upstream from Cairo, IL (River Mile 956) to the Gulf of Mexico. It varies in width from 20 to 80 mi with an average width of 45 mi(ref 1). During a flood, the river goes out of its banks in some areas and deposits sediment, forming banks generally 10 to 15 ft above the floodplain(ref 2). This building of natural levees occurred, for the most part, before the present levee system was constructed. The river has almost uninterrupted manmade levees on the west bank from Cape Girardeau to the Gulf. On the east side of the river, levees alternate with high bluffs from Cairo to Baton Rouge (River Mile 230); from Baton Rouge to the Gulf, there are continuous levees(ref 1).

The Floodway System

Considering all the control structures in the Mississippi River basin, the floodway system and associated structures in the river delta have the most direct bearing on river flood stage at the site. The system consists of three major floodways, which are the West Atchafalaya Floodway, the Morganza Floodway, and the Bonnet Carre Spillway plus the Atchafalaya River proper(ref 3). This system is shown in Fig. 2.4-2.

The Atchafalaya River is the continuation of the Red River. It starts at the latitude of Red River Landing (River Mile 302) and discharges into Atchafalaya Bay at the Gulf of Mexico. Acting as a distributary, it also receives water from the Mississippi River through the Old River control structures (River Mile 315)(ref 3).

The West Atchafalaya Floodway also starts at the latitude of Red River Landing and parallels the Atchafalaya River. The Morganza Floodway leaves the main stem of the Mississippi River at about River Mile 285. It flows west and then parallels the two Atchafalaya floodways and eventually merges with them to become the Lower Atchafalaya Floodway. The Bonnet Carre Spillway leaves the main stem of the Mississippi River at about River Mile 128 and directs floodwaters into Lake Pontchartrain and then to the Gulf(ref 3,4).

The chronological sequence of floodway operation during a severe flood would be as follows.

As the river discharge approaches 1,250,000 cfs, the Bonnet Carre Spillway is opened. The spillway is operated to prevent the Carrolton (New Orleans) stage from exceeding 20 ft . As the flow increases, the Old River control structures would be operated to allow water from the Mississippi River to flow into the Atchafalaya River. The Morganza Floodway is the next flood relief structure which would be operated(ref 3,4).

The West Atchafalaya Floodway is protected at its upper end by a fuse-plug dike that closes its entire length of about 7 mi. Water that cannot be immediately discharged by the Atchafalaya River proper is stored in the backwater areas of the Red and Ouachita Rivers. Backwater storage continues until floodwaters overtop and wash out the fuse-plug, making the West Atchafalaya Floodway operational. The remaining flood flow is discharged by the Mississippi River and the Bonnet Carre Spillway(ref 3,4).

The combined discharge of the three parallel floodways is on the order of one-half the Corps of Engineers project design flood (PDF) at the latitude of Red River Landing. The maximum postulated flood flow that has been calculated by the Corps is officially defined as a lower Mississippi River PDF (Section 2.4.3).

Dams

Dams are discussed in Section 2.4.4

2.4.1.2.2 Local Streams

The River Bend Station site is located above the Mississippi River floodplain on elevated, gently sloping terrain approximately 2 mi east of the Mississippi River at River Mile 262. Local streams are intermittent or have a low base flow with a tendency to rise and fall rapidly dependent upon local rainstorms. Peak flows in these streams are discussed in Section 2.4.2.

The site lies within the 15.6 sq mi Grants Bayou drainage basin, shown in Fig. 2.4-3. Approximately 8.4 sq mi of the basin (Upper Grants Bayou) lie upstream from the plant site. Just south of the site, a small tributary of Grants Bayou enters from the west. This stream, called West Creek, drains approximately 1 sq mi including portions of the plant site.

The reactor units are situated between Grants Bayou and West Creek (Fig. 2.1-2). Flooding of these two streams is the chief flooding concern for this site and is discussed in Section 2.4.3.

The adjacent Alligator Bayou drainage basin drains a portion of the site property. Above the river floodplain, this same stream is called Alexander Creek. The U.S. Geological Survey has maintained a crest stage gauge on Alexander Creek since 1953, and has collected peak stage data on an annual basis noncontinuously to the present. The area of the Alexander Creek basin above the gauge is about 23.9 sq mi. The area of the Alligator Bayou basin north of the southern Gulf States property line is approximately 30.4 sq mi.

Grants Bayou joins Alligator Bayou in the river floodplain just south of the Gulf States property. Alligator Bayou joins Thompson Creek about 3 mi above the point where Thompson Creek flows into the Mississippi River.

Stream Control Structures

There are no control structures on Grants Bayou. Plant construction has no significant effect upon flood flows or stages. West Creek has been confined in a 2,850-ft-long Fabriform-lined channel slightly west of the plant area. Storm runoff is directed into the channel through a concrete drop structure at the upstream end.

Bridges on local streams are discussed in Section 2.4.3.

2.4.1.2.3 Lakes

There are a number of small farm ponds in the local watershed area, but few natural lakes. Twenty-four ponds existed within the site boundary prior to construction (Fig. 2.4-4). Due to construction this has been reduced to 20 ponds, with an overall increase in total pond surface area from 28.6 acres to 61.1 acres due to the impoundment of Wildlife Management Lake (Fig. 2.4-4). The impact of local ponds on site flooding is discussed in Section 2.4.3.

The water level of pond No. 11 is 11.0 ft msl which is higher than the average site grade of 95.0 ft msl. The average depth of the pond is approximately 4 ft. The estimated storage capacity of this pond is 1,167,000 ft**3.

The pond may fail under the conditions of the PMF, the 1/2 PMF coincident with an OBE, or the 25-year flood coincident with an SSE. Since the pond is likely to have failed prior to the arrival of the PMF peak flow, the pond failure would not make the PMF more critical. However, if the pond fails during the OBE or SSE, the flood from the pond failure could directly add to the peak flow. Consequently, the latter case is analyzed for its safety implications.

The peak flow of instantaneous and complete failure of pond No. 11 can be conservatively estimated by the following relationship (Ref 62):

$$Q = \frac{8}{27} g^{1/2} y_o^{3/2} L$$

where:

Q	= Peak flow
g	= Gravity acceleration
y_o	= Water depth at breaching place
L	= Width

The estimated peak flow is 3,363 cfs for a breaching width of 250 ft and a water depth of 4 ft. This flood would enter Grants Bayou at a point upstream of East Creek. Due to the relatively large amount of channel storage in Grants Bayou and the high level of the divide between Grants Bayou and the site, the flow of pond No. 11 failure does not flood the plant area directly. Water levels would be increased in Grants Bayou for the 1/2 PMF and 25-yr. flood events, but would not exceed the estimated peak flood levels for the PMF event. An increase in West Creek water levels at cross section W1 would occur, but would not be as severe as postulated for the PMF. Therefore, failure of Pond No. 11 does not affect the design basis water level at River Bend Station.

2.4.1.2.4 Users of River Water

Industrial water users along the Mississippi River downstream of the site are listed in Table 2.4-3 and located in Fig. 2.4-5. Industrial water pumpage is shown in Table 2.4-3.

Domestic water use of the river water within 100 river miles of the station is limited to the Bayou Lafourche Fresh Water District. Intake pumps are located at River Mile 175.5 and river water is conveyed at a maximum rate of 259.2 mgd to the bayou. The first municipal intake on the bayou is People's Water Service Company, Inc., of Donaldsonville, LA.

A tabulation of groundwater users is contained in Section 2.4.13.

2.4.2 Floods

2.4.2.1 Flood History

2.4.2.1.1 Mississippi River

Major floods on the Lower Mississippi River (below confluence with the Ohio River) generally occur when floods of the major tributaries coincide. A substantial contribution from the Ohio River is required to produce a major flood.

Major floods on the Ohio River generally occur between the middle of January and the middle of April. On the Upper Mississippi and Missouri Rivers, floods occur between mid-April and July, and on the Arkansas and White Rivers, between April and June. The flood season on the Lower Mississippi includes the flood seasons of the individual influents and extends from mid-January to July.

The first Mississippi River flood in recorded history occurred in 1543. There are fragmentary records of great floods occurring in 1782, 1785, 1796, 1809, 1815, 1823, 1844, 1849, 1858, 1862, 1867, and 1882. In more recent times, there are records of floods occurring in 1903, 1912, 1913, 1916, 1922, 1927, 1937, 1945, 1950, 1973, and 1979. Most of the flood data collected before 1913 are of doubtful accuracy and therefore not useful for comparison with later data[5]. A detailed description of the modern floods, with the exception of the 1973 and 1979 floods, has been prepared by the U.S. Army Corps of Engineers[1]. A description is given in this section of the origins and course of the 1973 flood, for the purpose of presenting river flood response to a major storm. Although the 1979 flood produced a greater water level at the site, a description of this flood is not provided because this event has no bearing on design basis flooding at the site.

Table 2.4-4 lists the maximum computed or observed discharges at some locations on the Mississippi River and tributaries. Table 2.4-5 gives the annual maximum Mississippi River flows and levels near the River Bend Station site for the years 1900 to 1979. Table 2.4-6 shows the maximum confined discharges at selected stations on the Lower

Mississippi River. The confined discharge is the flow that would have been carried within the existing levees had they been high and strong enough to confine the flood.

The flood of 1927 was the most disastrous in the history of the Lower Mississippi Valley. It was the result of a series of storms from the fall of 1926 through April 1927. There were flood waves on the Lower Mississippi in January, February, and April, each increasing in magnitude. Approximately 14.7 million acres of the alluvial valley were inundated. The major storm occurred April 12 to 16 and produced extremely high stages on the Upper Mississippi and Missouri Rivers. The storm was even more severe over the Arkansas and Red River Basins. With the rivers on the rise, another intense storm followed April 18 to 24. Crevasses and breaks in the levees occurred all along the Lower Mississippi.

The 1973 flood was among the greatest recorded on the Mississippi River. During December 1972, over 4 in of precipitation fell over most of the Ohio Basin, and over 8 in over large portions of the Tennessee and Cumberland Basins. Widespread rainfall occurred throughout much of January 1973, and precipitation in the White, Lower Missouri, and Lower Arkansas Basins reached or exceeded 150 percent of normal.

During March, most of the area that contributes to Mississippi River flooding experienced precipitation in excess of 150 percent of normal, the principal exception being the Upper Ohio Basin. Large areas received over 200 percent excess, and areas of the Arkansas and Missouri Basins received over 400 percent of normal rainfall.

During April, precipitation in excess of 150 percent of normal fell over the Upper Mississippi, the Upper Ohio, and the Lower Mississippi Basins, and parts of the Arkansas-Red Basins. In May, heavy rains fell over the upper Mississippi Valley and over the Ohio River southward to the Gulf. There was also considerable above-normal precipitation in June.

It can be seen that the cause of flooding in 1973 was not one or two large storms, but rather a long, continued excess of precipitation.

In early December 1972, the Middle Mississippi was falling and the Lower Ohio, Tennessee, and Cumberland Rivers were rising. The crest inflow to the Cairo, Illinois reach at the upstream portion of the Lower Mississippi (River Mile 956) was about 1,100,000 cfs, of which the Middle Mississippi contributed only about 175,000 cfs.

By the end of January 1973, the Middle Mississippi had risen to a crest of about 450,000 cfs, but, owing to a flow reduction on the Lower Ohio and its tributaries, the simultaneous crest inflow to the Cairo reach was less than 900,000 cfs. The Mississippi River flow at Helena, Arkansas (River Mile 663) remained above 1,000,000 cfs until the middle of January, but the Arkansas and White Rivers were not unusually high, so that the crest inflow below the mouth of the Arkansas River (River Mile 584) did not exceed 1,200,000 cfs.

During February, the Middle Mississippi produced discharges which again exceeded 450,000 cfs. However, the Lower Ohio and its tributaries contributed only moderately, so that the crest inflow to the Cairo reach had a peak of about 950,000 cfs. At Helena, the crest discharge exceeded 1,000,000 cfs, but Arkansas-White contributions were not excessive and the peak inflow below the mouth of the Arkansas was less than 1,200,000 cfs.

By the beginning of March, flow from all the major tributaries was reduced, and the Middle Mississippi was discharging less than 250,000 cfs. The unusually small contributions from the Lower Ohio and tributaries brought the total main stem flow at Cairo to less than 450,000 cfs. At this time, a general rise began. By the end of March, the Middle Mississippi was discharging about 700,000 cfs, and other contributions brought the Cairo discharge to over 1,500,000 cfs. This proved to be the Cairo crest. However, the Middle Mississippi continued to rise, reaching a crest at St. Louis of about 850,000 cfs on April 29, resulting from nearly, but not quite, coincident crests of 450,000 cfs on the Missouri River and 530,000 cfs from the Upper Mississippi system.

In the middle of April, the Cairo discharges eased a little, but then began to rise again. In early May, a second crest occurred, nearly as great as the first. Meantime, during April, rises occurred on the Arkansas and White Rivers, culminating near the end of April in a combined discharge of about 540,000 cfs. These flood waves were so timed as to combine with the second Cairo crest to produce a crest discharge of about 1,880,000 cfs below the mouth of the Arkansas in early May.

The Red-Ouachita River system produced a flood wave with timing such as to combine with the Mississippi in early May to produce a crest flow of about 2,150,000 cfs below the latitude of the mouth of Red River. This flow was distributed to the Mississippi River and the Atchafalaya River and floodway system.

About 12 1/2 million acres were inundated along the Middle and Lower Mississippi. From St. Louis to New Orleans, the river was generally out of banks from mid-March until June 1973. The 1973 flood crest would be expected to recur about once in 20 yr at Cairo. Due to the coincidence of flooding from the St. Francis, White, Arkansas, and Yazoo Basins, the recurrence frequency at Vicksburg, Mississippi (River Mile 437) is estimated at 40 yr.

Except for some problems in the unleveed backwater areas and in the Atchafalaya Basin, the flood was successfully contained within the main stem levees. However, in the middle reach of the river where levee cutoffs were made in the 1930s and 1940s, there was evidence of reduction in channel capacity. Stages in this reach were considerably higher than had been expected for the discharges experienced. No abnormal trend of this sort was observed below Red River Landing (River Mile 301)(ref 6).

Record stages were not set near the River Bend Station site (River Mile 262.5) due to operation of the Bonnet Carre Spillway (River Mile 128) and the Morganza Spillway (River Mile 286), as well as the Old River Control Structures (River Mile 315). In 1973, the peak stage at Bayou Sara, about 2 mi upstream from the site, was 50.7 ft msl. This compares to the record of 55.5 ft msl in 1927, and to stages of 51.2, 53.2, 52.7, 53.7, 50.7 and 52.5 ft msl in 1912, 1922, 1937, 1945, 1950, and 1979, respectively(ref 7).

The stage remained continuously above bankfull stage of 32 ft msl at Bayou Sara from March 19 to July 5, 1973, a period of 109 days. Water was continuously present on floodplain portions of GSU property at River Bend Station for approximately this same period.

2.4.2.1.2 Streams

No flood records are available for streams that potentially could flood the site. The U.S. Geological Survey has collected data on two other watersheds in the region, Alexander Creek and West Fork Thompson Creek.

A crest-stage gauging station has been maintained on Alexander Creek from 1953 to the present by the U.S. Geological Survey. The flow on Alexander Creek has varied from 12,700 cfs, occurring in 1953, to zero, which occurred several times during the period of record. The stream is subject to periods of high runoff and extended drought periods of zero flow. Table 2.4-7 shows the maximum flows and gauge heights that occurred during the period of record.

The U.S. Geological Survey also had a water stage recorder on West Fork Thompson Creek near Wakefield, Louisiana, from 1949 to 1970. The peak recorded flow of 18,100 cfs occurred in May 1953. Table 2.4-8 shows the maximum flows that occurred during the period of record.

2.4.2.2 Flood Design Considerations

The Army Corps of Engineers has made extensive studies of Mississippi River flood hydrology and has determined a project design flood (PDF)(ref 1). The PDF is based upon floods predicted by the U.S. Weather Bureau as the "maximum possible" and by the Mississippi River Commission as the "maximum probable"(ref 8,9). The PDF constitutes the basis for a determination of the probable maximum flood (PMF) at the site (see Section 2.4.3). The coincident occurrences of severe winds or upstream dam failures have been considered. It is demonstrated in Sections 2.4.3 and 2.4.4 that the River Bend Station with grade at about 95 ft msl is well above flooding from the Mississippi River.

Because of the proximity of the site to local drainage courses, stream flooding impacts on plant safety were evaluated. Regulatory Guide 1.59, Revision 2, was applied to determine estimated flood flows and levels. A flood on Grants Bayou (and its tributary, West Creek) is potentially more severe than flooding of other area streams.

The PMF flows for Grants Bayou and West Creek were computed. These basins and the estimated design basis flood levels are described in Section 2.4.3. The Grants Bayou PMF was conservatively assumed to coincide with the PDF on the Mississippi River. The peak river stage elevation is 54.5 ft msl, as determined by the Army Corps of Engineers(ref 10). This was used as the starting elevation for Grants Bayou backwater profile calculations.

The flood flows for West Creek and Grants Bayou below West Creek were reduced in the upstream direction at each cross section to account for the reduction in contributing drainage area. The flow for Grants Bayou above West Creek was conservatively assumed to exist undiminished upstream.

An analysis of the computed flood hydrographs was conducted to determine the flow and water level in Grants Bayou that would occur simultaneously with the West Creek peak flow. This Grants Bayou level was used as the starting elevation for the West Creek backwater profile. A similar comparison was conducted to determine the flow contribution of West Creek for the times of peak flow in Grants Bayou at the West Creek Confluence and in Grants Bayou at the outlet to the river floodplain.

The local streams are spanned by railroad and road bridges with piers located adjacent to and in the stream bed. These streams are subject to debris accumulation. For these reasons, each bridge crossing downsteam of the station was assumed to be 50 percent clogged at the occurrence of the PMF and 1/2 PMF + OBE, and 100 percent clogged for the 25-yr. flood + SSE. Flood flow proceeding over a roadway or rail trestle was treated as broad crested weir flow.

Flow through bridges or embankment conveyances upstream of the station was conservatively assumed to enter the study area undiminished in magnitude. Backwater calculations were performed on West Creek flows assuming creek conditions as they will exist during plant operation.

Combinations of extreme local flooding and seismic events were also investigated. An operational basis earthquake (OBE) combined with a 1/2 PMF and a safe shutdown earthquake (SSE) combined with a 25-yr flood were assumed to occur. Neither occurrence would produce water levels higher than the PMF.

The SSE was assumed to:

1. Fail all local slopes to a maximum inclination of 20H:1V and fully clog all bridges downstream of the plant.

2. Leave bridges upstream of the plant intact, allowing floodwater to enter the site undiminished.

An examination of available literature was conducted to assess the impact on site topography of an occurrence of the OBE. It was determined that an OBE would not fail site area slopes. Specifically, the Donaldsonville earthquake of 1930 is used as the basis for determining OBE and SSE seismic intensities. This earthquake had an epicenter about 50 mi south-southeast of the River Bend Station and an epicentral intensity of VI (M.M.). The limit of bank caving is related to an intensity VII (M.M.), and there was no bank caving associated with the Donaldsonville earthquake. The foundation conditions at the River Bend Station are better than most areas that felt the Donaldsonville earthquake, and better than the recent floodplain deposits in the immediate Donaldsonville vicinity. Therefore, the intensity felt at Donaldsonville due to the Donaldsonville event would be more highly amplified than a similiar event occurring at the River Bend Station. It is therefore conservative to apply a Donaldsonville intensity VI (M.M.) at the site for determining OBE and SSE intensities. However, assuming an intensity VI earthquake did occur, no bank caving would result.

The Donaldsonville earthquake was determined to have a maximum ground motion of about 0.07 g. The OBE at the River Bend Station is conservatively assumed to have a maximum ground motion of 0.05 g. Therefore, it can be inferred that an OBE would not cause bank caving at the River Bend Station. It is unlikely that an SSE, with a conservatively assumed maximum ground motion of 0.1 g, would cause bank caving. However, this has been assumed for the flood analysis. Since the channel conditions for the 1/2PMF + OBE would be the same as for the PMF, it is seen that the PMF would be more severe. Therefore, the PMF is considered the design basis for the flooding ananlysis.

2.4.2.3 Effects of Local Intense Precipitation

An analysis of plant drainage was performed to determine whether safety-related equipment could be flooded during an occurrence of the probable maximum precipitation (PMP). The following discussion pertains to flooding in the immediate plant area. Flooding of local streams, in combination with severe seismic events, is discussed in Section 2.4.3.

Safety-related equipment at the River Bend Station is located in buildings protected from floodwater entry or situated at a minimum elevation of 98 ft msl. Finish grade at the edge of plant buildings is about 95 ft msl. The elevation of the road surrounding the buildings varies from 94 to 100 ft msl. Grassed areas between buildings and roads are at 93 to 94 ft msl. Railroad spurs in the plant area have a top-of-rail elevation of 95 ft msl.

Fig. 2.4-6 shows the immediate plant area and drainage patterns. The area that could produce runoff accumulation near plant building is outlined. Normal plant area drainage is effected by directing runoff into the storm drain system, drainage ditches and culverts. All local 1
runoff is conveyed to West Creek or East Creek. There are no onsite areas which could produce ponding of runoff to an elevation greater than 96 ft msl.

During an extreme meteorological event such as the PMP, storm runoff in the eastern area of the site would drain to the storm drain system. For a rainfall intensity greater than the storm drain capacity, water would pond to 94 ft msl, then overflow to the low area west of the cooling towers and east of the plant. Water ponded in this area would overtop the cooling water access road at 93 ft msl and flow directly to East Creek. Runoff from the eastern area of the site would not top the railroad track to the Unit 2 excavation.

A portion of the PMP runoff in the construction parking area, immediately north of the plant, would drain to West Creek via culvert or overland flow. The remainder of the area would drain to the ditch along the east, south, and west embankments. A single 24 in. diameter culvert currently provides conveyance of runoff from this ditch to West Creek. The culvert is located beneath the intersection of the plant ring road and the West Plant Access Road, at the northwest corner of the immediate plant area.

