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REELFOOT LAKE
TENNESSEE AND KENTUCKY

VOLUME 3

APPENDIX B:

SECTION 1: HYDRAULICS AND HYDROLOGIC

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APPENDIX B

SECTION 1:
HYDRAULICS AND HYDROLOGIC ANALYSIS

Section I
Hydraulic and Hydrologic
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Reelfoot Lake Feasibility Report

TOPIC A-INTRODUCTION

This section presents climatalogical, hydrologic and hydraulic data and assumptions applicable to the development of water resource alternatives for the Reelfoot Lake Drainage Basin.

The technical activities included in this appendix are related to the development and feasibility of improvements or modifications to existing features in the vicinity of Reelfoot Lake. The primary areas of concern were related to flood control, sediment control, water quality, water supply, fish and wildlife preservation and enhancement, recreation, regional development and allied purposes.

Investigations related to alternatives which addressed the above mentioned areas were conducted during the period 1985 to 1988. The results of these investigations were presented in a report entitled, "Reelfoot Lake, Tennessee and Kentucky, Reconnaissance Report," dated May 1988. A subsequent investigation, initiated in November 1992, resulted in a report entitled "Reelfoot Lake, Tennessee and Kentucky, Reconnaissance Report" dated November 1993. That investigation is the foundation for the present feasibility report. This report represents an update of the previous investigations along with additional features to further meet study objectives.

TOPIC B-REELFOOT LAKE ALTERNATE SPILLWAY

1-01. Description of Drainage Basin

The Reelfoot Lake Drainage Basin covers approximately 240 square miles in northwestern Tennessee, Lake and Obion Counties. A relatively small portion of the northern end of the lake lies in Fulton County, KY. The main body of the lake extends southward from Kentucky and is approximately 4 1/2 miles at its greatest width. The lake has a mean depth of 5.2 feet, a surface area of approximately 24.2 square miles, and a volume of approximately 80,300 acre-ft at a normal pool elevation of 282.2 feet National Geodetic Vertical Datum (NGVD). Reelfoot Lake lies within the Mississippi Embayment Section of the Gulf Coastal Plain. The Reelfoot Lake Basin is characterized by several physiographic features including the Mississippi River and floodplain, Tiptonville Dome, a bluff line which bisects the basin along a northeast-southwest axis, and the uplands east of the bluffs.

Reelfoot Lake has three major tributaries, Reelfoot Creek, Indian Creek, and Bayou du Chien (Running Slough); and one major outlet, Running Reelfoot Bayou. Surface outflow from Reelfoot Lake is regulated by a low-head spillway with gates (including a radial arm gate), with the discharge being a function of lake elevation and tailwater depth. The lowest the lake could be drawn with the present spillway structure is elevation 276.4 feet NGVD which would leave a surface area of approximately 8.3 square miles, a volume of approximately 19,700 acre-ft, and a mean depth of approximately 3.7 feet. Location maps of the Reelfoot Lake area are shown on Plates 1 and 2.

1-02. Climatology

a. <u>Climate</u>. The climate of the area may generally be classified as mild. The National Weather Service records at Union City and Samburg Wildlife Refuge, Tennessee, reveal that average monthly temperatures in the area range from 34 degrees Fahrenheit in January to 78 degrees Fahrenheit in July. The maximum observed temperature was 105 degrees Fahrenheit and the minimum was minus 19 degrees Fahrenheit.

b. <u>Precipitation</u>. Annual precipitation varies from 28 to 77 inches with a normal over the area of about 51 inches. The heaviest rainfall generally occurs in the period from March through May.

c. <u>Storms</u>. Selected major storms recorded at the Samburg Wildlife Refuge, Tennessee, station from the period 1956-1996 are shown below:

TABLE 1

SELECTED STORMS

MAXIMUM ACCUMULATED RAINFALL (inches)

Month	Year	1-Day	2-Day	3-Day	4-Day	5-Day	6-Day	7-Day
May	1957	3.75	5.45	5.50	5.50	8.19	8.24	8.24
Nov	1957	4.40	4.75	4.75	4.75	5.88	8.73	9.08
Jul	1972	5.12	7.52	7.52	7.52	7.52	7.52	7.52
Apr	1973	3.54	4.04	5.18	6.37	6.87	8.01	8.27
May	1973	4.41	4.70	4.71	4.71	4.71	4.81	6.16

TABLE 1 (con't)

Nov	1973	4.68	5.42	5.58	6.48	7.93	8.56	8.56
Mar	1975	4.68	6.34	6.54	6.68	6.68	6.95	7.09
Dec	1978	4.13	4.94	5.48	5.48	6.72	9.77	10.31
Apr	1979	2.90	4.90	4.90	5.60	5.60	5.60	5.60
Dec	1982	3.20	4.98	5.47	5.47	5.47	5.47	5.47
May	1983	5.00	5.21	5.21	6.96	6.96	6.96	9.56
Oct	1984	4.25	4.25	4.25	4.39	4.44	4.87	5.20
Nov	1988	0.50	1.00	1.00	5.80	8.10	8.20	8.20
Feb	1989	3.80	5.00	8.10	8.10	8.10	8.10	8.10
Dec	1990	0.70	0.90	3.10	4.40	4.40	4.40	9.10
Jul	1996	0.10	0.10	1.30	3.30	6.20	6.20	6.20

d. Winds and Growing Season. The prevailing winds are from the southwest. The growing season has a length of approximately 7 months, from May through November.

1-03. Stream Flow Data

The United States Geological Survey has published records on Reelfoot Lake near Tiptonville, Tennessee; Running Reelfoot Bayou near Owl City, Tennessee; North Reelfoot Creek at State Highway 22 near Clayton, Tennessee; South Reelfoot Creek near Clayton, Tennessee; and Running Slough near Ledford, Kentucky. Periods of record for all the above stations, except Reelfoot Lake near Tiptonville, Tennessee, are less than 10 years. There are 47 years of record for Reelfoot Lake stages.

The highest elevation recorded on the Reelfoot Lake Gage was 285.94 feet NGVD on 31 May 1957, and the lowest was 279.76 feet NGVD on 20-21 November 1953. The maximum and minimum annual stages at the Reelfoot Lake gage from 1953 to 1997 are shown in Table 2.

Maximum recorded discharge and stage for Running Reelfoot Bayou near Owl City, Tennessee, are as follows: 2400 cfs at a stage (elevation) of 18.25 ft (283.21 NGVD) on 21 Feb 1989.

TABLE 2

MAXIMUM AND MINIMUM OBSERVED ANNUAL STAGES - REELFOOT LAKE

Year	Maximum	Minimum
1953	283.60	279.76
54	282.57	280.13
55	283.03	280.83
56	284.22	280.84
57	285.94	280.84
58	285.10	281.68

TABLE 2 (con't)

59	285.02	281.90
60	282.92	281.14
61	282.90	281.30
62	283.54	281.66
63	282.78	280.72
64	283.40	280.86
65	283.44	281.46
66	284.46	281.44
67	283.36	281.66
68	283.06	281.40
69	283.42	281.14
70	283.30	281.96
71	283.30	282.10
72	283.76	281.86
73	285.56*	281.56
74	283.55	282.01
75	285.16	282.04
76	283.16	281.56
77	282.75	281.51
78	**	281.46
79	284.62	281.38
80	282.96	280.76
81	283.06	280.64
82	282.78	281.27
83	284.74	280.79
84	283.74	280.38
85	283.79	279.81***
86	283.33	280.82
87	283.06	280.65
88	283.44	**
89	285.06	**
90	284.28	281.15
91	284.11	281.35
92	283.17	281.70
93	283.69	281.80
94	283.40	281.46
95	283.28	281.81
96	283.39	282.14
97	284.27	281.97

* Highest Recorded for year; no record 27 Apr - 10 May.
** Record incomplete for maximum stage.
*** Occurred during attempted TWRA drawdown.

Note: Stages in feet NGVD.

1-04. Methodology

During the development of the 1988 and 1993 Reconnaissance Reports, Hydrologic and Hydraulic models were developed for various features under investigation. The current updated efforts utilized these models along with models developed for the additional features investigated. The following paragraphs present the methodology, assumptions and design criteria for each feature investigated.

1-05. Existing Conditions-Reelfoot Lake, Spillway & Outlet

a. HEC-1 Flood Hydrograph Package. The development of the lake stage frequency and spillway discharge frequency curves was accomplished by utilizing the HEC-1 Flood Hydrograph Package. The drainage area was divided into 113 subareas. A synthetic unit hydrograph was developed for each subarea. Hypothetical rainfall was applied to the synthetic unit hydrograph to develop a flood hydrograph for the subarea. In order to develop composite hydrographs at all pertinent points, the flood hydrographs were combined and routed using the modified Puls method of stream flow routing.

b. Rainfall. Rainfall data from the NWS Technical Papers No. 40 and No. 49 were used in the development of the synthetic storms. These data were adjusted from partial duration series to annual series. The rainfall was divided into 2-hour intervals to provide adequate definition of the runoff hydrographs for individual subareas. A total duration of ten days was chosen for the hypothetical storms because of the basin's long time of concentration. The HEC-1 model was executed using the precipitation depth-area relationship simulation, which interpolates a hydrograph at each computation point based on a rainfall depth consistent with the drainage area at that point.

c. Unit Hydrograph Parameters. The study area was divided into 113 subareas with an average subarea size of 1.65 square miles. Synthetic unit hydrograph parameters were developed for each subarea using Snyder's Unit hydrograph relationships. Coefficients required in Snyder's relations varied depending on individual drainage basin characteristics. The criteria used to select C_t and C_p values were developed during a previous analysis of several gaged basins in the Memphis District. A relationship was developed between C_t and weighted stream slope and is shown on Plate 3. Snyder's lag time, t_p, was calculated for each subarea from measured values of L and L_{ca} based on weighted stream slope. Values of C_p were found to vary from 0.5 for flat land to 0.65 for hilly land in the gaged basins. Subareas which contained urban development were adjusted to reflect the accelerated runoff associated with urban areas. The percent of urbanization was obtained by measuring the urban area and then adjusting the unit hydrograph parameter t_p by using the following relation:

$$t_{pu} = \frac{t_p}{1 + \frac{\%urban}{100}}$$

This relation is based on the assumption that maximum urbanization will result in a 50% reduction in the time to peak of a subarea hydrograph. A summary of the unit hydrograph parameters is shown on Plates 4 through 7. The calculated values of t_p and selected values of C_p for each subarea were used in the HEC-1 model.

d. Hydrograph Construction for the Lake. A hydrograph for the lake was input directly into the model by converting point rainfall to discharge. A partial-to-annual series conversion was made using partial duration to equivalent annual series conversion factors given in TP-40. This point rainfall was then adjusted to the area of the drainage basin using the following equation:

FACTOR = 1-BV*(1- EXP(-.015*AREA))

where FACTOR is the coefficient to adjust point rainfall, BV is the maximum reduction of point rainfall (from Table 3.4 of the HEC-1 Flood Hydrograph Package Users Manual), and AREA is the drainage area in square miles. Incremental precipitation was computed and rearranged to develop the hydrograph. The rainfall depths were converted to discharge based on a lake area of 23.2 square miles and this discharge hydrograph was input directly into the model. Zero losses were assumed for the lake.

e. Spillway Discharge. The existing spillway configuration for Reelfoot Lake is shown on Plate 8. The following equation for computing discharge over a weir was used to quantify spillway outflow for various water surface elevations:

$$Q = CLH^{3/2}$$

where

Q = discharge in cfs
C = discharge coefficient (2.63)
L = length of weir in feet
H = head over weir in feet

The flow through the radial arm gate was computed separately using the above equation and then added to the spillway outflow for a total outflow. When operated the radial arm gate is fully opened and flow conditions reflect those of a broad crested weir. Stage (elevation) and discharge frequency curves for the existing spillway for various starting pool levels are shown on Plates 9 and 10 respectively. Existing storage relationship is shown on Plate 11.

f. Calibration. Because of the short period of record for the streamflow stations within the Reelfoot Lake Basin, the HEC-1 model was calibrated by comparing predicted lake stages with observed lake stages. A period of 41 years (1953-1993) was used in a frequency analysis of observed maximum annual lake stages. The observed lake elevations versus probability of occurrence are plotted in the curve shown on Plate 12. Loss rates and the lake stage-outflow relationship were adjusted to achieve calibration. The predicted lake elevations show close correlation with the observed frequency analysis (Plate 12). Initial loss rates varied from 0 to 1.0 inch and uniform loss rates varied from 0 to 0.1 in/hr.

g. Reelfoot Lake Outlet. Outflows from Reelfoot Lake are conveyed by Running Reelfoot Bayou (RRB). Existing condition flowlines were developed for RRB utilizing the HEC-2 Water Surface Profile program. Geometric input data was based on surveyed channel cross sections which were extended into the overbank using 1:24000 quadrangle maps of the area. Channel and overbank roughness values were based on visual inspection of the channel and overbank areas. The roughness values selected for RRB ranged from 0.045 to 0.075 for the channel and 0.07 to 0.125 for overbank areas.

Starting water surface elevations for the flowline computations were based on hydraulic data developed on the Obion River for General Design Memorandum Supplement for the West Tennessee Tributaries Project (July 1983). Flows for the HEC-2 model were based on spillway outflows computed by the HEC-1 model along with the intervening drainage area runoff between the spillway and the mouth of RRB. The intervening drainage runoff was computed using regression equations developed by the USGS for the West Tennessee area. This method of developing channel discharge results in the most conservative estimate of discharge downstream of the lake since the peak discharges computed for the intervening areas were added to the peak outflows from the spillway. The backwater model was not calibrated due to insufficient gage data for RRB.

h. Future Conditions. Changes in land use and urbanization were determined to be hydrologically insignificant over the project life. Therefore, future conditions with and without project were assumed to be identical to existing conditions with and without project.

1-06. Alternate Spillway Design Criteria

The design of the existing spillway is such that a very narrow range of acceptable water level management options is available with the structure. This prevents adjustment of the spillway operation to offset the slowly increasing flood damages resulting from the deposition of sediment in the flood storage zone of the lake. In addition to the design limitations of the structure, the normal aging process in combination with underseepage has resulted in differential settlement, cracking and leakage of the structure. This deterioration is jeopardizing the structure's existing water level management capability.

a. Previous Studies. The alternate spillway design considered in the 1988 Reconnaissance Study was to be located east of the existing structure. This structure was to consist of four 11-foot by 40-foot drum gates, 8000 feet of outlet channel to connect the spillway to Running Reelfoot Bayou, and vegetative clearing with selective cleanout on Running Reelfoot Bayou to the Obion River. The 1993 Reconnaissance Study consisted of a structure with 8 tainter gates which were 20 feet in width. The existing structure was to be reinforced and used as an overflow spillway. The proposed structure was east of the existing structure with approximately 6000 feet of outlet channel.

b. Current Study. As part of the current update of the Reelfoot Feasibility Study, the design of the alternate spillway structure was revised. The revisions included the use of sluice gates in lieu of the previously recommended drum and tainter gates. Sluice gates were recommended over tainter gates since they were more economical and required less maintenance. The new structure will provide the capability for a program of dynamic water level fluctuation combined with periodic major drawdowns.

The feasibility level requires an optimization process be conducted in order to select the location of the alternate spillway site. Several locations were examined and the selected location of the structure was in an area west of the existing spillway. This new location was chosen after a constructibility and/or environmental problem was cited for the area in the previous reconnaissance reports. Since the recommended structure will possess the capability to pass similar flows, the existing structure will be filled in and abandoned as an outlet for Reelfoot Lake. The final criteria for design of the structure is as follows:

(1) Maintain normal pool elevation 282.2 NGVD.

(2) Provide seasonal fluctuation capabilities between

elevation 284 NGVD and elevation 280 NGVD.

(3) Conduct periodic major drawdown (every 5-10 years) to elevations ranging from 274.2 NGVD to 278.0 NGVD. Drawdown depths will be based on TWRA desired surface water area.

(4) Drawdown to occur in a 30 to 45 day period during the time frame June 1 - July 15, followed by at least 120 days of drying with refilling to start by 15 November.

(5) Following a drawdown, the lake will be gradually refilled and held at an elevation of 283.2 NGVD until June 1 of the following year.

c. Structure size. Three different structural sizes were examined in an effort to optimize. These sizes consisted of structures with clear openings of 120, 140 and 160 feet. The individual sluice gate dimension for each bay remained 12 foot tall by 20 foot wide for each alternative. The recommended plan consists of the 120-foot structure. Plates 13 and 14 show the location deemed the most feasible location for the new structure.

1-07. Alternate Spillway Hydraulic Analysis

The previous reconnaissance structure was used for the preliminary design in an effort to meet the required lake drawdown and to comply with hydraulic design criteria. This structure consisted of eight 20-foot vertical lift gates, seven 4-foot wide piers, and two 4-foot wide spillway abutments. Final details of structural design were conducted by Memphis District Corps of Engineers Structural Engineering Section. Based on the drawdown criteria, the discharges are limited to approximately 1,200 cfs, which is bankfull capacity for Running Reelfoot Bayou. A rating curve was developed to show a relationship between discharge and tailwater elevation. This rating curve and the target discharge are shown on Plate 15. At this discharge, the lake level would be drawn down to the spillway crest elevation at 274.2 NGVD within 45 days and the tailwater elevation would not exceed bank elevation at 280.0 NGVD.

Additional alternatives of six 20-foot gates and seven 20-foot gates were then analyzed to optimize the drawdown capability with the number of gates. As stated earlier, the recommended spillway was the six gated structure and is shown on Plates 16 and 17.

Additional runoff such as base flow or excess rainfall was not included in the necessary volume to evacuate. The results of the drawdown routings for the 120-foot structure are shown on Plate 18.

1-08. Spillway Flow Conditions During Drawdown

The new spillway is considered to be a low head spillway with a net head at normal pool of only 7.8 feet. Due to the small differential in head, the tailwater conditions will affect lake releases. Gate opening sequences were established to simulate the lake drawdown with consideration to downstream conditions. The number of gates and gate openings were correlated to maintain bankfull discharges on Running Reelfoot Bayou or approximately 1,200 cfs. Weir and orifice flow conditions, which were affected by outlet channel depths, were corrected for submergence. The initial discharges were computed using free orifice and submerged orifice equations as presented in EM-1110-2-1603 "Hydraulic Design of Spillways" and EM-1110-2-1605 "Design of Navigation Dams".

$$Q = CA\ (2gh)^{1/2}$$

where

Q = discharge in cfs
C = discharge coefficient
A = area of gate opening in square feet
h = available head at the gate in feet
g = acceleration of gravity in ft/sec/sec

Weir flow conditions over the ogee crest spillway were evaluated by the following relationship:

$$Q = CLH^{3/2}$$

where

Q = discharge in cfs
C = discharge coefficient
L = length of weir in feet
H = head over weir in feet

The submerged weir equation was not used until all gates were out of the water. The gates were kept in the water to maintain the target discharge until the pool elevation equaled the tailwater elevation, at which time all gates would be lifted completely out of the water. At this point the flow is submerged weir flow. The gate opening sequences and subsequent discharge through six 20-foot vertical lift gates for the drawdown of Reelfoot Lake can be found on Plate 19.

The spillway configuration will include abutments of some type, and include intermediate piers dependent on the number of gates/bay. The abutment and number of piers reduces the length of the spillway as computed using the equation below from EM 1110-2-1603:

$$Le = L - 2 (n Kp + Ka) He$$

where

Le = effective crest length in feet
L = net length of crest, gross length - width of piers in feet
n = number of piers
Kp = pier contraction coefficient
Ka = abutment contraction coefficient
He = total specific energy above the crest in feet

The discharge coefficient of the free orifice condition was estimated at 0.7. It was derived by using methods described in the Hydraulic Design Criteria (HDC), Sheet 320-1. Coefficients of discharge for the submerged orifice flow conditions were computed by using an equation from the HDC 512-1. The discharge coefficient of the submerged weir flow was adjusted using a relationship developed in "Handbook of Hydraulics" by King and Brater.

1-09. Spillway Crest Shape

The shape of the crest was designed based on hydraulic design criteria in Engineering Manual (EM) 1110-2-1605 'Hydraulic Design of Navigation Dams". For a net head of less than 20 feet, the minimum radius connecting the upstream face with the horizontal portion of the broad-crested weir is 3 feet. According to the engineering manual, the length of the horizontal crest from the upstream face to the beginning of downstream face is 110 percent (%) of the net head on the crest. Since the net head is 7.8 feet, assuming a crest elevation of 274.2 NGVD, the width of the broad-crested weir was approximated at 8.5 feet. Additional width was added to provide sufficient spacing for stoplogs. The total width of the broad-crested weir was estimated at 19 feet. The downstream face of the spillway will follow a trajectory governed by $X^2=40Y$. The variable x is measured horizontally to the right and the variable y is measured vertically downward from the spillway crest. This shape was chosen to minimize the negative pressures against the downstream face of the spillway.

1-10. Stilling Basin

The entire range of gate operations was explored to determine the length of basin necessary to dissipate energy. The final stilling basin was sized to control the discharge for the worst case scenario, which are the gates open to one foot, with a resulting velocity of 14.9 fps. As the tailwater conditions increase, this results in a reduction in head, spillway flows, and velocities.

Froude numbers for the discharges for this structure vary from a value of F=1.5 for the 100 year flow to a maximum of F=5.13 for the initial gate opening of 1 foot. The energy dissipater was designed for a Froude number ranging from 2.5 to 4.5 and was checked for flows with Froude numbers in the range of 1.5 to 2.5. Further examination revealed the tailwater rating curve is always higher than the hydraulic jump rating curve (theoretical D2) so the jump will be forced upstream and drowned out, becoming a submerged jump.

The USBR STILLING BASIN IV (U. S. Bureau of Reclamation Engineering Nomograph No. 25) was used, without the chute blocks. The high tailwater conditions below this structure, in excess of twice the depth of the jump, will sufficiently drown out the jump and eliminate the need for floor blocks. The maximum basin length needed to contain the jump will be 28 feet.

1-11. Inlet Channel Design

While the desired drawdown elevation is 274.2 NGVD, it was requested to also investigate crest elevations of 273.0 NGVD, 272.0 NGVD and 271.0 NGVD in an effort to optimize the structure using the smallest spillway possible. This required investigating four different channels, one for each sill elevation. Upon further investigation it was determined there was no advantage to lowering the sill crest elevation, so additional inlet channel alternatives were not investigated. An approach depth of five feet was recommended for the inlet channel, so a 140-foot bottom width channel at an elevation of 269.2 NGVD was added to the HEC-2 model to check for erosive velocities and the need for riprap. The model showed no velocities in excess of 2 fps so riprap would not be needed in the inlet channel except in the vicinity of the structure.

1-12. Outlet Channel Design

An HEC-2 model was prepared for the new channel starting at mile 17.8 of Running Reelfoot Bayou. A 140-foot bottom width channel with 3H to 1V side slopes was used to match the 140 feet total structural width. The water surface elevations at mile

19.124 (the downstream end of the sill), were used as the tailwater conditions for the spillway design. Although channel velocities were minor (less than 2 FPS), turbulent conditions downstream of the structure required a 100 foot riprap blanket. Plates 20 and 21 show a layout of downstream channel protection.

1-13. Induced Damages

Although the structure's main purpose is to drawdown lake levels in a specified time period, a hydraulic analysis was conducted to ensure the proposed structure possessed the capability to evacuate frequency storm events of the same magnitude as the existing stoplog structure. With the correct operation, the structure will not induce damages upstream around the lake or downstream in Running Reelfoot Bayou over damages presently experienced. The downstream base condition was established by developing an elevation versus discharge relationship on Running Reelfoot Bayou using HEC-2. This relationship determined necessary gate openings for a given pool level to maintain acceptable releases by the structure.

As shown on Plate 10, the 500 year storm event on a normal pool elevation of approximately elevation 282.2 NGVD produced a discharge of 4900 cfs. The analysis revealed six gates opened 9.0 feet will maintain this discharge. The frequency storms (1.05- to 500-year) were routed through Reelfoot Lake and the proposed 120 foot structure for pool levels ranging from 282 NGVD to 284 NGVD (reference Plates 9 and 10). The 500 year flood was also selected as the Inflow Design Flood (IDF). The IDF governs final structural dimensions and is discussed later in paragraph 1-23.

Topic C-Lake Isom Refuge

1-14. Lake Isom Existing Conditions

The Lake Isom Refuge is located downstream of the Reelfoot Lake area as shown on Plates 22 and 23. It is formed by a low area west of Running Reelfoot Bayou (RRB), an earthen dam on the southern end and excavated material from the enlargement of RRB on the east. Pertinent existing data is shown Table 3.

Table 3

Lake Isom Existing Pertinent Data

Normal Pool Elevation: 280.0 NGVD
Max Stop Log Elevation: 280.4 NGVD
Outlet Structure Upstream Invert: 274.8 NGVD
Outlet Structure Downstream Invert: 272.4 NGVD
Outlet Structure Weir Length: 5.0 feet
Earthen Spillway Width: 90 feet
Earthen Spillway Elevation: 280.4 NGVD (low point)
Length of Dam: 1225 feet
Dam Elevation: 282.8 NGVD - 284 NGVD
Dam Crown Width: 10 feet
Surface Area at Normal Pool: 1100 acres
Storage at Normal Pool: 2750 ac-ft

An HEC-1 model of the watershed above the Lake Isom Dam embankment was developed. Frequency rainfall events were applied to the watershed and then routed through the lake area and embankment structures. The storage relationship is shown on Plate 24. Results of the routings for the 100 year flood are shown in Table 4.

Table 4

100-Year Existing Conditions

Peak Inflow: 2702 cfs
Peak Outflow (Total): 112 cfs
Peak Outflow Outlet Structure: 17 cfs
Peak Outflow Spillway: 95 cfs
Maximum Pool Elevation: 281.1 NGVD
Storage at Maximum Pool Elevation: 4204 ac-ft
Surface Area at Maximum Pool Elevation: 1225 acres
Free Board at Maximum Pool Elevation: 1.7 feet at low point

1-15. Water Management Plans (WMP)

a. Existing Water Management Plan. The existing Water Management Plan at the Lake Isom Refuge is as follows:

Table 5

Period	Lake Level (NGVD)
Oct 15 - Nov 1	279.0 to 280.5
Nov 1 - Mar 1	280.5

Table 5 (con't)

Mar 1 - Mar 15	280.5 to 279.0
Mar 15 - Oct 15	279.0

b. Preferred Water Management Plan. The U.S. Fish and Wildlife Service desires a revised water management plan. The revised plan referred to as the Preferred Water Management Plan (PWMP) would provide for lake levels as shown in Table 6:

Table 6

Period	Lake Levels (NGVD)
Oct 15 - Nov 1	279.0 to 282.0
Nov 1 - Mar 1	282.0
Mar 1 - Mar 15	282.0 to 279.0
Mar 15 - Oct 15	279.0

Also included in the Preferred Water Management Plan is the capability to draw the lake down to elevation 274.0 NGVD and pumping capability to raise the lake 3.0 feet in a period of 14 to 21 days.

c. Modifications to Achieve Preferred Water Management Plan. In order to achieve the lake level fluctuations outlined in the PWMP, it will be necessary to modify the existing dam embankment and spillway and provide additional outlet capacity. Due to the size, purpose, remote location and minimal consequences associated with overtopping or dam failure, less stringent criteria than normally used in the design of impoundment structures were used. The design criteria adopted required the spillway and outlet structure to have the capability to pass the 100-year exceedence frequency event with 2.0 feet of freeboard. The risks associated with the use of this criteria is considered acceptable due to the topography of the area and the absence of roads, houses or major physical improvements, which would be placed in a high degree of risk, if the dam overtopped or breached. Specifics of inflow design flood selection are discussed in paragraph 1-28. The resulting modifications necessary to achieve the PWMP objectives are shown in the following table and on Plate 25. Resulting frequency elevations on Lake Isom for both existing and preferred plans are shown on Plate 26.

Table 7

Preferred Water Management Plan Pertinent Data

Normal Pool Elevation: 282.0 NGVD

Table 7 (con't)

Max Stop Log Elevation: 282.0 NGVD
Outlet Structure Upstream Invert: 274.0 NGVD
Outlet Structure Weir Length: 15.0 (3 @ 5 feet)
Outlet Structure Length: 77 feet
Earthen Spillway Length 125 feet
Earthen Spillway Elevation: 282.5 NGVD
Dam Embankment Length: 1240 feet
Dam Elevation: 285.0 NGVD
Crown Width: 15 feet
Surface Area at Normal Pool: 1350 acres
Storage at Normal Pool: 5620 ac-ft

As shown on Plate 27, an outlet structure configuration similar to the existing structure will be used. Control of lake levels will be by the addition or removal of stop logs. Routings indicated that a 30-foot weir length was required to meet drawdown requirements, however, considering the cost and the fact that a 15-foot weir length generally meets these requirements, a 15-foot length of weir was selected for economy. Plates 28 and 29 show the riprap limits downstream of the stilling basin. Other pertinent data concerning the PWMP are shown in the following table:

Table 8

100-Year Preferred Management Plan

Peak Inflow: 2702 CFS
Peak Outflow (Total): 166 cfs
Peak Outflow Outlet Structure: 45 cfs
Peak Outflow Spillway: 121 cfs
Maximum Pool Elevation: 282.96 NGVD
Storage at Maximum Pool Elevation: 6996 ac-ft
Surface Area at Maximum Pool Elevation: 1460 acres
Free Board at Maximum Pool Elevation: 2.04 feet NGVD

Drawdown Time Required:
282 - 280 : 12 days
282 - 279 : 36 days
Exit Velocity at Peak Outflow: 6.6 fps
Peak Outflow: 234 cfs

d. Lake Isom Refill. A feature of the preferred water management plan was to refill the lake from elevation 279.0 to 282.0 in approximately 2 to 3 weeks. This requirement will be satisfied by the use of 6 groundwater pumps with each pump possessing a 2750 gallons per minute rate. Hypothetical rainfall amounts were not included in the refill analysis.

Topic D-Dredged Circulation Channels

Reelfoot Lake is composed of three major basins. The circulation of flow between these basins has degraded over a period of time. As part of this study, dredged circulation channels have been investigated. These channels connect the three major basins to each other and the proposed alternate spillway. These channels would provide improved circulation between major pools during normal and high lake levels and minimize isolation of major pools in extremely low stages. The proposed location of the dredged circulation channels is shown on Plate 30. These channels will have bottom widths of approximately 30 feet with depths of 6 feet and side slopes of 1 on 4. The bottom widths of these channels were based on past experience with previously constructed channels within the Lake. These channels generally had top widths of 75 to 80 feet.

Topic E-Reelfoot Creek Sediment Retention Basin

1-16. Hydrology

The proposed sediment retention basin is shown on Plate 31 and 32. The contributing drainage area of 112.1 square miles for Reelfoot Creek was determined from U.S.G.S. 7.5 minute topographic quadrangle maps of the area. The HEC-1 (flood hydrograph package) model developed during previous analysis was used to compute and route runoff hydrographs for the watershed. Modifications were made to this model to incorporate the sediment retention basin. Hypothetical rainfall with durations of 1 to 10 days was obtained from Technical Papers No. 40 and 49. The hypothetical storms were applied to synthetic unit hydrographs to produce runoff hydrographs for each subarea. The runoff hydrographs were then combined and routed using normal depth channel routing methods to produce composite hydrographs upstream of the sediment retention basin. All contributing areas upstream of the retention basin were combined and routed through the basin and basin outlet using the modified-puls method of storage routing. The storage relationship for the retention basin was developed from one-foot topographic maps. The HEC-1 model was used to analyze information necessary to determine the elevation of the dam crest and corresponding spillway requirements.

The HEC-1 model was calibrated to the Samburg, TN U.S.G.S stream gage using a period of record from 1951 to 1972. This time period was prior to the construction of 10 SCS reservoirs in the Reelfoot Creek drainage basin and is labeled 1975 conditions for this report. The existing conditions (1995) HEC-1 model incorporates the SCS flood retention structures. Design information for the flood retention structures is shown in Plate 33 and 34. The locations of the SCS retention structures are shown on Plate 35.

The 1975 Conditions (without SCS structures) HEC-1 model was changed from a "normal depth" routing model to a Modified Puls storage volume versus outflow routing model using the results from the HEC-2 storage models. Initial and uniform loss rates were adjusted until the frequency flows from HEC-1 were in agreement with the flows at the Reelfoot Creek near Samburg, TN stream gage. The best results were obtained using initial and constant loss rates of 0.5 inch and .05 inches per hour, respectively. A discharge-frequency curve showing the comparison is shown on Plate 36. The 1995 Existing Conditions (with SCS structures) HEC-1 model uses the same storage versus elevation relationship as the 1975 Conditions. Plates 37 through 41 show the difference between initial storage and storage after 50 years.

The HEC-1 model was used to check that the outlets met the following hydrologic criteria at the end of the 50-year period:

(1) Evacuate the 1.05-year flood within 10 days, excluding the capacity below the inlet elevation of the low level outlets.
(2) Pass the 5-year flood without overtopping the emergency spillway.
(3) Pass the 100-year flood below elevation 305.0 NGVD.

For HEC-1 analysis, the culvert riser elevation was set at elevation 301.0 NGVD, the estimated sediment pool elevations after 50 years, and no storage or riser orifice flow was included below this elevation. The combined rating curve for the low level outlet, primary spillway, and emergency spillway used in HEC-1 is shown on Plate 42 and 43 for initial conditions and after 50 years.

Final HEC-1 frequency curves, developed in this study for North and South Reelfoot Creek are shown on Plates 44 to 46 for the following cases:

(1) 1975 Conditions
(2) Existing Conditions (1995)
(3) With Sediment Retention Basin, Initial Conditions
(4) With Sediment Retention Basin, After 50-year Design Life

1-17. Hydraulics

The HEC-2 models were constructed using 1995 survey data, supplemented by 7.5 minute USGS quadrangles for elevations in the overbank areas. River miles were marked every mile with zero starting at the retention structure on Reelfoot Creek. South Reelfoot Creek was modeled as a tributary to North Reelfoot Creek, with mile zero at the confluence with North Reelfoot Creek (NRC mile 1.31). The starting conditions were established using the slope-area method. Bridges were modeled with the Special Bridge routine in HEC-2.

Cowan's procedure as outlined in Chow's "Open Channel Hydraulics," 1988 was used to determine Manning's n values. The channel n value was estimated to range from 0.055 to 0.063 for both North and South Reelfoot Creeks. Overbank n values for both creeks ranged from about 0.080 for cropland to about 0.105 for wooded areas.

HEC-2 stage-discharge results were compared to the stage-discharge data from stream flow gages on North Reelfoot Creek at State Highway 22 near Clayton, TN and South Reelfoot Creek near Clayton, TN for the period 1984 to 1989. Calibration was achieved by adjusting the n values. This was accomplished using a comparison of the overbank conditions to photographs with published Manning's n values in the document: "Guide for Selecting Manning's Roughness Coefficients for Natural Channels and Flood Plains," Federal Highway Administration Report No. FHWA-TS-84-204 April 1984.

The stage records for Reelfoot Lake near Tiptonville, TN were checked for backwater influences for the years 1984 to 1989. From examination of the data is was concluded Reelfoot Creek gage data were not affected by backwater from the lake during their period of record. Water surface elevation profile plots obtained using HEC-2 are shown on Plates 47 through 52.

1-18. Sediment Analysis

A sedimentation analysis was conducted for the design of a sediment retention structure for Reelfoot Creek. The design life of the retention structure is 50 years. The computer program HEC-6 was used to determine the amount of sediment volume that would accumulate in the retention basin during this time period by running a 50-year sediment routing simulation.

As stated earlier, the inlet of the low level outlets is to be set at the elevation of the sediment pool after 50 years. The low level outlets also contain openings for staged releases that can be closed off as sediment accumulates. A preliminary estimate of the elevation of the sediment pool after 50 years is needed initially in order to prepare rating curves for HEC-6. An analysis was conducted to confirm that elevation 301.0 NGVD was acceptable as an estimate of the sediment pool elevation after 50 years. This elevation was used for the inlet elevation of the low level outlets and for rating curves used in HEC-6 and HEC-1 to check the performance of the outlets against the hydrologic criteria. Final HEC-6 results presented below indicate an elevation of the sediment pool after 50 years of 299.2 feet. This is believed to be within tolerance to confirm that elevation 301.0 was acceptable as the initial estimate.

a. HEC-6 Model Development. The HEC-2 model was used to build the HEC-6 model for Reelfoot Creek. The HEC-6 model was initially set up as two separate models, one for North Reelfoot Creek (NRC) and one for South Reelfoot Creek (SRC). After the models were calibrated, they were combined into a single HEC-6 model with NRC as the main stem, SRC as a tributary, and local inflows on NRC at mile 6.79 and on SRC at mile 2.98. Minor adjustments were made at bridges, where the special bridge routines were replaced with a single cross-section at the upstream face. The geometry of this cross-section was modified for each bridge until the water surface for all cross-sections was within about 0.5 foot and the velocity for all cross-sections was within 0.5 foot/second of the HEC-2 results. Manning's n values were slightly adjusted to meet the conditions.

b. Boundary Conditions. At the downstream boundary of the model, the rating curve at the retention structure outlet works was used to start the backwater computations. For the SRC model, the HEC-2 rating curve for SRC mile 0.01 was used. Inflowing sediment loads were applied at the upstream boundaries of both NRC and SRC and at the local inflow location to each stream.

c. Inflowing Load Curve. Sediment data at the gages on North Reelfoot Creek at Highway 22 and South Reelfoot Creek near Clayton included five years of record covering water years 1985 to 1989. For this time period, all SCS structures upstream were constructed and online. The rainfall from the same five years was applied to the 1995 conditions unit hydrograph just downstream of upper Highway 22 (mile 3.59) in a period of record simulation program entitled HUXRAIN, and the results were input to the histogram generator. HUXRAIN was developed by the Memphis District Corps of Engineers. For the initial attempt at estimating the inflowing load, the inflowing load curve from the USGS water investigations report titled "Water Budget and Estimated Suspended-Sediment Inflow for Reelfoot Lake, Obion and Lake Counties, Northwestern Tennessee, May 1984-April 1985" was input in the histogram generator. This resulted in annual sediment yields much lower than those actually measured at the gage. Since the period was limited to 5 years, the inflowing sediment load curve was formed by positioning the curve within the scatter of the data covering the full five years of record. The load curve was then adjusted such that the total average volume of sediment recorded at the gage during the five year period was in agreement (within two percent) with the total average volume of sediment reported by the histogram generator at the same location. Plates 53 and 54 show inflowing sediment loads developed from the gage data, from Yang's and Ilo's formulas, and the curve adopted for use in HEC-6.

d. Gradation of Inflowing Load and Bed. Due to the sparse nature of the sediment data available for North and South Reelfoot Creeks, an assumption was made that 80% to 95% of the sediment being transported was silts and finer, and the rest (5% to 20%) were sands. These estimates were based on analysis of existing conditions sediment sample data, observations from the field, and general sediment characteristics of the region. For lower flows, a gradation with a higher percentage of silt content was used (95% silt and 5% sand), and for higher flows, a lower percentage of silt content was used (80% silt and 20% sand). Bed gradation data for North Reelfoot Creek was based on a boring sample provided at mile 5.42. Bed gradation data for South Reelfoot Creek was based on a boring sample at mile 3.38. The bed gradations used and interpolated in HEC-6 are shown on Plates 55 and 56 titled "Gradation vs. Distance". The Yang sediment transport function was used in the model.

e. Hydrology. A unit hydrograph was developed at mile 0.88 (Highway 22) using the 5-year frequency discharge hydrograph computed with the 1975 conditions HEC-1 model. A period of record rainfall database was developed for the years 1951 to 1972 and applied to the unit hydrograph at node 332 in the HUXRAIN program to optimize API coefficients in HUXRAIN. A comparison was made of frequency flows from HUXRAIN to the Reelfoot Creek near Samburg, TN stream flow gage data for the same time period (1951 to 1972). The comparison plot is shown on Plate 57.

A 50-year period of record rainfall database was prepared for 1944 to 1993. This database was used to simulate the 50-year future sediment routing hydrology and was applied to unit hydrographs at selected nodes with the HUXRAIN program.

The sediment-weighted histogram generator program was used to put the synthetic hydrology data in the proper format for HEC-6.

f. Channel Maintenance Dredging. Dredging records were used in HEC-6 in order to determine annual maintenance requirements for Reelfoot Creek in the vicinity of State Highway 22 (to maintain bridge conveyance) and for North Reelfoot Creek and South Reelfoot Creek in the fringe areas of the retention basin backwater area.

This dredged volume computed by HEC-6 was removed from the model although the dredged material will probably be spread out within the sediment basin and remain in the system.

g. Sediment Routing Results. Sediment retention performance for 50 years was evaluated for three alternative locations to achieve the plan formulation optimization requirements. The largest basin, Alternative 1, was determined to provide the optimal sediment trap efficiency and was used for finalization of spillway and outlet designs. From HEC-6 results, the elevation of the sediment pool in the basin at the end of 50 years was estimated to be elevation 301.0. This elevation is based on a low level outlets consisting of six 72-inch culverts, a primary spillway with width of 135 feet and crest elevation at 302.2 NGVD and an emergency spillway with width of 800 feet and crest elevation at 303.9 NGVD.

Plate 58 entitled "50-Year Sediment Routing Results" shows the 50 year sediment routing results (sediment volume inflow/outflow values and trapping efficiency). The value for total volume trapped is the sum of the difference of sand and silt inflow and outflow values plus the dredging result. The results indicate a 56% trapping efficiency for silts, 100% trapping efficiency for sands, a total volume trapped within the stream system of 3,064 acre-feet, and a total volume trapped below NRC river mile 1.31 of 1,740 acre-feet. The elevation-volume curve for the sediment retention structure extending up to river mile 1.31, yields a sediment elevation of 299.2 NGVD, confirming that the selection of initial estimate for the sediment pool elevation at the end of 50-years of 301.0 is acceptable. Dredging quantities were obtained from the HEC-6 output file and summed. The total dredging volume of 288 acre-feet over 50 years represents about 9% of the total volume trapped by the structure. Annual dredging requirements translate to 5.8 acre-feet per year in the channels.

Plates 59 and 60 show the change in average bed elevation for North and South Reelfoot Creeks between existing conditions and after 50 years. The bed elevations are adjusted to reflect the channel maintenance dredging required to maintain existing channel bed elevations between NRC river mile 0.89 to 5.42 and between SRC river mile 0.01 to 3.38. Also, the average bed elevations shown for NRC river mile 0.00 to 0.89 do not reflect the sediment deposition elevations in the overbanks within the sediment retention basin. They only represent the main channel elevation at a flow of 200 cfs.

1-19. Sediment Basin Spillway Analysis

a. <u>Low Level Outlet Design</u>. The elevation of the low level outlet inlet structure is set at the elevation of the sediment pool after 50 years (el. 301.0, NGVD). A sketch of the structure is shown on Plate 61. The low level outlet was designed to evacuate the 1-year exceedance flood in less than 10 days excluding the capacity of the outlets below the elevation of the 50-year sediment pool. Final design of stilling basins for the low level outlets was accomplished using the Waterway Experiment Station CORPSLIB program H2261 entitled "Stilling Basin Design for Conduit Outlet Works".

Table 9

<u>Low Level Outlet Pertinent Data</u>

Conduit diameter	6.0 ft
Conduit invert slope	0.01 ft/ft
Design discharge (Q_{500} for one pipe)	468 cfs
Outlet portal invert elevation	287.8 ft NGVD
% of d_2 to intersect the tailwater surface elevation(from EM-1603)	85%
Tailwater surface elevation	292.6 ft NGVD
Conduit roughness height	0.0000048 ft
Water temperature	60 degrees Fahrenheit

The design discharge represents the culvert flow through one pipe given a head on the retention structure for the water surface elevation of the 500-yr flood, equal to 305.5 NGVD. This discharge was determined using the program entitled CULVERW which was developed by the St. Louis District Corps of Engineers. CULVERW develops a headwater rating curve for both free and submerged conditions. Tailwater elevations were determined using the HEC-RAS program developed by the Hydrologic Engineering Center.

The conduit's roughness height was determined through an iterative procedure using Plate C-4 of EM-1110-2-1602, "Hydraulic Design of Reservoir Outlet Works". The Reynolds number was computed using the hydraulic radius (4RV/v) for free surface flow instead of the pipe diameter (VD/v) for pressure flow, since the conduit is not flowing full. With the roughness height determined, CORPSLIB program H2261 was re-run and the output was used to compute the Reynolds number again.

The low-level spillway inlet was set at the elevation of the estimated submerged sediment pool. The low level inlet design provides for staged releases which results in maximum detention of the most frequent events while allowing full capacity for less frequent events. The staged inlet ensures that the basin is evacuated to the lowest possible level between events and provides a means to raise the intake level as the sediment pool fills. No permanent pool will be maintained by this inlet structure. The detention time of the 1-year exceedence frequency event is less than 10 days. Based on the drawdown criteria and spillway capacity requirements, the conduits and inlet structures were accordingly sized. The resulting conduit outlet required 6-72 inch conduits, 2 conduits located at 3 positions along the dam. Each pair of conduits would share a 20 feet by 11 feet rectangular inlet with 5-36 inch openings to provide staged outflow and a common stilling basin.

b. Low Level Outlet Stilling Basin. The CORPSLIB program H2261 and guidance provided in EM 1110-2-1602, Hydraulic Design of Reservoir Outlet Works was used for stilling basin design.

Riprap design for the downstream end of the stilling basin was based on the velocity of flow over the end sill computed by H2261 to be 6.7 feet per second. A tailwater elevation of 293.9 NGVD was established using the HEC-RAS program for a 30 ft. trapezoidal channel. The constructed channel will transition from the structure and taper to natural ground approximately 1000 feet from the structure. Final design will be conducted during Pre-Engineering and Design (PED) phase after more detailed surveys are available.

The riprap table adapted from the Corps of Engineers LMVD Report on Standardization of Riprap Gradations, November 1981, revised March 1989, was used as a guide for riprap sizing. A final riprap gradation of R650 was selected due to the anticipated scour downstream of the structure. Channel side slope protection is provided to a minimum distance of $10d_2$ downstream of the stilling basin, or 80 feet, according to EM-1110-2-1602. Outlet channel riprap protection details and dimensions are shown on Plate 62 and 63.

c. Primary Spillway Design. The crest of the primary spillway is set at the peak elevation of the 1.05-year frequency flood at the end of 50 years (elevation 302.2 NGVD). The primary spillway is sized to prevent floods of magnitude less than a 5-year frequency event from overtopping the emergency spillway. Using a primary spillway final width of 135 feet and an emergency spillway crest elevation of 303.9 NGVD, results in a peak elevation of the 5-year flood of elevation 303.2 NGVD initially and elevation 304.0 NGVD after 50 years. This is accepted as meeting the primary spillway hydrologic criteria. Final design data is shown in Table 10.

Table 10

Primary Spillway Pertinent Data

Spillway design flood, 500-year return period	3,200 cfs
Spillway crest elevation	302.2 NGVD

Table 10 (con't)

Maximum reservoir pool elevation	305.5 NGVD
Max. pool elev.-crest elev., He	3.3 ft NGVD
Design ratio, He/Hd	1.33
Design head, Hd	2.5 ft
Crest elevation minus approach channel invert, P = 302.2 - 301.0 (at the end of 50-yr life)	1.2 ft
P/Hd = 1.2 / 2.5	0.48
Tailwater elevation, 500-year HEC-RAS results	293.0 NGVD
Spillway piers	none
Spillway upstream face slope	Vertical, 0H:1V
Spillway downstream face slope	1H:1V
Spillway net width (gross width less piers)	135 ft

d. Primary Spillway Effective Length. The primary spillway will have an ogee crest. Discharge over the spillway is limited by the basic weir equation shown below and presented in EM 1110-2-1603, "Hydraulic Design of Spillways".

$$Q = C\ Le\ (He)^{1.5}$$

where

Q = rate of discharge, cfs
C = coefficient of discharge
Le = effective length of crest, ft
He = total specific energy above crest, ft

All spillways include abutments of some type, and many include intermediate piers. The primary spillway will not have piers but the effect of the abutments must be accounted for by modifying the crest length using the following equation:

$$Le = L - 2\ (\ n\ Kp + Ka)\ He$$

where

Le = effective crest length in feet
L = net length of crest, gross length - width of piers in feet
n = number of piers
Kp = pier contraction coefficient
Ka = abutment contraction coefficient
He = total specific energy above the crest in feet

The resulting effective length was approximately 135 foot. The CORPSLIB program H1107 was used to compute a discharge rating curve for an elliptical shaped spillway crest. The rating curve is shown on Plate 64.

e. Primary Spillway Crest Shape. The complete shape of the lower nappe, which is also the spillway crest surface, is described by separating it into two quadrants upstream and downstream from the high point (apex) of the lower nappe. The equation for the downstream quadrant as expressed in EM 1110-2-1603 is shown below:

$$X^n = K\ Hd^{n-1}\ Y$$

where

- X = horizontal coordinate, positive to the right, feet
- n = variable, usually set equal to 1.85
- K = variable dependent upon P/Hd
- Hd= design head
- P = depth of spillway approach channel below the spillway crest
- Y = vertical coordinate positive downward, feet

The equation of the upstream elliptical shape as presented EM 1110-2-1603 is as follows:

$$X^2/A^2 + (B-Y)^2/B^2 = 1$$

where

- X = horizontal coordinate, origin at crest axis, positive to the right, feet
- A = one-half horizontal axis of ellipse, feet
- B = one-half vertical axis of ellipse, feet
- Y = vertical coordinate, origin at crest axis, positive downward, feet

Upper nappe profiles are included in program H1108 output. The final design was checked for cavitation potential using the relationship He/Hd=1.33 and found to be insignificant. The final crest shape is shown on Plate 65.

f. Primary Spillway Stilling Basin.

As presented in EM 1110-2-1603 , well stabilized hydraulic jumps form when the Froude number is between 4.5 and 9.0. Stilling basin apron length was determined using equation 7-1, in EM 1110-2-1603 as listed below:

$$L_b = K\ d_1\ F_1^{1.5}$$

where

- L_b = stilling basin length, ft
- K = stilling basin length coefficient from Table 7-1, EM 1110-2-1603
- d_1 = depth of flow entering stilling basin, ft
- F_1 = Froude number for flow entering stilling basin
- K = 1.4 for stilling basin with a vertical, stepped, or sloping end sill and one or two rows of baffles.

This resulted in a stilling basin of 20 feet with baffle blocks and an end sill. A drawing of the final structure and stilling basin is shown on Plate 66 and 67.

Riprap design for the downstream end of the stilling basin was based on the velocity of flow above the end sill. The tailwater elevation of 293.9 NGVD was established using the HEC-RAS program for a 145 ft. trapezoidal channel. The constructed channel will transition from the structure and taper to natural ground approximately 1300 feet from the structure. Final design also will be conducted during Pre-Engineering and Design (PED) phase after more detailed surveys are available.