PMP runoff in the area of the Unit 2 excavation would be partially intercepted by the storm drain system (flowing to West Creek), and partially drain to the excavation or enter the excavation as direct rainfall.

As discussed in Section 2.4.3, there is no flooding condition on West Creek which produces a water elevation above the top of the Fabriform Channel, and water does not enter the plant area from any postulated condition of stream flooding.

Precipitation that falls on the roofs of onsite buildings is collected in gutters along the roof edge and discharged via downspouts to the plant yard adjacent to buildings. Overflow from the roof gutters spills directly onto the plant yard. All building roofs are sloped, and no potential exists for significant ponding of rainfall on the roofs. No parapet walls exist on plant buildings which would encourage rainfall ponding. Safety-related equipment is not jeopardized by roof drainage during even the most severe postulated rainfall event.

It is clear from a review of plant drainage that runoff from the PMP could not pond above 98 ft msl and jeopardize plant safety-related equipment. Ponding above 95 ft msl would produce flow into the Unit 2 excavation or into West or East Creeks. The analysis of site drainage will now focus on ponding in the Unit 2 excavation, and the potential exceedence of the design basis groundwater level of 70 ft msl (see Section 2.4.13.5).

A 72-hr storm was assumed to occur in the site area. Direct rainfall to the 730,000 sq ft excavation (area at 94 ft msl) was computed. For the 16 acres between the excavation and the surrounding plant ring road and railroad, it was assumed that 50 percent of the runoff would be handled by the storm drain system, and 50 percent would drain to the excavation, with the exception of the peak hour of storm runoff. A comparison of rainfall intensity and storm drain capacity indicated that only a small portion of the runoff for this period would be accepted by the storm drain system. The design storm intensity for the drain system is 5.5 in/hr. It was conservatively assumed that all runoff from this maximum 1 hr period would enter the excavation.

Lastly, it was assumed that the 24 in. diameter culvert providing conveyance of construction area runoff to West Creek is in in a clogged condition for the duration of the PMP storm. Runoff from this 22.5 acre area would collect along the north side of the plant ring road, just south of the construction parkng area. Runoff would pond above the elevation of this road and flow west to West Creek, south to the Unit 2 excavation, and south to the Unit 1 plant area. It was assumed that 75 percent of the runoff from the construction parking area would enter the Unit 2 excavation.

The excavation has a bottom area of about 550,000 sq ft at 66 ft msl, and a top area of about 730,000 sq ft at 94 ft msl. The average porosities for the backfill and Terrace Sands in the excavation area are 0.31 and 0.24, respectively. The groundwater level beneath the excavation is at 57 ft msl, and the Terrace Aquifer extends to approximately-40 ft msl.

Table 2.4-36 presents the rainfall-runoff relationship and estimated accumulated runoff volume in the Unit 2 excavation.

PMF Condition

Runoff entering the excavation would initially infiltrate, filling the soil voids from 57 ft msl, the groundwater level, to 66 ft msl, the bottom of the excavation. Assuming no lateral spreading of infiltration, the infiltration volume would be about 1,534,500 cu ft. Since the infiltration capacity is greater than the runoff inflow rate, little or no ponding would occur during this period. After Hr 13 (see Table 2.4-26), infiltration would cease and runoff would begin to pond. It was assumed that runoff continued to pond through the hour of maximum rainfall intensity (hr 16). Ponding in the excavation would reach 73.2 ft msl at this time.

Seepage from the excavation banks was then calculated for the remainder of the PMP storm using Ref. 68. The seepage profile in the embankments surround the excavation can be calculated by

$$h^2(x,t) = h_i^2 + (h_o^2 - h_i^2)\ \text{erfc}\ (x/2(vt)^{\frac{1}{2}})$$

where h_o = Vertical distance from bottom of the Terrace Aquifer (-40 ft msl) to the water level in the excavation

h_i = Vertical distance from the bottom of the Terrace Aquifer to the groundwater level (57 ft msl)

x = Horizontal distance from the excavation bank to point of interest in the embankment where h is determined

h = Vertical distance from the bottom of the Terrace Aquifer to the groundwater level due to seepage

erfc = Complementary error function

$v = \dfrac{K\bar{h}}{S}$

K = Horizontal permeability (0.0025 cfs/ft, average for backfill and aquifer)

S = Specific yield of aquifer (0.27, average for backfill and aquifer)

$\bar{h}$ = Weighted average value of h for the seepage profile

t = Time from the beginning of seepage

The seepage flow is computed from Ref. 68 also:

$$Q = \frac{K\,(h_o{}^2 - h_i{}^2)}{2\,(\pi\, v\, t\,)^{\frac{1}{2}}}$$

Where Q = Flow per ft of embankment

It was determined that subsequent to Hr 16, seepage would exceed inflow to the excavation, causing a reduction in ponded level. Table 2.4-37 presents the estimated seepage rates and reduction in ponding. It was found that the water level drops from 73.2 ft msl at Hr 16 to 70 ft msl, the design basis groundwater level, in about 15 hours. It was noted that at a distance of 50 ft. into the excavation embankment, the groundwater reaches a maximum level of 68.8 ft msl, 1.2 ft below the design basis groundwater level.

Buildings bordering the excavation have been checked for potential bouyancy during the brief period of design basis groundwater level exceedence, and have been found to be stable. No impacts would occur to other plant buildings, as the groundwater level would not exceed 70 ft msl.

Summary of Plant Area Flooding

It has been demonstrated that ponding of runoff in the immediate plant area could not exceed 98 ft msl, the elevation of safety-related equipment.

It has also been demonstrated that runoff to the Unit 2 excavation could pond to a maximum elevation of 73.2 ft msl during the PMF and would remain above the design basis groundwater level of 70 ft msl for about 15 hr. The groundwater level 50 ft. into the excavation embankment would not exceed 69 ft msl. Plant buildings bordering the excavation have been checked for bouyancy and found to be stable.

Neither the ½PMF + OBE nor the 25-Yr Flood + SSE would produce a groundwater level which exceeds the RBS design basis groundwater level.

2.4.3 Probable Maximum Flood (PMF) on Rivers and Streams

2.4.3.1 Probable Maximum Precipitation (PMP)

The PMF analysis for the Mississippi River did not involve a PMP determination (Section 2.4.3.4). The following discussion pertains to precipitation in local drainage basins which produces the design basis flooding condition for the plant.

PMP values for the Grants Bayou basin and sub-basins were based on data contained in Hydrometeorological Reports 51 and 52 (11,63). All-season PMP values for a variety of storm durations and drainage area sizes are based on a nationwide analysis of storm characteristics such as dew points, land contours, and historical rainfall. PMP values for the local basins are presented in Table 2.4-9.

Based on drainage characteristics of the basins, rainfall durations and storm distributions were selected. Fig. 2.4-7 shows the basins that were analyzed to determine the impact of extreme local flooding on plant safety. The basins include: Grants Bayou, 15.6 sq mi; Grants Bayou above confluence with West Creek, 8.4 sq mi; and West Creek, 1.0 sq mi. Storm durations were selected such that the shortest time interval in the rainfall distribution corresponded to the unit rainfall duration (the time of runoff-producing rainfall) as calculated for each sub-basin (Section 2.4.3.3).

A storm duration of 24 hr was selected for the entire Grants Bayou basin while durations of 12 and 6 hr were applied to Grants Bayou above West Creek, and West Creek, respectively.

PMP storm values were distributed in accordance with NRC Regulatory Guide 1.59, Rev. 2, and standard procedures(ref 14,16). The storm for each basin was divided into four equal time periods and ordered 1 through 4 in decreasing rainfall magnitude. The storm sequence of these periods was arranged 4, 2, 1, 3. Within the maximum rainfall time period, six additional equal time periods were established and ordered 1 through 6 in decreasing rainfall magnitude. These periods were arranged 6, 4, 2, 1, 3, 5. Rainfall values for each interval were determined from rainfall-duration relationships presented in Table 2.4-9. Storm distributions for the local basins are given in Table 2.4-10.

2.4.3.2 Precipitation Losses

The soil in Grants Bayou basin belongs to essentially two soil associations(ref 17). About 75 percent of the basin is Memphis-Loring Association and the remaining 25 percent is Vicksburg-Collins-Waverly Association. The component soils of each association were identified by soil group, as shown in Table 2.4-11. About 71 percent of the basin is of Type B drainage, and this soil type was used to evaluate runoff characteristics for all sub-basins.

Field inspection showed the site area to b composed mostly of forest with some gently sloping pasture and meadow. Good pasture and forest drainage conditions exist. This was combined with the assumption of nearly saturated soil conditions due to heavy antecedent rainfall at the time of the PMP. Under these conditions, the runoff curve numbers from the U.S. Bureau of Reclamation for pasture and forest are 60 and 55, respectively (ref 12). Curve number 60 was selected to apply to all sub-basins, with the exception of West Creek, where curve number 65 was selected to account for modified drainage conditions in the plant area. Runoff was computed from rainfall using the formula:

$$Q = \frac{(P - 0.2S)^2}{(P + 0.8S)} \qquad \text{(ref 12)}$$

where:

Q = Direct runoff, in

P = Rainfall, in

S = Maximum potential difference between P and Q at the beginning of the storm, in

The value of S for runoff curve 60 is 6.67 in, and for runoff curve 65 is 5.38 in (12).

Minimum soil retention rates were also used to evaluate storm runoff. A rate of 0.2 in/hr was adopted and applied when the above runoff formula predicted a storm runoff rate with less than 0.2 in/hr retained by the soil(ref 12).

Minimum soil retention rates are dependent on types of soil in the drainage basin. The soils map (Ref 17) reveals that the distribution of soil in the Grants Bayou drainage basin is about 75 percent Memphis-Loring Association and 25 percent Vicksburg-Collins-Waverly Association. Based on the hydrologic soil group classification (Ref 12), Memphis, Loring, and Vicksburg belong to the hydrologic soil group B. Collins is soil group C and Waverly is soil group D. The range of minimum retention rates for hydrologic soil groups B, C, and D is 0.15 to 0.30 in/hr, 0.08 to 0.15 in/hr, and 0.02 to 0.08 in/hr, respectively. The retention rate of 0.2 in/hr was chosen for the Grants Bayou drainage basin.

The use of minimum soil retention rates when computing runoff is further discussed in Reference 17, which is cited in Section 5.2.9 of ANS-2.8 (Ref 16) as an acceptable source for determining precipitation losses.

From Reference 17, it was assumed that the initial moisture losses were equal to 0.2S.

Table 2.4-12 presents the PMP rainfall and runoff values.

2.4.3.3 Runoff and Stream Course Models

For the purpose of computing flood flow from runoff, runoff models were prepared for each basin. Unit hydrographs were determined by applying drainage area runoff characteristics to three independent methods of unit graph preparation, and obtaining an average unit graph for each basin (18,19). Table 2.4-13 lists basin characteristics used in the study. Basin lag time (the time from the midpoint of unit rainfall duration to the peak of the unit hydrograph) were estimated from local basin data and several independent empirical methods. (12,18,19,64).

Lag Time Estimates

Method 1. Ref. 18 presents the Snyder formula

$$T = C_t \, (L \, L_{ca})^{0.3}$$

where T = Lag time, hr
C_t = Basin characteristic
L = Channel length, mi
L_{ca} = Channel length to centroid of basin, mi

Ref. 18 also presents a correlation of C_t vs. $S^{\frac{1}{2}}$ for small basins in Texas. For the present study, this was supplemented with data from 16 small Louisiana basins. While Ref. 18 showed fairly good correlation for C_t vs. $S^{\frac{1}{2}}$, substitution of the Louisiana basin data showed good correlation only in the region of $S^{\frac{1}{2}}$ greater than 0.04. Fortunately, this includes the site basins.

Method 2. A comparison of site basin data with the known lag times and basin characteristics for 16 small Louisiana basins.

Method 3. Ref. 18 presents a relationship for basin lag vs. $L/S^{\frac{1}{2}}$ for small basins in Texas. Lag times for the site basins were initially determined by this method. A modification to the relationship was then made by supplementing the data base with information from 16 small Louisiana basins, and new estimates of the lag times for the local basins were made. Additional basin lag vs. $L/S^{\frac{1}{2}}$ relationships developed by Ramser and Chow were also used to determine site basin lag time [65].

Method 4. An empirical formula for time of concentration is presented in Ref. 19:

$$T = 5.33 \, L^{0.62} \, S_{10/85}^{\;-.448} \, S_t^{\;0.231}$$

where L = Length of stream, mi
$S_{10/85}$ = Channel slope between 10 and 85 percent of the watercourse, ft/mi
S_t = Percent of basin as swamp, lake, or pond

An additional relationship without the S_t factor was also found to be applicable through a check of correlation with data for 16 small Louisiana basins.

Method 5. Ref. 64 contains an empirical lag time method:

$$T = \frac{L^{0.8} \, (S+1)^{0.7}}{1900 \, Y^{0.5}}$$

where L = Length of channel, ft
Y = Average watershed slope, percent
$S = \frac{1000}{\text{Runoff Curve No.}} - 10$

This reference also contains a method for reduction of computed lag time based on upgrade of the existing channel due to construction, which is applicable to West Creek.

A summary of the lag time values derived for the site basins is provided in Table 2.4-14. The selected lag times are 5.0 hr for the entire Grants Bayou basin, 2.5 hr for Grants Bayou upstream of the West Creek confluence, and 1.2 hr for West Creek.

Unit rainfall duration (the time of runoff-producing rainfall) for each basin was obtained from the Army Corps of Engineers formula:

$$T_r = T_p / 5.5 \qquad \text{(ref 20)}$$

where:

T_r = Unit rainfall duration, hr

T_p - Lag Time, hr

These values were rounded to the nearest quarter hour for use in storm distribution and rainfall determination.

Unit Hydrographs

Unit hydrographs were developed by three independent empirical methods. An average unit graph was then computed and adjusted as necessary to ensure the unit hydrograph represented 1 in. runoff volume.

Method 1. Ref. 19 presents a method for unit hydrograph construction based on inputs of lag time, storage, runoff, and duration of rainfall excess. A regression formula for the computation of basin storage, similar to the formula for lag time previously cited from this source, is included in Ref. 19.

Method 2. Ref. 18 presents dimensionless unit hydrographs for small basins in Texas, which can be converted to unit hydrographs with inputs of lag time, runoff, and duration of rainfall excess.

Method 3. The Snyder method from Ref. 20 identifies the peak unit hydrograph flow as:

$$Q = \frac{A(640\ C_p)}{T}$$

where Q = Peak flow, cfs
A = Drainage area, sq mi
T = Lag time, hr
$640C_p$ = Basin characteristic

To determine the value 640 Cp for each basin, a review was made of computed $640C_p$ values for 13 small Texas basins and 16 small Louisiana basins. It was found that the 640 C_p values ranged from 194 to 785, with an average value of 472. In an effort to more precisely identify $640C_p$, a Ref. 18 correlation of $640C_p$ and $C_t\ (LL_{ca})^{0.3}/A$ was used. This was supplemented with the small Louisiana basins data, and a curve best fit was established. The values of $640C_p$ for the local basins were then selected. These are 480 for the entire Grants Bayou basin, 469 for Grants Bayou upstream of the West Creek confluence, and 370 for West Creek.

Ref. 20 was then employed to determine the unit hydrograph widths at 50 and 75 percent of peak flow. An excellent correlation of Louisiana and Texas basin known unit hydrograph widths with the Ref. 20 values was found. A relationship relating hydrograph width at 10 percent of peak flow was then developed from the Louisiana and Texas data, and used to define local basin hydrograph shape for low flow. From Ref. 18, the widths are positioned such that one-third of the width is to the left of the peak flow.

Table 2.4-13 presents the unit graphs derived from the above three methods along with the average unit graphs which have been modified as necessary to ensure that 1 in. runoff volume is represented.

Maximization of Unit Hydrographs

Investigations have shown that peak flow values from major storms in large basins are generally 25 to 50 percent higher than values computed using a unit hydrograph computed from data for minor storms(ref 20). This is probably due to two separate events:

1. The minor floods analyzed resulted from rainfall of approximately uniform areal distribution. Precipitation during major floods usually covers the entire drainage area, but in most instances the intensity and accumulated amounts vary over the area. If the volume of runoff during a major storm is proportionately heavier in the lower portion of the basin, or near the principal stream channels, the concentration of runoff would be higher than represented by the unit hydrograph derived from minor floods.

2. During minor floods, the hydraulic gradients in natural streams are usually relatively low, because of the series of pools that exist in the channel. As the stage increases during major floods, the pools tend to drown out and the channel conveyance is usually substantially increased.

Neither of the above conditions is applicable to extreme flooding conditions in the area of the plant and no further adjustment to the computer unit hydrographs was made. A real rainfall distribution is assumed to be uniform throughout the small basins. The discussion regarding increased channel conveyance and drowned pools is not appropriate for the very small site streams.

Unit hydrographs for the local basins are shown in Fig. 2.4-11 through 2.4-14. Local streams flow intermittently, and base flow for all unit hydrographs was assumed to be zero.

Based on suggestions from Reference 16, a 1/2 PMF antecedent storm was assumed to occur

1 day prior to the PMF. As can be seen from the sub-basin unit hydrographs, no overlap would occur from these two storms and the peak PMF flows would be unaffected. This antecedent storm could saturate the soil, producing a maximized runoff condition, which has been considered in the calculation of runoff.

According to the basic theory of unit hydrographs, hydrograph shape is independent of rainfall intensity. Thus, successive runoff estimations from rainfall of varying intensity may be combined using a unit hydrograph to approximate the actual storm hydrograph. Guidelines for application of this approach were obtained from Reference 12.

The values of peak flow used in the combined events analysis of 25-yr. flood & SSE were not determined by the unit graph/runoff method, but were developed through regression analysis, as presented in Section 2.4.3.4.

2.4.3.4 Probable Maximum Flood Flow

2.4.3.4.1 Mississippi River

The largest flood flow calculated for the Mississippi River in the site region is the Project Design Flood (PDF). The flood estimation was performed by the Army Corps of Engineers(ref 1). The PDF has an estimated frequency of occurrence of greater than 100 yr, but no more exact frequency determination is available(ref 8). The occurrence of a greater flood would be very rare, and could be estimated only by using very improbable intensities of rainfall, runoff, and storm sequences. The PDF is based on tributary and main stem floods predicted by the U.S. Weather Bureau as "maximum possible" and by the Mississippi River Commission as "maximum probable"(ref 9).

The Army Corps of Engineers defines a Standard Project Flood (SPF) as follows:

> A Standard Project Flood hydrograph represents critical concentrations of runoff from the most severe combination of precipitation that is considered "reasonably characteristic" of the drainage basin involved. The SPF peak discharge and volume is usually equal to about 40 to 60 percent of the PMF estimate for the same drainage basin when the comparison is related to rainfall concentrated in approximately four days or less(ref 22).

The Army Corps considers the PMF to be the most severe flood "reasonably possible" at a particular location(ref 22). A PMF for the Lower Mississippi River has not been defined because there are no recent criteria available for such a determination on a basin of this size and complexity(ref 9).

Considering the above flood descriptions, it is concluded that the PDF could be considered to be of the same order of magnitude as a PMF, and certainly is larger than an SPF as the PDF exceeds a "reasonably characteristic" flood.

A PMF for the river at the site was determined for this study by considering that the PDF is 60 percent of the PMF. This situation applies between a PMF and an SPF when the SPF rainfall occurs in 4 days or less. From the flood study by the Army Corps of Engineers, the PDF rainfall lasts for 2.5 months(ref 1). However, considering the descriptive definitions of the design floods and the probable similarity of the PDF and PMF, the PMF estimate can be considered to be reasonably conservative. The unregulated (not taking into account the existence of upstream reservoir storage) PDF discharge at the latitude of Red River Landing (River Mile 305) is 3,330,000 cfs, and the estimated PMF discharge at this point is 5,500,000 cfs. The PMF estimation is made for Red River Landing because flood controls exist between this location and the site, and the mitigating effect of these controls is evaluated for computation of the PMF level near the site (Section 2.4.3.5).

2.4.3.4.2 Local Streams

The PMP estimates from Section 2.4.3.1 were applied to the runoff characteristics of Section 2.4.3.2 and the unit hydrographs of Section 2.4.3.3, and the PMF runoff hydrographs for the local basins were determined. Tables 2.4-18 through 2.4-21 present the calculated hydrographs and peak flows. Peak flow values are: entire Grants Bayou basin - 42,690 cfs; Grants Bayou above West Creek confluence - 35,346 cfs; West Creek - 6,699 cfs.

The 25-yr peak flows for the local basins were determined from Refs. 21 and 66. These sources provide regression relationships based on a large amount of Louisiana data. Inputs are drainage area, annual precipitation, and channel slope. Ref. 21 applies to basins 10 sq mi and smaller, and was used to determine 25-yr peak flows of 842 cfs for West Creek and 4,364 cfs for Grants Bayou above the West Creek confluence. Ref. 66 applies to basins larger than 10 sq mi and was used to determine a peak 25-yr flow for the entire Grants Bayou basin of 6,760 cfs.

Several small farm ponds are located in the Grants Bayou basin. Failure of one or more of these ponds concurrently with a design flooding condition would have no significant impact on peak flood flows due to the small volume of storage in the ponds and the relatively large amount of channel storage available during extreme flooding (see Section 2.4.1.2.3).

The 32-acre Wildlife Management Lake is situated more than 40 ft below plant grade, and failure of the dike at the lake would not affect the plant.

2.4.3.5 Water Level Determinations

2.4.3.5.1 Mississippi River

The anticipated flow distribution to the Mississippi River floodway system during a PDF, utilizing upstream reservoir storage, is shown on Fig. 2.4-16. The total unregulated PDF flow at the latitude of Red River Landing is 3,330,000 cfs. Upstream reservoir storage would provide a reduction in peak flow to about 3,030,000 cfs. About 1,530,000 cfs would be diverted to the Morganza and West Atchafalaya Floodways and the Atchafalaya River upstream of the site, and about 1,500,000 cfs is estimated to pass the site(ref 1,23). The estimated flood level at the site for this flow is 54.5 ft msl, about 40 ft below plant grade(ref 10). The PDF is confined between the manmade levee on the west bank of the river and the eastern edge of the river floodplain, as shown in Fig. 2.4-17. The levee elevation along the west bank opposite the site is about 57.5 ft msl, 3 ft above the PDF crest level.

In the estimation of PDF flow reduction due to reservoir storage, storms used to construct the PDF were assumed to be generally located near the downstream portions of the major tributaries where the maximum effectiveness of reservoir storage capability would not be obtained. The reduction computed for reservoirs appears reasonably certain of attainment. The maximum reduction in flow from reservoir storage using this approach is about 510,000 cfs, and the minimum reduction is about 250,000 cfs(ref 1). It is assumed that reservoir storage provides a reduction in peak flow of 300,000 cfs during the PDF, and 400,000 cfs during the PMF.

A minimum freeboard of 2-3 ft above the PDF crest exists on the floodway system and main stem levees(ref 1). It is assumed that flow diversion upstream of the site during the PMF would increase from the PDF confined discharge of 1,530,000 cfs. The main stem confined discharge at the site would increase from 1,500,000 cfs at PDF stage of 54.5 ft msl, because bankfull stage is 57.5 ft msl. Considering the peak flow at Red River Landing for the PMF to be 5,500,000 cfs and reservoir storage to be 400,000 cfs, the confined discharge in the floodway system at bankfull stage would be 1,500,000 to 2,000,000 cfs less than the peak flow.

At this point in the flood, flow would overtop the levees and enter the vast low-lying storage area between the main stem of the river near the site and the western extent of the floodway system (Fig. 2.4-18). The ground level in the overbank area is 20-35 ft msl. Considering the volume of this storage area together with the fact that it would not become flooded until just prior to peak flood flow, it is extremely unlikely that the peak water level near the site would ever exceed 60 ft msl, about 35 ft below plant grade. No plant safety-related equipment would be jeopardized in the event of a Mississippi River PMF. Section 2.4.3.6 presents the effects on water level of the combined occurrence of the PMF and the 2-yr extreme wind speed.

2.4.3.5.2 Local Streams

Water levels in Grants Bayou and West Creek were computed through the use of the HEC-2 Water Surface Profiles computer program developed by the Army Corps of Engineers.[67]

Manning's roughness coefficient, n, has been determined based on observations at the site and experience in Louisiana by a consultant(ref 25). The channel and overbank n values for the existing topography and subsequent to an SSE are presented with the cross section data in Tables 2.4-24 and 2.4-25. A portion of West Creek in the plant area has been lined with Fabriform to provide channel stability and increase conveyance. While the manufacturers suggested roughness coefficient is 0.012-0.015, the roughness coefficient was conservatively assumed to be 0.03 to account for possible debris accumulation.

Cross section data were obtained from a consultant survey of the onsite streams and United States Geological Survey topographic maps(ref 21,25).

Flow and water level in Grants Bayou and West Creek is affected by road and railroad bridges, all with bridge piles located adjacent to and in the stream bed. Some moderate debris accumulation has occurred historically at these locations, but there is no record of a debris jam causing higher than anticipated flood levels or bridge washout. However, for the PMF it was conservatively assumed that each bridge was 50 percent clogged. The cross section data for the bridges is presented in Tables 2.4-24 and 2.4-25.

Two flood conditions were analyzed for the local streams. These include the PMF, and a 25-yr flood + safe shutdown earthquake (SSE). A discussion of the potential effects of an OBE are presented in Section 2.4.2.2.

For the SSE, it was assumed that slopes failed to a maximum of 20H:1V. Bridges downstream of the plant were assumed to remain standing in a fully clogged condition after an earthquake, which would produce higher water levels than a washout condition. Bridges upstream of the plant were assumed to be unaffected by an earthquake, allowing flood flow to enter the site area unmitigated.

A comparison of PMF and 1/2 PMF flows shows the PMF to be the more severe flood condition. Since stream channel conditions are assumed to be the same for both cases, flood levels from the PMF condition would be greater. The 1/2 PMF + OBE condition was eliminated from further consideration.

The starting elevation for the Grants Bayou backwater profile for both PMF and 25-yr + SSE conditions is conservatively assumed to be the Mississippi River PDF level, 54.5 ft msl(ref 10). It is highly unlikely that the river PDF would coincide with the PMF on the local basins.

Fig. 2.4-21 shows cross section locations for the PMF and 25-yr + SSE flooding conditions. Cross section data are presented in Tables 2.4-24 and 2.4-25. Applicable channel and overbank Manning's n values are also presented in these tables. As noted in the tables, vertical walls were assumed to exist at either end of some sections to limit the spread of water and channel conveyance. Conservative water levels would result from this approach.

As discussed previously, bridges were assumed to be partially or fully clogged with debris, and overflow can be treated as for a broad-crested weir. Applicable weir widths and configurations are presented in Fig. 2.4-22 through 2.4-28.

Normal sediment accumulation in the West Creek Fabriform channel will have not significant impact on the conveyance of flood flow past the plant area. The predicted PMF water level is more than 1 foot below the channel crest. Due to the comparatively larger conveyance at the top of the channel cross section, it is estimated that more than 2 feet of sediment could accumulate before the PMF water level would reach the channel crest.

It was assumed for the ½PMF + OBE that landslides caused a substantial loss of West Creek channel conveyance. Given the conveyance of the revised (post-OBE) cross section, as noted in Table 2.4-25, pre-OBE sediment accumulation would have no significant impact on the ½PMF water level.

The computed backwater profiles for Grants Bayou and West Creek are presented in Table 2.4-26. The peak flooding condition occurs during the PMF.

The maximum water level on Grants Bayou near the plant occurs between Sections 10 and 11 (Fig. 2.4-21), where the water level varies from 95.3 to 101.8 ft msl, respectively. The adjacent cooling tower yard is at about 104 ft msl, above the flood level. Additionally, no safety related equipment is located in this area. The maximum water level on West Creek near the plant occurs at about Section W9 (Fig. 2.4-21), where the peak water level is about 92.7 ft msl. This is below the top of the Fabriform channel (94.0 ft msl) and the adjacent railroad spur at 95.0 ft msl, and plant area flooding would not occur.

2.4.3.6 Coincident Wind Wave Activity

2.4.3.6.1 Mississippi River

An estimated PMF level of 60 ft msl was combined with the 2-yr extreme wind speed to determine the maximum water level at the site due to river flooding. Based on Regulatory Guide 1.59, Rev 2, a wind speed of 50 mph was selected. Standard methods were used to determine the wave height, period, and runup(ref 28).

It is assumed that the entire alluvial valley is flooded at the time of the PMF crest elevation. The average water depth west of the plant within about 20 mi was estimated to be about 25 ft, based on a water level of 60 ft msl and an average ground surface elevation of 35 ft msl.

Plant grade and any safety-related equipment are well above any wind-wave water level. Plant safety is not jeopardized by even the most extreme conditions of Mississippi River flooding.

2.4.3.6.2 Local Streams

The design flooding level in the plant area would not be increased by coincident wind wave activity. No substantial fetch could be generated to affect Grants Bayou flood levels due to the dense vegetation surrounding the stream. Additionally, no safety-related equipment exists in the cooling tower area which could be affected by any possible wave action between Grants Bayou cross sections 10 and 11.

For West Creek, the PMF is contained within the Fabriform channel and would not be substantially affected by high winds. During the postulated 25-yr flood + SSE, the water level on West Creek near the plant (cross sections W7 to W9) would be only 3 to 4 ft above the channel bottom, and could not generate a significant wave.

2.4.4 Potential Dam Failures

2.4.4.1 Mississippi River

The effect of the failure of dams located in the Mississippi River Basin from both flood and seismic action has been considered. The basin encompasses about 41 percent of the conterminous United States, and includes many dams. There are no dams on the river affecting plant safety. Those dams in the states of Texas, Mississippi, and Louisiana on tributaries to the river upstream of the site are presented in Table 2.4-27. There are no dams on the river main stem between the site and confluence with the Ohio River (691 river miles).

The dams nearest the site are located on Indian Creek and Cotile Creek near Alexandria, LA, about 75 air miles northwest of the site. These streams are tributaries to Red River, which feeds into the Atchafalaya River in confluence with Lower Old River near Mississippi River Mile 304. The dams are more than 100 river miles from the site. Total dam storage in this area (4 dams) is about 123,275,000 cu m (100,000 acre-ft)(ref 29). Flow from the Red River-Atchafalaya River does not enter the Mississippi River; however, extreme flood flow from this source may affect the floodplain of the Mississippi River near the site in the unlikely event that the Atchafalaya River and Morganza Floodway levees are overtopped. Levee elevations are 2-3 ft above the estimated Project Design Flood level (Section 2.4.3)(ref 1).

Plant grade is about 95 ft msl, and safety-related equipment is positioned at a minimum elevation of 98 ft msl or is located in buildings protected from floodwater entry. The normal river water level at the site is about 20.4 ft msl, and the highest recorded water level since installation of numerous upstream river control structures is about 52.1 ft msl, which occurred in April 1979(ref 30). The natural levee on the east (plant) side of the river varies in elevation between 37 and 45 ft msl, and on the west side of the river the manmade levee is at about 57.5 ft msl. The Mississippi River Project Design Flood level at the site has been estimated by the U.S. Army Corps of Engineers to be about 54.5 ft msl(ref 10). The return period for this event is estimated to be much greater than 100 yr (Section 2.4.3)(ref 8). The river floodplain at the site is more than 30 mi wide. The postulated PMF level, from Section 2.4.3, is about 60 ft msl.

Considering the distance of dams from the site (greater than 100 river miles), the elevation of the site with respect to surrounding topography and the river floodplain, and the broad expanse of tributary and river floodplain available to overbank flows, it is extremely unlikely that a flood wave or flood flows generated by a dam failure or series of failures anywhere in the basin could affect safety-related equipment at the site. All safety-related equipment is more than 35 ft above the PMF peak level, well above any potential effect from dam failures.

2.4.4.2 Local Streams

There are no dams or similar water control structures on the local streams. The impact of bridge clogging or stream bank failure on local flooding and floodwater levels is discussed in Section 2.4.3. Failure of the Fabriform-lined portion of West Creek is postulated for the SSE condition discussed in Section 2.4.3. Failure of the drop structure at the upstream end of the lined portion of West Creek could possibly reduce flood flow and water level in West Creek. Manmade and natural topography in that area ensures the direction of flood flow along a watercourse west of the plant. 1

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Insert for page 2.4-72

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TABLE 2.4-9

PMP VALUES AT RIVER BEND STATION SITE

Duration	West Creek PMP 1 sq mi (in)	Grants Bayou PMP above West Creek Confl. 8.4 sq mi (in)	Grants Bayou PMP 15.6 sq mi (in)
15 min	9.7	8.2	-
30 min	14.2	11.9	-
1 hr	19.4	16.3	14.7
6 hr	32.0	32.0	31.0
12 hr	38.7	38.7	37.8
18 hr	-	43.7	42.6
24 hr	47.1	47.1	46.3
48 hr	-	51.8	50.8
72 hr	-	55.7	-

Sources: U.S. Dept. of Commerce. Probable Maximum Precipitation Estimates, United States East of the 105th Meridian. Hydrometeorological Report No. 51, Washington, DC, 1978.

U.S. Dept. of Commerce. Application of Probable Maximum Precipitation Estimates, United States East of the 105th Meridian. Hydrometeorological Report No. 52, Washington, DC, 1952.

TABLE 2.4-10

PMP STORM DISTRIBUTION

Grants Bayou

Time (hr)	Incremental Rainfall (in)
0-6	3.7
6-12	6.8
12-13	1.7
13-14	3.3
14-15	5.4
15-16	14.7
16-17	4.0
17-18	1.9
18-24	4.8
	46.3

Grants Bayou above Confluence with West Creek

Time (hr)	Incremental Rainfall (in)
0-3	2.8
3-6	7.2
6-6.5	1.8
6.5-7	2.1
7-7.5	4.4
7.5-8	11.9
8-8.5	2.6
8.5-9	2.0
9-12	3.9
	38.7

West Creek

Time (hr)	Incremental Rainfall (in)
0-1.5	2.8
1.5-3.0	4.4
3.0-3.25	1.0
3.25-3.5	2.3
3.5-3.75	4.5
3.75-4.0	9.7
4.0-4.25	2.9
4.25-4.5	1.1
4.5-6.0	3.3
	32.0

Sources: U.S. Dept. of Commerce. Probable Maximum Precipitation Estimates, United States East of the 105th Meridian. Hydrometeorological Report No. 51, Washington, DC, 1978.

U.S. Dept. of Commerce. Application of Probable Maximum Precipitation Estimates, United States East of the 105th Meridian. Hydrometeorological Report No. 52, Washington, DC, 1982.

TABLE 2.4-11

LOCAL SOILS CATEGORIZED BY HYDROLOGIC SOIL GROUPS

Association	Soil	Percent of Association	Hydrologic Soil Group
Memphis-Loring (75% of Grants Bayou)	Memphis	55	B
	Loring	30	B
	Miscellaneous	15	10 percent C 5 percent D
Vicksburg-Collins-Waverly (25% of Grants Bayou)	Vicksburg	30	B
	Collins	30	C
	Waverly	25	D
	Miscellaneous	15	7.5 percent C 7.5 percent D

Key: B = 71% of drainage area
C = 17% of drainage area
D = 12% of drainage area

Sources: Bureau of Reclamation. Design of Small Dams. U.S. Dept. of the Interior, Washington, DC, 1974.

Soil Conservation Service. Map 4-R-29109-A, General Soil Map for West Feliciana Parish, LA. U.S. Dept. of Agriculture, Washington, DC, 1970.

TABLE 2.4-12

RAINFALL - RUNOFF RELATIONSHIPS

Basin	Time (hr)	Incremental Rainfall (in)	Accumulated Rainfall (in)	Accumulated Runoff (in)	Incremental Runoff (in)	Incremental Loss (in)
Grants Bayou	0-6	3.7	3.7	0.62	0.62	3.08
	6-12	6.8	10.5	5.30	4.68	2.12
	12-13	1.7	12.2	6.73	1.43	0.27
	13-14	3.3	15.5	9.63	2.90	0.40
	14-15	5.4	20.9	14.59	4.96	0.44
	15-16	14.7	35.6	28.68	14.09	0.61
	16-17	4.0	39.6	32.59	3.88	0.20[1]
	17-18	1.9	41.5	34.44	1.70	0.20
	18-24	4.8	46.3	39.16	3.60	1.20
Grants Bayou above Confluence with West Creek	0-3	2.8	2.8	0.26	0.26	2.54
	3-6	7.2	10.0	4.90	4.64	2.56
	6-6.5	1.8	11.8	6.39	1.49	0.31
	6.5-7	2.1	13.9	8.21	1.82	0.28
	7-7.5	4.4	18.3	12.18	3.97	0.43
	7.5-8	11.9	30.2	23.45	11.27	0.63
	8-8.5	2.6	32.8	25.96	2.50[1]	0.10[1]
	8.5-9	2.0	34.8	27.90	1.90	0.10
	9-12	3.9	38.7	31.71	3.30	0.60
West Creek	0-1.5	2.8	2.8	0.31	0.31	2.49
	1.5-3.0	4.4	7.2	3.26	2.95	1.45
	3.0-3.25	1.0	8.2	4.06	0.80	0.20
	3.25-3.5	2.3	10.5	6.00	1.94	0.36
	3.5-3.75	4.5	15.0	10.04	4.04	0.46
	3.75-4.0	9.7	24.7	19.24	9.20	0.50
	4.0-4.25	2.9	27.6	22.05	2.81	0.09
	4.25-4.5	1.1	28.7	23.12	1.05[1]	0.05[1]
	4.5-6.0	3.3	32.0	26.34	3.00	0.30

[1] Minimum retention rate of 0.2 in/hr applies from this point to end of storm.

TABLE 2.4-13

HYDROLOGIC CHARACTERISTICS OF SMALL BASINS

Texas Basins	L(1) (mi)	L_{ca}(2) (mi)	S(3) (ft/mi)	Area (sq mi)	Lag Time (hr)
1	0.96	0.33	62.8	0.48	0.3
2	1.53	0.91	79.2	1.26	1.0
3	2.23	0.87	32.3	1.73	1.4
4	1.84	1.04	58.1	2.14	1.4
5	2.22	1.33	70.2	3.29	0.75
6	2.69	0.90	100.3	3.42	1.25
7	4.79	1.70	63.4	4.32	0.75
8	4.11	1.70	88.2	5.25	1.25
9	3.50	1.70	41.9	7.01	2.25
10	6.78	3.93	19.9	9.16	3.4
11	7.92	3.75	9.1	17.60	5.1
12	19.2	8.50	12.0	70.00	8.5
13	25.0	14.00	6.4	75.50	13.1
Louisiana Basins					
1	5.4	2.7	15.9	12.1	2.5
2	15.1	7.6	11.8	35.3	3.5
3	21.2	11.0	7.1	103.0	12.0
4	16.9	8.6	8.7	89.7	22.5
5	30.9	16.8	6.6	79.5	16.5
6	7.6	4.0	15.3	21.4	4.5
7	3.9	2.1	25.9	5.3	3.5
8	19.2	9.2	8.4	68.3	30.0
9	10.8	5.8	2.2	37.1	9.0
10	11.3	5.9	1.8	19.0	14.0
11	12.3	5.8	2.7	25.7	18.0
12	25.4	12.6	5.4	94.2	27.0
13	23.2	10.2	6.4	82.2	13.5
14	19.2	10.3	7.2	96.5	30.0
15	4.2	2.3	24.4	3.2	3.5
16	8.5	4.9	11.4	13.1	7.5
Local Basins					
GB	7.04	4.81	22.7	15.55	5.5(4)
GBA	4.73	3.64	30.1	8.45	2.5(4)
WC	2.13	1.04	38.8	0.96	1.2(4)

(1) Length of longest watercourse from the point of interest to the watershed divide.

(2) Length of longest watercourse from the point of interest to the centroid of the basin.

TABLE 2.4-13 (Cont.)

(3)Slope of the longest watercourse from the point of interest to the watershed divide. For basins other than Texas Basins, this is $S_{10/85}$, the slope between 10 and 85 percent of the watercourse.

(4)Estimated from Table 2.4-14.

Key: GB = Grants Bayou
GBA = Grants Bayou above Confluence with West Creek
WC = West Creek

Sources: Hudlow, M.D. Techniques for Hydrograph Synthesis Based on Analysis of Data from Small Drainage Basins in Texas. Water Resources Institute, Texas A&M University, 1966.

United States Geological Survey. Topographic Maps of Louisiana. Elm Park, 1961, Port Hudson, 1963. Dept. of the Interior, Washington, DC.

U.S. Dept. of the Interior, Geological Survey, and Louisiana Dept. of Transportation. Unit Hydrographs for Southwestern Louisiana, Technical Report No. 2D, Baton Rouge, 1969.

U.S. Dept. of the Interior, Geological Survey, and Louisiana Dept. of Transportation. Unit Hydrographs for Southeastern Louisiana and Southwestern Mississippi, Technical Report No. 2B, Baton Rouge, 1967.

TABLE 2.4-14

COMPARISON OF LAG TIME ESTIMATES

Method	GB[1]	GBA[1]	WC[1]
Snyder $T = C_t(LL_{ca})^{0.3}$	6.1	3.4	1.4
Direct Comparison of Basin Characteristics	6.0	3.0	1.5
T vs. $L/S^{1/2}$			
Based on Data from Small Texas Basins	4.4	2.3	1.1
Based on Data from Small Texas and Louisiana Basins	4.8	2.5	1.1
Based on Data from Ramser	4.4	2.3	1.1
Based on Data from Chow	3.5	1.8	1.0
Mitchell $T= 5.33L^{.602} S_{10/85}^{-.448} S_t^{.231}$	6.1	3.0	1.4
$T= 5.02L^{.65} S_{10/85}^{-.46}$	6.4	2.9	1.5
Soil Conservation Service $T= \frac{L^{.8} (S+1)^{.7}}{1900\ Y^{.5}}$	5.0	3.2	1.2
Average	5.2	2.5	1.2
Range	3.5 - 6.4	1.8 - 3.4	1.0 - 1.5

(1) GB = Grants Bayou
GBA = Grants Bayou above West Creek confluence
WC = West Creek

Sources: Hudlow, M.D. Techniques to Hydrograph Synthesis Based on Analysis of Data from Small Drainage Basins in Texas Water Resources Institute, Texas A&M University, 1966.

U.S. Dept. of the Interior, Geological Survey and Louisiana Dept. of Transportation Unit Hydrographs for Southeastern Louisiana and Southwestern Mississippi, Technical Report No. 2B, Baton Rouge, 1967.

U.S. Dept. of the Interior, Geological Survey, and Louisiana Dept. of Transportation. Unit Hydrographs for Southwestern Louisiana, Technical Report No. 2D, Baton Rouge, 1969.

TABLE 2.4-14 (Cont.)

Chow, V.T. Hydrologic Determination of the Waterway Areas for the Design of Drainage Structures in Small Drainage Basins, University of Illinois Bulletin, Vol. 59, No. 65, Urbana, IL, 1962.

U.S. Dept. of Agriculture, Soil Conservation Service. Urban Hydrology for Small Watersheds, Technical Release No. 55, 1975.