The riprap table adapted from the Corps of Engineers LMVD Report on Standardization of Riprap Gradations, November 1981, revised March 1989, was used as a guide for riprap sizing. A final riprap gradation of R650 was selected due to the anticipated scour downstream of the structure. Channel side slope protection is provided to a minimum distance of $10d_2$ downstream of the stilling basin, or 80 feet, according to EM-1110-2-1602. Outlet channel riprap protection details and dimensions are shown in Plates 68 and 69.

g. <u>Emergency Spillway Design</u>. The emergency spillway width and crest elevation were determined such that the peak elevation of the 100-year flood does not exceed elevation 305.0 NGVD. Using a spillway width of 800 feet and a crest elevation of 303.9 NGVD, pool elevation results shown on Table 11 indicate that the peak elevation of the 100-year flood is el. 304.8 NGVD initially and el. 305.0 NGVD after 50 years. This is accepted as meeting the emergency spillway hydrologic criteria. With this configuration, three feet of freeboard is added to the elevation of the 100-year pool elevation to set the top of the dam at elevation 308.0.

Table 11

<u>Emergency Spillway Pertinent Data</u>

Spillway design flood, 500-year return period	4,800 cfs
Spillway crest elevation	303.9 NGVD
Maximum reservoir elevation	305.5 NGVD
Max. pool elev. - crest elev., He	1.6 ft
Design ration, He/Hd	1.33
Design head, Hd	1.2 ft
Crest elevation minus approach channel invert, P = 303.9 - 301.0 (at the end of 50-yr life)	2.9 ft
P/Hd = 2.9 / 1.2	2.42
Tailwater elevation, 500-year HEC-RAS results	292.2 NGVD
Spillway piers	None

Table 11 (con't)

Spillway upstream face slope	4H:1V
Spillway downstream face slope	4H:1V

h. Emergency Spillway Effective Length. The proposed emergency spillway will be a trapezoidal, broad-crested weir with the overflow capacity computed by the basic weir equation:

$$Q = C\ Le\ (He)^{1.5}$$

where

Q = rate of discharge, cfs
C = coefficient of discharge
Le = effective length of crest, ft
He = total specific energy above crest, ft

The emergency spillway will not have piers but the effect of the abutments must be accounted for by modifying the crest length using the following equation:

$$Le = L - 2\ (\ n\ Kp + Ka)\ He$$

where

Le = effective crest length in feet
L = net length of crest, gross length - width of piers in feet
n = number of piers
Kp = pier contraction coefficient
Ka = abutment contraction coefficient
He = total specific energy above the crest in feet

Using a coefficient of discharge (C = 3.0), a net length of 800 feet was determined to meet the required design criteria. The discharge coefficient is based on comparison to flow over approach roadways as described in HEC-RAS River Analysis System Hydraulic Reference Manual, July 1995, page 5-19.

i. Emergency Spillway Stilling Basin. The design of the stilling basin floor elevation required several iterations. Initial estimates were determined from evaluation of the energy difference between the maximum reservoir level and the assumed apron elevation. After the final basin floor elevation was determined, the stilling basin width, baffle block design, and end sill design were determined.

A percentage of the sequent depth (0.85 d_2 from EM 1603) was compared to the available tailwater elevation for the assumed stilling basin apron elevation in order that the most optimum energy dissipation performance would occur for flows relatively more frequent than the spillway design flood (500-year return period) as presented in paragraph 7-5b of EM 1110-2-1603.

Satisfactory hydraulic jumps form when the Froude number is between 4.5 and 9, when using stilling basins with end sill and a tailwater which produces 85% of the theoretical d_2 as presented in EM 1110-2-1603.

The length of the stilling basin was computed using the following equation as presented in EM 1110-2-1603:

$$L_b = K\, d_1\, F_1^{1.5}$$

where

- L_b = stilling basin length, ft
- K = stilling basin length coefficient from Table 7-1, EM 1110-2-1603
- d_1 = depth of flow entering stilling basin, ft
- F_1 = Froude number for flow entering stilling basin
- K = 1.7 for stilling basin with a vertical, stepped, or sloping end sill only

The Froude number for the flow entering the basin is 9.9. The USBR recommends not using baffle blocks when the entering Froude number is greater than 5.8. Baffle blocks were not used because of the lower frequency of use expected for the emergency spillway ands the large width (800 ft).

Without baffles and end sill, it is recommended to pave the apron for a distance of $4d_2$ to $5d_2$ (p. 5-7, EM 1110-2-1602). Since baffle blocks will not be used, but an end sill will be needed, the final length of 20 feet was chosen. A plan and profile view of the emergency spillway and stilling basin is shown on Plate 70.

A defined exit channel will not be excavated for the emergency spillway but a 2 foot berm will be constructed to aid in training flow departing the stilling basin.

A HEC-RAS model was used to determine the tailwater conditions. Using the riprap table adapted from the Corps of Engineers LMVD Report on Standardization of Riprap Gradations, November 1981, revised March 1989, a minimum riprap designation of R-25 was selected (see attached table) for pre-formed scour hole protection. Exit channel side slope protection is not required because of the low velocity in the exit channel.

Although this velocity is less than the average velocity shown for R-25 class riprap (5.3 ft/s), rock outlet protection is recommended due to potential turbulence at the end of the basin and to protect the basin from scour experienced at regulating structures within the Memphis District COE.

1-20. Wave-Wash Protection and Freeboard

No wave-wash protection was provided for the dry sediment retention basin. Reelfoot Creek discharge records indicated that this is an ephemeral stream with periods of no flow during part of the year. Any wind induced erosion would be repaired during annual maintenance activities. A minimum free board of three feet was added to the 1 percent annual chance of exceedance pool elevation to obtain the top of dam elevation. The resulting top of dam elevation was 308.0 NGVD. Since the 100-year surcharge pool elevations will not be within 3 feet of the maximum pool level for a substantial period of time, 36 hours or more, the minimum freeboard of three feet was considered adequate. The top of dam elevation provided 2.5 feet of freeboard above the 500-year flood pool.

1-21. Operation and Maintenance

Operation of the sediment retention basin will consist of placing stoplogs at the low level inlet structures as the sediment pool fills with deposited material. Maintenance will be required to ensure adequate performance of outlet facilities, to repair wind and flow induced erosion of the embankment, to repair erosion downstream of the emergency spillway outlet, to provide sufficient channel capacity in the upper reaches of the basin and through the basin, and to reshape deposited material in the vicinity of bridges and roadways.

a. Channel maintenance and reshaping deposited material. The estimated annual volume of sediments requiring removal from channels is shown in Table 12. Annual channel maintenance and projected overbank deposition will require acquisition of permanent right-of-way 300 feet left and right of the existing channel top banks.

Table 12

Estimated Maintenance

1. Channel Maintenance

Reach	Annual Volume
Reelfoot Creek Dam to Hwy 22 (Mi. 0.0 - 1.31)	5060 C.Y.
Reelfoot/North Reelfoot Creek Above Hwy 22 (Mi. 1.31 - 4.365)	2588 C.Y.
South Reelfoot Creek Above confluence with N. Reelfoot Creek (Mi. 0.01 - 2.98)	956 C.Y.

2. Reshaping Deposited Material

Reach	Annual Volume
Above Hwy 22*	25,000 C.Y.
Below Hwy 22	5,000 C.Y.

*Assumes ownership of all lands below el. 305.1 NGVD.

b. Structural Maintenance. Repair of the area below the emergency spillway caused by erosion may require replacement of the original riprap protection twice during the 50-year design life. The embankment could also require reforming the upstream dam face to the original grade due to wind and flow induced erosion. This maintenance item is anticipated to be minor.

Maintenance of the outlet facilities will require annual removal of debris from the inlets and periodic repair of the structures and outlets if necessary. The debris removal is expected to be minor. Estimated repair of the low level and principal spillway outlets could consist of repairing or replacing the riprap protection once during the 50 year design life.

Topic F-Shelby Lake and Waterfowl Management Areas

Waterfowl Management Areas are proposed to be constructed in the area east of Running Reelfoot Bayou and south of Highway No. 22. The conceptual design was developed by the project sponsor, the Tennessee Wildlife Resource Agency. Reelfoot Lake will be the source of water to Shelby lake and the surrounding waterfowl areas. The water will be conveyed by gravity flow through two 24 inch conduits under Highway 22 to the area. The structure is shown on Plate 71.

These waterfowl areas will consist of a series of terraces along 1 to 1.5 foot contours, up to the 285 NGVD elevation contour. The terraces would have 25 to 35 foot bottom widths, a height of 2 to 3 feet and a 12 foot crown. Borrow areas would be adjacent and on both sides of the terraces. The area inside the terraces would be either reforested or planted in crops and seasonally flooded. Water sources for the seasonal flood will accomplished by wells and gravity flow from the Reelfoot Lake. Water level management would be achieved by simple water control structures located in the terraces. These structures would consist of culverts with stop log risers. Plate 72 shows the locations of the proposed management areas. Effects of the terraces on overbank flow from Running Reelfoot Bayou will be minor, with potential flooding only in the area of the proposed new spillway structure outlet channel.

Topic G-Inflow Design Floods (IDF)

1-22. General

The purpose of this section is to present the rationale and analysis for the development of the Inflow Design Flood for the impoundment structures within the Reelfoot Project area. Impoundment structures include the Reelfoot Lake Outlet Structure (spillway), Reelfoot Creek Sediment Retention Basin, and the Lake Isom Embankment and Outlet. The IDF for each structure has been approved by Mississippi Valley Division and is included in this report only for informative purposes.

1-23. Inflow (Spillway) Design Flood Criteria

ER-1110-8-2(FR), "Inflow Design Flood For Dams and Reservoirs", dated March 1991 states that it is the Corps of Engineers policy that dams designed, constructed, or operated by the Corps will not create a threat of loss of life or inordinate property damage. Although the subject structures are not traditional dams/reservoirs, the ER defines a dam as a barrier, usually built across a stream, that obstructs, directs, retards, or stores water. The selection of Inflow Design Floods (IDF)s and the design of dam/structure elements necessary to meet safety requirements will conform to one of four Corps of Engineers safety standards.

a. Corps of Engineers Safety Standards.

Standard No. 1: Standard No. 1 applies to the design of dams capable of placing human life at risk or causing a catastrophe, should they fail. Dam height with appropriate freeboard, spillways, regulating outlets, and structural designs will be such that the dams will safely pass an IDF computed from probable maximum precipitation (PMP) occurring over the watershed above the dam site.

Standard No. 2: Standard No. 2 applies principally to the design of run-of-river hydroelectric power or navigation dams, diversion dams, and similar structures where relatively small differentials between headwater and tailwater elevations prevail during major floods. While no unique IDF needs to be established, the structure should be able to safely pass major floods, typical of the region, without excessive structural damage and remain operable. Project design will be based on upstream impact, sediment, dredging, life cycle cost, operation, and other considerations.

Standard No. 3: Standard No. 3 applies to dams where an analysis clearly demonstrates that failure could be tolerated at some flood magnitude. The recommended plan should be for a dam which meets or exceeds a base safety standard. The base safety standard will be met when a dam failure related to hydraulic capacity will result in no measurable increase in population at risk and negligible increase in property damages over that which would have occurred if the dam had not failed. Determination of the IDF that identifies the base safety standard will require definition of the relationship between flood flows and adverse impacts (population at risk and property damages) with and without dam failure for a range of floods up to the Probable Maximum Flood (PMF). Appropriate freeboard will be included for all evaluations. Selection of a base condition predicated on the risk to life from dam failure will require supporting information to demonstrate the increment of population that would actually be threatened. The evaluation should distinguish between population downstream of a dam and the population that would likely be in a life threatening situation given the extent of prefailure flooding, evacuation opportunities, and other factors that might affect the occupancy of the incrementally inundated area at the time the failure occurs. The occurrence of overtopping floods must be relatively infrequent to make Standard No. 3 acceptable. One-half of the PMF is the minimum acceptable IDF for Standard No. 3 dams.

Standard No. 4: Standard No. 4 is applicable to many small recreational type lakes and farm ponds generally containing twenty acre-feet or less of storage. IDF's for small projects corresponding to Standard No. 4 are usually based on rainfall-runoff probability analyses and may represent events of fairly frequent occurrence. In such cases it is often preferable to keep freeboard allowances comparatively small, in order to assure that the volume of water impounded will never be large enough to create a major flood wave if the dam is overtopped and fails. In some instance adaption of Standard No. 4 may be mandatory in spite of the owner's desire for a higher dam to reduce the frequency of damage to the structure due to overtopping floods.

b. Dam Classification. Dams or structures are classified in accordance with size and hazard potential in order to determine the appropriate design flood. The classification for size based on the height of the dam and/or its storage capacity is outlined below in Table 13 and is based on data presented in Corps of Engineers guidance entitled "Recommended Guidelines for Safety Inspection of Dams."

TABLE 13

SIZE CLASSIFICATION

Category	Storage (Ac-Ft.)	Height (Ft.)
Small	<1000 and >50	<40 and >25
Intermediate	>1000 and <50,000	>40 and <100
Large	>50,000	>100

As presented in the referenced document, the height of the dam is determined with respect to maximum storage potential measured from the natural bed of the stream or watercourse at the downstream toe of the barrier, or if it is not across a stream or watercourse, the height from the lowest elevation of the outside limit of the barrier to the maximum water storage elevation. Size classification may be determined by either storage or height, whichever gives the larger size classification.

c. Base Safety Standard. The selection of the IDF is also based on consideration of the upstream impact of imposed ponding and downstream consequences of dam failure which determines the base safety standard. For impoundment structures, the downstream consequences latter consideration typically dictates the dam safety requirements and determines the proper IDF. Downstream impacts were addressed using Bureau of Reclamation guidance in "Training Aids for Dam Safety (TADS), Module: Evaluation of Hydrologic adequacy," dated 1990. Table 14 classifies downstream hazard potential.

TABLE 14

HAZARD POTENTIAL CLASSIFICATION

Category	Loss of Life	Economic Loss
Low	None expected (No permanent structures for human habitation	Minimal (Undeveloped to occasional structure or agriculture)
Significant	Few (No urban developments and no more than a small number of inhabitable structures)	Appreciable (Notable agriculture, industry or structures)
High	More than a few	Excessive (Extensive community, industry or agriculture)

Table 15 presents information from "Recommended Guidelines for Safety Inspection of Dams" and was used as guidance in determining an Inflow Design Floods. The discharge capacity and/or storage capacity should be capable of safely handling the recommended inflow design flood for the size and hazard potential classifications of the dams presented in Table 13 and Table 14.

Table 15

HYDROLOGIC EVALUATION GUIDELINES

Recommended Spillway Design Floods

Hazard	Size	Inflow Design Flood
Low	Small	50 to 100-yr Frequency
	Intermediate	100-yr to 1/2 PMF
	Large	1/2 PMF to PMF

Table 15 (con't)

Significant	Small	100-yr to 1/2 PMF
	Intermediate	1/2 PMF to PMF
	Large	PMF
High	Small	1/2 PMF to PMF
	Intermediate	PMF
	Large	PMF

1-24. PMF AND FREQUENCY FLOOD DEVELOPMENT

Hypothetical storms were developed for the range of frequency events (1.01-500 year) using the frequency-duration data published in the National Weather Service Technical Paper No. 40. The PMF was developed due to the uncertainty related to hazard standards classification for the subject sites. Probable Maximum Precipitation (PMP) estimates, necessary to develop the PMF, were obtained from Hydrometeorlogical Report (HMR) No. 51, Probable Maximum Precipitation, United States East of the 105th Meridian, dated June 1978 and published by the U. S. Weather Bureau. The PMP rainfall estimate was used to compute the PMF. Critical arrangement of the rainfall values for the PMF was in accordance with the NWS publication Hydrometeorological Report No. 52 entitled "Application of Probable Maximum Precipitation Estimates-United States East of the 105th Meridian," dated August 1982. The arrangement was applied using the Hydrologic Engineering Center's computer program, HMR52, dated March 1984. HMR52 was also used to adjust precipitation based on drainage area size and to adjust the orientation of the storm over the basin. Runoff hydrographs for the range of frequency floods and PMF at each structure in the Reelfoot Lake Project and downstream on Running Reelfoot Bayou drainage basin were computed using the HEC-1 model with Snyder's unit hydrograph method.

1-25. Reelfoot Lake Alternate Spillway Feature

Although neither the feasibility study nor the structure is authorized for flood control purposes, the proposed gated structure will possess the capability to better control releases downstream. The existing uncontrolled spillway will be abandoned after alternate spillway construction is complete. Both structures' releases are restricted and controlled by tailwater downstream on

Running Reelfoot Bayou which is critical in determining the dam base safety standard and inflow design flood. During normal gate operations, the lake elevation will fluctuate between elevations 282.0 NGVD and 283.0 NGVD. Assuming that normal gate releases will be restricted to bankfull conditions on Running Reelfoot results in an outlet channel water surface elevation of approximately elevation 280.0. Should a major storm event occur in Running Reelfoot Bayou with similar structure releases, the differential head would be less than 2.0 feet. On occasion locals have witnessed flows from Running Reelfoot Bayou backing into Reelfoot Lake signifying the small differential in heads experienced at the structure.

Since the area downstream of the existing and proposed spillways is primarily agricultural and rural, a structural (embankment) failure would result in damage, if any occurred, primarily to agricultural land or country roads. Field reconnaissance and aerial photography inspection indicates several dwellings east of the proposed structure. The houses are not downstream of the proposed structure and during the occurrence of a major flood, the dwellings would already experience flooding from Running Reelfoot Bayou. Using size and hazard potential classification presented in Tables 14 and 15, the alternate spillway is a large size (80,300 acre-feet), low hazard potential structure based on the downstream undeveloped area. As stated earlier the existing and proposed structure discharges are restricted by tailwater on Running Reelfoot Bayou. Considerations of downstream population at risk and the small differentials between headwater and tailwater elevations at the structure resulted in hazard evaluation conforming to either Standard No. 2 or Standard No. 3.

To substantiate selection of a hazard standard, an analysis was made using guidance from "Training Aids for Dam Safety, Module: Evaluation of Hydrologic Adequacy" (TADS), to determine any incremental increase in water surface elevations. A 500-year event was analyzed initially and considered to be the maximum frequency storm of interest since it determined maximum gate openings necessary to reproduce or maintain existing lake releases. In accordance with standard flood control studies, the 1.01-500 year events were developed for economic and hydraulic evaluations needed for completion of the entire Reelfoot Lake Feasibility Study. During the occurrence of major frequency storms, the proposed structure must maintain identical discharges as the proposed abandoned existing spillway structure for the respective hypothetical storm to avoid project induced damages on Reelfoot Lake and downstream on Running Reelfoot Bayou. The 500-year event was routed through the proposed structure and to downstream areas, with the assumption that the proposed structure remains in place as described in TADS. The same flood was analyzed and routed through the structure assuming the embankment fails. Dam break information is presented in the following table:

Table 16

DAM BREACH DATA

Total Length of Embankment	800 Ft.
Embankment Height	10 Ft.
Width of Breach	400 Ft.
Side Slopes	1V on 1H
Breach Start Elevation	285.0 NGVD
Breach Bottom Elevation (Nat. Grd.)	281.0 NGVD

The incremental increase in downstream water surface elevation between the with-failure and without-failure conditions was determined and the potential damage was quantified. This procedure was then conducted for a larger flood, the One-Half PMF. Computed results also reflect dam break conditions on the sediment retention basin on Reelfoot Creek. Computed water surface elevations do not reflect attenuation through Lake Isom. ER 1110-8-2 (FR) has the One-Half PMF design minimum for Standard NO. 3, but also states the base safety standard will be met when a dam failure related to hydraulic capacity will result in no measurable increase in population at risk and negligible increase in property damages over that which would have occurred if the dam had not failed. Should the 500-year flood or the One-Half PMF occur on the Reelfoot Lake drainage area and downstream on Running Reelfoot Bayou the entire area would be inundated. Under these conditions, outflows are restricted by tailwater which would reduce gravity flow from the lake. The computed One-Half PMF elevation downstream was approximately elevation 288.0 NGVD which equates to the computed lake level. It should also be noted that at elevation 288.0 NGVD, the lake has additional outlets not included in the analysis, which provide relief and would decrease the lake elevations.

Based on incremental water surface increase analysis, a dam failure related to hydraulic capacity will result in no measurable increase in population at risk and a negligible increase in property damages over that which would have occurred if the dam had not failed. Due to the incremental increase in water surface elevations being larger under the 500 year event conditions, it was selected rather than the One-Half PMF. Also, small differentials between headwater and tailwater elevations will exist at the structure during events in the Reelfoot Lake and Running Reelfoot Bayou drainage basins as stated for Corps of Engineers Standard No. 2. Corps of Engineers Standard No. 2 applies principally to the design of run-of-river hydroelectric power of navigation dams, diversion dams, and similar structures with relatively small differentials in headwater and tailwater elevations. The standard states no unique IDF needs to be established but the structure should be able to safely pass major floods (i.e., 500 year event) typical of the region, without excessive structural damage and remain operable

1-26. Lake Isom Feature

Based on the proposed project storage capacity, the structure is classified as an intermediate size dam. The hazard potential is identical to the proposed Reelfoot Lake alternate spillway structure resulting in a low hazard potential due to the unpopulated area downstream. During major events, a small differential between Lake Isom pool levels and downstream tailwater would exist which conforms to Corps of Engineers Standard No. 2. Should an event occur in the magnitude of a 500-year event or the One-Half PMF, Running Reelfoot Bayou floodplain would be inundated and incremental increases in water surface elevations due to a structural failure would be at a minimal. It is also assumed that a dam (embankment) failure of an impoundment structure of this storage capacity would not result in a distinct hazard due to the unpopulated area downstream.

1-27. Sediment Retention Basin Feature

In selecting a hazard classification for a dam, the classification is based on failure consequences resulting from the failure condition that will result in the greatest hazard potential for loss of life and property damage. A comparison of the storage capacity of 1105 acre-feet and a dam height of 13 feet results in an intermediate size classification based on design guidance presented in Table 14. The low hazard potential classification was determined and is based on the sediment basin discharging directly into Reelfoot Lake. The inflow design floods for a low hazard/ intermediate size structure range from the 100-year flood to One-Half the Probable Maximum Flood (PMF).

A worst case scenario was assumed and an incremental water surface elevation increase analysis was initiated for the 500 year event and One-Half the PMF for the Reelfoot Creek basin and downstream on Reelfoot Lake to support the hazard classification. Information describing the necessary hydrologic analysis is outlined in the design reference entitled "Training Aids for Dam Safety (TADS), Module: Evaluation of Hydrologic Adequacy." As outlined in TADS and Corps Safety Standard No. 3, the analysis was required to evaluate the incremental increase in consequences resulting from a failure in order to identify the flood level above which the consequences of failure become acceptable. In other words, the floodflow condition above which the incremental increase due to failure is no longer considered to present an unacceptable threat to downstream life and property. The selection of the appropriate IDF for a dam is the result of this hazard evaluation. Differences between pre-dam break and post-dam break were computed on Reelfoot Lake for incremental increases in water surface elevation determinations. The dam break was simulated using the HEC-1 computer program. Dimensions for the dam break on the Sediment Retention Basin are shown in Table 17.