TABLE 2.4-15

UNIT HYDROGRAPHS FOR LOCAL BASINS

Grants Bayou

Method 1		Method 2		Method 3		Final Version	
Time (hr)	Flow (cfs)	Time (hr)	Flow (cfs)	Time (hr)	Flow (cfs)	Time (hr)	Flow (cfs)
0.0	0.0	0.0	0.0	0.0	0	0.0	0.0
0.5	14	1.92	106	1.0	45	1.0	64
1.0	107	2.70	321	2.0	245	2.0	352
1.5	329	3.46	1047	3.0	500	3.0	850
2.0	666	4.24	1401	4.0	900	4.0	1443
2.5	1096	5.00	1536	5.0	1420	5.0	1620
3.0	1572	5.39	1569	6.0	1460	6.0	1415
3.5	1950	6.16	1441	7.0	1270	7.0	1090
4.0	2118	7.32	1047	8.0	1040	8.0	720
4.5	2093	8.86	724	9.0	800	9.0	510
5.0	1913	10.78	445	10.0	600	10.0	380
5.5	1623	13.48	241	11.0	420	11.0	290
6.0	1313	17.32	97	12.0	320	12.0	245
6.5	1051	21.18	33	13.0	265	13.0	205
7.0	842	25.41	0	14.0	200	14.0	180
7.5	674			15.0	180	15.0	160
8.0	540			16.0	120	16.0	140
8.5	432					17.0	115
9.0	346					18.0	90
9.5	277					19.0	70
10.0	222					20.0	45
10.5	178					21.0	40
11.0	142					22.0	30
11.5	114					23.0	20
12.0	91					24.0	10
						25.0	5
						26.0	0

TABLE 2.4-15 (Cont.)

Grants Bayou Above West Creek Confluence

Method 1		Method 2		Method 3		Final Version	
Time (hr)	Flow (cfs)	Time (hr)	Flow (cfs)	Time (hr)	Flow (cfs)	Time (hr)	Flow (cfs)
0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
0.25	12	0.96	115	0.5	50	0.5	57
0.5	90	1.35	349	1.0	255	1.0	320
0.75	279	1.73	1138	1.5	550	1.5	840
1.0	572	2.12	1523	2.0	905	2.0	1435
1.25	955	2.50	1670	2.5	1450	2.5	1667
1.5	1390	2.70	1705	3.0	1545	3.0	1522
1.75	1755	3.08	1566	3.5	1375	3.5	1175
2.0	1953	3.66	1138	4.0	1090	4.0	855
2.25	1986	4.43	787	4.5	825	4.5	615
2.5	1880	5.39	484	5.0	600	5.0	475
2.75	1669	6.74	262	5.5	425	5.5	360
3.0	1423	8.66	105	6.0	350	6.0	285
3.25	1205	10.59	36	6.5	275	6.5	240
3.5	1020	12.70	0	7.0	200	7.0	200
3.75	863			7.5	160	7.5	150
4.0	731					8.0	130
4.25	618					8.5	115
4.5	524					9.0	90
4.75	443					9.5	70
5.0	375					10.0	60
5.25	318					10.5	50
5.5	269					11.0	40
5.75	228					11.5	30
6.0	193					12.0	20
						12.5	10
						13.0	0

TABLE 2.4-15 (Cont.)

West Creek

Method 1 Time (hr)	Method 1 Flow (cfs)	Method 2 Time (hr)	Method 2 Flow (cfs)	Method 3 Time (hr)	Method 3 Flow (cfs)	Final Version Time (hr)	Final Version Flow (cfs)
0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
0.12	2	0.46	27	0.25	15	0.25	14
0.24	18	0.65	82	0.5	70	0.5	81
0.36	58	0.83	268	0.75	170	0.75	212
0.48	119	1.02	359	1.0	245	1.0	341
0.60	200	1.20	394	1.25	290	1.25	362
0.72	292	1.30	402	1.5	285	1.5	305
0.84	372	1.48	369	1.75	265	1.75	240
0.96	418	1.76	268	2.0	235	2.0	190
1.08	431	2.13	186	2.25	210	2.25	146
1.20	415	2.60	114	2.5	170	2.5	120
1.32	375	3.25	62	2.75	135	2.75	102
1.44	328	4.17	25	3.0	115	3.0	82
1.56	284	5.10	8			3.25	70
1.68	246	6.12	0			3.5	54
1.80	213					3.75	43
1.92	185					4.0	33
2.04	160					4.25	23
2.16	139					4.5	18
2.28	120					4.75	12
2.40	104					5.0	9
2.52	90					5.25	7
2.64	78					5.5	5
2.76	68					5.75	3
2.88	59					6.0	2
						6.25	1
						6.5	0

TABLE 2.4-16

DELETED

TABLE 2.4-17

DELETED

TABLE 2.4-18
PMF HYDROGRAPH FOR GRANTS BAYOU

Time Interval Number	Time (hr)	Unit Hydrograph (cfs)	Incremental Runoff (in)	Total PMF Hydrograph (cfs)
0	0.0	0	0.0	0
1	1.0	64	0.10	0
2	2.0	352	0.10	6
3	3.0	850	0.10	42
4	4.0	1443	0.10	127
5	5.0	1620	0.11	271
6	6.0	1415	0.11	439
7	7.0	1090	0.78	579
8	8.0	720	0.78	739
9	9.0	510	0.78	1061
10	10.0	380	0.78	1698
11	11.0	290	0.78	2717
12	12.0	245	0.78	3842
13	13.0	205	1.43	4822
14	14.0	180	2.90	5619
15	15.0	160	4.96	6446
16	16.0	140	14.09	8009
17	17.0	115	3.80	11777
18	18.0	90	1.70	19465
19	19.0	70	0.60	29918
20	20.0	45	0.60	39808
21	21.0	40	0.60	42690
22	22.0	30	0.60	38721
23	23.0	20	0.60	31701
24	24.0	10	0.60	24182
25	25.0	5	0.00	19014
26	26.0	0	0.00	15625
27	27.0			13242
28	28.0			11443
29	29.0			9618
30	30.0			7970
31	31.0			6553
32	32.0			5345
33	33.0			4306
34	34.0			3405
35	35.0			2665
36	36.0			2006
37	37.0			1613
38	38.0			1240
39	39.0			900
40	40.0			593
41	41.0			376
42	42.0			213
43	43.0			137
44	44.0			90

TABLE 2.4-18 (Cont.)

Time Interval Number	Time (hr)	Unit Hydrograph (cfs)	Incremental Runoff (in)	Total PMF Hydrograph (cfs)
45	45.0			63
46	46.0			39
47	47.0			21
48	48.0			9
49	49.0			3
50	50.0			0

TABLE 2.4-19

DELETED

TABLE 2.4-20

PMF HYDROGRAPH FOR GRANTS BAYOU
ABOVE CONFLUENCE WITH WEST CREEK

Time Interval Number	Time (hr)	Unit Hydrograph (cfs)	Incremental Runoff (in)	Total PMF Hydrograph (cfs)
0	0.0	0	0.0	0
1	0.5	57	0.04	0
2	1.0	320	0.04	2
3	1.5	840	0.04	15
4	2.0	1435	0.04	49
5	2.5	1667	0.05	106
6	3.0	1522	0.05	173
7	3.5	1175	0.77	237
8	4.0	855	0.77	334
9	4.5	615	0.77	613
10	5.0	475	0.77	1259
11	5.5	360	0.78	2326
12	6.0	285	0.78	3553
13	6.5	240	1.49	4672
14	7.0	200	1.82	5583
15	7.5	150	3.97	6472
16	8.0	130	11.27	7765
17	8.5	115	2.50	10531
18	9.0	90	1.90	10108
19	9.5	70	0.55	24334
20	10.0	60	0.55	32278
21	10.5	50	0.55	35346
22	11.0	40	0.55	33202
23	11.5	30	0.55	27971
24	12.0	20	0.55	22566
25	12.5	10	0.00	18208
26	13.0	0		15243
27	13.5			12884
28	14.0			10956
29	14.5			9164
30	15.0			7415
31	15.5			5786
32	16.0			4664
33	16.5			3793
34	17.0			3029
35	17.5			2419
36	18.0			1978
37	18.5			1606
38	19.0			1275
39	19.5			969
40	20.0			699
41	20.5			453
42	21.0			250
43	21.5			167

TABLE 2.4-20 (Cont.)

Time Interval Number	Time (hr)	Unit Hydrograph (cfs)	Incremental Runoff (in)	Total PMF Hydrograph (cfs)
44	22.0			116
45	22.5			83
46	23.0			55
47	23.5			33
48	24.0			17
49	24.5			6
50	25.0			0

TABLE 2.4-21

PMF HYDROGRAPH FOR WEST CREEK

Time Interval Number	Time (hr)	Unit Hydrograph (cfs)	Incremental Runoff (in)	Total PMF Hydrograph (cfs)
0	0.0	0	0.0	0
1	0.25	14	0.05	0
2	0.50	81	0.05	1
3	0.75	212	0.05	5
4	1.00	341	0.05	15
5	1.25	362	0.05	32
6	1.50	305	0.06	50
7	1.75	240	0.49	66
8	2.00	190	0.49	85
9	2.25	146	0.49	131
10	2.50	120	0.49	233
11	2.75	102	0.49	389
12	3.00	82	0.50	553
13	3.25	70	0.80	691
14	3.50	54	1.94	804
15	3.75	43	4.04	933
16	4.00	33	9.20	1188
17	4.25	23	2.81	1832
18	4.50	18	1.05	3152
19	4.75	12	0.50	4963
20	5.00	9	0.50	6431
21	5.25	7	0.50	6699
22	5.50	5	0.50	6027
23	5.75	3	0.50	5109
24	6.00	2	0.50	4305
25	6.25	1	0.00	3656
26	6.50	0		3203
27	6.75			2833
28	7.00			2432
29	7.25			2028
30	7.50			1614
31	7.75			1272
32	8.00			987
33	8.25			749
34	8.50			577
35	8.75			434
36	9.00			330
37	9.25			251
38	9.50			183
39	9.75			128
40	10.00			87
41	10.25			55
42	10.50			32
43	10.75			20

TABLE 2.4-21 (Cont.)

Time Interval Number	Time (hr)	Unit Hydrograph (cfs)	Incremental Runoff (in)	Total PMF Hydrograph (cfs)
44	11.00			14
45	11.25			9
46	11.50			6
47	11.75			3
48	12.00			2
49	12.25			1
50	12.50			0

TABLE 2.4-22

DELETED

TABLE 2.4-23

DELETED

TABLE 2.4-24

PMF CROSS SECTION DATA[1]

Grants Bayou

Distance from Left Bank (ft)	Elevation (ft msl)	Distance from Left Bank (ft)	Elevation (ft msl)
Cross Section No. 1			
0	56.6	427	36.7
50	56.5	435	43.1
100	56.5	469	43.5
150	56.7	488	49.3
194	56.2	500	51.4
254	56.4	540	56.5
292	56.5	573	56.5
316	52.4	600	56.9
326	49.3	658	57.8
334	43.0	668	58.4
359	40.6	684	59.4
362	36.4	700	59.2
380	36.6	751	60.5
400	36.7	800	60.8

channel n = 0.04; overbank n = 0.13

Distance from Left Bank (ft)	Elevation (ft msl)	Distance from Left Bank (ft)	Elevation (ft msl)
Cross Section No. 2			
0	75.0	830	40.0
40	50.0	950	45.0
290	45.0	1000	50.0
730	45.0	1120	75.0
745	40.0		

channel n = 0.05; overbank n = 0.13

Distance from Left Bank (ft)	Elevation (ft msl)	Distance from Left Bank (ft)	Elevation (ft msl)
Cross Section No. 2a			
0	75.0	935	43.0
180	55.0	970	45.0
865	50.0	1430	50.0
900	45.0	1590	75.0

channel n = 0.05; overbank n = 0.13

---

(1) All cross sections looking upstream.

TABLE 2.4-24 (Cont)

Distance from Left Bank (ft)	Elevation (ft msl)	Distance from Left Bank (ft)	Elevation (ft msl)
Cross Section No. 3			
0	75.0	745	46.0
80	60.0	790	50.0
680	55.0	900	55.0
700	50.0	980	75.0

channel n = 0.05; overbank n = 0.13

Distance from Left Bank (ft)	Elevation (ft msl)	Distance from Left Bank (ft)	Elevation (ft msl)
Cross Section No. 4			
0	67.3 (RR)	193	56.7
13	63.8	242	55.0
28	58.3	251	59.9
58	56.8	261	65.5
74	54.9	319	70.4
102	55.4	351	70.8
112	49.4	395	71.2
127	47.7	451	71.8
141	49.8	501	72.4
151	46.5	551	73.1
158	48.6	601	73.6
180	48.3	651	74.2
		701	74.8
		751	75.3 (RR)

Bridge assumed clogged at el. 61.5 ft msl.
channel n = 0.09; overbank n = 0.13

Bridge piles (12 in diam) are located at the following distances from the left bank:

14ft	144
28	157
41	169
54	182
66	195
79	208
92	221
105	234
118	247
131	261

TABLE 2.4-24 (Cont)

Distance from Left Bank (ft)	Elevation (ft msl)	Distance from Left Bank (ft)	Elevation (ft msl)

Cross Section No. 4a

0	90.0	585	50.0
50	85.0	620	55.0
90	80.0	645	60.0
140	75.0	880	65.0
220	70.0	910	70.0
310	65.0	930	75.0
360	60.0	970	80.0
540	60.0	1060	85.0
550	55.0	1120	90.0

channel n = 0.06; overbank n = 0.13

Cross Section No. 5

0	66.2 (RR)	122	51.1
29	63.2	140	53.3
43	60.5	160	54.6
77	58.7	173	59.2
93	56.6	189	63.0
102	52.2	190	66.1 (RR)
112	50.8		

Bridge assumed clogged at el 61.5 ft msl.
channel n = 0.10; overbank n = 0.13

Bridge piles (12 in diam) are located at the following distances from the left bank:

29	119 (Double pile)
43	133
57	147
71	161
86 (Double pile)	176
	189

Cross Section No. 5a

0	90.0	340	55.0
70	85.0	420	55.0
120	80.0	460	60.0
180	75.0	830	65.0
210	70.0	870	70.0
250	65.0	910	75.0
300	60.0	930	80.0
		960	85.0
		990	90.0

TABLE 2.4-24 (Cont)

Distance from Left Bank (ft)	Elevation (ft msl)	Distance from Left Bank (ft)	Elevation (ft msl)

channel n = 0.094; overbank n = 0.13

Cross Section No. 6

0	83.0 (RR)	470	66.6
14	78.9	488	61.3
23	77.0	498	54.8
36	74.3	514	55.3
49	73.3	536	55.5
71	69.5	545	58.9
88	66.0	558	66.8
114	64.4	584	67.0
137	63.9	614	67.1
178	63.7	664	67.4
185	57.3	714	68.4
203	64.3	760	69.3
264	64.5	778	70.7
314	65.0	793	71.3
364	65.2	804	75.6
387	65.4	814	80.2
421	64.8	823	84.4 (RR)
436	66.0		

Bridge assumed clogged at el 62.0 ft msl.
channel n = 0.094; overbank n = 0.13

Bridge piles (12 in diam) are located at the following distances from the left bank:

469	500	530	561
485	514	545	

Cross Section No. 6a

0	85.0	460	65.0
120	75.0	590	70.0
250	60.0	650	75.0
285	55.0	690	80.0
320	60.0		

channel n = 0.094; overbank n = 0.13

TABLE 2.4-24 (Cont)

Distance from Left Bank (ft)	Elevation (ft msl)	Distance from Left Bank (ft)	Elevation (ft msl)
Cross Section No. 7			
0	70.6 (RR)	170	56.3
81	67.1	179	56.1
103	62.1	185	59.6
116	57.8	200	62.6
124	56.5	206	65.5
132	55.7	230	64.6
143	56.5	233	69.9
152	56.9	275	69.9 (RR)
160	55.0		

Bridge assumed clogged at el 65.0 ft msl.
channel n = 0.09; overbank n = 0.13

Bridge piles (12 in diam) are located at the following distances from the left bank:

82	164
95	177
109	191
122	205
136	219
151	231

Cross Section No. 8

0	100.0	600	65.0
180	75.0	630	70.0
280	70.0	720	75.0
460	65.0	900	100.0
500	60.0		
580	60.0		

channel n = 0.06; overbank n = 0.13

Cross Section No. 9

0	100.0	710	65.0
260	80.0	750	75.0
470	75.0	790	100.0
630	65.0		

channel n = 0.07; overbank n = 0.13

TABLE 2.4-24 (Cont)

Distance from Left Bank (ft)	Elevation (ft msl)	Distance from Left Bank (ft)	Elevation (ft msl)

Cross Section No. 10

0	100.0	470	75.0
120	90.0	530	80.0
350	80.0	700	90.0
370	75.0	750	100.0

channel n = 0.07; overbank n = 0.13

Cross Section No. 11

0	110.0	425	80.0
50	100.0	450	85.0
150	95.0	460	90.0
280	90.0	545	95.0
320	85.0	580	100.0
350	80.0	660	110.0

channel n = 0.07; overbank n = 0.13

Cross Section No. West Creek

0	72.2 (RR)	117	58.9
69	72.6	123	61.9
72	66.9	129	62.5
76	65.3	139	65.0
84	60.0	153	68.5
91	58.3	155	73.2
100	57.6	200	73.8 (RR)
109	58.4		

Bridge assumed clogged at el 67.5 ft msl.
channel n = -.11; overbank n = 0.12

Bridge piles (12 in diam) are located at the following distances from the left bank:

71	98	126	153
84	112	140	

TABLE 2.4-24 (Cont)

Distance from Left Bank (ft)	Elevation (ft msl)	Distance from Left Bank (ft)	Elevation (ft msl)
Cross Section No. W1a			
0	90.0	210	70.0
20	85.0	230	75.0
30	80.0	300	80.0
80	75.0	320	85.0
90	70.0	330	90.0
130	65.0		
180	65.0		

channel n = 0.05; overbank n = 0.12

Distance from Left Bank (ft)	Elevation (ft msl)	Distance from Left Bank (ft)	Elevation (ft msl)
Cross Section No. W2			
0	86.5 (RR)	124	64.3
78	85.4	139	66.0
79	81.3	146	68.6
92	74.5	156	79.3
104	70.4	158	84.6
109	66.0	250	83.6 (RR)

Bridge assumed clogged at el 77.0 ft msl.
channel n = 0.10; overbank n = 0.12

Bridge piles (12 in diam) are located at the following distances from the left bank:

89	105	130	157
102	118	144	

Distance from Left Bank (ft)	Elevation (ft msl)	Distance from Left Bank (ft)	Elevation (ft msl)
Cross Section No. W3			
0	100.0	250	75.0
40	90.0	265	80.0
90	85.0	360	85.0
120	80.0	400	90.0
150	75.0	440	95.0
200	71.0	470	100.0

channel n = 0.05; overbank n = 0.12

TABLE 2.4-24 (Cont)

Listance from Left Bank (ft)	Elevation (ft msl)	Distance from Left Bank (ft)	Elevation (ft msl)
Cross Section No. W4			
0	100.0	190	80.0
50	95.0	220	85.0
80	90.0	370	90.0
110	85.0	400	95.0
155	80.0		

channel n = 0.05; overbank n = 0.12

Cross Section No. W4a			
0	90.0 (Wall)	300	80.0
220	90.0	330	90.0
250	80.0	510	92.0

channel n = 0.03; overbank n = 0.12

Cross Section Nos. W5 through W9 (Fabriform Channel)

See Figure 2.4-28

TABLE 2.4-25

SSE CROSS SECTION DATA[1]

GRANTS BAYOU

Distance from Left Bank (ft)	Elevation (ft msl)	Distance from Left Bank (ft)	Elevation (ft msl)
Cross Section No. 1			
0	56.5	245	44.5
		540	59.0
n for entire channel = 0.13			
Cross Section No. 2			
0	69.0	830	43.5
480	45.0	1240	64.0
820	45.0		
n for entire channel = 0.13			
Cross Section No. 3			
0	89.0	990	53.5
620	58.0	1400	74.0
980	55.0		
n for entire channel = 0.13			
Cross Section No. 4			
See Fig. 2.4-22.			
Cross Section No. 4a			
0	82.0	670	60.5
440	60.0	1120	83.0
645	60.0		
n for entire channel = 0.13			
Cross Section No. 5			
See Fig. 2.4-23.			

TABLE 2.4-25 (Cont)

Distance from Left Bank (ft)	Elevation (ft msl)	Distance from Left Bank (ft)	Elevation (ft msl)
Cross Section No. 5a			
0	80.0	495	60.5
440	58.0	680	63.0
		990	78.5
n for entire channel = 0.13			
Cross Section No. 6			
See Fig. 2.4-24.			
Cross Section No. 6a			
0	70.0	460	65.0
340	60.0	690	76.0
390	62.5		
n for entire channel = 0.13			
Cross Section No. 7			
See Fig. 2.4-25.			
Cross Section No. 8			
0	88.0	900	88.0
450	66.0		
n for entire channel = 0.13			
Cross Section No. 9			
0	105.0	1320	106.0
650	72.0		
n for entire channel = 0.13			
Cross Section No. 10			
0	103.0	1160	112.0
490	79.0		
n for entire channel = 0.13			

TABLE 2.4-25 (Cont)

Distance from Left Bank (ft)	Elevation (ft msl)	Distance from Left Bank (ft)	Elevation (ft msl)
Cross Section No. 11			
0	125.0	1250	115.0
60	123.0		
740	89.0		

n for entire channel = 0.13.

WEST CREEK

Cross Section No. W1

See Fig. 2.4-26.

Cross Section No. W1a			
0	86.0	500	91.0
200	76.0		

n for entire channel = 0.12

Cross Section No. W2

See Fig. 2.4-27.

Cross Section No. W3			
0	121.0	980	80.0
320	105.0	1480	100.0
590	105.0	1540	100.0
		1610	105.0

n for entire channel = 0.12

Cross Section No. W4			
0	99.0	400	87.0
310	83.5	540	94.0
350	85.5	800	105.0

n for entire channel = 0.12

Cross Section No. W4a			
0	106.5	660	92.0
310	90.0	680	95.0
340	90.0	790	95.0
430	84.0	990	105.0
590	91.0		

TABLE 2.4-25 (Cont)

Distance from Left Bank (ft)	Elevation (ft msl)	Distance from Left Bank (ft)	Elevation (ft msl)
n for entire channel = 0.05			
Cross Section No. W5			
0	106.0	690	95.0
410	83.5	720	95.0
		880	103.0
560	91.0	940	105.0
640	92.0		
n for entire channel = 0.05			
Cross Section No. W6			
0	115.0	990	95.0
610	84.5	1050	95.0
740	91.0		
980	92.0	1260	106.0
n for entire channel = 0.05			
Cross Section No. W7			
0	105.0	690	95.0
230	94.5	960	95.0 (Wall)
360	93.0		
510	86.0		
n for entire channel = 0.05			
Cross Section No.W8			
0	127.0	1210	95.0 (Wall)
760	87.5		
930	95.0		
n for entire channel = 0.05			
Cross Section No. W9			
0	113.5	700	95.0
400	94.5	1000	95.0 (Wall)
420	94.5		
550	88.0		
n for entire channel = 0.05			

(1) All cross sections looking upstream. Maximum channel side slope after SSE = 20H: 1V.

TABLE 2.4-26

DESIGN FLOOD PROFILES

Station	Cross Section	Grants Bayou PMF Flow (cfs)	Grants Bayou PMF Elevation (ft msl)	Grants Bayou 25-yr Flood + SSE Flow (cfs)	Grants Bayou 25-yr Flood + SSE Elevation (ft msl)
0'	No. 1	2208[4]	54.5[1]	4760	54.5[1]
2000'	No. 2	7927	54.7	4692	55.9
6150'	No. 3	19794	63.1	4550	59.0
7400'	No. 4	23368	75.3	4507	71.8
9600'	No. 4a	28229	79.3	4449	73.0
9600'	No. 5	29659	81.6	4432	73.1
10300'	No. 5a	31661	84.1	4408	73.3
11000'	No. 6	33662	84.5	4384	73.5
11600'	No. 7	35378	85.8	4364	74.1
13000'	No. 8	35346	89.2	4364	78.7
15200'	No. 9	35346	91.1	4364	82.6
17700'	No. 10	35346	95.3	4364	89.9
19750'	No. 11	35346	101.8	4364	98.1

Station	Cross Section	West Creek PMF Flow (cfs)	West Creek PMF Elevation (ft msl)	West Creek 25-yr Flood + SSE Flow (cfs)	West Creek 25-yr Flood + SSE Elevation (ft msl)
625'	No. W1	6699	79.1	842	73.5
1325'	No. W1a	6314	80.2	794	81.7
1925'	No. W2	5984	87.7	752	86.0
2625'	No. W3	5598	88.8	703	87.3
3425'	No. W4a	5158	89.9	648	88.4
3550'	No. 5	5062	90.4	636	88.5
4070'	No. W6	4668	90.9	587	88.9
4670'	No. W7	4212	91.6	529	89.6
5190'	No. W8	3818	92.2	480	90.8
5850'	No. W9	3392	92.7	427	91.8

---

(1)Mississippi River PDF crest level.
(2)Control Section at W1.
(3)Measured from Grants Bayou cross section No. 7.
(4)Two cases were evaluated for the peak Grants Bayou water level, one at Time = Grants Bayou peak flow at West Creek confluence, and a second at Time = Grants Bayou peak flow at outlet to river flood plain. The first case was found to produce higher water levels in the plant area, and is presented here.

TABLE 2.4-36
PMP RUNOFF TO UNIT 2 EXCAVATION

Time (HR)	Accum Rain (1) (in)	Runoff From Const Pkg(2) (in)	Runoff From Area Within RR and Ring Road (in)	Vol From Dir Rain (A=16.76 across) (FT3)	Vol. From Const Pkg (FT3)	Volume From Area Within RR (FT3)	Total Runoff Volume (FT3)
0-6	3.1	1.32	.11	188,583	80,858	3,194	272,635
6-12	9.8	7.33	3.42	596,167	449,008	99,317	1,144,492
12-13	11.7	9.16	4.78	711,750	561,107	138,811	1,411,668
13-14	13.9	11.29	6.47	845,583	691,583	187,889	1,725,055
14-15	17.7	15.02	9.59	1,076,750	920,069	278,494	2,275,313
15-16	37.1	34.26	28.32	2,256,917	2,098,639	1,308,252	5,663,808
16-17	39.8	36.76	29.62	2,421,167	2,251,780	1,375,044	6,047,991
17-18	41.8	38.56	31.82	2,542,833	2,362,041	1,421,508	6,326,382
18-24	47.1	42.66	34.12	2,865,250	2,613,192	1,505,724	6,984,166
24-48	51.8	42.66	34.12	3,151,167	2,613,192	1,505,724	7,270,083
48-72	55.7	42.66	34.12	3,388,417	2,613,192	1,505,724	7,507,333

(1) Storm arranged in conservative sequence pattern. See Section 2.4.3.

(2) Construction parking area 22.5 acres. Area within ring road and railroad = 16 acres.

TABLE 2.4 - 37

Ponding and Seepage in the Unit 2 Excavation

Time (hr)	Total Runoff to Excavation (ft^3)	Runoff Inflow (cfs)	Seepage Flow (cfs)	Ponded Volume (ft^3)	Ponded Elevation (ft msl)	Groundwater Level 50 ft Into Embankment (ft msl)
0-6	272,635	12.6	(1)	-	-	(5)
6-12	1,144,492	40.4	(1)	-	-	(5)
12-13	1,411,668	74.2	(1)	-	-	(5)
13-14	1,725,055	87.0	(3)	190,555	66.3	(5)
14-15	2,275,313	152.8	(3)	740,813	67.3	(5)
15-16	5,663,808	941.2	(3)	4,129,308	73.2	(5)
16-17	6,047,991	106.7	195.6	3,809,160	72.7	65.8
17-18	6,326,382	77.3	108.4	3,697,308	72.5	676
18-24	6,984,166	30.4	64.7	2,957,076	71.2	68.8
24-48	7,270,083	3.3	33.1 (4)	2,100,276 (2)	69.8 (2)	68.3 (2)
48-72	7,507,333	2.7	-	-	-	-

NOTES:

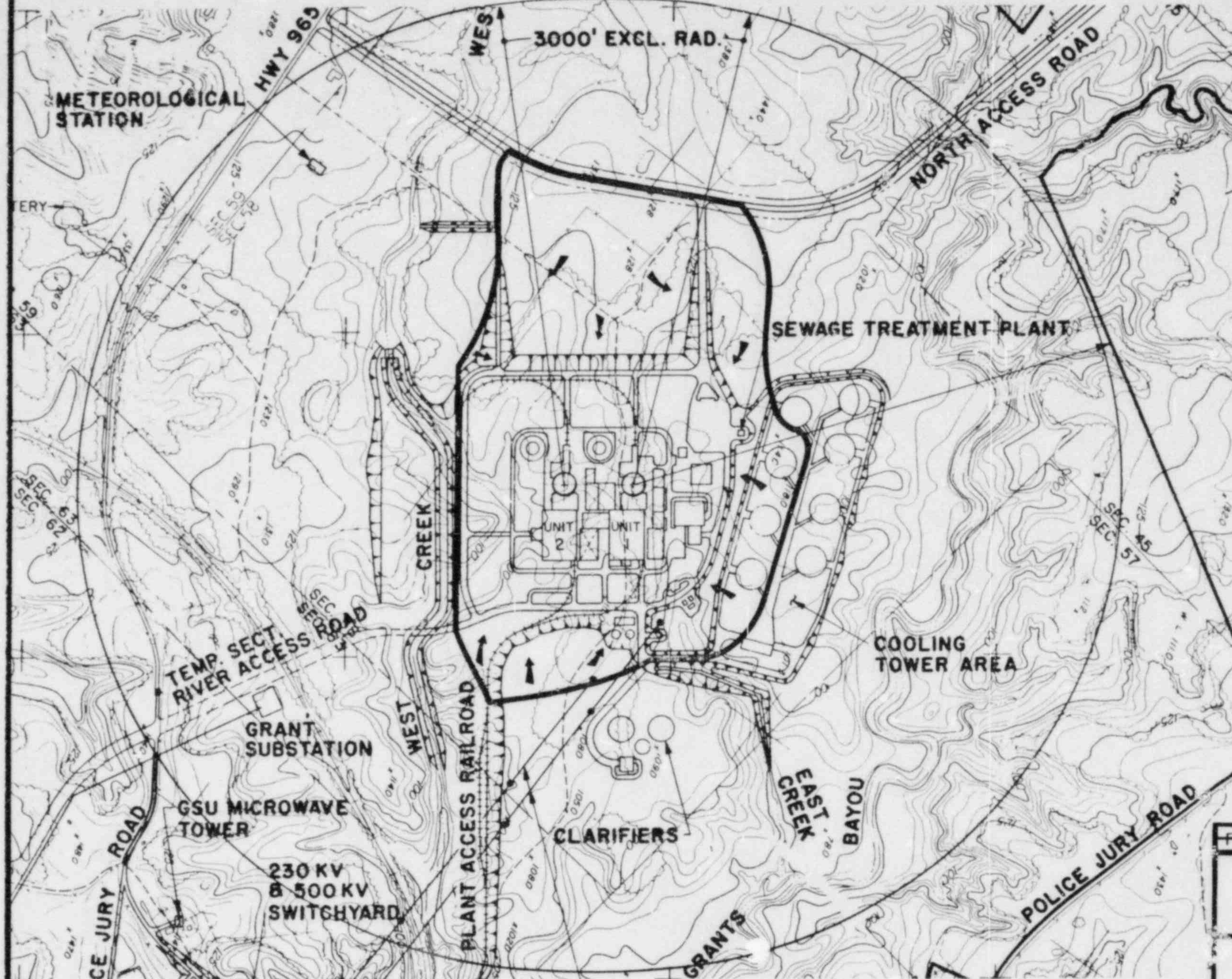
(1) Initial infiltration = 1,334,500 cu ft.

(2) At Hr 32

(3) Assumed = 0 after infiltration period is completed.

(4) For Hr 24-32

(5) Assumed = 57 ft msl, normal groundwater level.



LEGEND:

- DRAINAGE AREA BOUNDARY
- DRAINAGE DIRECTION

REPLACE WITH THE ATTACHED FIGURE.

~~FIGURE 2.4-6~~

~~PLANT DRAINAGE AREA BOUNDARY~~

RIVER BEND STATION
FINAL SAFETY ANALYSIS REPORT

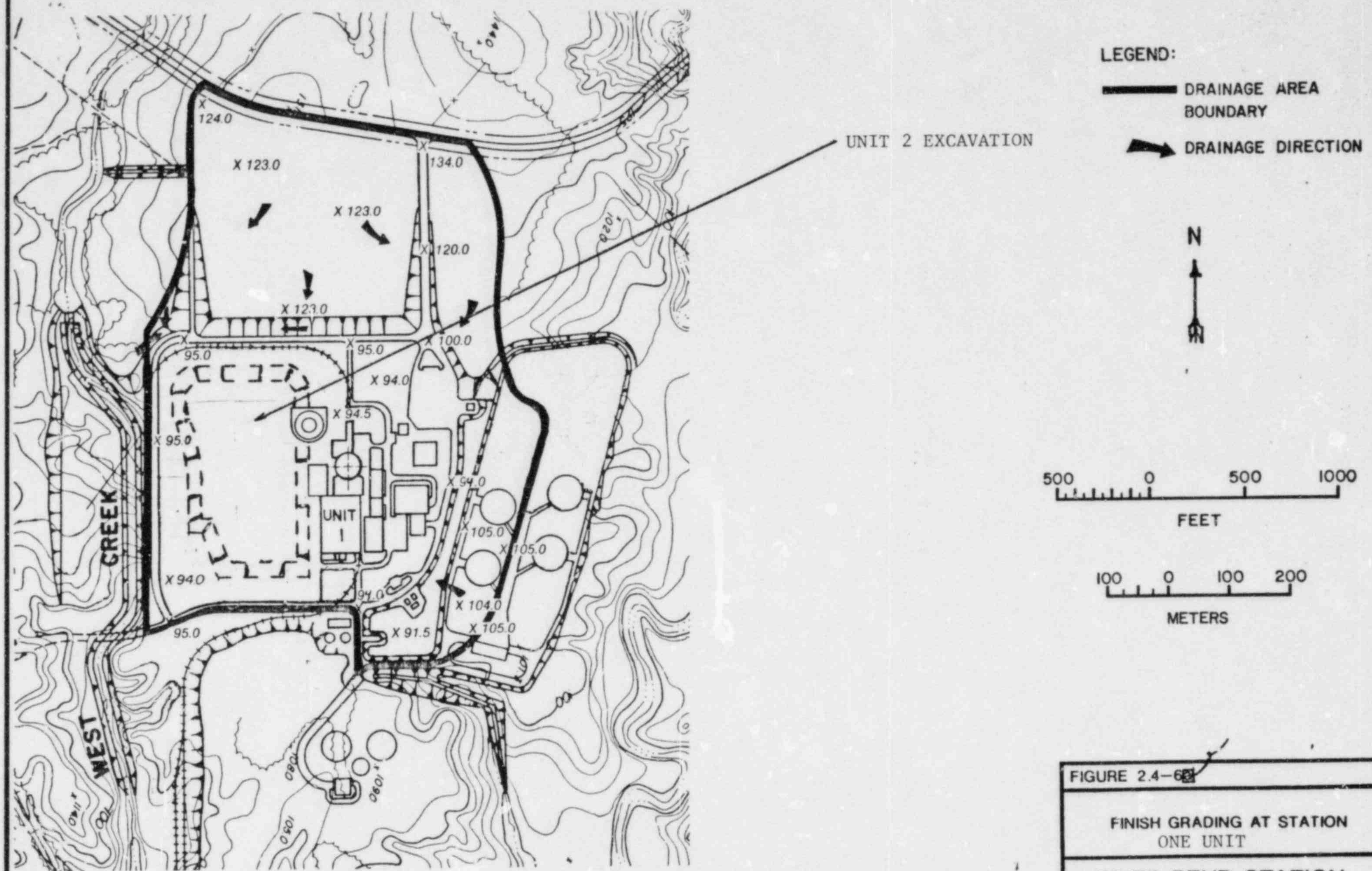


FIGURE 2.4-6

FINISH GRADING AT STATION
ONE UNIT

RIVER BEND STATION
FINAL SAFETY ANALYSIS REPORT

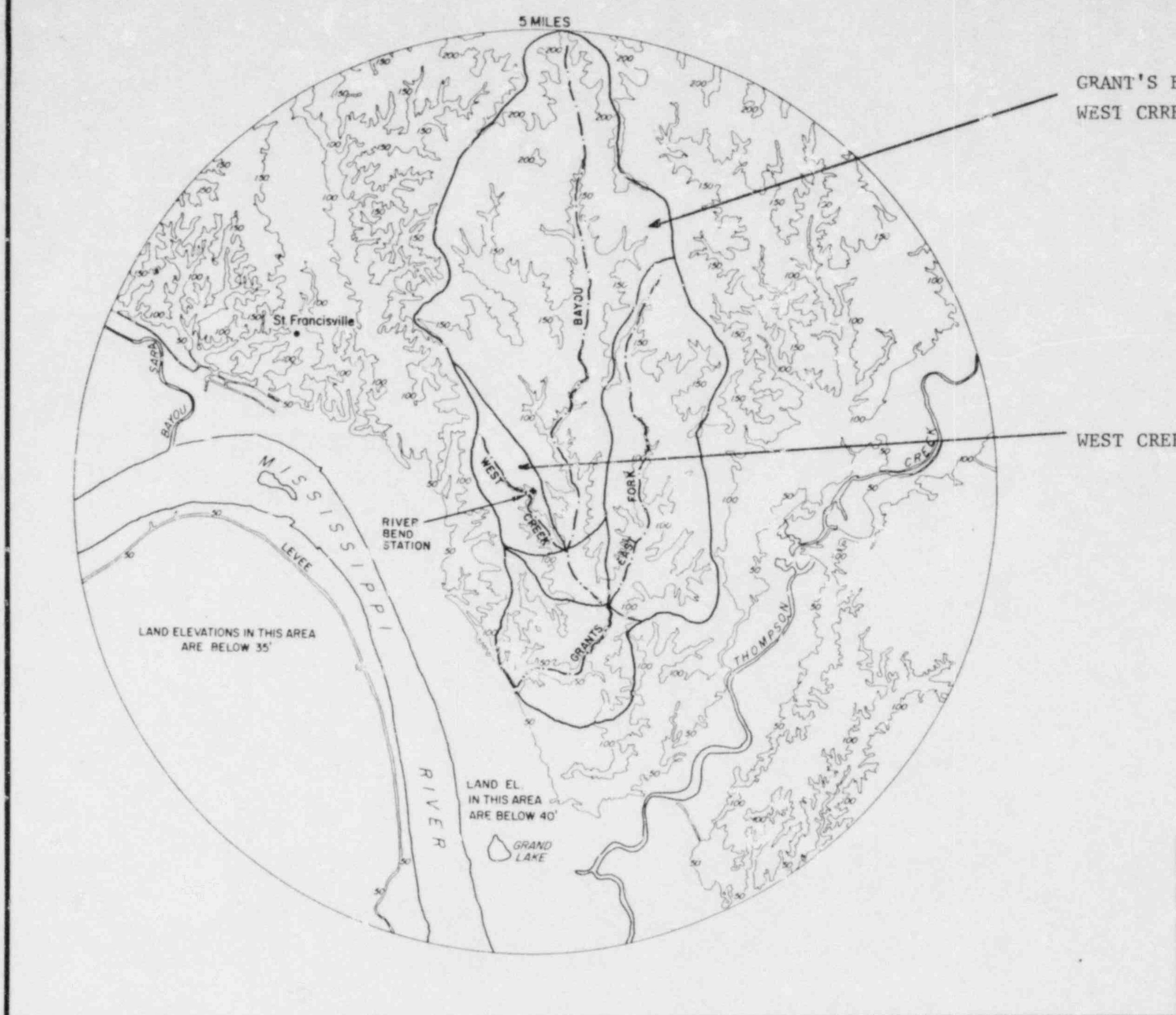
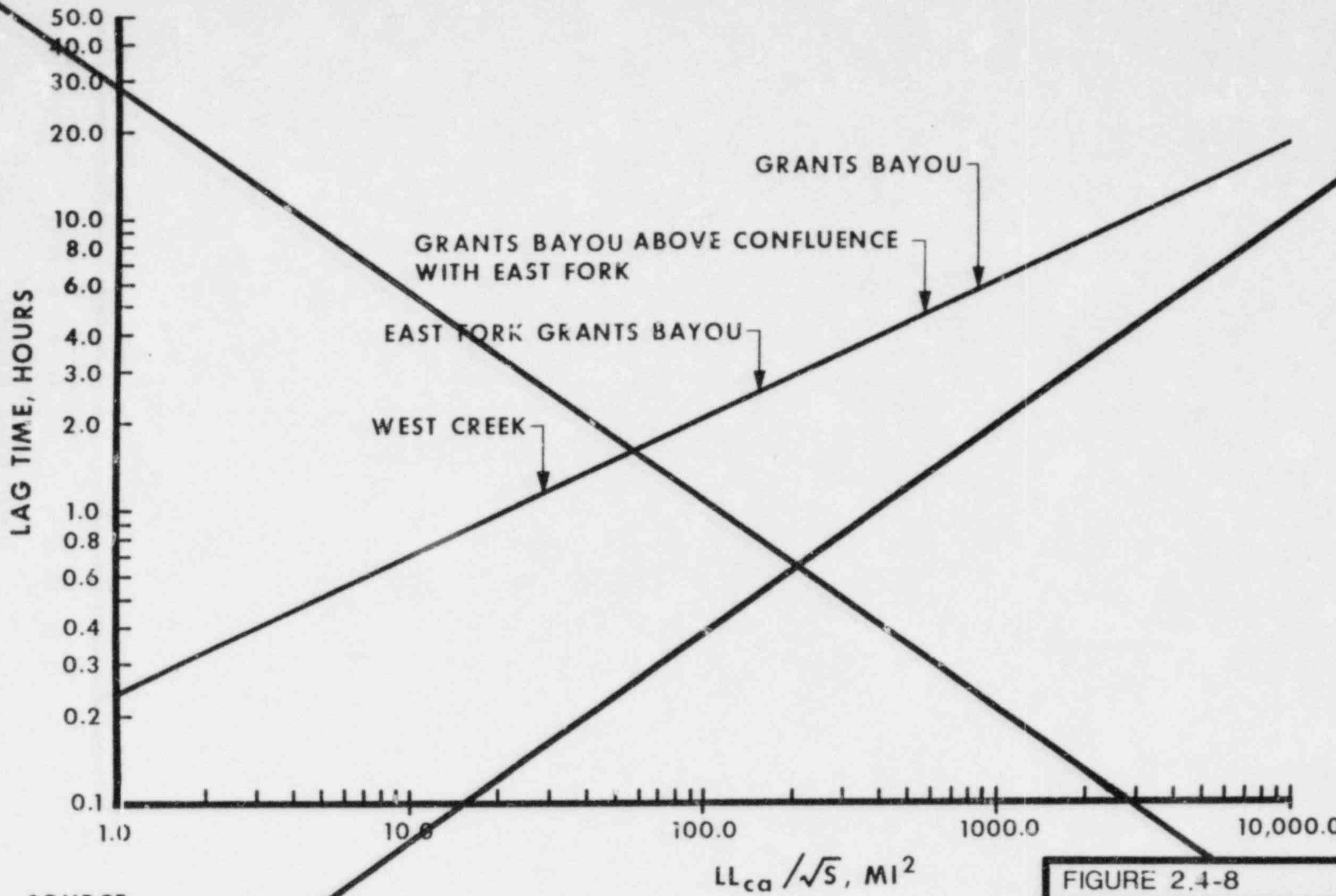