Table 17

DAM BREACH DATA

Total Length of Embankment	+/- 13600 Ft.
Width of Breach	1000 Ft.
Side Slopes	1V on 1H
Breach Start Elevation	308.2 NGVD
Breach Bottom Elevation (Nat. Grd)	298.7 NGVD

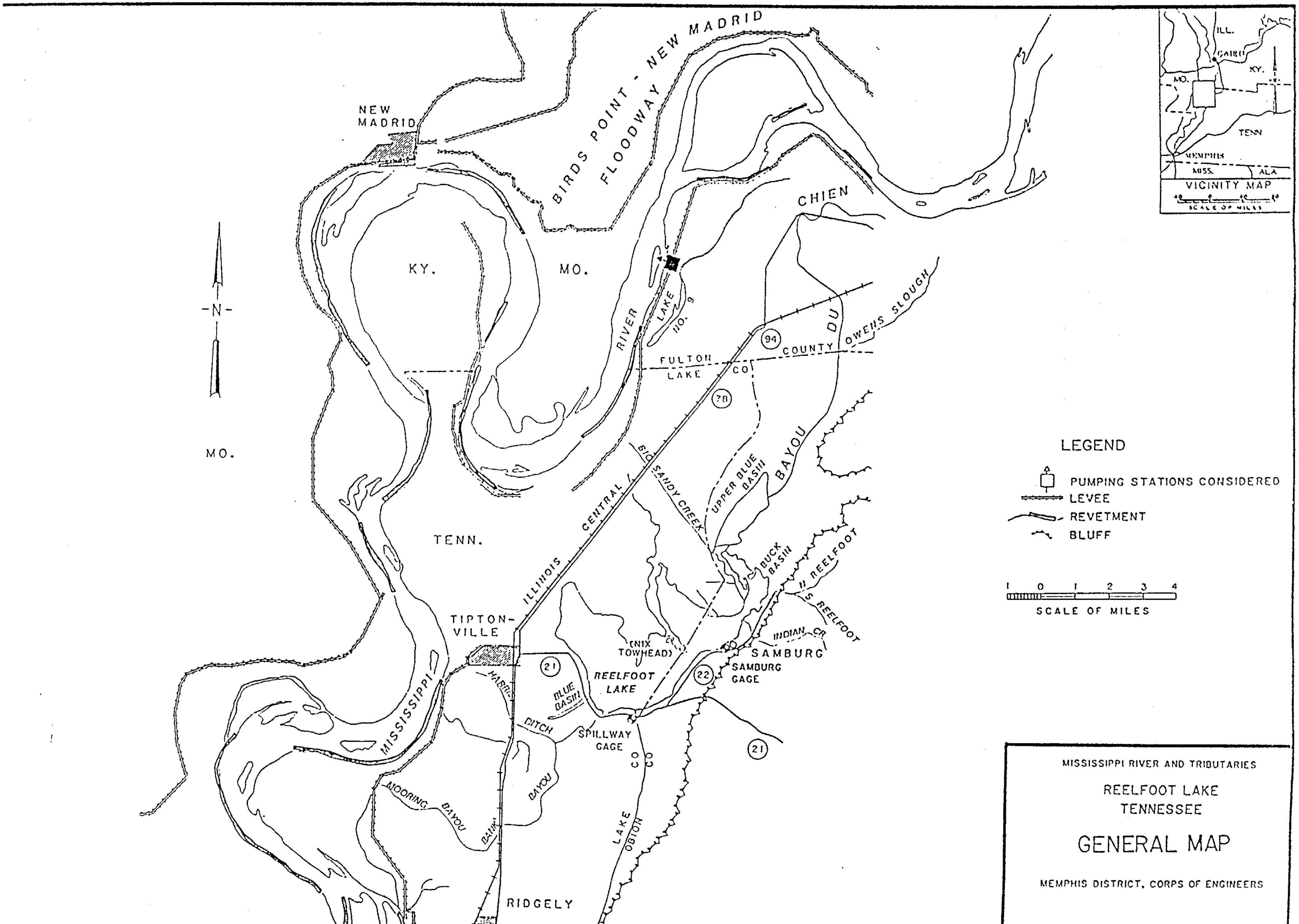
The 500-year event was routed through the proposed structure and to downstream areas, with the assumption that the proposed structure remains in place as described in TADS. The same flood was re-analyzed and routed through the structure assuming the embankment fails. Both pre-dam break and post-dam break conditions were analyzed for the 500-year flood and One-Half the PMF. Comparisons under both conditions made downstream on Reelfoot Lake at the existing spillway structure revealed no measurable difference because of attenuation due to the vast surface area of the lake. The incremental increase in Reelfoot Lake water surface elevations due to the dam break was determined to be approximately 0.1 foot for both conditions. Criteria for Standard No. 3 applies to dams which impound water upstream from populated areas which would be susceptible to hazardous conditions should a dam fail. This shows the results of a dam break would not cause a further threat to life or property over pre-failure conditions so selection of the IDF was based on a lesser flood (500-year event) instead of the one-half PMF. Also, since the sediment retention basin will discharge directly into Reelfoot Lake, the probability for placing life or property at risk below the sediment retention basin would be relatively low or non-existent. TADS also states dambreak routings may be terminated at a point where damage caused by routing flood flows appears limited (i.e., a large downstream storage area).

1-28. Summary

It is the Corps of Engineers policy, as stated in ER 1110-8-2 (FR), that dams designed, constructed, or operated by the Corps will not create a threat of loss or life or inordinate property damage. The structures for the Reelfoot Lake Project are categorized as low hazard dams; meaning failure will result in no adverse impacts downstream. ER-1110-8-2 (FR) was used as guidance for determining base safety standards for each structure, and incremental increases in water surface elevations were analyzed to ensure population(s) downstream are not at risk should a failure occur.

Aerial photography and field reconnaissance reveals the downstream areas for the Reelfoot Lake and Lake Isom structures are largely unpopulated and resulting consequences of failure would not be catastrophic. Damages will be primarily isolated to county roads and bridges. The magnitude of the selected IDF will produce downstream tailwater conditions which greatly reduce outflows from each structure should a failure occur. Also, the presence of sufficient tailwater creates a small differential between lake elevations and Running Reelfoot Bayou flood elevations. This tends to make Corps of Engineers Standard No. 2 applicable. It is felt the selection of the 500-year event as the IDF is appropriate since no unique IDF needs to be established under this standard.

Consequences of a failure at the Reelfoot Creek sediment retention basin also would not present a hazard to human life due to the storage available downstream on Reelfoot Lake. Routing of the dam break flood wave under the 500 year event and the One-Half PMF resulted in no measurable increase in water surface elevations at the existing spillway on Reelfoot Lake which is a reflection of the vast storage area. The structure also will not maintain a permanent pool which decreases the probability of having a failure occur presenting a hazard downstream. Selection of a 500-year event as the IDF was based on the low hazard potential, small incremental increase in water surface elevations and absence of a traditional permanent pool.



MISSISSIPPI RIVER AND TRIBUTARIES

REELFOOT LAKE
TENNESSEE

GENERAL MAP

MEMPHIS DISTRICT, CORPS OF ENGINEERS

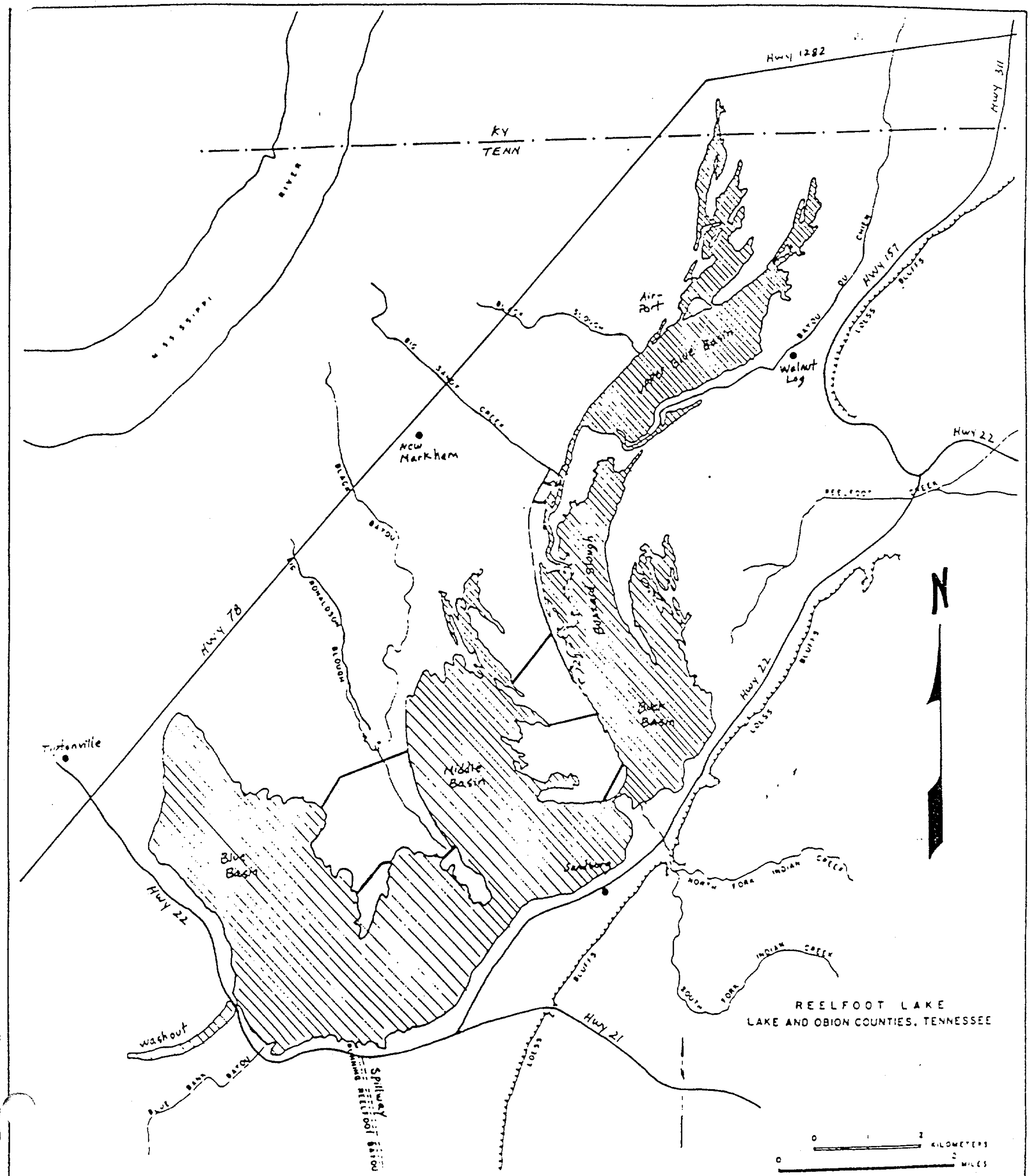
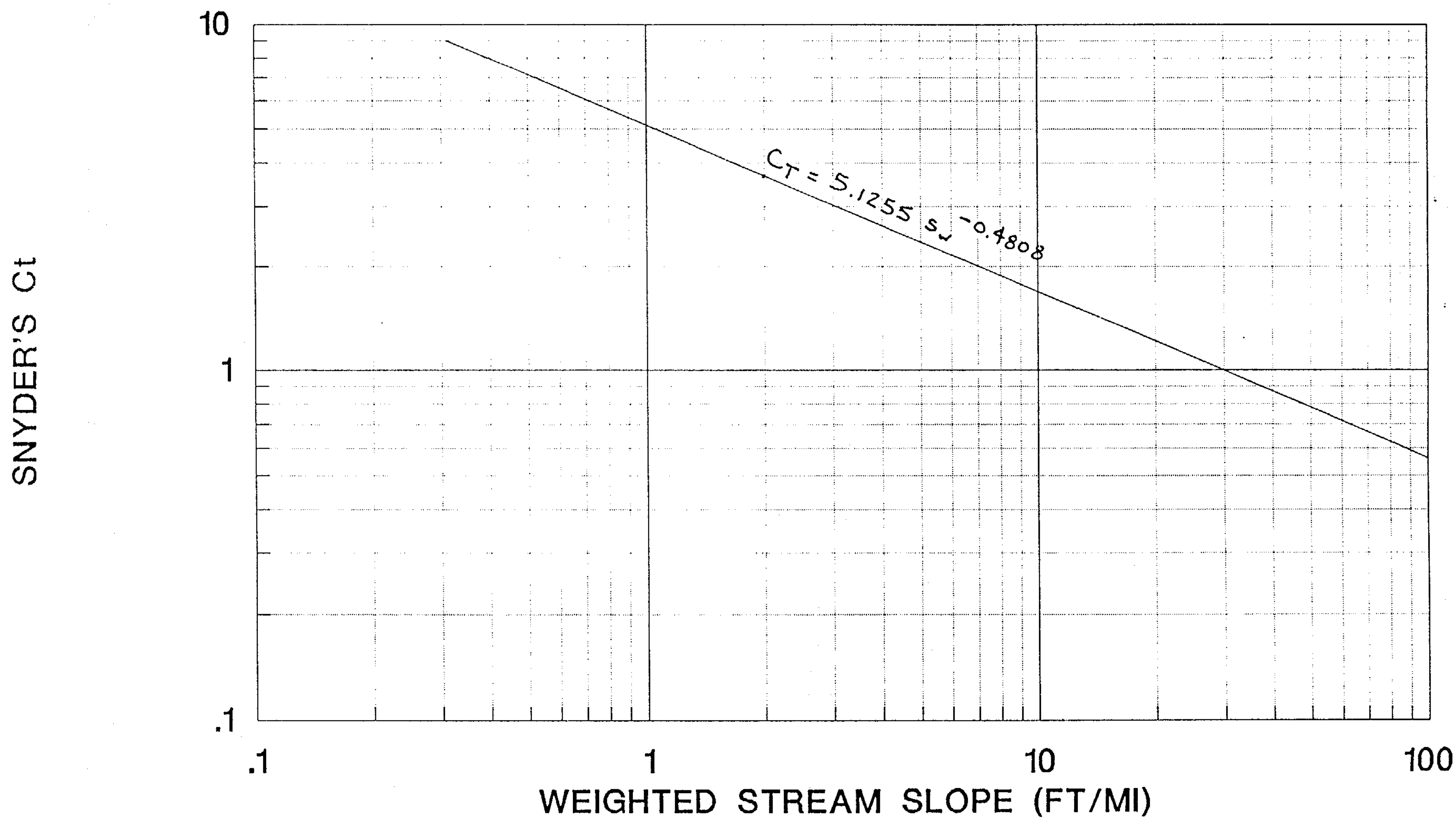


Figure 1. Map depicting Reelfoot Lake and subbasins, Lake - Obion Counties, Tennessee.

Ct vs. WEIGHTED STREAM SLOPE
MEMPHIS DISTRICT



SUMMARY OF SYNTHETIC UNIT HYDROGRAPH PARAMETERS

Subarea No.	Drainage Area Rural mi^2	Drainage Area Urban mi^2	L mi	L_{ca} mi	S_{st} ft/mi	c_t	c_p	t_p	Percent Urban	t_{pu} hrs
North Reelfoot Creek										
2	2.13		2.25	1.25	6.04	2.18	0.50	2.97		
4	1.95		2.08	0.92	1.23	4.70	0.50	5.72		
6	0.85		1.72	0.72	17.23	1.30	0.50	1.39		
8	0.39		1.15	0.30	29.45	1.01	0.55	0.73		
10	2.83		2.68	1.20	12.27	1.55	0.50	2.20		
12	1.50		2.20	1.05	3.96	2.67	0.50	3.43		
14	1.27		2.08	1.12	14.15	1.45	0.50	1.87		
16	1.60	0.04	2.30	1.08	10.12	1.60	0.50	2.10	2.5	2.05
18	1.78	0.07	2.80	1.45	16.98	1.31	0.50	1.99	3.9	1.92
20	3.13	0.08	6.15	3.28	12.03	1.57	0.50	3.87	2.6	3.77
22	0.96		1.40	0.58	19.50	1.23	0.50	1.15		
24	0.62		2.08	0.95	23.99	1.11	0.55	1.36		
26	0.21		0.98	0.32	30.40	0.99	0.55	0.70		
28	0.76		2.06	1.06	26.01	1.08	0.55	1.36		
30	0.89		2.00	1.20	11.72	1.58	0.50	2.05		
32	3.78		4.75	2.32	15.13	1.38	0.50	2.83		
34	2.46		2.55	1.35	14.06	1.45	0.50	2.10		
36	1.64		3.00	1.54	20.61	1.20	0.50	1.90		
44	0.24		1.15	0.61	24.53	1.10	0.55	0.99		
38	2.44		2.73	1.28	21.42	1.18	0.50	1.72		
40	1.21		2.95	1.56	27.15	1.05	0.55	1.66		
42	0.28		1.15	0.60	9.17	1.78	0.50	1.59		
45	0.09		0.60	0.24	25.72	1.08	0.55	0.60		
46	0.10		0.62	0.16	47.75	0.80	0.60	0.40		
48	1.62		2.60	1.25	33.42	0.95	0.55	1.35		
50	1.83		2.70	1.18	25.47	1.09	0.55	1.54		
52	2.34		3.33	1.92	14.38	1.43	0.50	2.50		
54	0.14		0.65	0.47	25.10	1.09	0.55	0.76		
56	0.74		2.05	0.65	25.91	1.08	0.55	1.18		
58	0.83		1.84	0.87	42.37	0.85	0.55	0.98		
60	0.05		0.42	0.25	11.90	1.58	0.50	0.80		
68	3.67		3.95	2.23	23.90	1.11	0.55	2.13		
62	0.67		1.21	0.39	16.18	1.35	0.50	1.08		
59	3.77		3.23	1.25	29.06	1.01	0.55	1.54		
64	1.73		2.65	0.88	18.55	1.26	0.50	1.62		
63	0.16		0.85	0.40	17.65	1.29	0.50	0.93		

SUMMARY OF SYNTHETIC UNIT HYDROGRAPH PARAMETERS

Subarea No.	Drainage Area Rural mi^2	Drainage Area Urban mi^2	L mi	L_{ca} mi	S_{st} ft/mi	c_t	c_p	t_p	Percent Urban	t_{pu} hrs
North Reelfoot Creek (Continued)										
65	0.22		0.76	0.30	23.02	1.14	0.50	0.73		
70	2.78		2.73	1.72	38.87	0.88	0.55	1.40		
66	0.31		1.10	0.58	22.63	1.15	0.50	1.00		
72	3.75		3.28	1.07	19.61	1.23	0.50	1.79		
74	1.91		3.35	1.73	13.96	1.46	0.50	2.47		
76	0.32		1.05	0.38	14.94	1.40	0.50	1.06		
80	2.09		2.65	1.62	37.93	0.89	0.55	1.38		
78	0.16		0.68	0.41	4.39	2.51	0.50	1.71		
82	2.28		2.48	1.34	3.30	1.52	0.50	2.18		
84	2.16		3.55	1.82	17.47	1.09	0.50	1.91		
South Reelfoot Creek										
112	4.01	0.24	2.90	1.14	8.72	1.80	0.50	2.57	6.0	2.42
114	1.92		1.90	1.15	3.25	2.93	0.50	3.70		
108	0.76		2.25	1.02	46.00	0.81	0.55	0.96		
106	3.88		2.75	1.31	16.81	1.33	0.50	1.96		
110	1.71		1.82	0.58	13.12	1.98	0.50	2.01		
116	0.22		0.75	0.53	29.69	1.00	0.55	0.76		
118	1.84		2.60	1.38	37.59	0.90	0.55	1.32		
120	3.35		2.52	1.75	4.37	2.54	0.50	3.97		
122	1.42		2.28	1.46	2.94	2.94	0.50	4.22		
104	2.89		4.18	2.30	16.93	1.32	0.50	2.60		
86	2.95		2.38	1.19	29.61	1.00	0.55	1.36		
88	2.50		2.55	0.75	31.99	0.92	0.55	1.12		
89	0.70		1.55	0.60	11.77	1.58	0.50	1.54		
92	1.67		2.70	1.68	49.92	0.79	0.60	1.24		
91	0.07		0.28	0.18	49.56	0.78	0.60	0.32		
90	0.67		1.78	0.78	42.58	0.85	0.55	0.94		
94	2.24		2.42	0.70	43.26	0.84	0.55	0.98		
96	1.91		2.58	1.35	24.98	1.09	0.55	1.58		
98	0.03		0.45	0.13	94.18	0.57	0.65	0.24		
100	0.67		2.95	1.70	4.87	2.40	0.50	3.89		
102	1.03		2.45	1.34	46.39	0.82	0.55	1.17		
124	1.71		1.58	0.55	3.54	2.81	0.50	2.69		
126	2.16		3.05	1.80	4.44	1.44	0.50	2.40		
128	1.08		3.15	1.63	24.68	1.10	0.55	1.80		
130	3.16		1.98	1.42	1.04	2.49	0.50	3.40		

SUMMARY OF SYNTHETIC UNIT HYDROGRAPH PARAMETERS

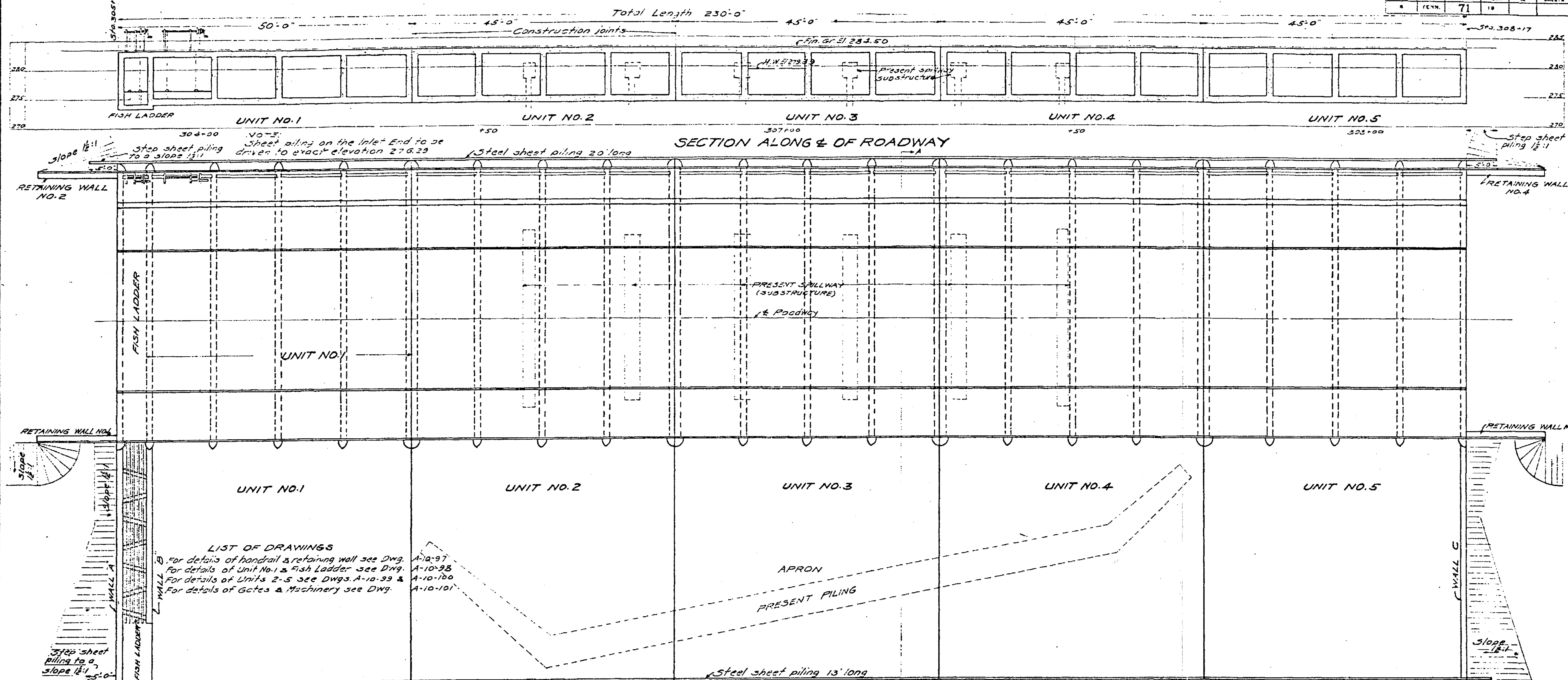
ubarea No.	Drainage Area Rural mi^2	Drainage Area Urban mi^2	L mi	L_{ca} mi	S_{st} ft/mi	c_t	c_p	t_p	Percent Urban	t_{pu} hrs
rea surrounding the lake										
44	3.25		3.37	1.70	31.57	0.98	0.50	1.65		
48	1.65		2.73	1.30	17.52	1.30	0.50	1.90		
46	3.18		3.85	2.11	35.02	0.93	0.50	1.74		
50	3.65	0.25	2.82	1.80	5.11	2.35	0.50	3.82	6.9	3.57
42	0.93		1.60	0.84	35.31	0.91	0.55	0.99		
40	1.07		0.88	0.66	12.30	1.54	0.50	1.31		
38	0.59		2.30	1.48	44.78	0.82	0.55	1.19		
36	0.17		0.60	0.20	2.43	3.37	0.50	1.78		
34	0.61		1.02	0.34	1.74	3.95	0.50	2.88		
32	0.14		0.48	0.13	2.08	3.67	0.50	1.60		
52	1.38		1.95	0.95	52.92	0.76	0.60	0.91		
54	2.79		3.20	1.40	7.44	1.98	0.50	3.10		
22	3.04		3.35	1.74	9.62	1.72	0.50	2.92		
24	0.48		1.45	0.88	6.12	2.17	0.50	2.34		
26	2.25		3.05	2.00	1.39	4.40	0.50	7.57		
28	2.07		3.25	1.87	1.61	4.10	0.50	7.04		
12	4.80		4.10	2.84	0.59	6.62	0.50	13.82		
14	0.11		0.72	0.40	1.39	4.40	0.50	3.03		
156	0.22		0.68	0.24	3.36	2.88	0.50	1.67		
00	3.39	0.60	2.98	1.47	25.71	1.80	0.55	2.80	17.7	2.38
02	1.04		1.72	0.70	1.43	4.35	0.50	4.60		
04	1.69		2.74	1.40	1.73	3.97	0.50	5.39		
06	2.15		1.80	1.07	1.11	5.15	0.50	1.58		
08	0.78		1.08	0.64	5.55	2.28	0.50	2.04		
10	2.34		3.18	1.30	0.28	9.50	0.50	14.54		
30	2.36		2.55	1.32	0.35	8.49	0.50	12.23		
158	0.73		1.30	0.84	3.02	3.04	0.50	3.12		
16	0.92		2.00	0.92	2.67	3.20	0.50	3.84		
18	3.57		3.30	1.31	1.39	4.40	0.50	6.83		
20	1.77		2.25	1.00	0.44	7.65	0.50	9.76		
160	1.26		2.25	1.02	3.90	2.67	0.50	3.43		
162	0.89		1.90	0.88	0.53	7.00	0.50	8.17		
164	1.57		2.32	1.63	0.79	5.78	0.50	8.62		

SUMMARY OF SYNTHETIC UNIT HYDROGRAPH PARAMETERS

Subarea No.	Drainage Area Rural mi^2	Drainage Area Urban mi^2	L mi	L_{ca} mi	S_{st} ft/mi	c_t	c_p	t_p	Percent Urban	t_{pu} hrs
Area surrounding the lake (Continued)										
168	3.56		3.05	0.90	0.56	6.80	0.50	9.21		
170	3.15		3.30	1.15	1.52	4.40	0.50	6.57		
172	1.22		2.62	1.28	0.30	9.20	0.50	13.23		
174	2.88		3.38	1.50	1.30	4.55	0.50	7.40		
176	0.57		1.80	0.89	0.74	5.95	0.50	6.85		
178	4.10		2.72	1.75	0.41	7.97	0.50	12.73		
180	1.50		1.63	0.66	7.13	2.00	0.50	2.05		
182	0.58		0.55	0.28	6.34	2.12	0.50	1.21		
184	0.93	0.10	0.88	0.40	8.41	1.85	0.50	0.11	10.75	0.10

Date of Last Revision - 5/11/31

FED. AID DIST. NO.	STATE	FED. AID PROJ. NO.	FISCAL YEAR	SHEET NO.	TOTAL SHEETS
8	TENN.	71	19		



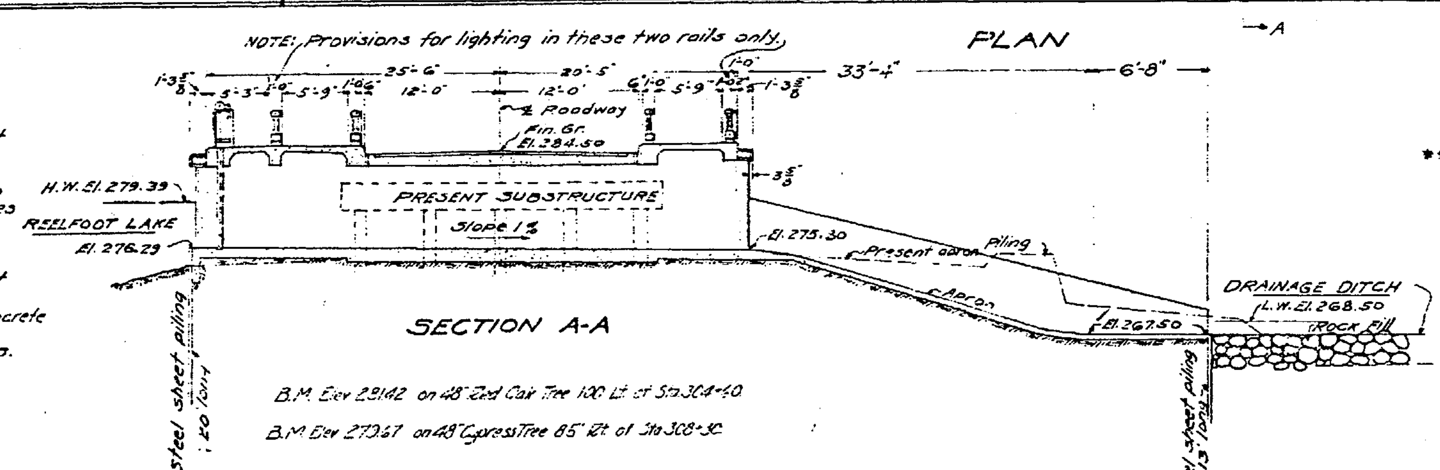
LIST OF DRAWINGS

For details of handrail & retaining wall see Dwg. A-10-97

For details of Unit No. 1 & Fish Ladder see Dwg. A-10-98

For details of Units 2-5 see Dwgs. A-10-99 & A-10-100

For details of Gates & Machinery see Dwg. A-10-101



General Notes

Specifications: Standard Road and Bridge Specifications of the Tennessee Department of Highways and Public Works except as noted below.

Machinery: Material and workmanship to be as per Specifications for movable bridges prepared by the A.R.E.A. 1922 editions.

Structural Steel: 3½" rails @ 50#/yd. see A.S.T.M. Specifications.

Sheet Piling: M104 - Carnegie or equivalent section.

Concrete in handrail to be Class "S" concrete in remainder of structure to be Class "A".

Reinforcing Steel: See Specifications.

Forms and Finish: See Specifications.

TOTAL ESTIMATED QUANTITIES

*Excavation (Unclass)	7400	Cu. Yds.
Concrete Class "S"	65.8	Cu. Yds.
Concrete Class "A"	1349.9	Cu. Yds.
Reinforcing Steel	141,927	Lbs.
** Structural Steel	8250	Lbs.
Sheet piling	7200	Lin. Ft.
Rock Fill	333	Cu. Yds.
Removal of present bridge	Lump Sum ⊙	
Machinery	Lump Sum ▪	

** Structural steel includes only the 3½" rails in the fish ladder and fastenings.

* Unclassified excavation includes the removal of present apron and pile wall to an elevation so as not to interfere with the new construction.

⊙ Lump sum includes the complete removal of about 92 feet of present bridge, consisting of concrete pile bents and timber superstructure (roadway 22' & 2' sidewalk). The concrete piles shall be removed to a point so as not to interfere with the new construction.

▪ Lump sum includes the supporting frames, the bearings, all revolving parts, chains, fastenings to the gates, the gates proper, the guides for the gates and the placing of the above items.

SPECIAL NOTE

Wearing surface to be 1½" of Class "A" concrete poured monolithic with roadway slab.

Rock fill shall be placed at lower end of apron wherever it is necessary.

Second and Third Field Coats of Paint for all Structural Steel, including the 3½" Rails, may be Aluminum Paint as per Specification M-22, Rev. Mar. 1, 1930.

STATE OF TENNESSEE
DEPARTMENT OF HIGHWAYS AND PUBLIC WORKS
NASHVILLE

LAYOUT OF SPILLWAY

STA. 305+87
LAKE-OBION CO'S
1930

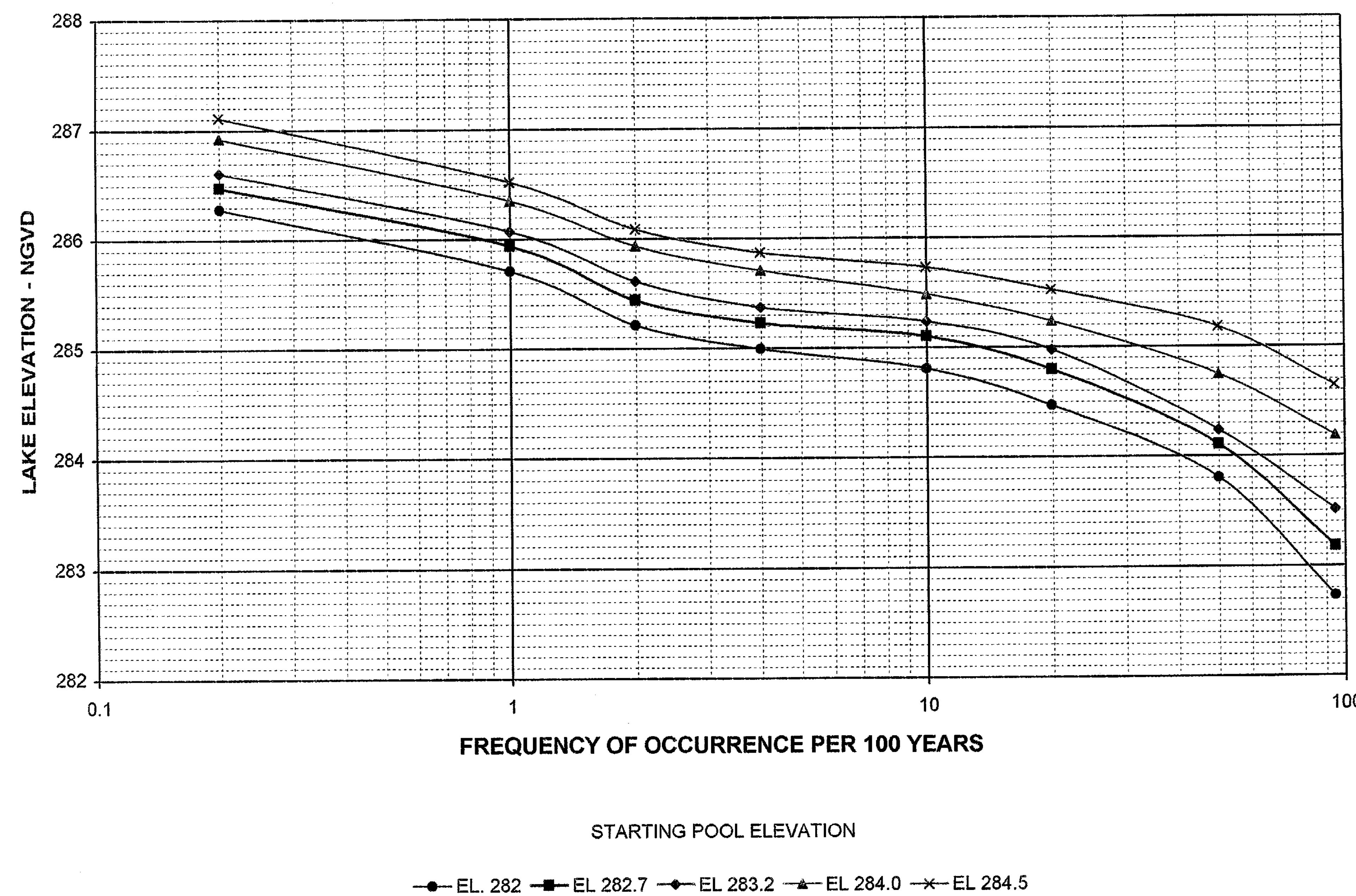
CORRECT L. N. Kennickson, BRIDGE ENGINEER
APPROVED C. M. ..., STATE HIGHWAY ENGINEER

A-10-96

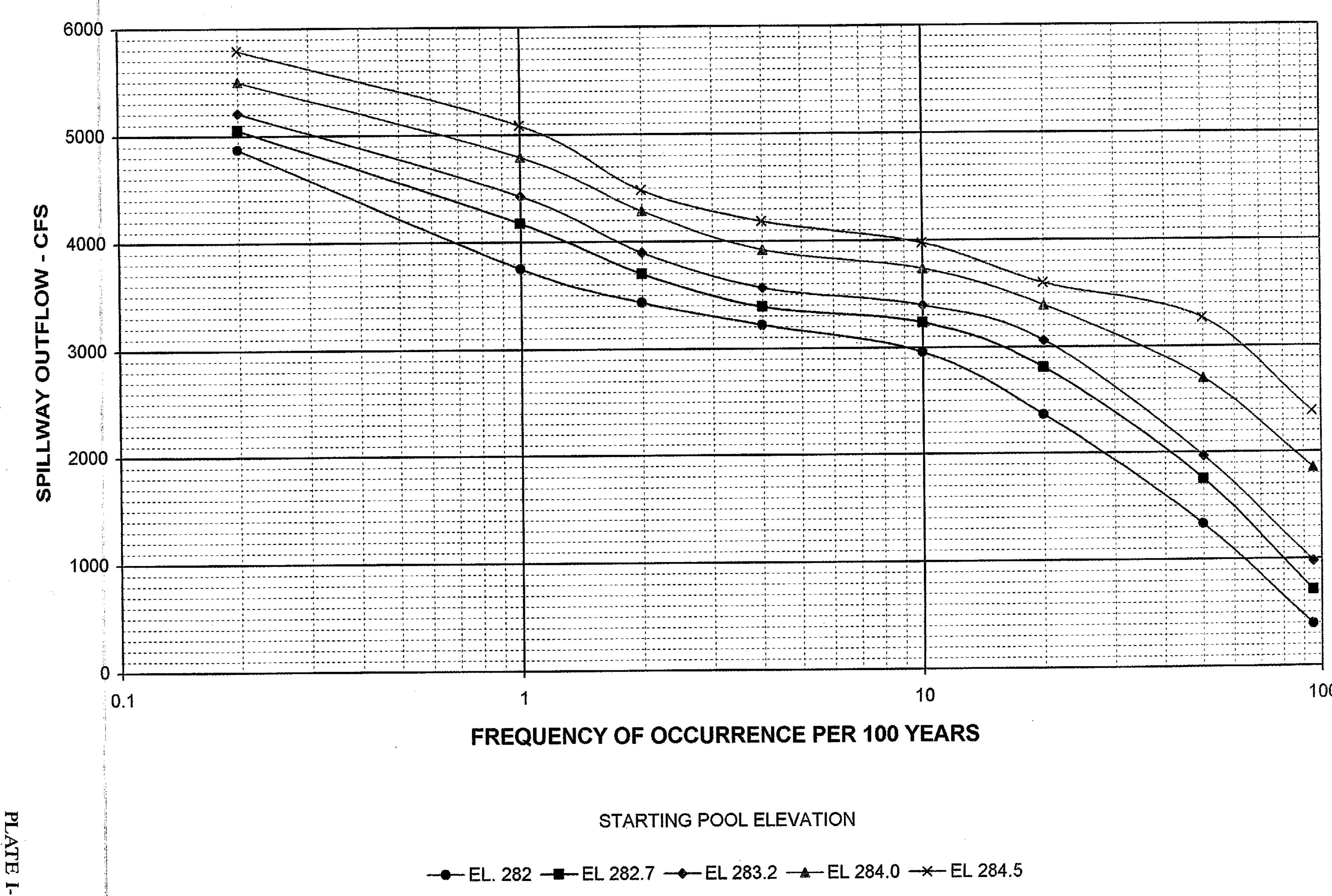
DESIGNED BY Will Lich DATE 10-28-30
DRAWN BY ... DATE ...
TRACED BY Warner DATE 11-13-30
CHECKED BY M. Sch. DATE ...

Handrail at 6" from curb 5-11-31
Elevation of inlet raised 1'-0" - 3/12/31

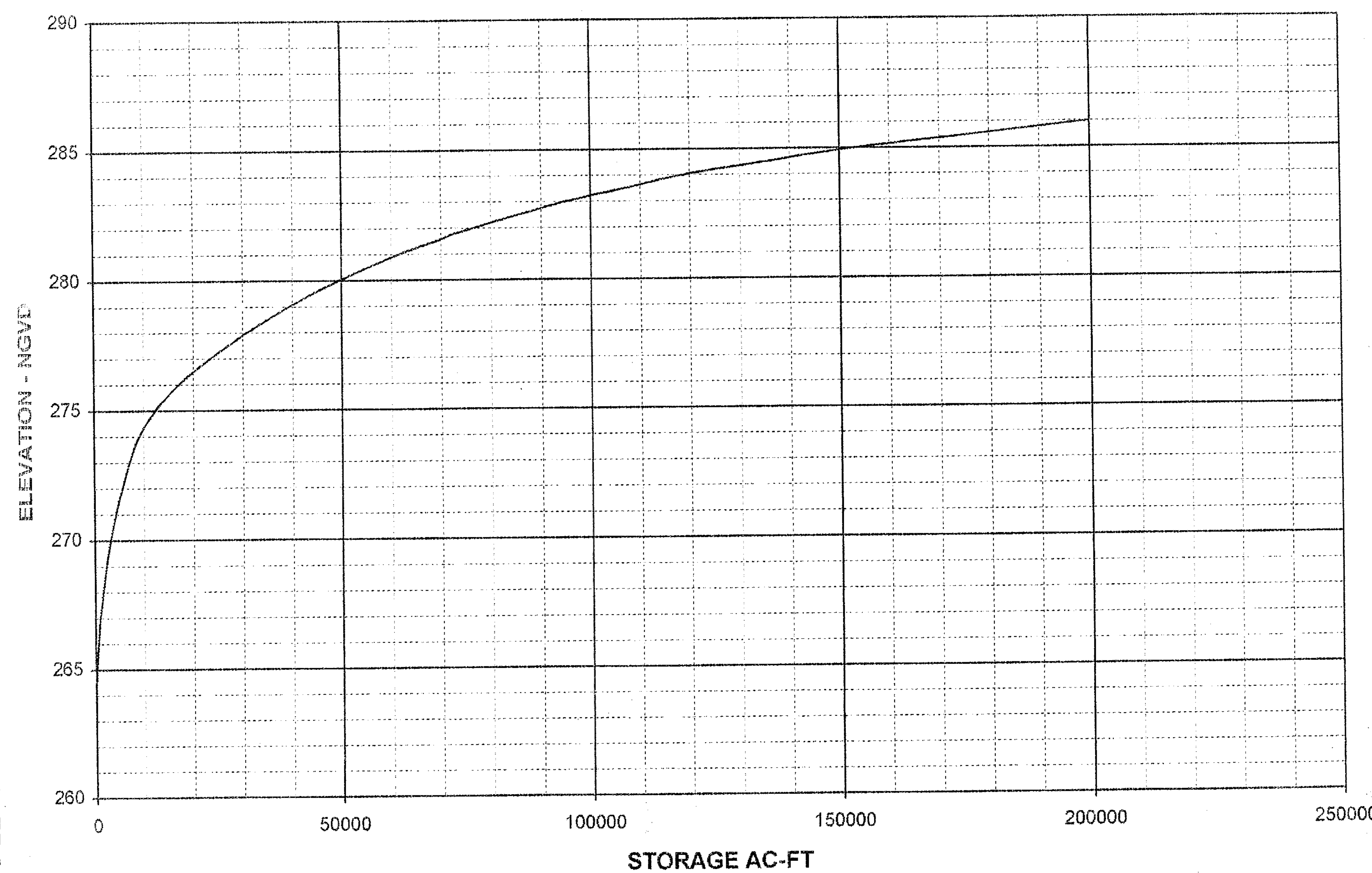
REELFOOT LAKE FEASIBILITY STUDY
REELFOOT LAKE
ELEVATION VS. FREQUENCY



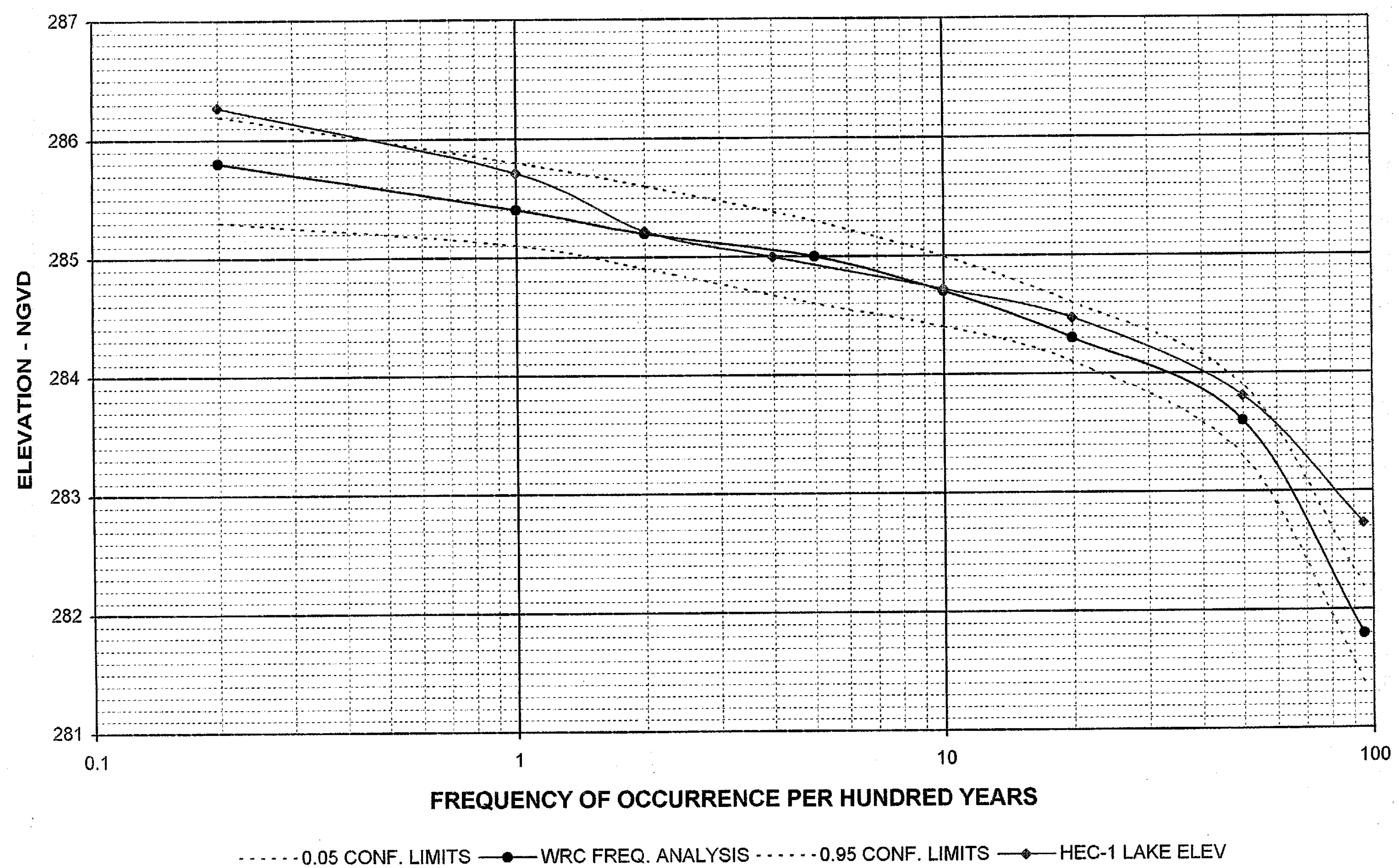
REELFOOT LAKE FEASIBILITY STUDY
REELFOOT LAKE
OUTFLOW VS. FREQUENCY



REELFOOT LAKE FEASIBILITY STUDY
REELFOOT LAKE
STORAGE CURVE



REELFOOT LAKE FEASIBILITY STUDY
REELFOOT LAKE GAGE
HEC-1 MODEL VERIVICATION
PERIOD OF RECORD 1953 - 1993



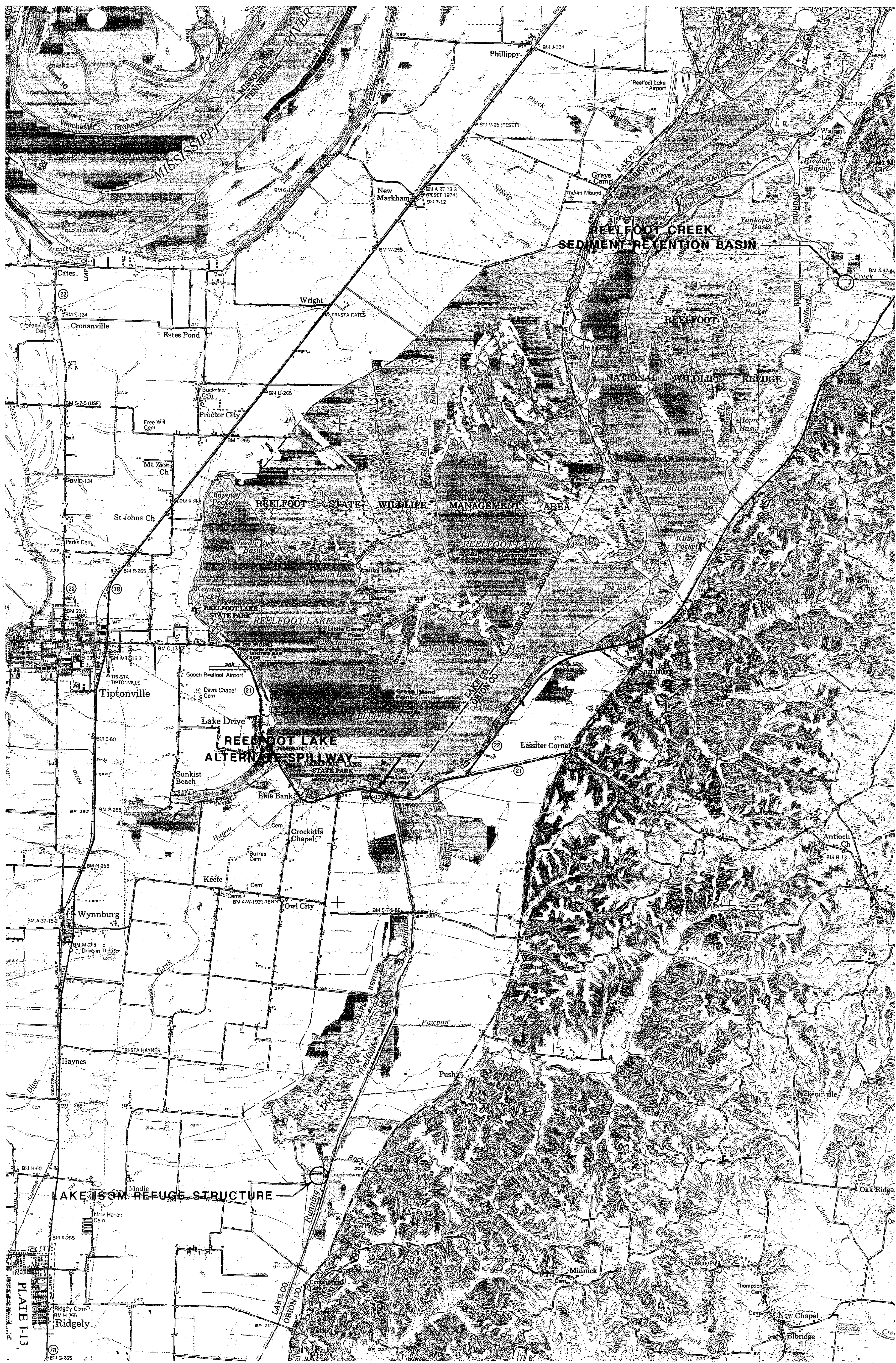
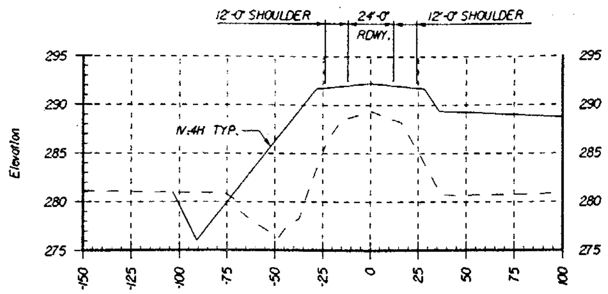
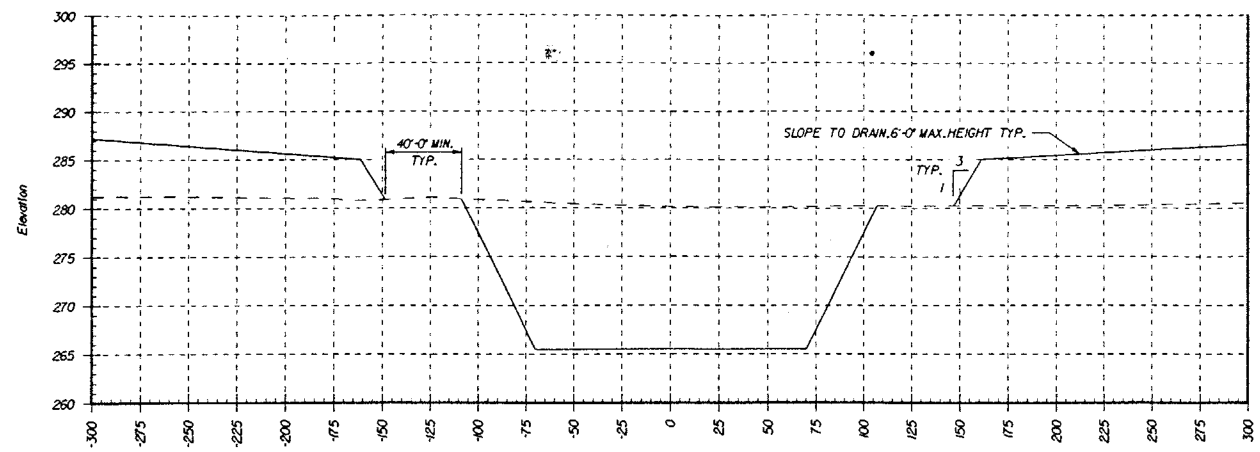

MISSISSIPPI
RIVER
MISSOURI
TENNESSEE
Island 10
Winchester
Phillippy
Reelfoot Lake Airport
Black Slough
Grays Camp
Indian Mound
New Markham
LAKE CO.
OBION CO.
UPPER BLUE BASIN
REELFOOT STATE WILDLIFE MANAGEMENT
Walnut Log
Mud Basin
Yankapin Basin
REELFOOT CREEK
SEDIMENT RETENTION BASIN
Cates
Wright
Cronanville
Cronanville Cem
Estes Pond
Reelfoot
Rat Pocket
NATIONAL WILDLIFE REFUGE
Proctor City
Free Will Cem
Mt Zion Ch
St Johns Ch
Parks Cem
Champey Pocket
REELFOOT STATE WILDLIFE MANAGEMENT AREA
BUCK BASIN
Needle Eye Basin
REELFOOT LAKE
Swan Basin
Caney Island
Keystone Pocket
REELFOOT LAKE STATE PARK
Joe Basin
Kirby Pocket
Little Caney Point
Choctaw Basin
Samburg
Gooch Reelfoot Airport
Davis Chapel Cem
Tiptonville
Green Island Point
BLUE BASIN
Lake Drive
REELFOOT LAKE
ALTERNATE SPILLWAY
Lassiter Corner
Sunkist Beach
Blue Bank
Crocketts Chapel
Burrus Cem
Antioch Ch
Keefe
Owl City
Wynnburg
Drive-in Theater
Haynes
Pawpaw
Push
Jacksonville
Rock
LAKE ISOM REFUGE STRUCTURE
Madie
New Haven Cem
Oak Ridge
Minnick
Thompson Cem
New Chapel
Elbridge
Ridgely Cem
Ridgely
Running
LAKE CO.
OBION CO.