FIGURE 2.4-7

LOCAL DRAINAGE BASINS

RIVER BEND STATION
FINAL SAFETY ANALYSIS REPORT



SOURCE:

TECHNIQUES FOR HYDROGRAPH SYNTHESIS BASED ON ANALYSIS OF DATA FROM SMALL DRAINAGE BASINS IN TEXAS. HUDLOW, M.D. WATER-RESOURCES INSTITUTE, TEXAS A&M UNIVERSITY, 1966

FIGURE 2.4-8

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RIVER BEND STATION

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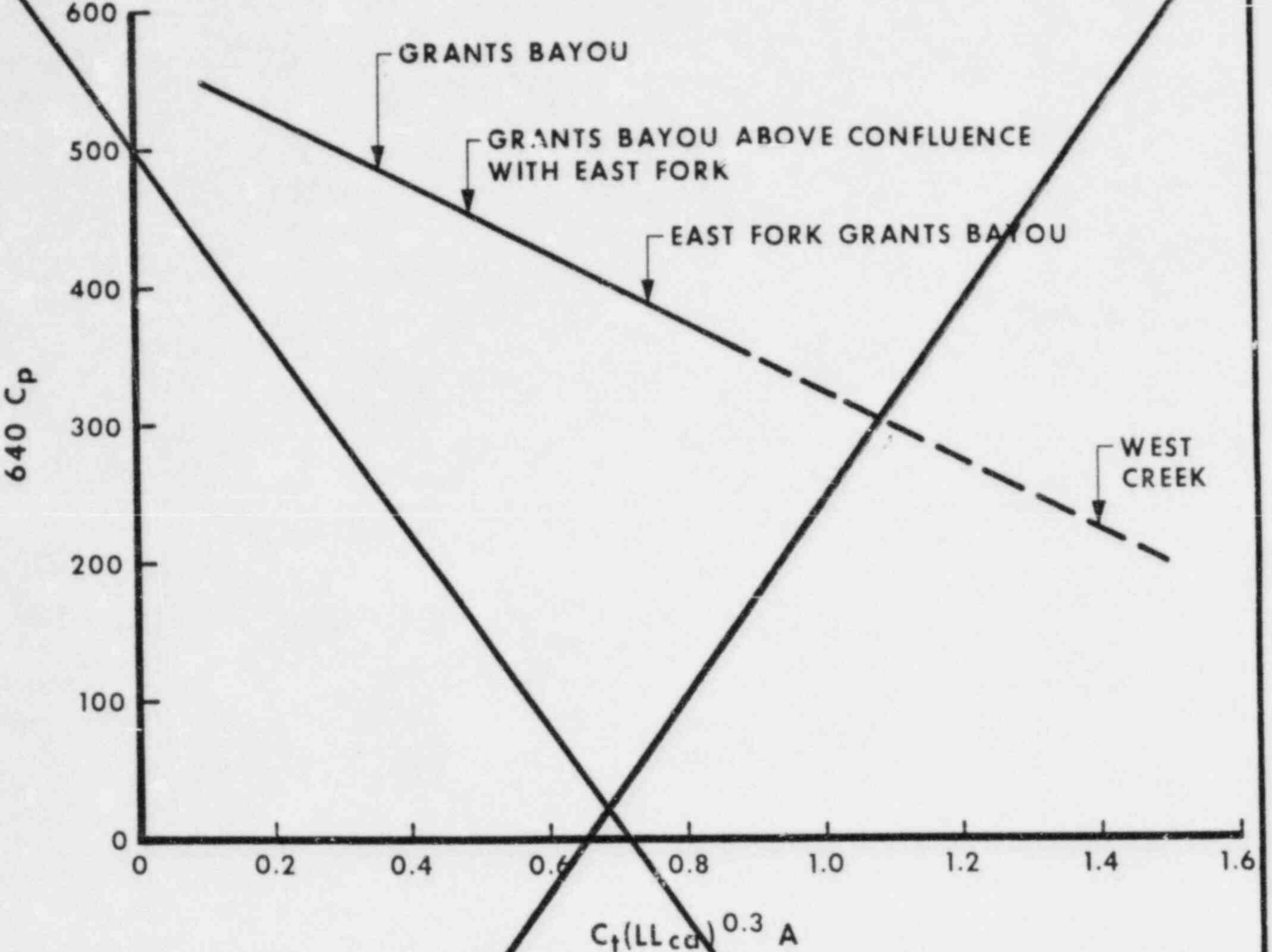

SOURCE:

TECHNIQUES FOR HYDROGRAPH SYNTHESIS BASED ON ANALYSIS OF DATA FROM SMALL DRAINAGE BASINS IN TEXAS. HUDLOW, M.D. WATER-RESOURCES INSTITUTE, TEXAS A&M UNIVERSITY, 1966

FIGURE 2.4-9

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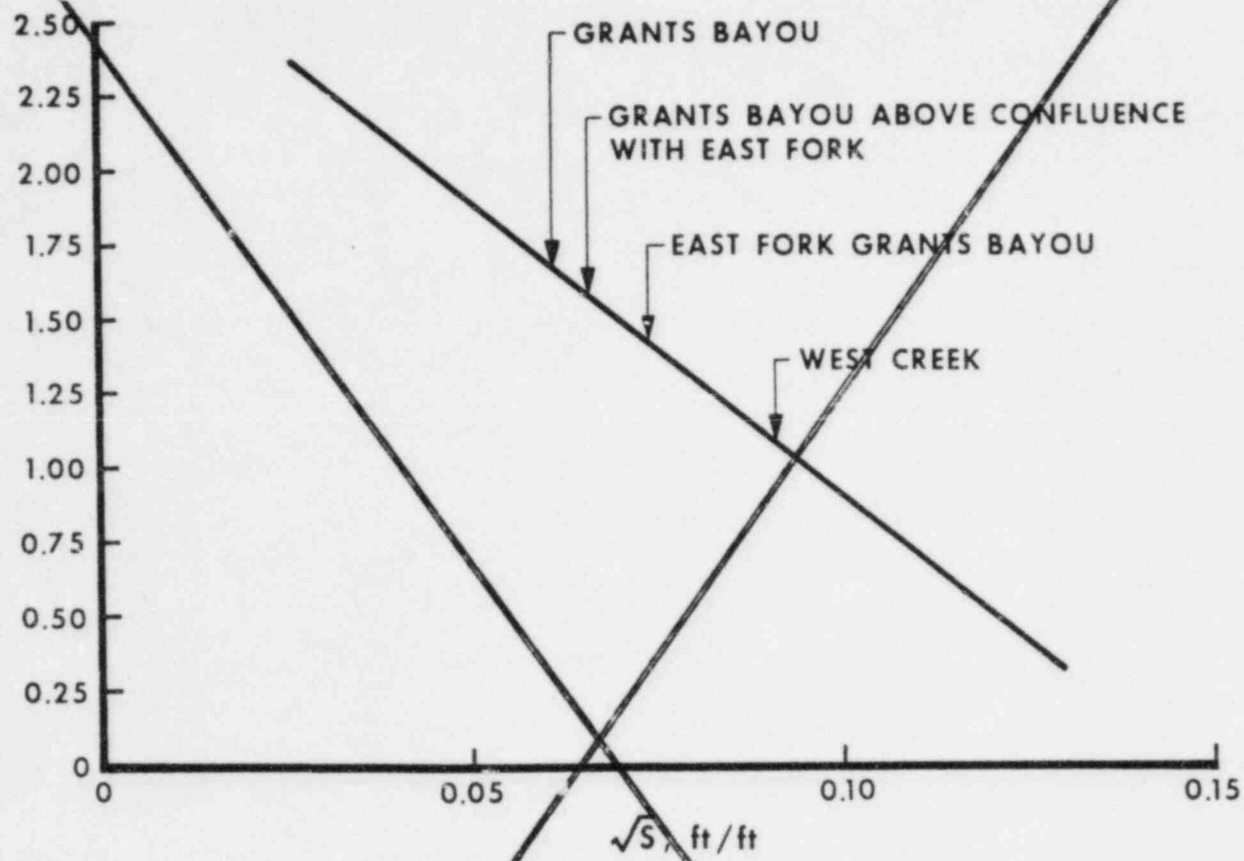
FINAL SAFETY ANALYSIS REPORT



SOURCE:

TECHNIQUES FOR HYDROGRAPH SYNTHESIS BASED ON ANALYSIS OF DATA FROM SMALL DRAINAGE BASINS IN TEXAS. HUDLOW, M.D. WATER-RESOURCES INSTITUTE, TEXAS A&M UNIVERSITY, 1966

FIGURE 2.4-10

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FINAL SAFETY ANALYSIS REPORT

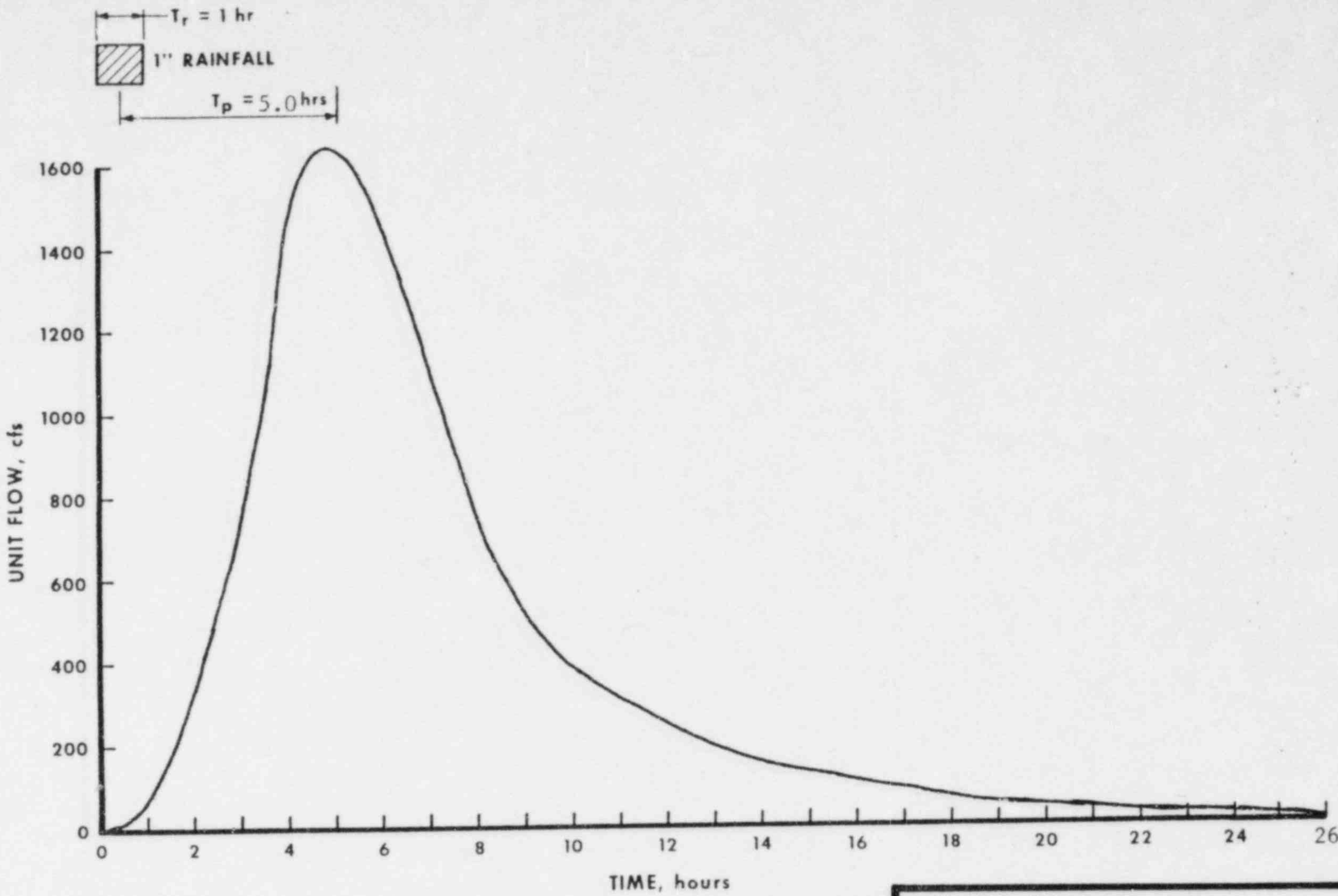

FIGURE 2.4-11

GRANTS BAYOU UNIT HYDROGRAPH

RIVER BEND STATION
FINAL SAFETY ANALYSIS REPORT

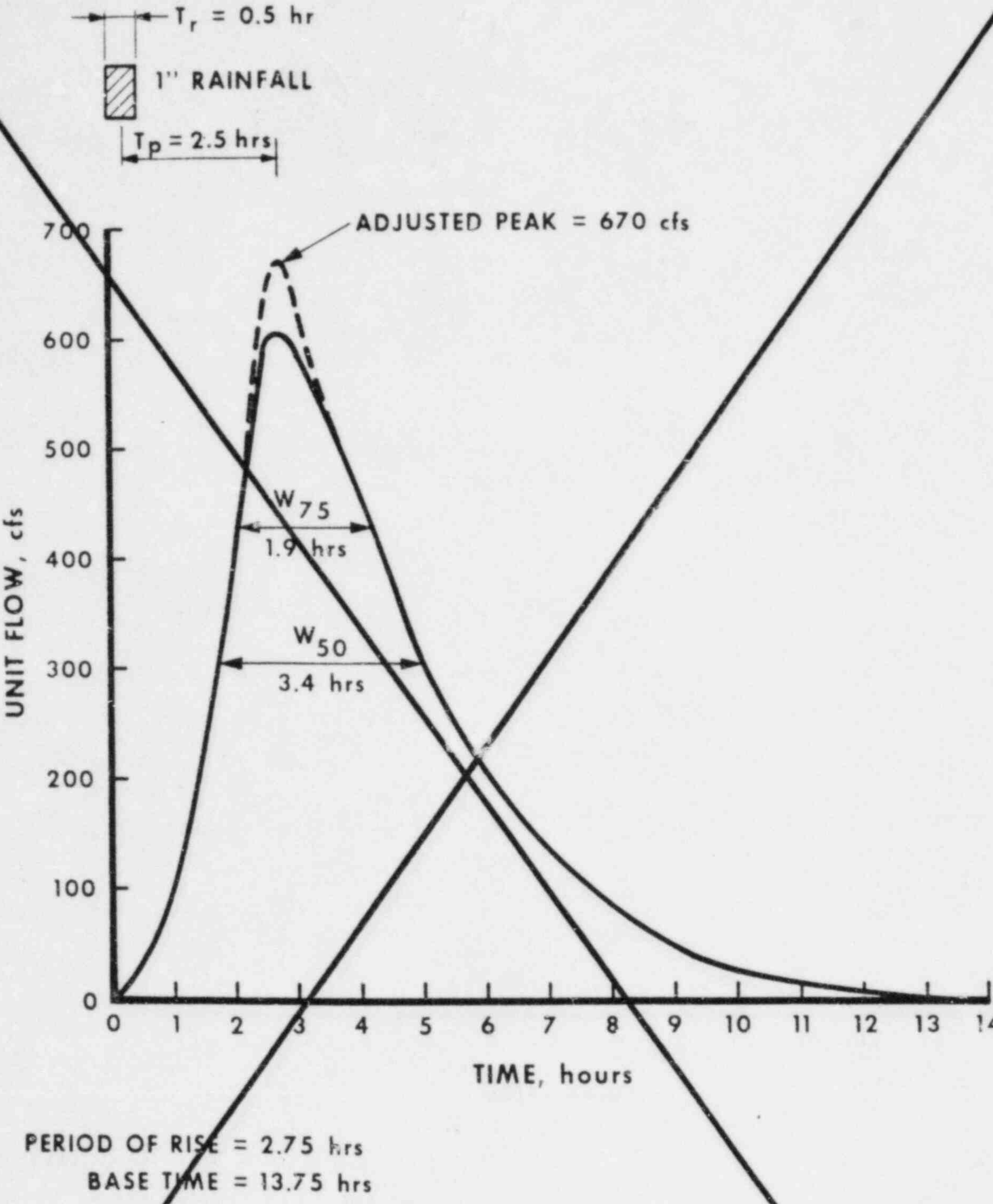

FIGURE 2.4-12

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RIVER BEND STATION

FINAL SAFETY ANALYSIS REPORT

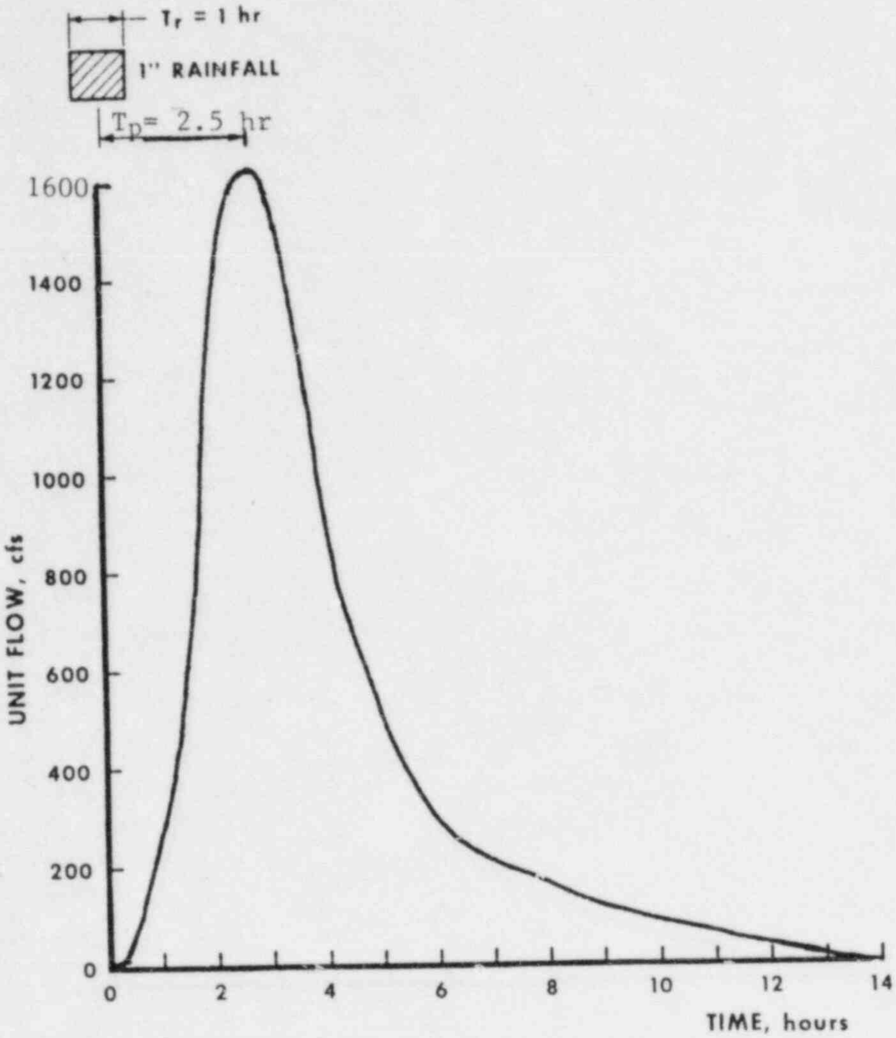

FIGURE 2.4-13

GRANTS BAYOU ABOVE CONFLUENCE WITH ~~EAST FORK~~ WEST CREEK UNIT HYDROGRAPH

RIVER BEND STATION
FINAL SAFETY ANALYSIS REPORT

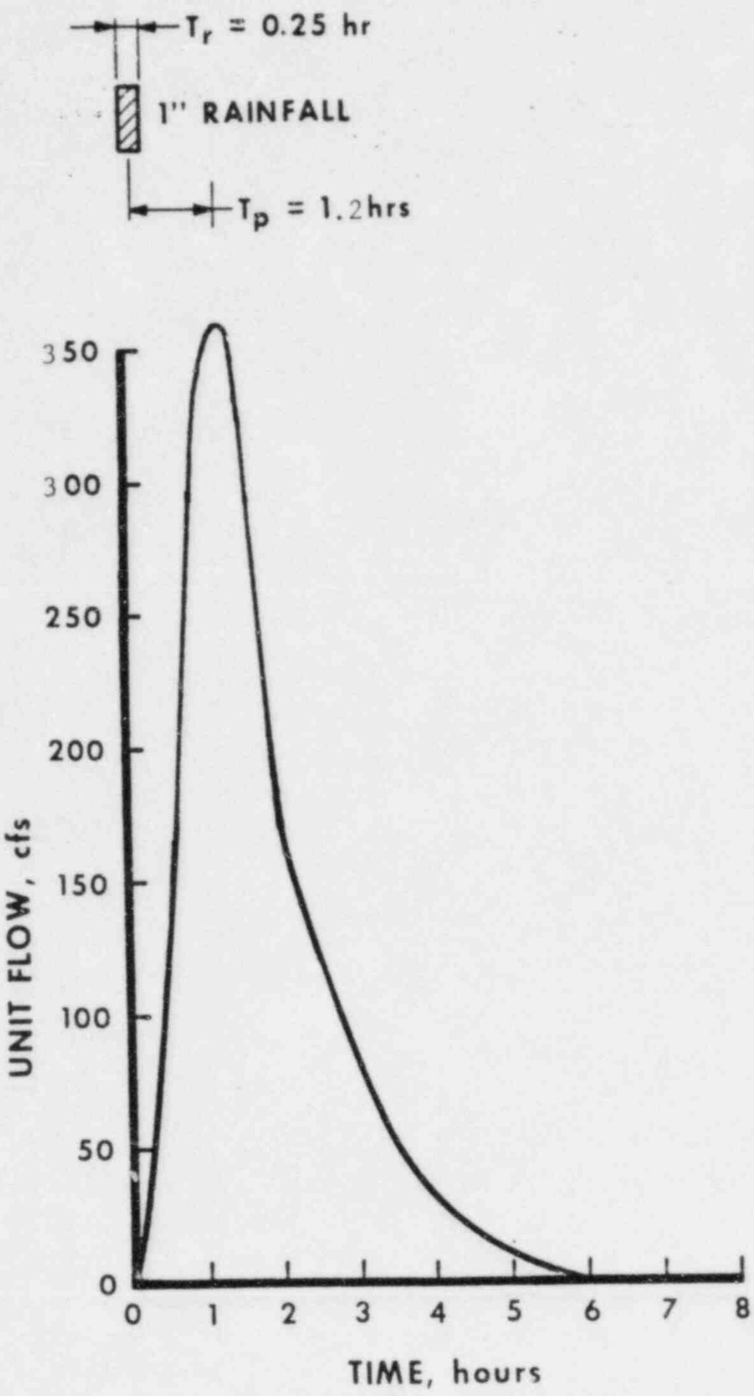

FIGURE 2.4-14

WEST CREEK UNIT HYDROGRAPH

RIVER BEND STATION
FINAL SAFETY ANALYSIS REPORT

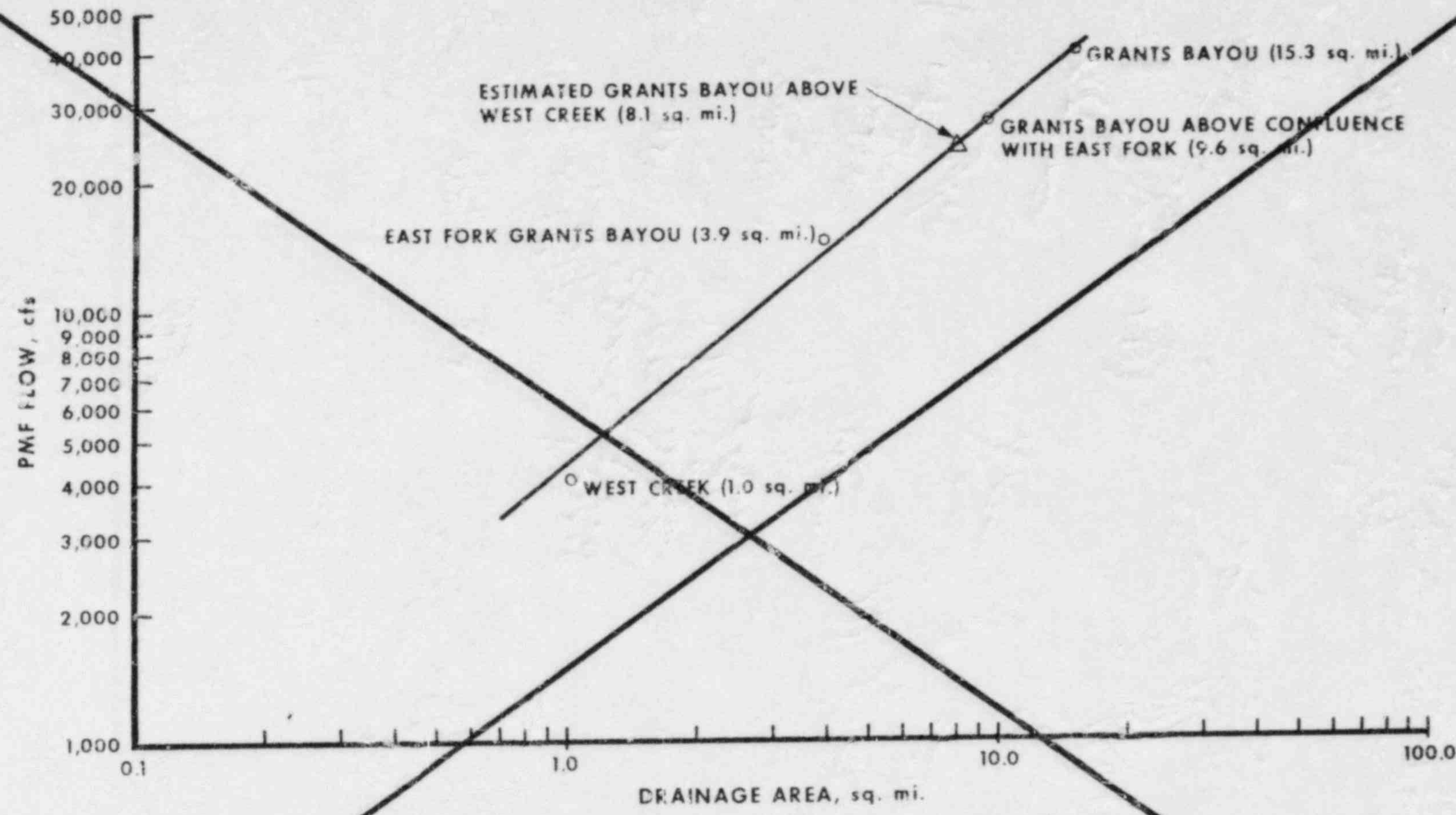

FIGURE 2.4-15

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RIVER BEND STATION

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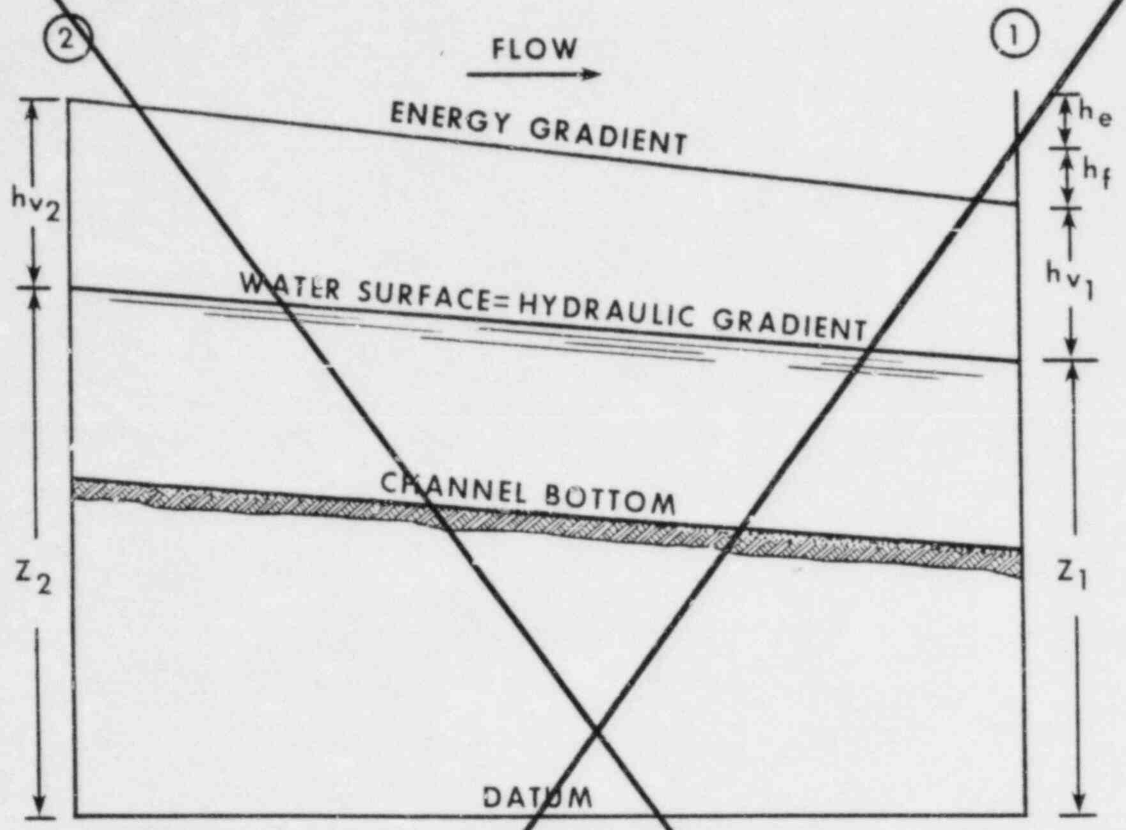

$$Z_2 = Z_1 + h_f + h_{v_1} - h_{v_2} + h_e$$

SOURCE:

BACKWATER CURVES IN RIVER CHANNELS. U.S. ARMY CORPS OF ENGINEERS, EM 1110-2-1409, 1959.

FIGURE 2.4-19

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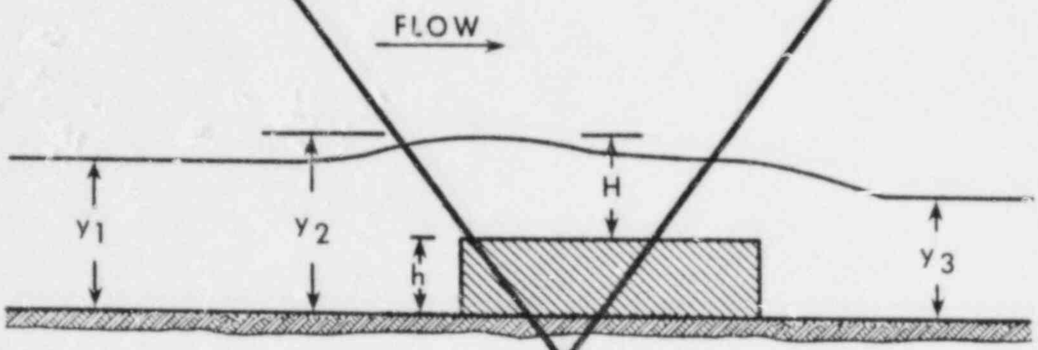
RIVER BEND STATION
FINAL SAFETY ANALYSIS REPORT



IF $y_3 < \frac{(2y_2+h)}{3}$ THEN THE DISCHARGE/HEADWATER RELATIONSHIP BASED ON THE BROAD-CRESTED WEIR FORMULA REMAINS UNAFFECTED BY BACKWATER.

SOURCE:

OPEN-CHANNEL HYDRAULICS. CHOW, V.T., McGRAW-HILL, NEW YORK, 1959.

FIGURE 2.4-20

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RIVER BEND STATION

FINAL SAFETY ANALYSIS REPORT

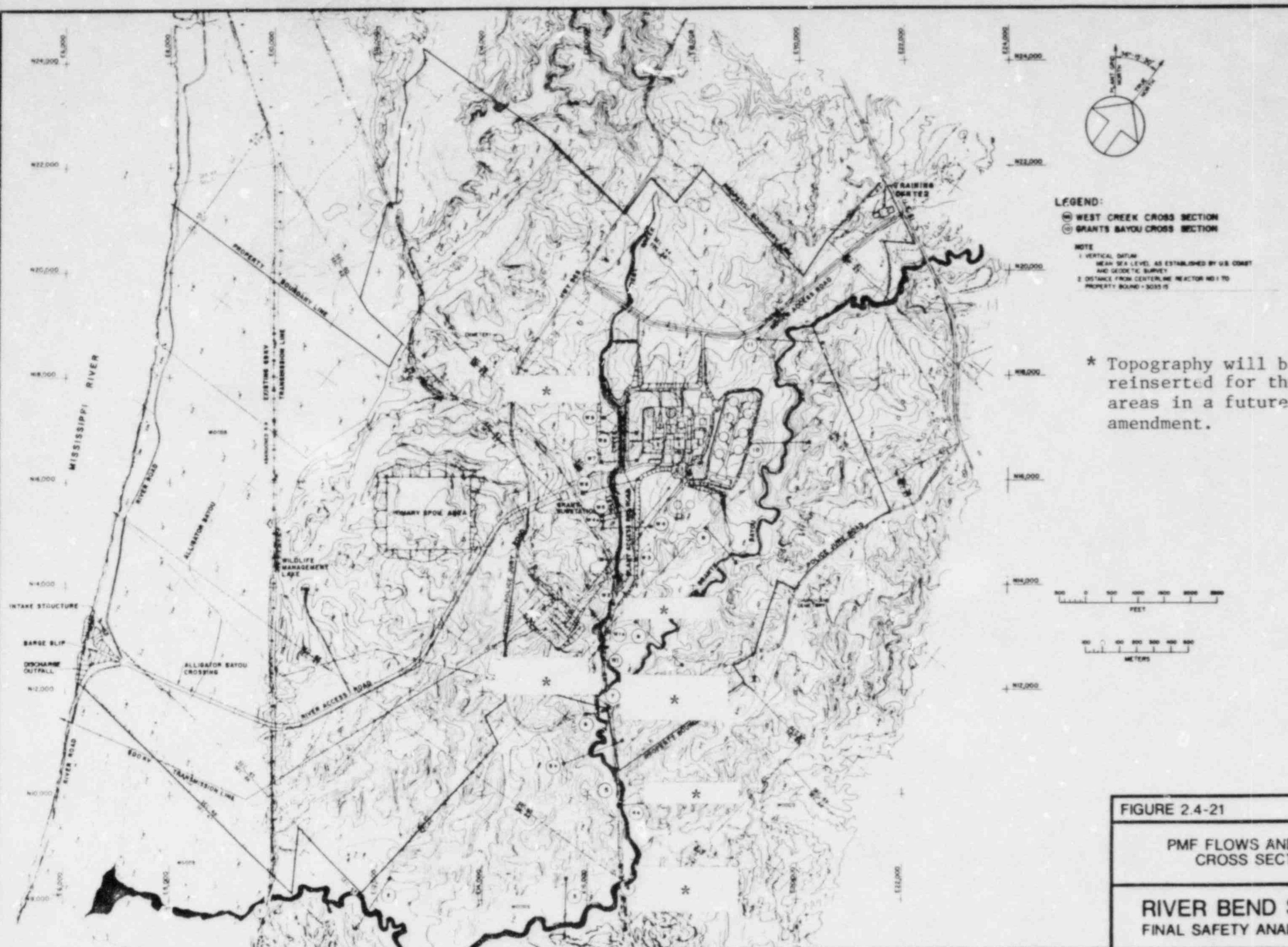
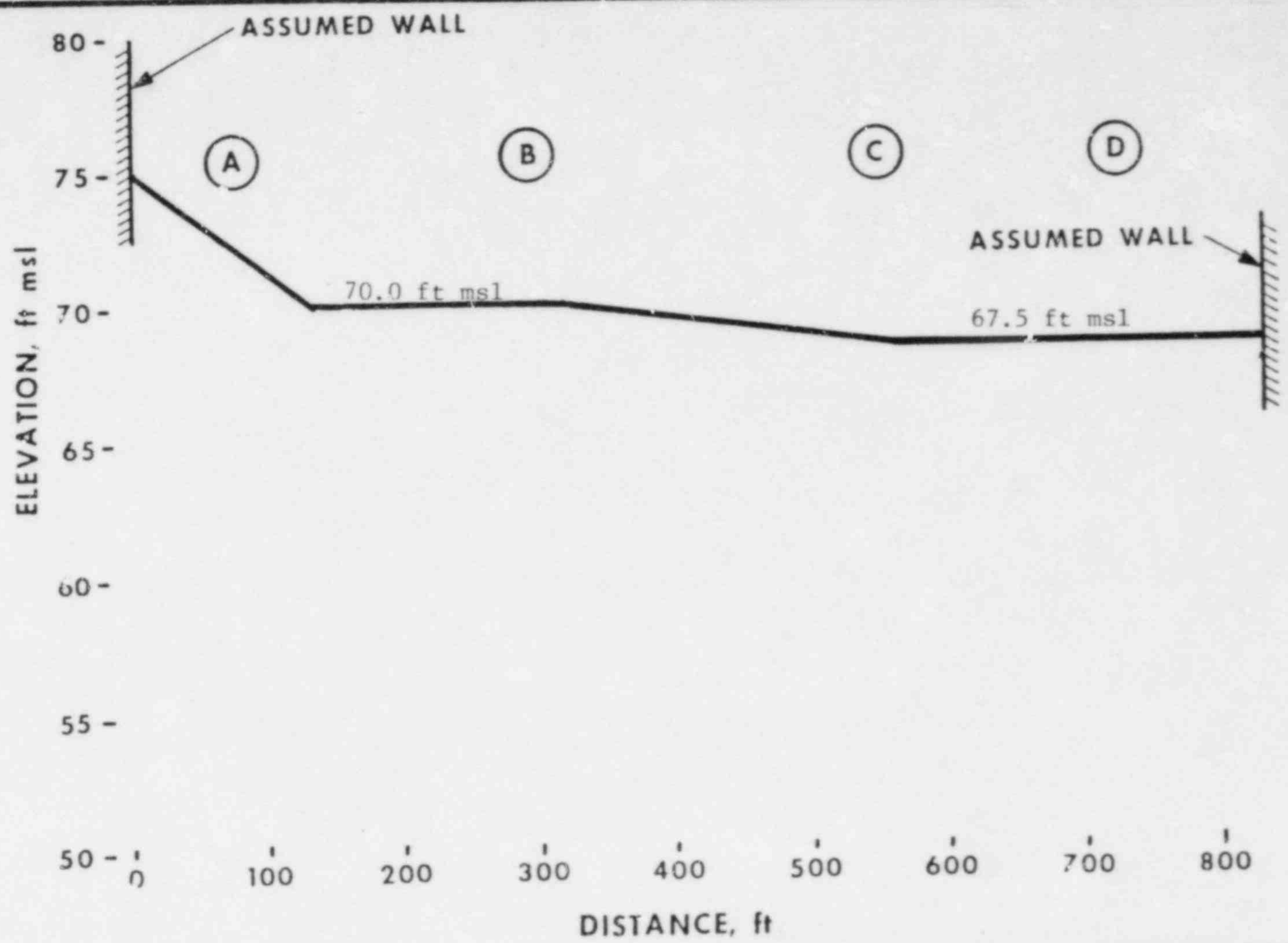