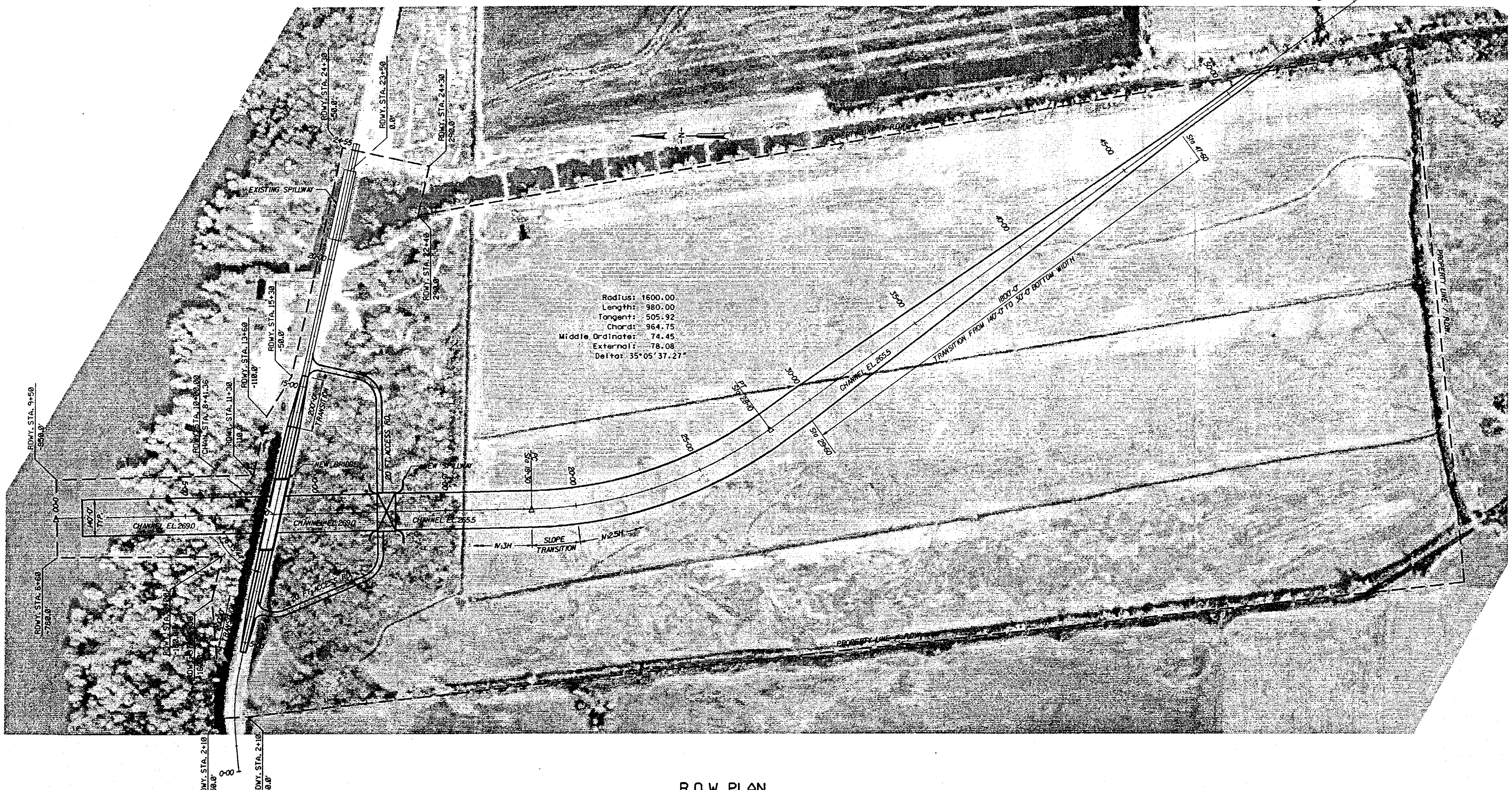
PLATE I-13



TYPICAL HIGHWAY SECTION



TYPICAL CHANNEL CROSS SECTION



R.O.W. PLAN
SCALE 1" = 400'

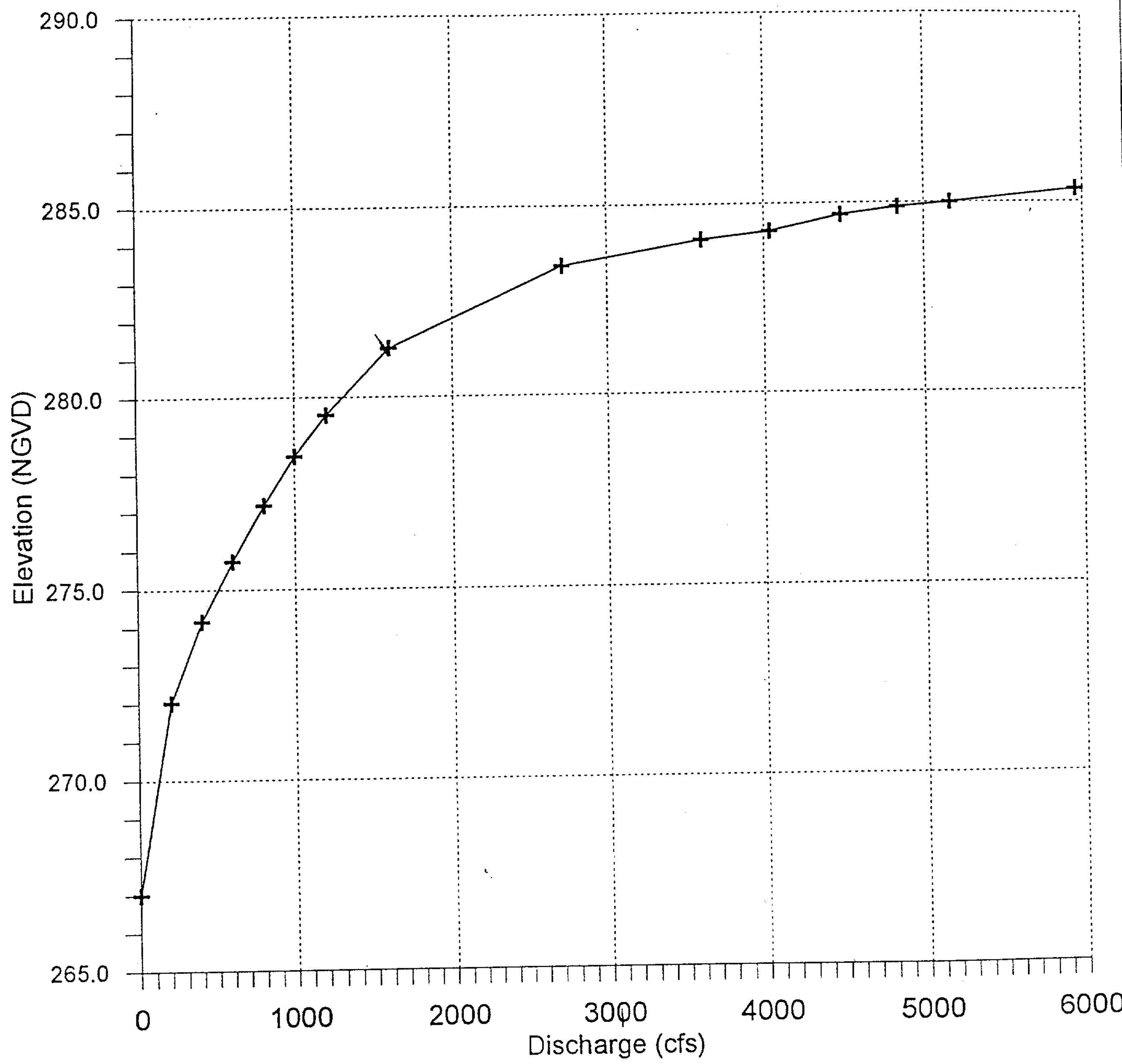
US Army Corps of Engineers
Memphis District

U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
MEMPHIS, TENNESSEE

ENVIRONMENTAL RESTORATION
NATURAL RESOURCES PROJECT
MISSISSIPPI RIVER AND TRIBUTARIES
CONSTRUCTION

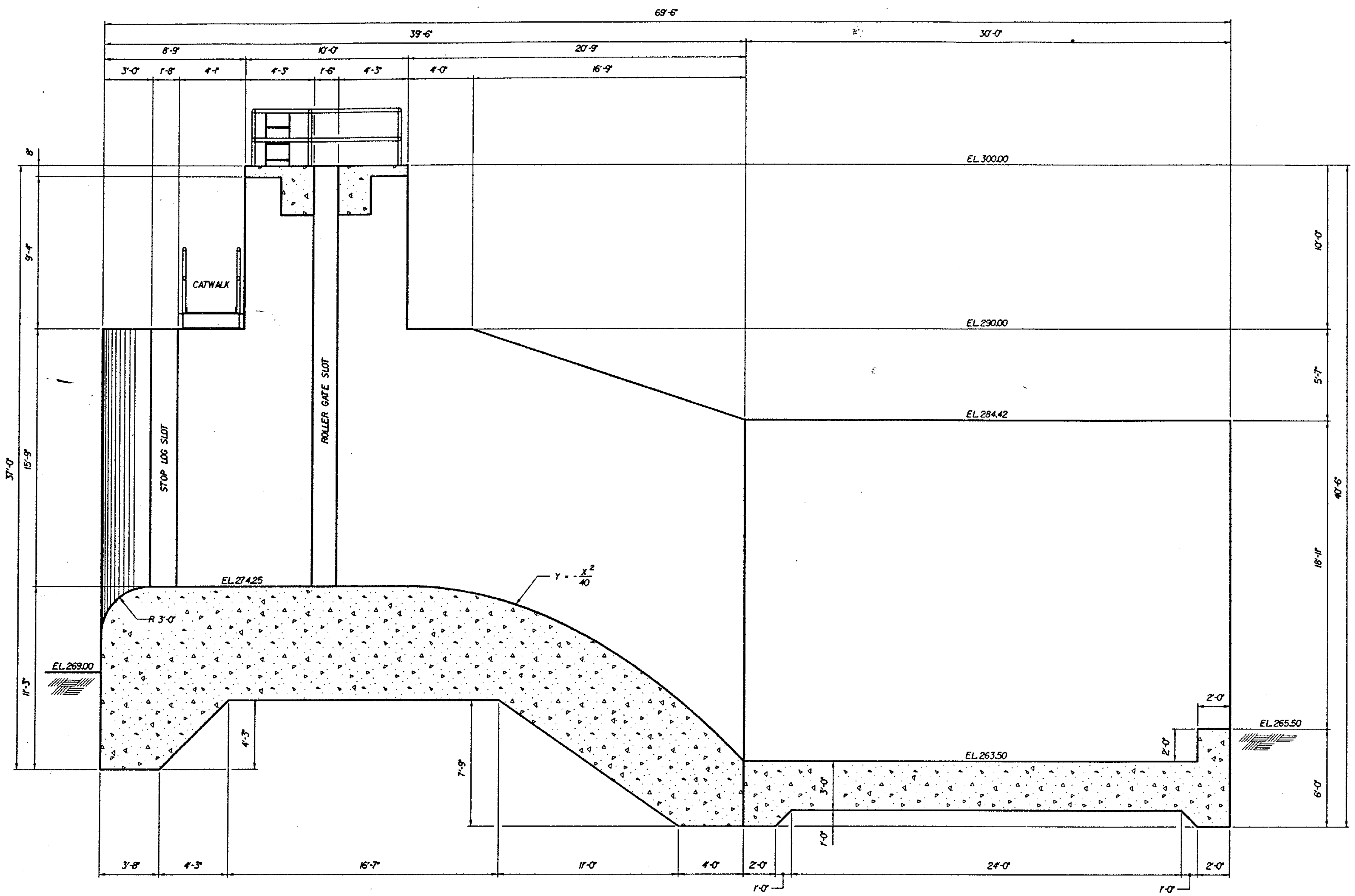
REELFOOT LAKE, TENNESSEE & KENTUCKY
REELFOOT LAKE ALTERNATIVE SPILLWAY
RIGHT-OF WAY & TYPICAL SECTIONS

PLATE
I-14

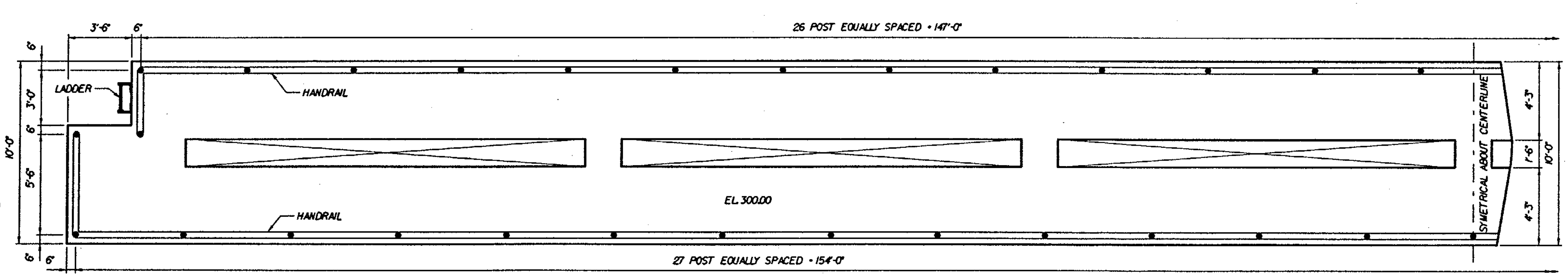


REELFOOT LAKE, TN & KY

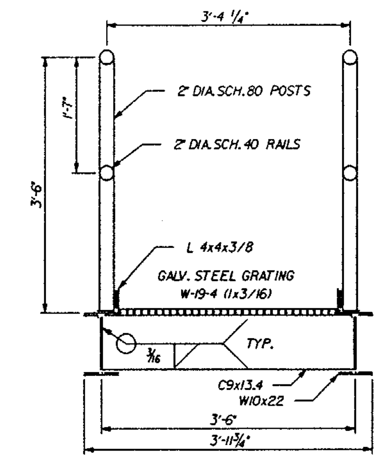
TAILWATER RATING CURVE
OF NEW SPILLWAY STRUCTURE



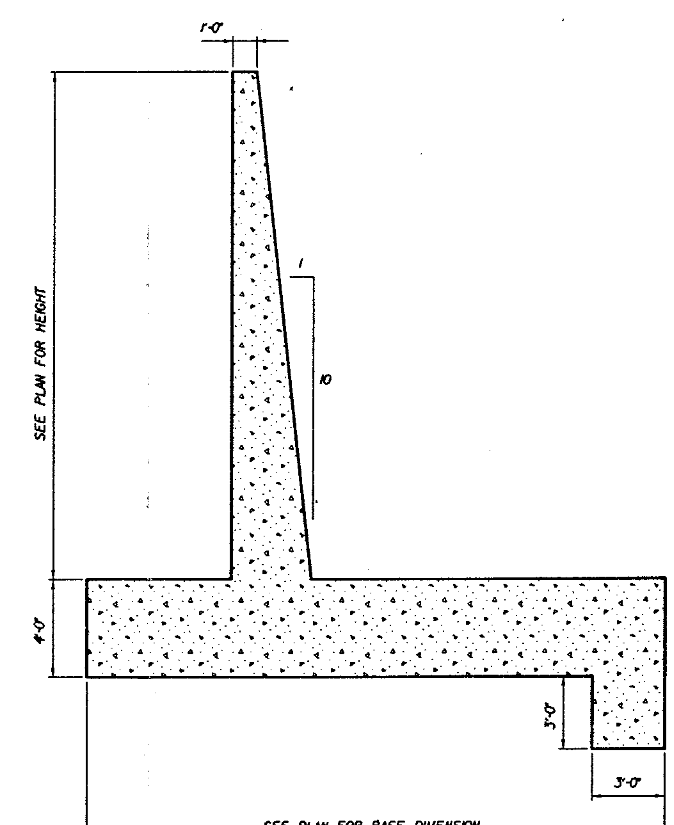
SECTION (A / S-IS-2)
SCALE 1/8" = 1'-0"



OPERATING PLATFORM
SCALE 1/8" = 1'-0"



CATWALK SECTION
SCALE 1 1/2" = 1'-0"



TYPICAL WING WALL SECTION
SCALE 1/8" = 1'-0"

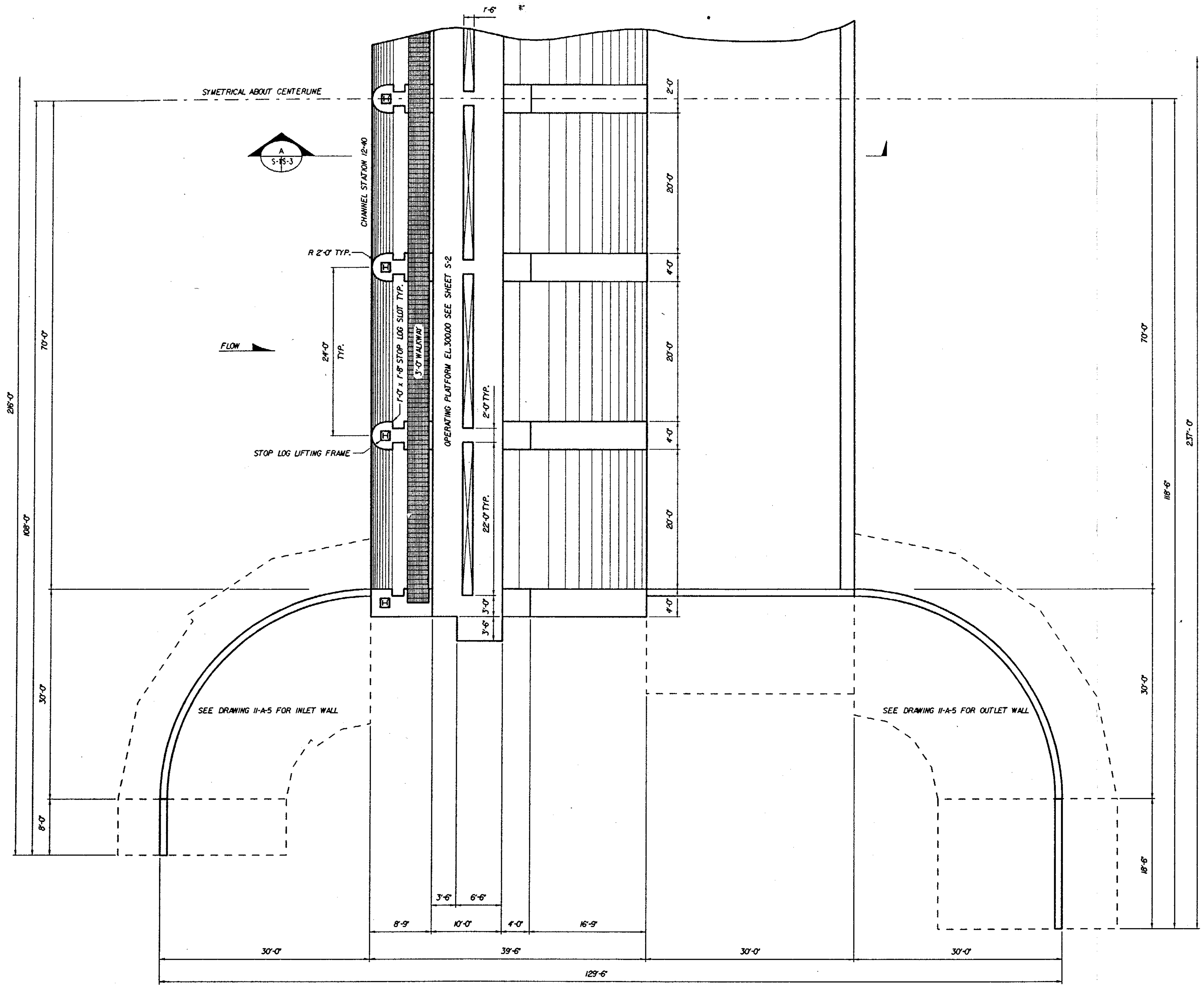
US Army Corps of Engineers Memphis District

U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
MEMPHIS, TENNESSEE

ENVIRONMENTAL RESTORATION
NATURAL RESOURCES PROJECT
MISSISSIPPI RIVER AND TRIBUTARIES
CONSTRUCTION

REELFOOT LAKE, TENNESSEE & KENTUCKY
REELFOOT LAKE ALTERNATIVE SPILLWAY
SPILLWAY SECTIONS

PLATE
I-16



PLAN

SCALE 1/16" = 1'-0"