FIGURE 2.4-21

PMF FLOWS AND STREAM CROSS SECTIONS

RIVER BEND STATION
FINAL SAFETY ANALYSIS REPOR

AMENDMENT 7 FEBRUAR

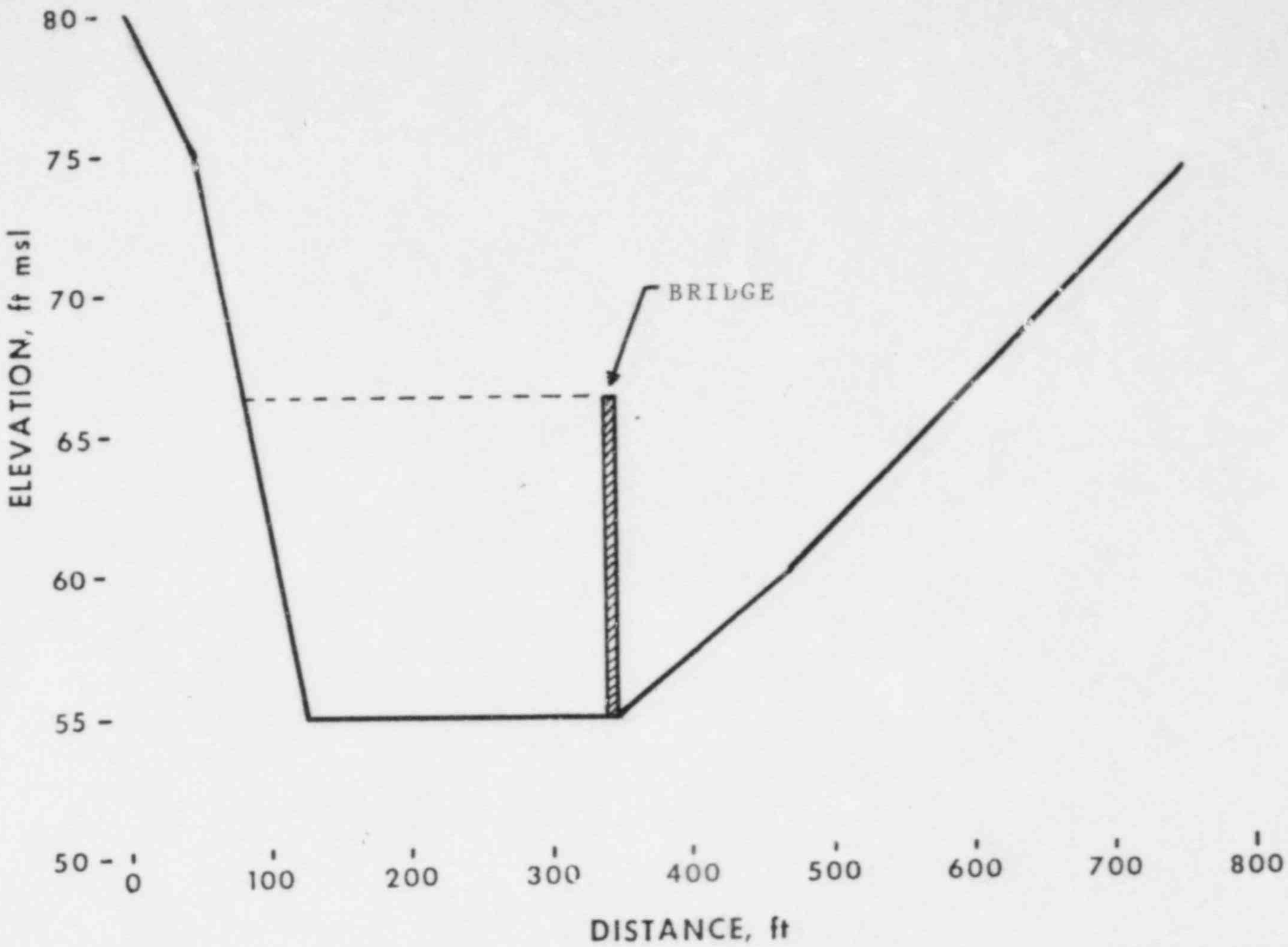


NOTES:
LOOKING DOWNSTREAM.
SEE SECTION LOCATION ON FIGURE 2.4-21

FIGURE 2.4-22

SIMULATED WEIR FOR BRIDGE AT CROSS SECTION 4 ON GRANTS BAYOU, SSE CONDITION

RIVER BEND STATION
FINAL SAFETY ANALYSIS REPORT

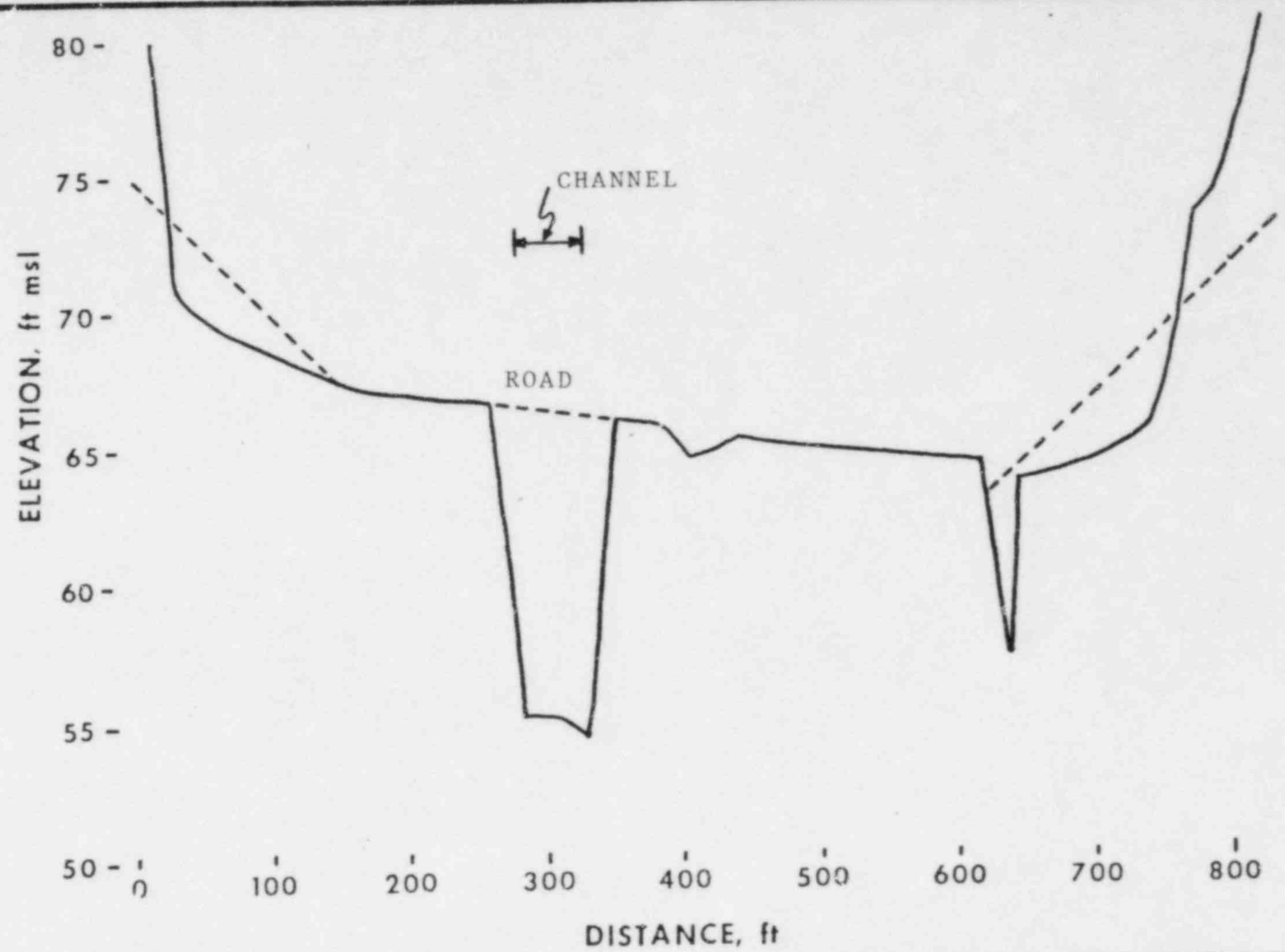


NOTES:
LOOKING DOWNSTREAM.
SEE SECTION LOCATION ON FIGURE 2.4-21

FIGURE 2.4-23

SIMULATED WEIR FOR BRIDGE AT CROSS SECTION 5 ON GRANTS BAYOU, SSE CONDITION

RIVER BEND STATION
FINAL SAFETY ANALYSIS REPORT

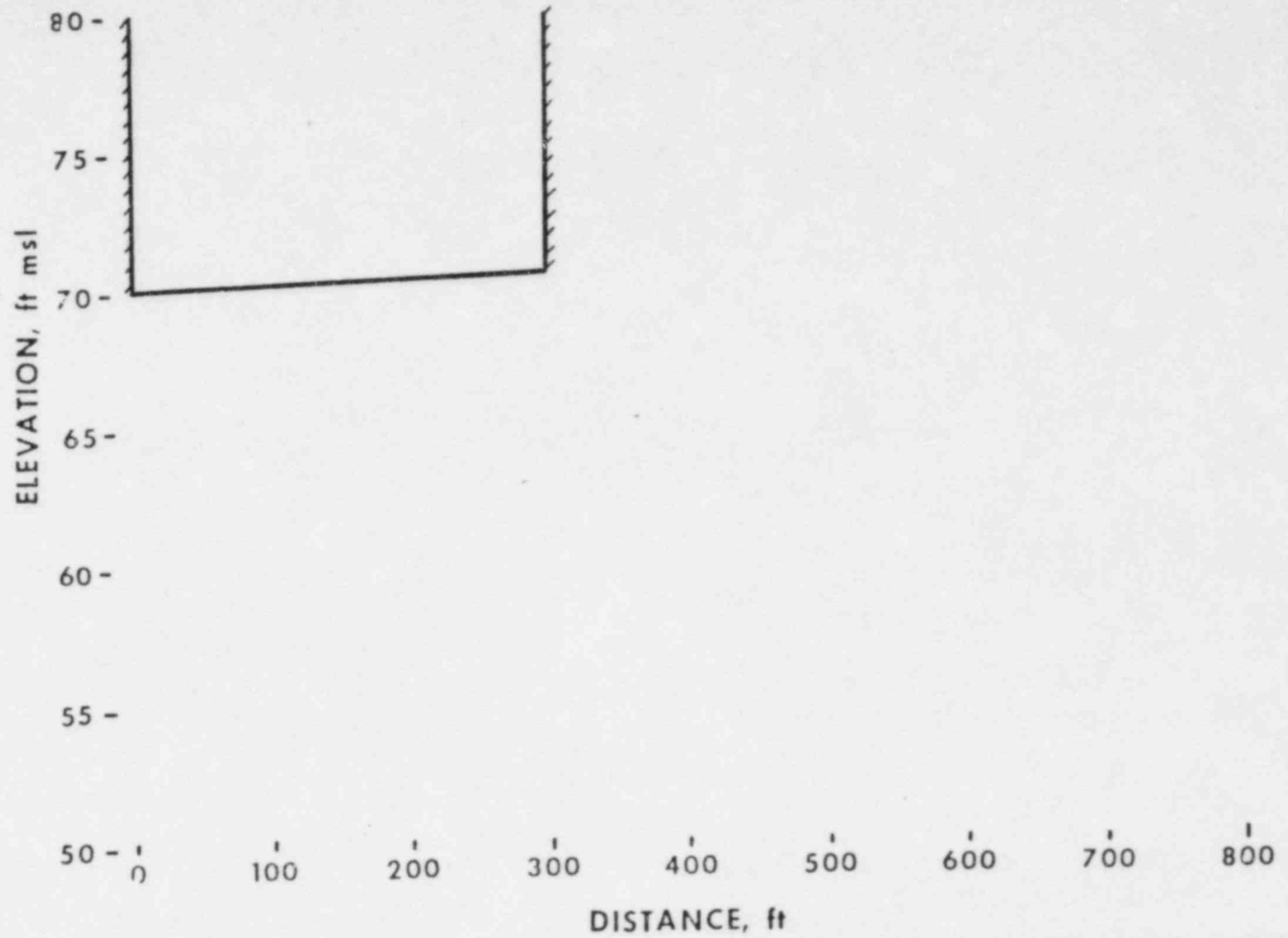


NOTES:
LOOKING DOWNSTREAM.
SEE SECTION LOCATION ON FIGURE 2.4-21
---- ASSUMED SSE PROFILE

FIGURE 2.4-24

SIMULATED WEIR FOR BRIDGE AT CROSS SECTION 6 ON GRANTS BAYOU,
SSE CONDITION

RIVER BEND STATION
FINAL SAFETY ANALYSIS REPORT

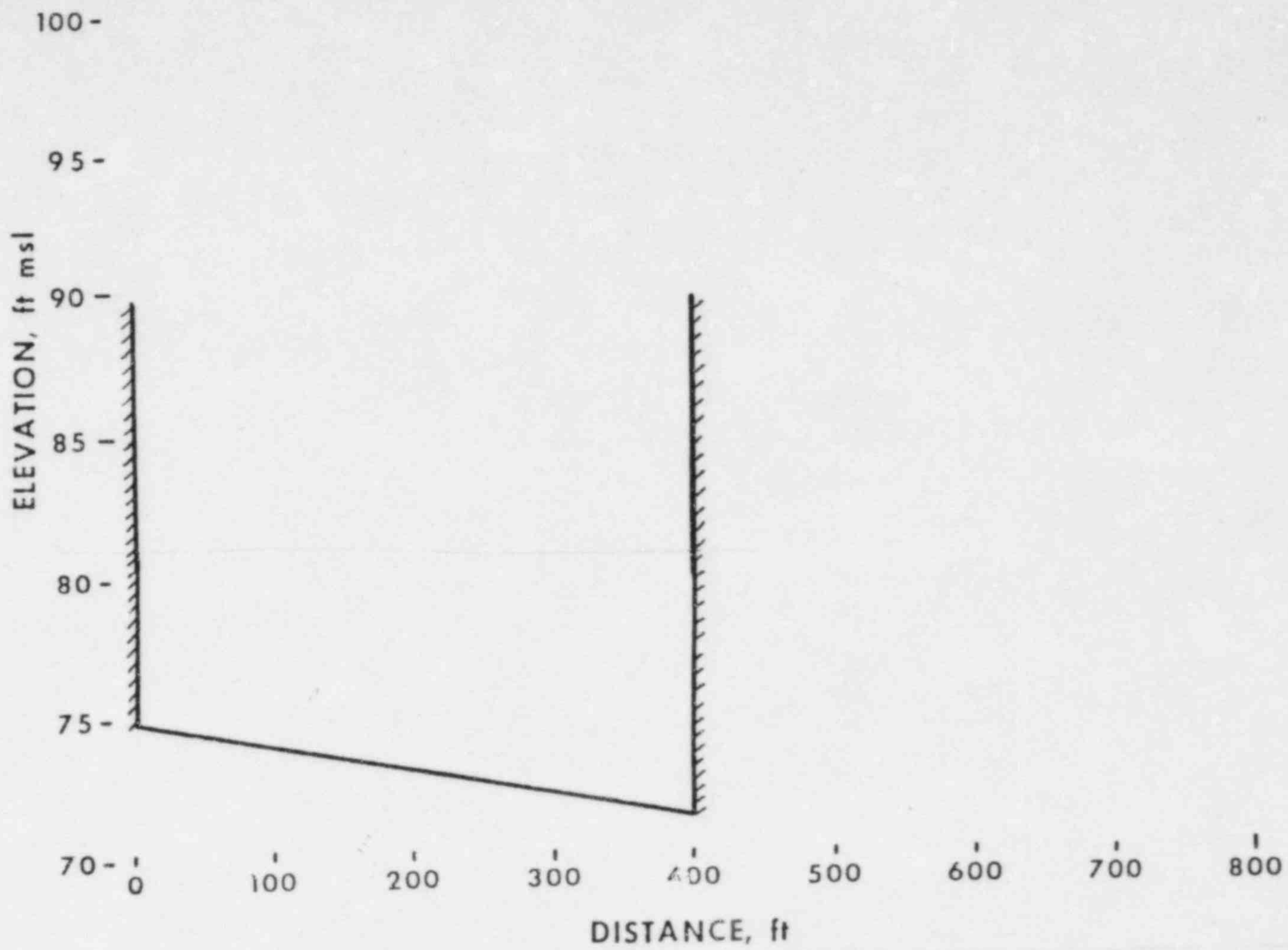


NOTES:
LOOKING DOWNSTREAM.
SEE SECTION LOCATION ON FIGURE 2.4-21

FIGURE 2.4-25

SIMULATED WEIR FOR BRIDGE AT CROSS SECTION 7 ON GRANTS BAYOU, SSE CONDITION

RIVER BEND STATION
FINAL SAFETY ANALYSIS REPORT

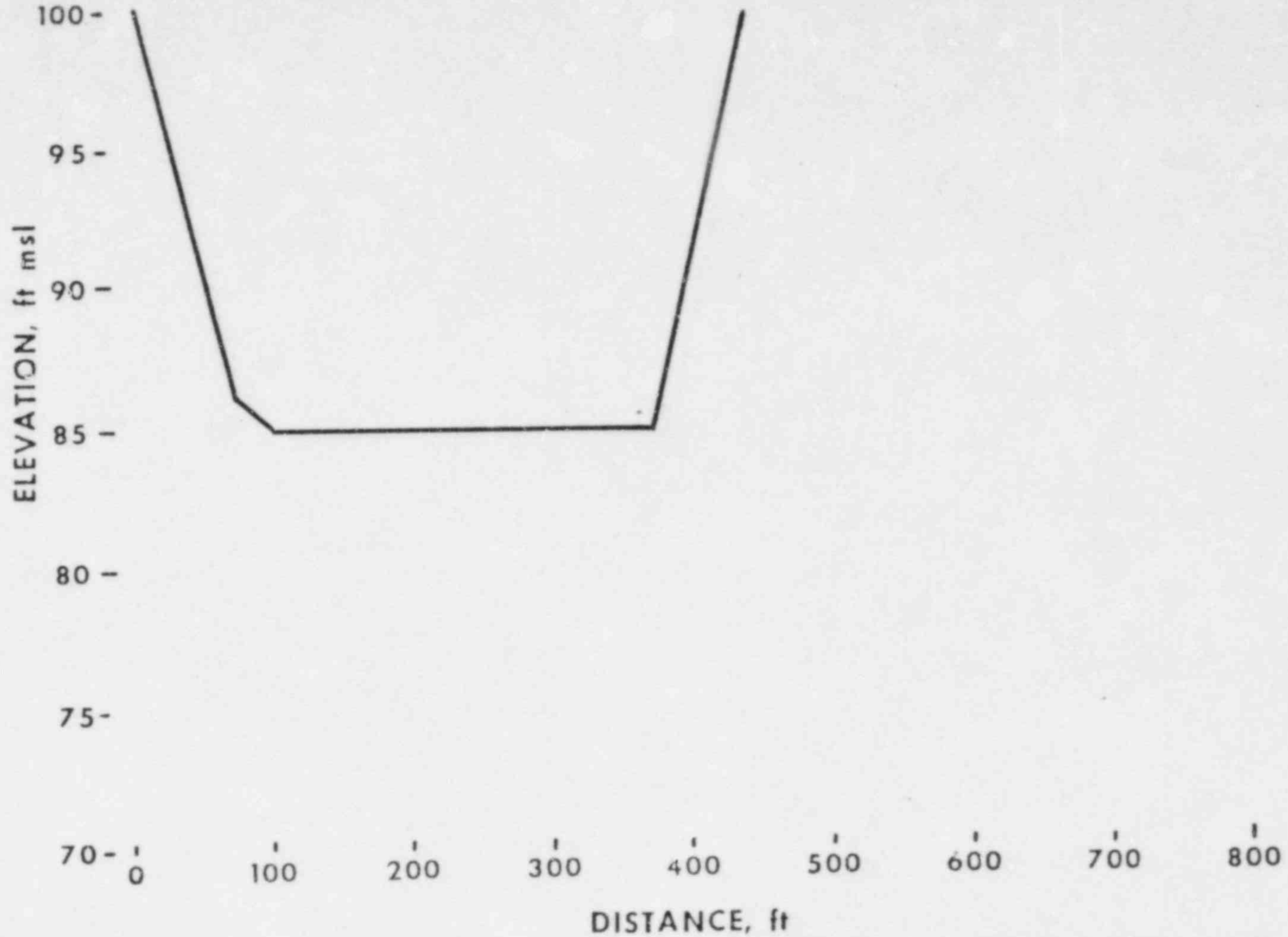


NOTES:
LOOKING DOWNSTREAM.
SEE SECTION LOCATION ON FIGURE 2.4-21

FIGURE 2.4-26

SIMULATED WEIR FOR BRIDGE AT CROSS SECTION W1 ON WEST CREEK, SSE CONDITION

RIVER BEND STATION
FINAL SAFETY ANALYSIS REPORT

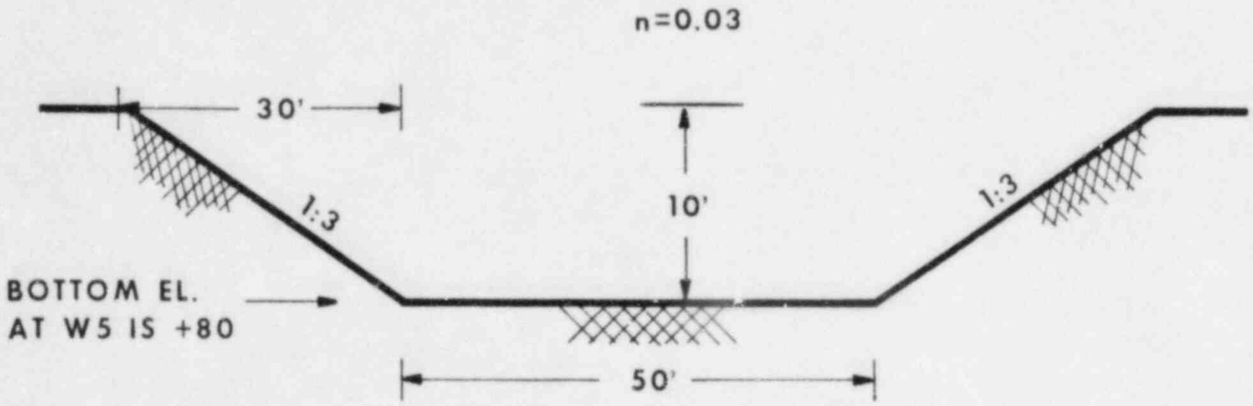


NOTES:
LOOKING DOWNSTREAM.
SEE SECTION LOCATION ON FIGURE 2.4-21

FIGURE 2.4-27

SIMULATED WEIR FOR BRIDGE AT CROSS SECTION W2 ON WEST CREEK, SSE CONDITION

RIVER BEND STATION
FINAL SAFETY ANALYSIS REPORT



CROSS SECTION	BOTTOM ELEVATION (ft msl)
W5	80
W6	81
W7	82
W8	83
W9	84

FIGURE 2.4-28

FABRIFORM CHANNEL CROSS SECTION

RIVER BEND STATION

FINAL SAFETY ANALYSIS REPORT

QUESTION 240.7 (2.4.2.2)

At the construction permit stage the results of the analysis of flood conditions on West Creek indicated the peak water level could reach an elevation of 98.49 ft MSL. It is stated in Section 2.4.2.2 of the FSAR that the maximum water elevation that would be produced is 95.1 ft MSL. Also, the SER for the construction permit stage discussed a PMF peak discharge of 5460 cfs for West Creek. Section 2.4.3.4.2 of the FSAR indicates a West Creek PMF of 4000 cfs. Provide a detailed discussion of the reasons for the modifications in water surface elevation and PMF discharge estimates. The data to be provided in your discussion should include channel cross sections, assumptions, and calculations to allow for an independent staff evaluation.

RESPONSE

The difference of PMF flow in the PSAR and FSAR is due to a change in calculation methodology. The modifications in water surface elevation in the PSAR and FSAR are due to the change in PMF estimates.

In the PMF analysis at the construction permit stage, the West Creek basin was divided into the upper and lower basins with the north plant road as the dividing line. The upper basin was treated as one basin and the unit hydrograph derived from Hudlow's Average dimensionless hydrograph was used to estimate the PMF. The lower basin was further subdivided into 18 areas. The PMF in each subarea was analyzed using the rational method. The peak PMF flow of the entire drainage basin was then determined by combining all PMF peak flows.

The methodology used in the PSAR to calculate the PMF in West Creek basin was an ultra-conservative approach. In an effort to establish a more reasonable methodology, the entire West Creek basin was treated as one basin in the FSAR and the same Hudlow's method was used to estimate PMF. This modification greatly reduced the peak PMF flow at the site. The approach of determining the surface water elevations is the same in both the PSAR and FSAR.

INSERT 1

INSERT 2

The assumptions and calculations of the latest PMF are discussed in Section 2.4.3. The locations of the cross sections used in the determination of the surface water elevations are shown in Figure 2.4-21. The detailed data of these channel cross sections are presented in Tables 2.4-24 and 2.4-25 and Figures 2.4-22 through 2.4-28.

INSERT 1 for Question 240.7

and the PMF was estimated using an average unit hydrograph developed as discussed in Revised Section 2.4.3.3.

INSERT 2 for Question 240.7

was updated to reflect the Army Corps of Engineers' current computer model as discussed in Revised Section 2.4.3.5.2.

QUESTION 240.10 (2.4.2.3)

The characteristics of the site drainage network is highly dependent on grading and railroad track rail elevations. Provide detailed drawings of drainage areas that includes ponding locations, direction of flow, and a profile of the top of railroad track steel rails.

RESPONSE

The response to this request is provided in revised Section 2.4.2.3 and $\boxed{\text{new Fig. 2.4-6a.}}$

revised Figure 2.4-6.