US Army Corps of Engineers Memphis District

U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
MEMPHIS, TENNESSEE

ENVIRONMENTAL RESTORATION NATURAL RESOURCES PROJECT

MISSISSIPPI RIVER AND TRIBUTARIES CONSTRUCTION

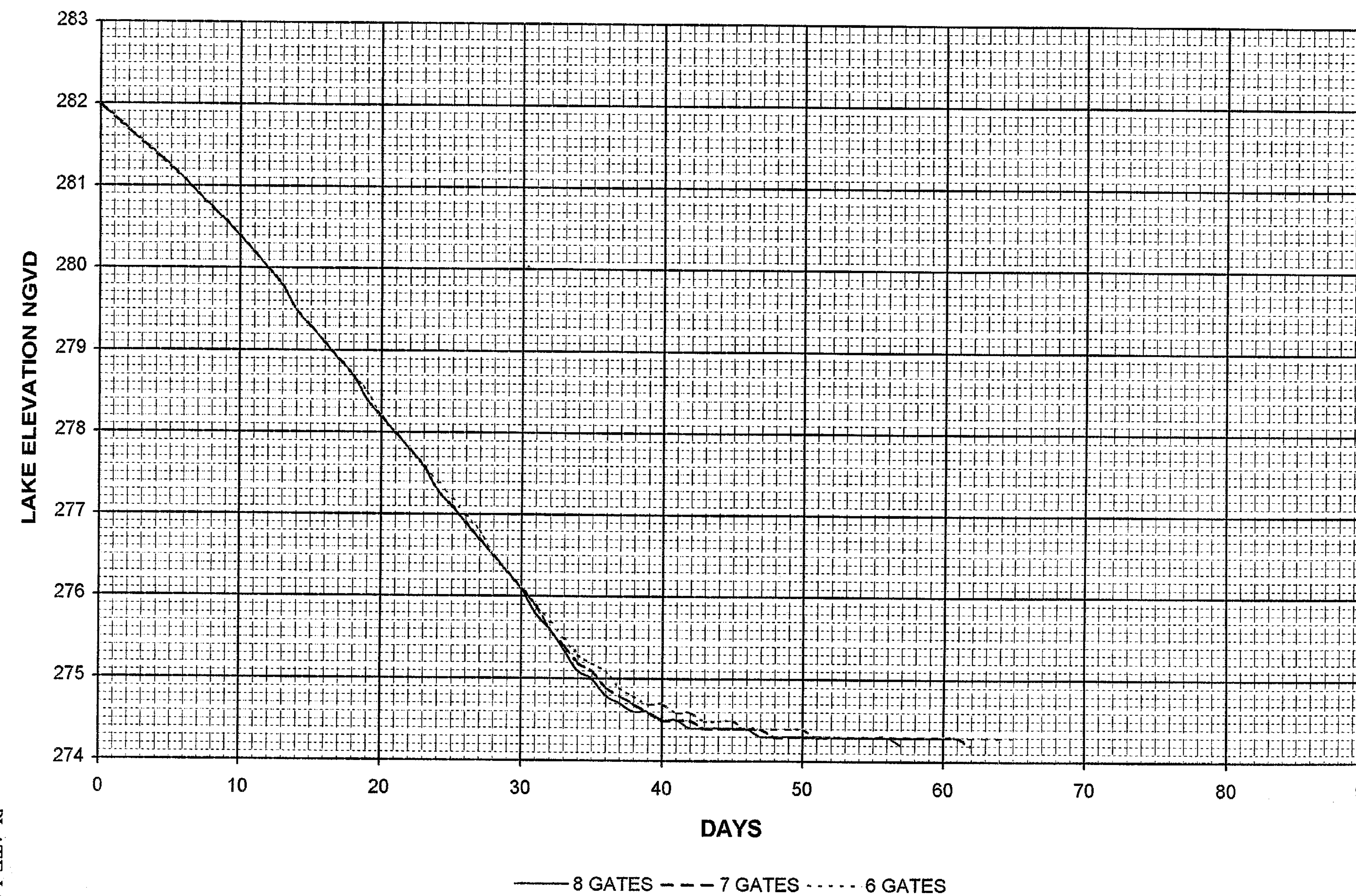
REELFOOT LAKE, TENNESSEE & KENTUCKY

REELFOOT LAKE ALTERNATIVE SPILLWAY

SPILLWAY PLAN

PLATE I-17

REELFOOT LAKE
FEASIBILITY STUDY
LAKE DRAWDOWN

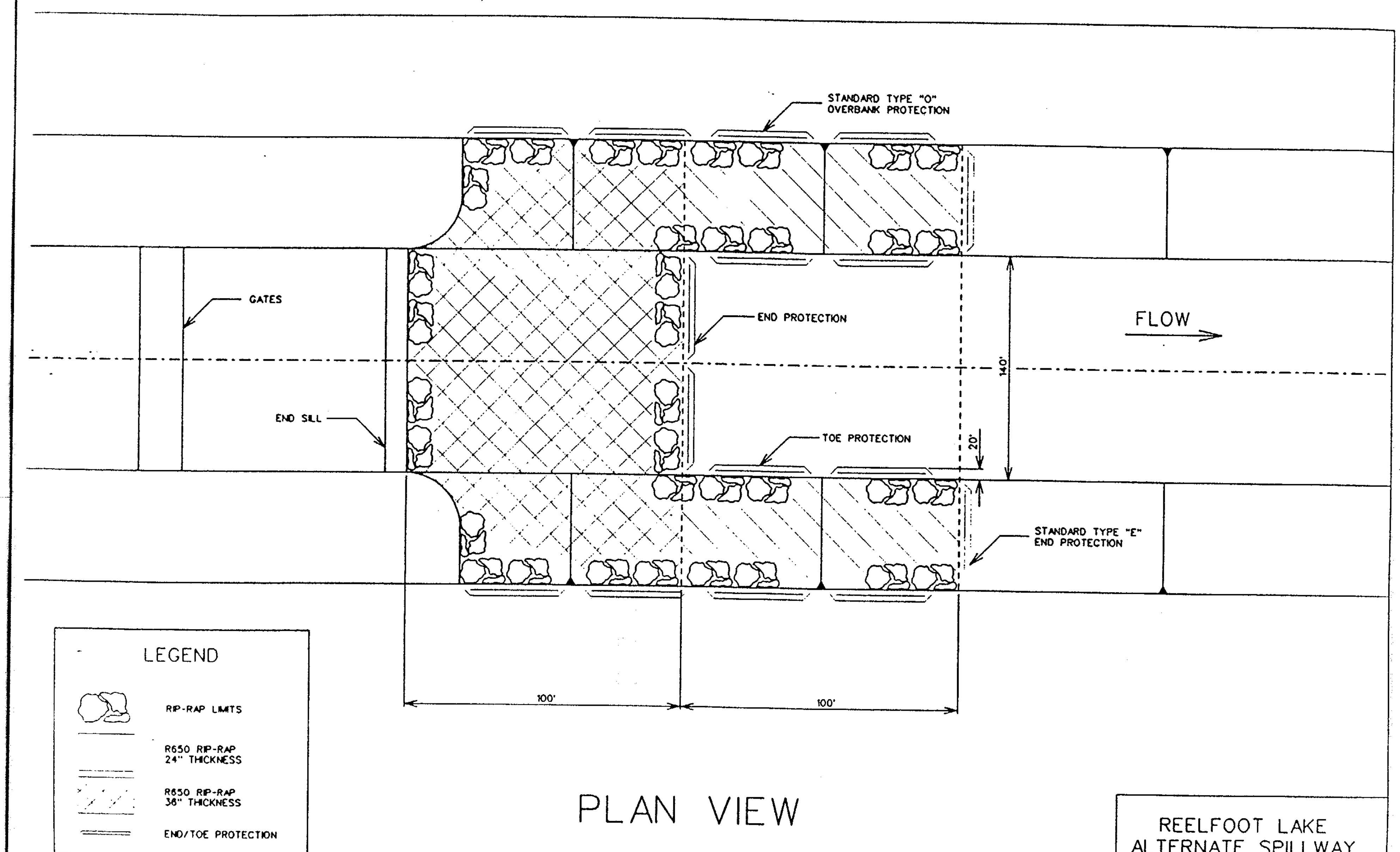


REEELFOOT LAKE, TN AND KY
POOL DRAWDOWN TABLE

VERTICAL GATE OPENING IN FEET

POOL EL.	STEP	GATE 1	GATE 2	GATE 3	GATE 4	GATE 5	GATE 6	Q (cfs)	TW EL.
282.00	1	1	0	0	0	0	0	298	273.0
	2	1	1	0	0	0	0	596	275.6
	3	1	1	1	0	0	0	813	277.4
	4	1	1	1	1	0	0	914	278.2
	5	1	1	1	1	1	0	1046	279.0
	6	1	1	1	1	1	1	1106	279.4
	7	2	1	1	1	1	1	1212	279.9
281.75	8	2	2	2	1	1	1	1294	280.3
281.50	9	2	2	2	2	2	1	1269	280.2
281.25	10	2	2	2	2	2	1	1194	279.8
281.00	11	2	2	2	2	2	1	1262	280.2
280.75	12	3	3	3	2	2	2	1205	279.9
280.50	13	3	3	3	2	2	2	1237	280.1
280.25	14	5	5	4	4	4	4	1212	279.9
280.00	15	12	12	12	12	12	12	1212	279.9
280-274.2	16	12	12	12	12	12	12	*	*
274.20	17	0	0	0	0	0	0	0	0.0

* See Drawdown Curve for discharges and tailwater elevations.



PLAN VIEW

REELFOOT LAKE
ALTERNATE SPILLWAY
RIP - RAP DETAIL
NOT TO SCALE

PLATE I-20

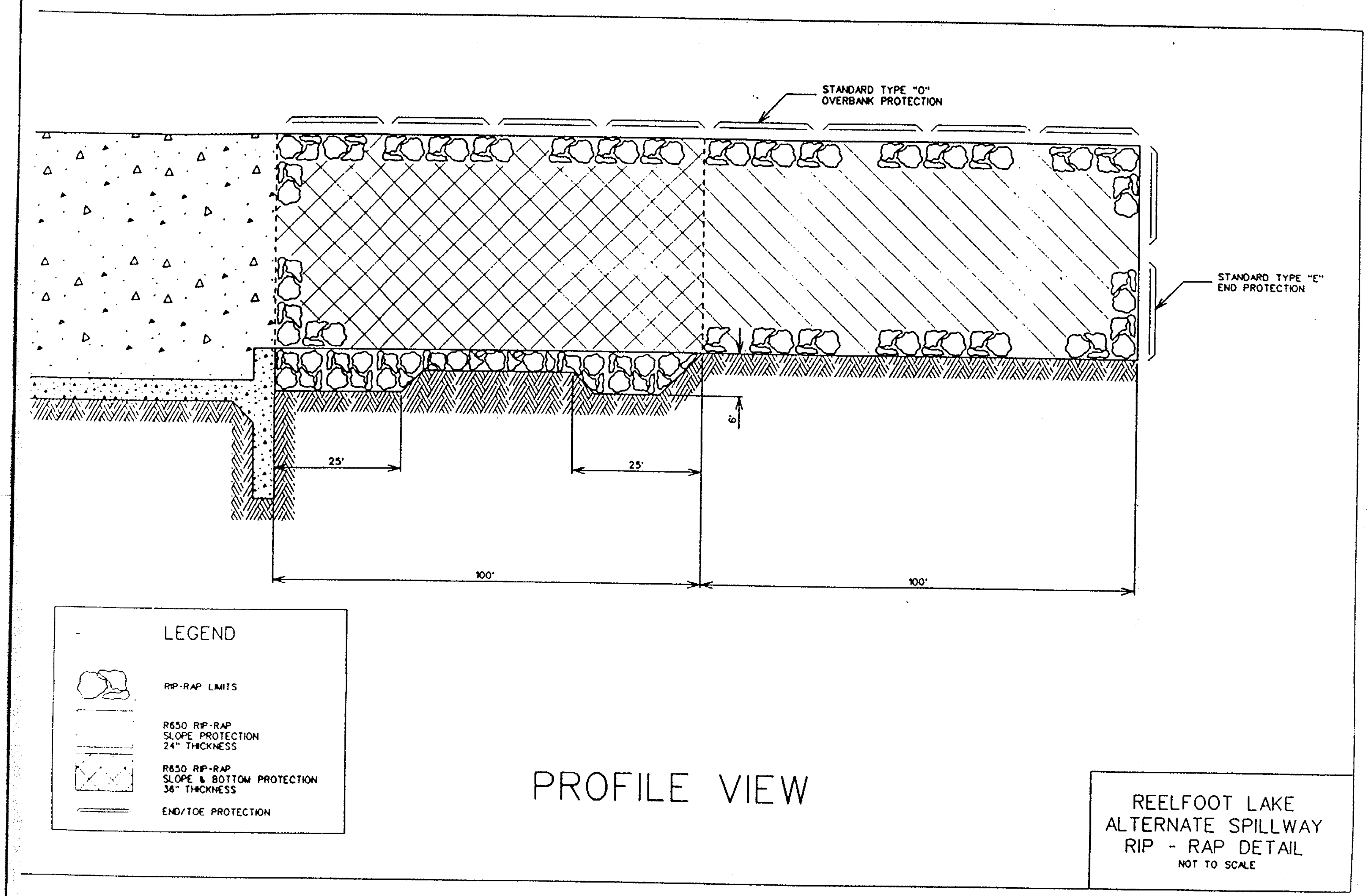

STANDARD TYPE "O"
OVERBANK PROTECTION
STANDARD TYPE "E"
END PROTECTION
6'
25'
25'
100'
100'
LEGEND
RIP-RAP LIMITS
R650 RIP-RAP
SLOPE PROTECTION
24" THICKNESS
R650 RIP-RAP
SLOPE & BOTTOM PROTECTION
36" THICKNESS
END/TOE PROTECTION
PROFILE VIEW
REELFOOT LAKE
ALTERNATE SPILLWAY
RIP - RAP DETAIL
NOT TO SCALE

PLATE
I-2

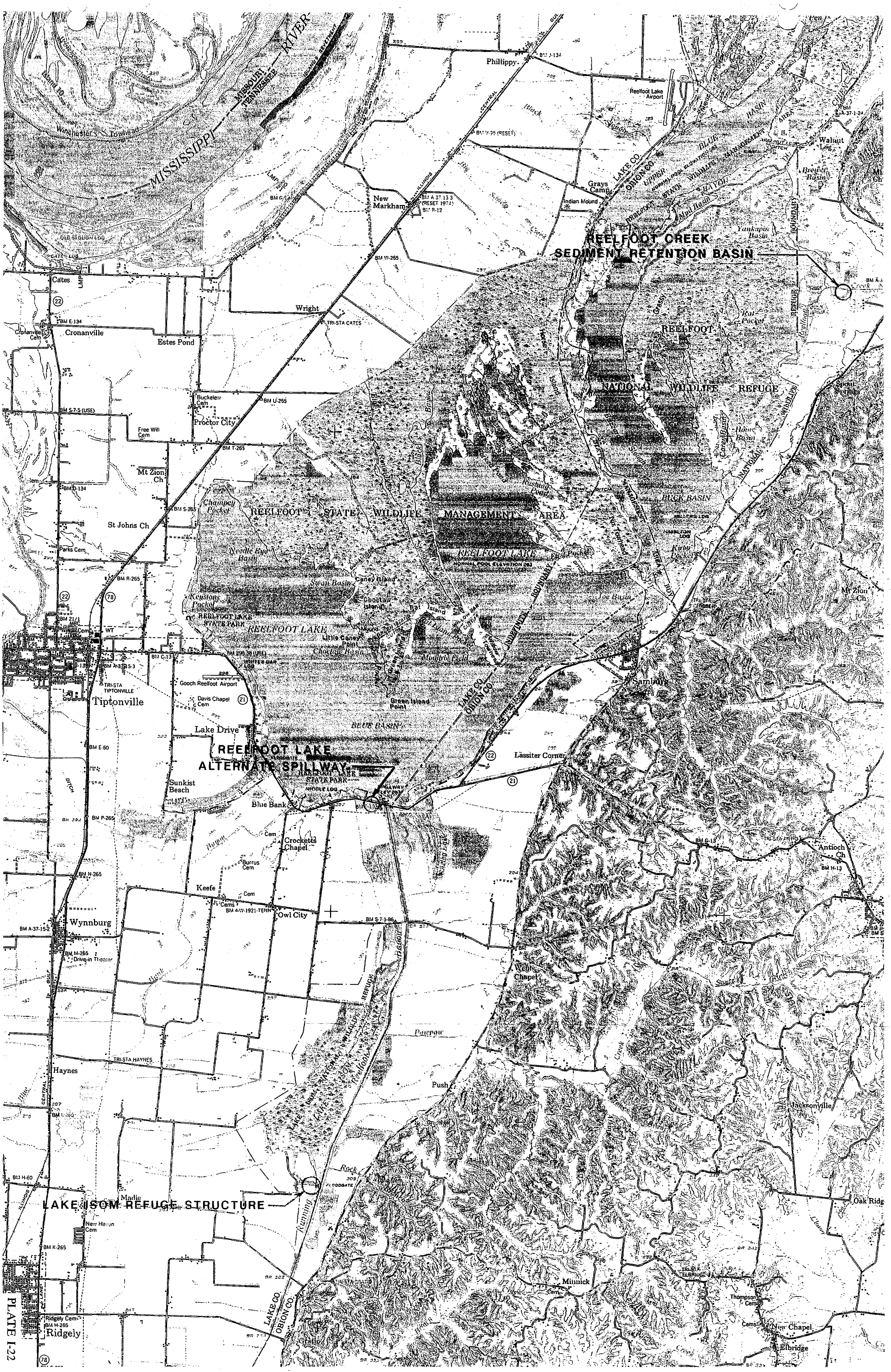

REELFOOT CREEK
SEDIMENT RETENTION BASIN
REELFOOT LAKE
ALTERNATE SPILLWAY
LAKE ISOM REFUGE STRUCTURE
MISSISSIPPI
RIVER
MISSOURI
TENNESSEE
REELFOOT NATIONAL WILDLIFE REFUGE
REELFOOT STATE WILDLIFE MANAGEMENT AREA
REELFOOT LAKE
BLUE BASIN
BUCK BASIN
LAKE CO.
OBION CO.
Tiptonville
Wynnburg
Ridgely
Cronanville
Cates
Estes Pond
Proctor City
Wright
New Markham
Phillippy
Grays Camp
Indian Mound
Reelfoot Lake Airport
Walnut Log
Samburg
Lassiter Corner
Blue Bank
Sunkist Beach
Lake Drive
Gooch Reelfoot Airport
Keefe
Owl City
Haynes
Push
Minnick
New Chapel
Elbridge
Jacksonville
Antioch Ch
Mt Zion Ch
Oak Ridg
St Johns Ch
Parks Cem
Crocketts Chapel
Burrus Cem
New Haven Cem
Ridgely Cem
Thompson Cem
Free Will Cem
Bucklew Cem
Davis Chapel Cem
Drive-in Theater
Green Island Point
Swan Basin
Caney Island
Choctaw Island
Little Caney Point
Choctaw Basin
Keystone Pocket
Needle Eye Basin
Champey Pocket
REELFOOT LAKE STATE PARK
Yankapin Basin
Brewer Basin
Rat Pocket
Kiwa Pocket
Joe Basin
Home Basin
Mud Basin
Winchester's Towhead
Island 10

PLATE I-22

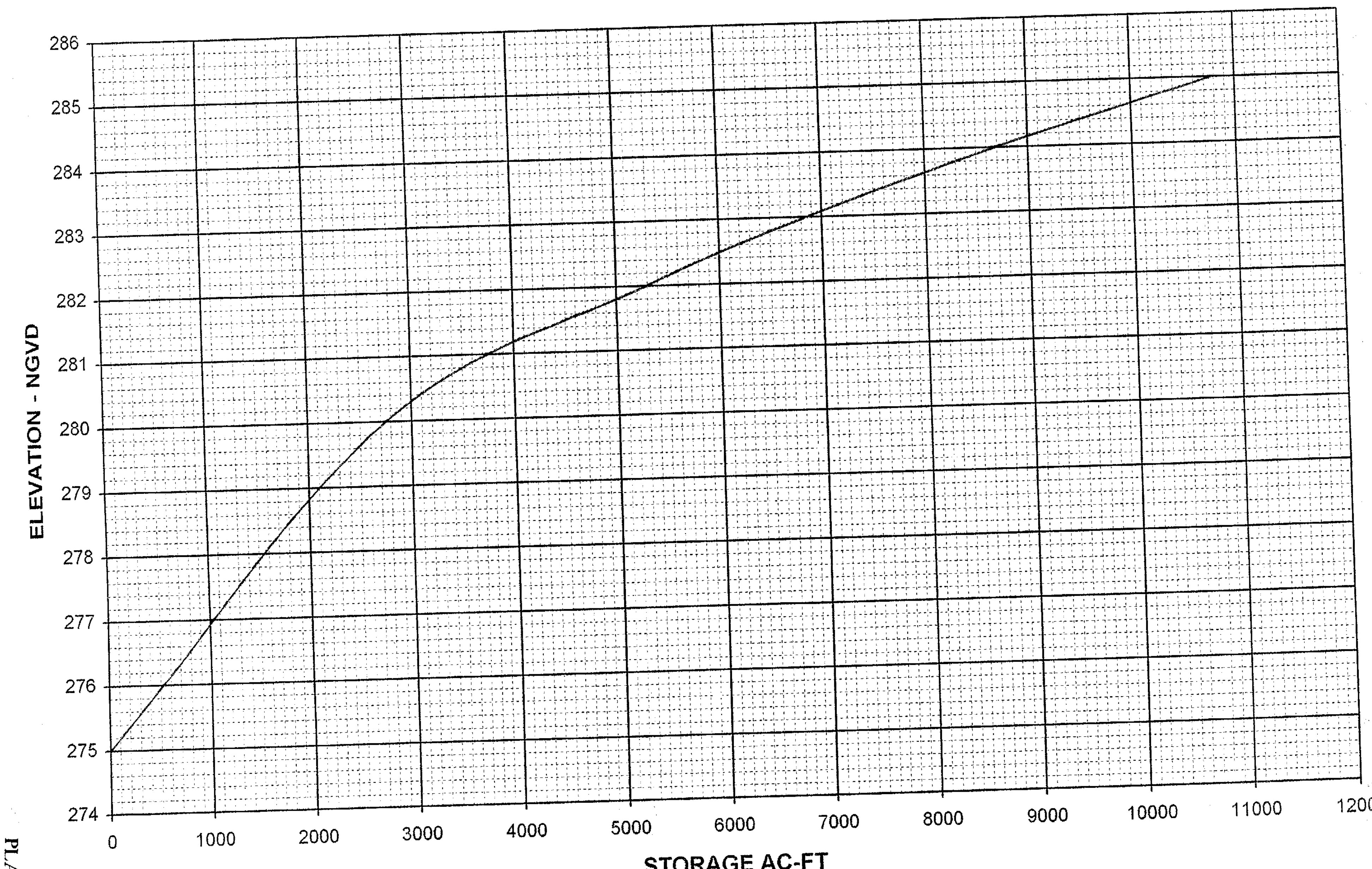
US Army Corps of Engineers Memphis District

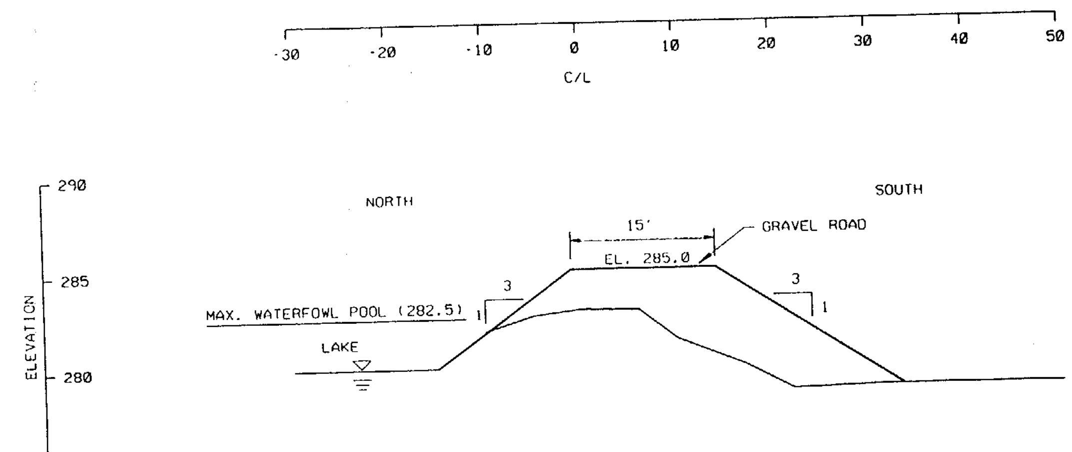
U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
MEMPHIS, TENNESSEE

REELFOOT LAKE, TENNESSEE & KENTUCKY
LAKE ISOM RESTORATION

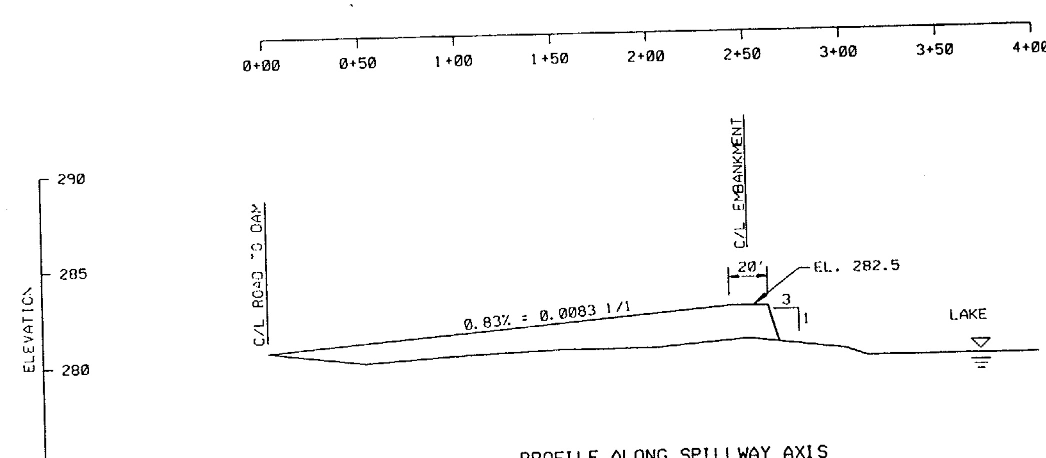
PLATE
I-23

REELFOOT LAKE FEASIBILITY STUDY
LAKE ISOM
STORAGE CURVE





EMBANKMENT SECTION @ 8+50
SECTION A-A



PROFILE ALONG SPILLWAY AXIS
(2+50 PROFILE = 1+00 EMBANKMENT)
SECTION B-B

ENVIRONMENTAL RESTORATION
NATURAL RESOURCES PROJECT

MISSISSIPPI RIVER AND TRIBUTARIES
CONSTRUCTION

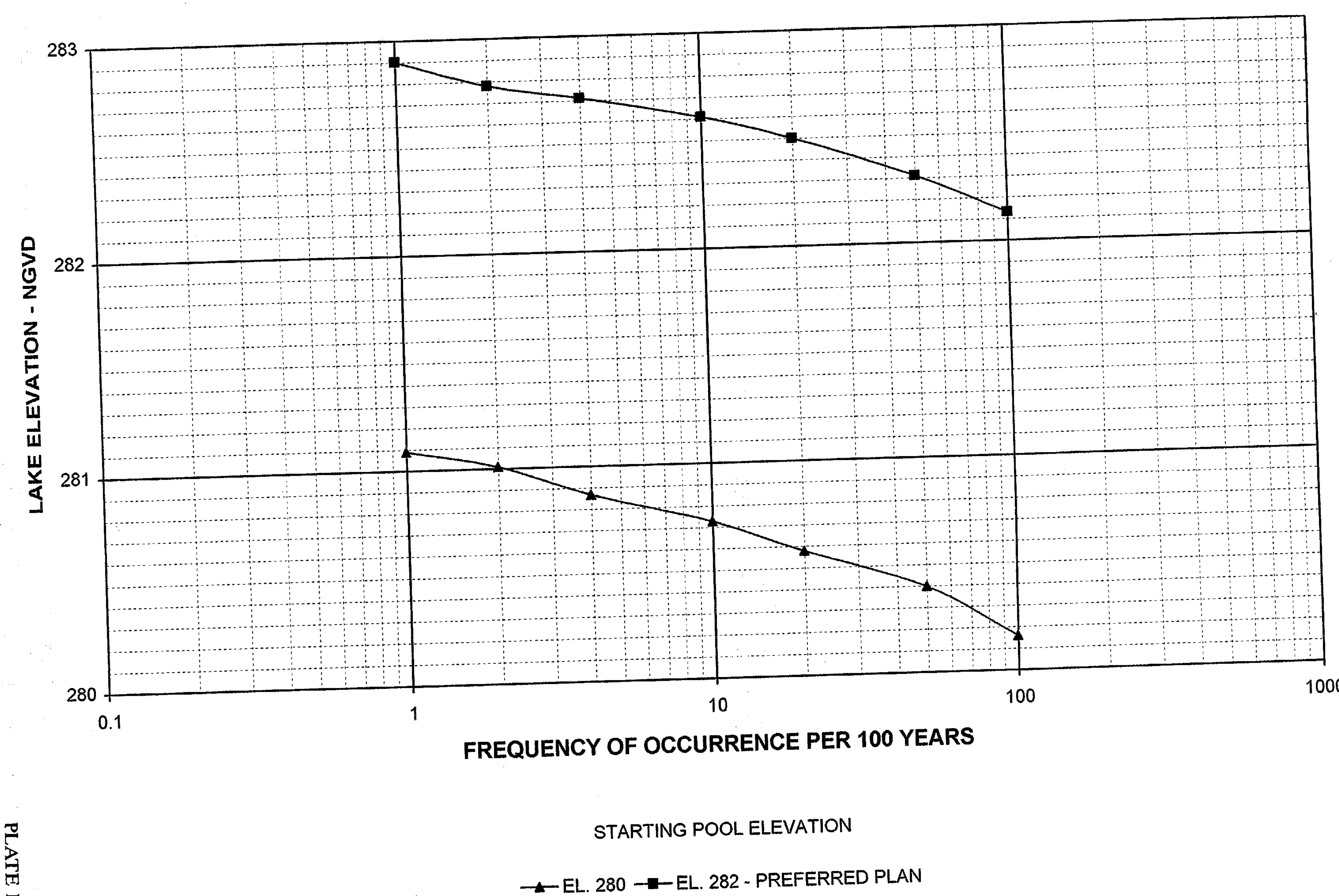
REELFOOT LAKE, TENNESSEE & KENTUCKY
LAKE ISOM RESTORATION
CROSS SECTIONS

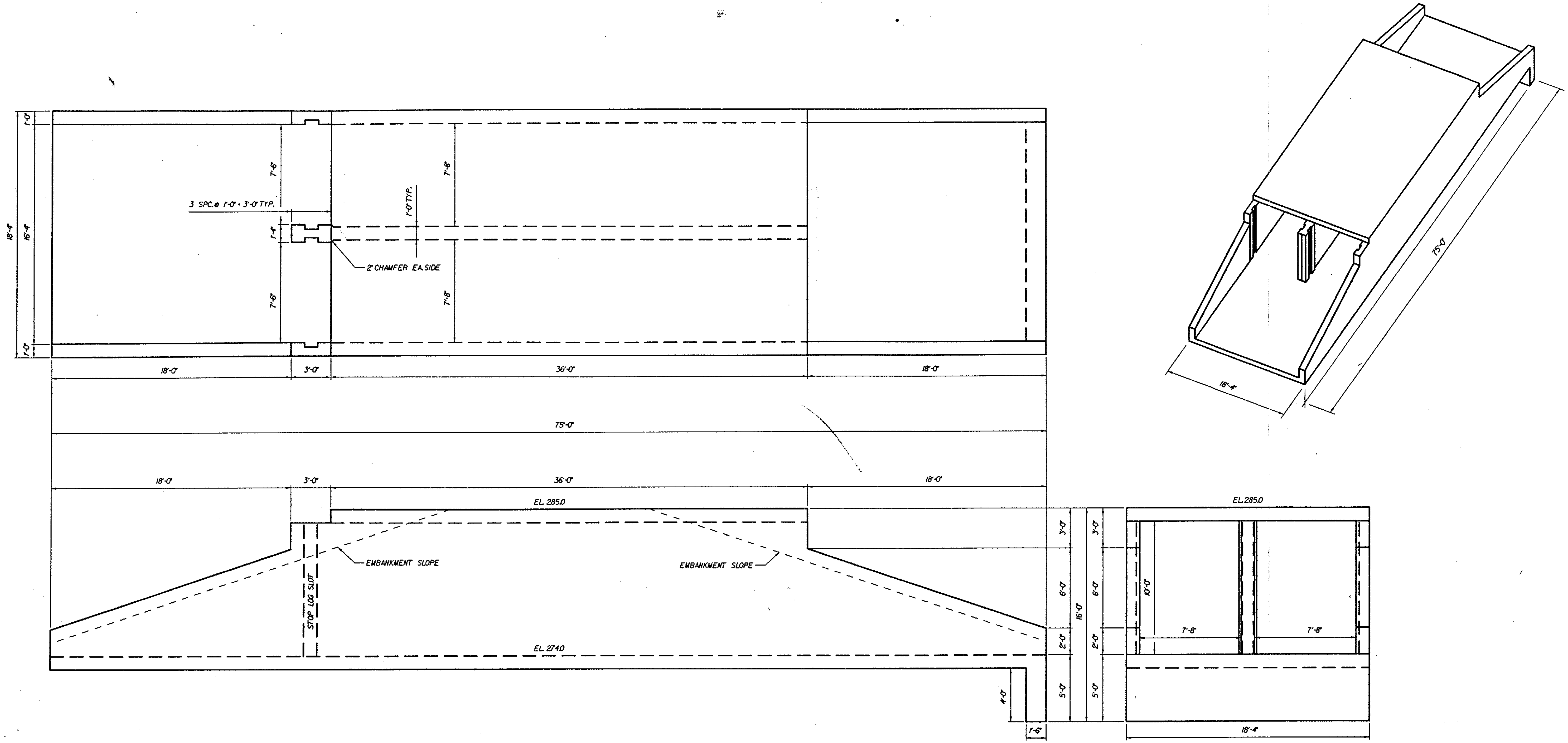
U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
MEMPHIS, TENNESSEE

US Army Corps
of Engineers
Memphis District

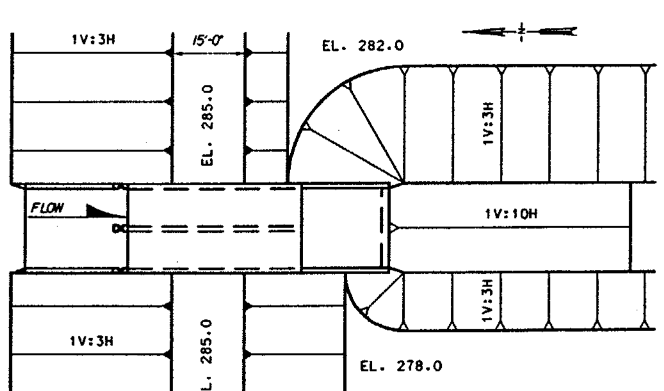
PLATE
I-25

REELFOOT LAKE FEASIBILITY STUDY
LAKE ISOM FEATURE
ELEVATION VS. FREQUENCY





OUTLET STRUCTURE
SCALE 1/8" = 1'-0"



SITE PLAN
SCALE 1" = 40'-0"

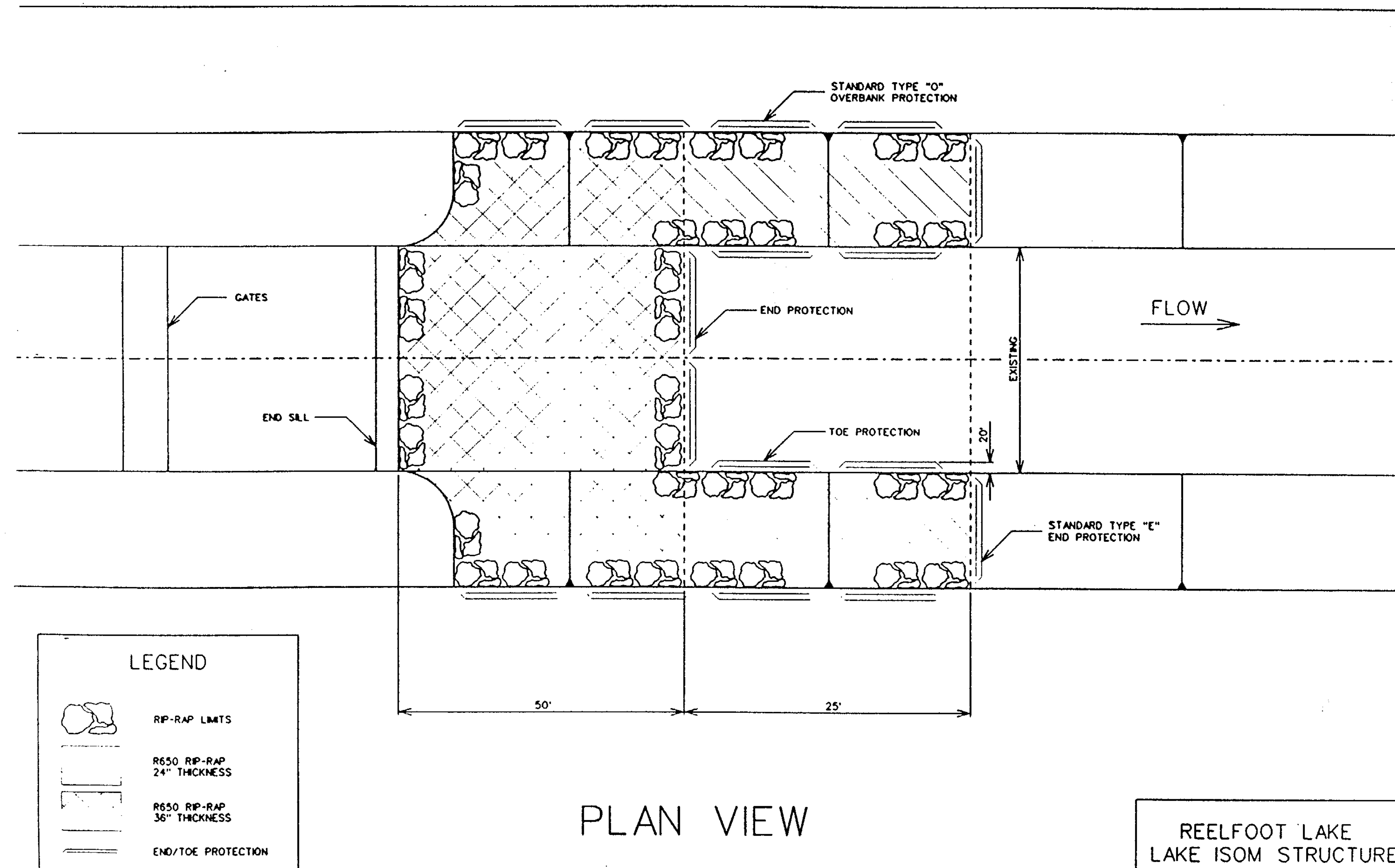
US Army Corps of Engineers Memphis District

U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
MEMPHIS, TENNESSEE

MISSISSIPPI RIVER AND TRIBUTARIES CONSTRUCTION
ENVIRONMENTAL RESTORATION NATURAL RESOURCES PROJECT

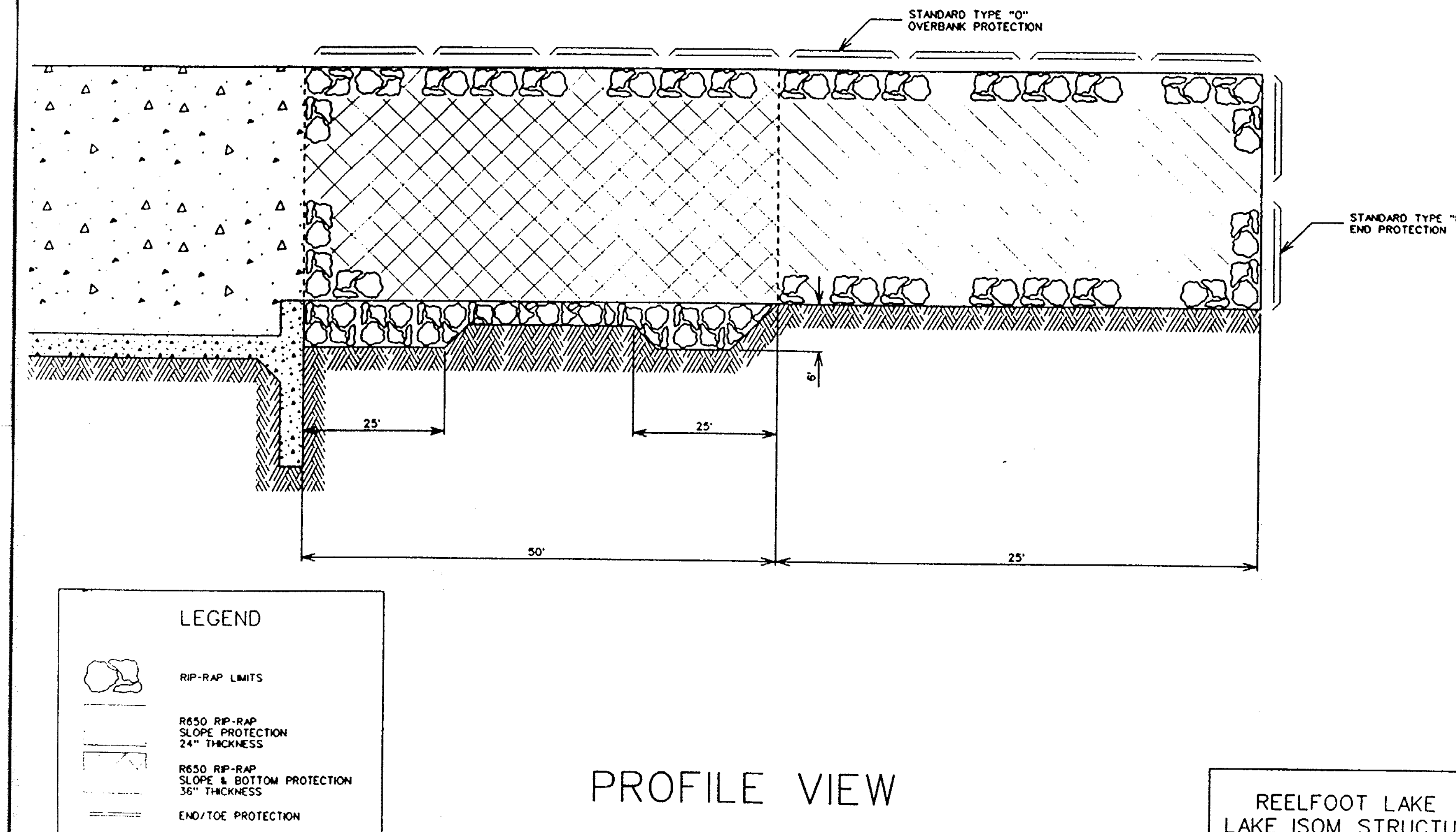
REELFOOT LAKE, TENNESSEE & KENTUCKY
LAKE ISOM RESTORATION
OUTLET STRUCTURE

PLATE I-27



PLAN VIEW

REELFOOT LAKE
LAKE ISOM STRUCTURE
RIP - RAP DETAIL
NOT TO SCALE



PROFILE VIEW

REELFOOT LAKE
LAKE ISOM STRUCTURE
RIP - RAP DETAIL
NOT TO SCALE

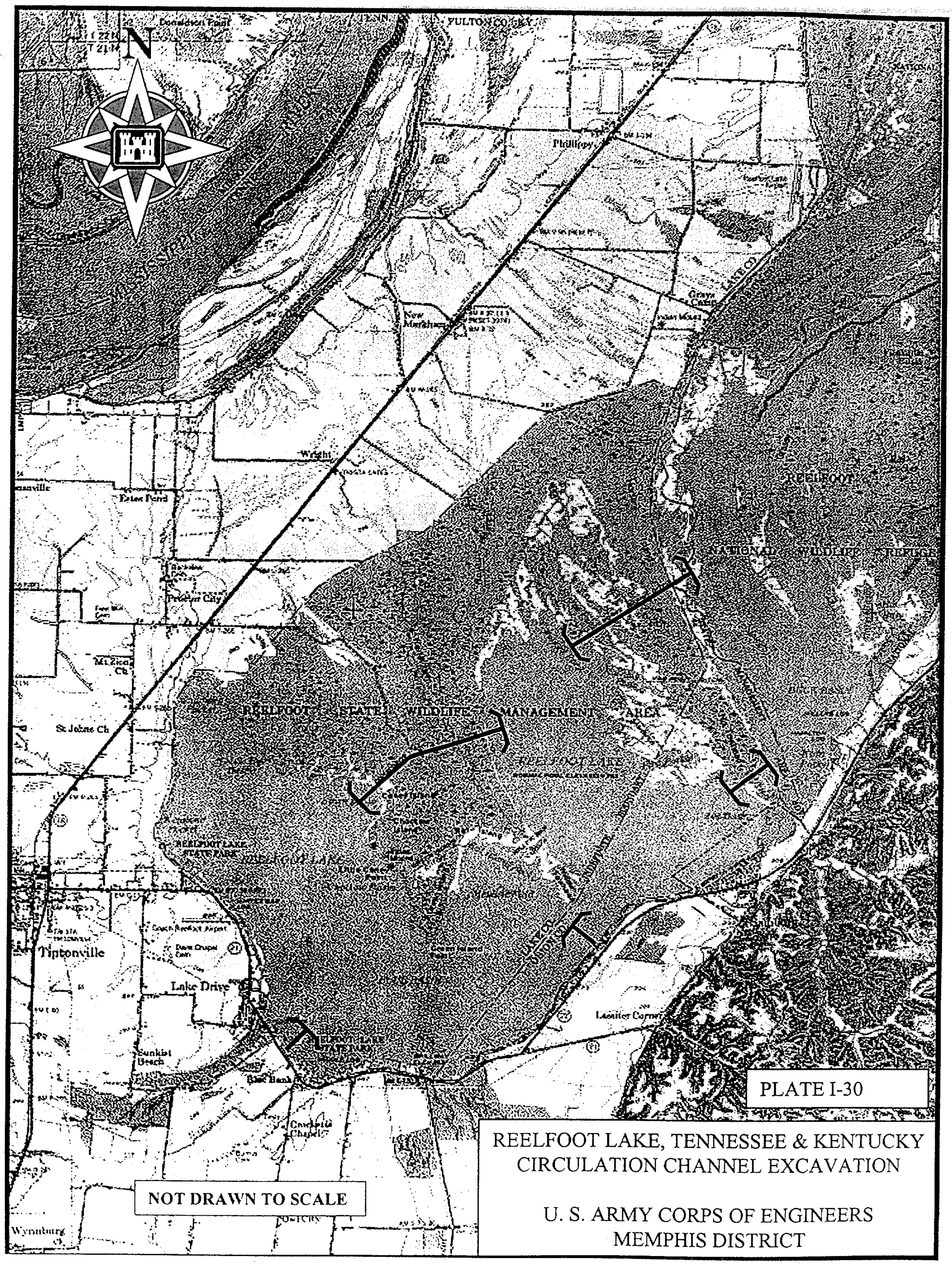


PLATE I-30

REELFOOT LAKE, TENNESSEE & KENTUCKY
CIRCULATION CHANNEL EXCAVATION

U. S. ARMY CORPS OF ENGINEERS
MEMPHIS DISTRICT

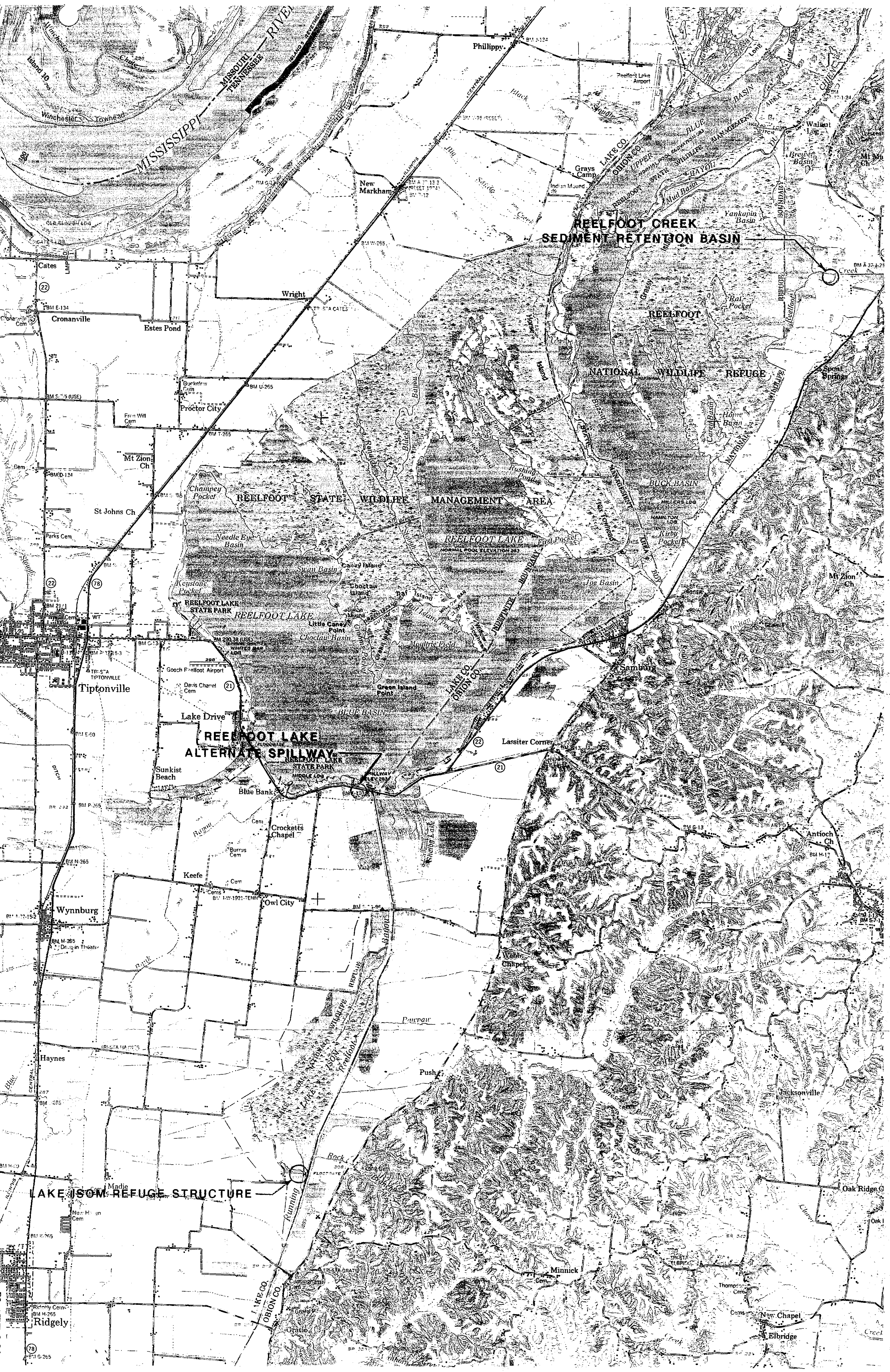

REELFOOT CREEK
SEDIMENT RETENTION BASIN
REELFOOT LAKE
ALTERNATE SPILLWAY
LAKE ISOM REFUGE STRUCTURE
REELFOOT NATIONAL WILDLIFE REFUGE
REELFOOT STATE WILDLIFE MANAGEMENT AREA
REELFOOT LAKE
REELFOOT LAKE STATE PARK
MISSISSIPPI RIVER
MISSOURI
TENNESSEE
LAKE CO.
OBION CO.
Tiptonville
Ridgely
Wynnburg
Haynes
Cronanville
Cates
Wright
Phillippy
New Markham
Proctor City
Samburg
Lassiter Corner
Blue Bank
Lake Drive
Sunkist Beach
Keefe
Owl City
Crocketts Chapel
Jacksonville
Minnick
Antioch Ch
Webb Chapel
Push
Oak Ridge
New Chapel
Elbridge
Spout Springs
Mt Zion Ch
Walnut Log
Grays Camp
Island 10

PLATE I-31

PLATE I-32

ENVIRONMENTAL RESTORATION NATURAL RESOURCES PROJECT — MISSISSIPPI RIVER AND TRIBUTARIES CONSTRUCTION

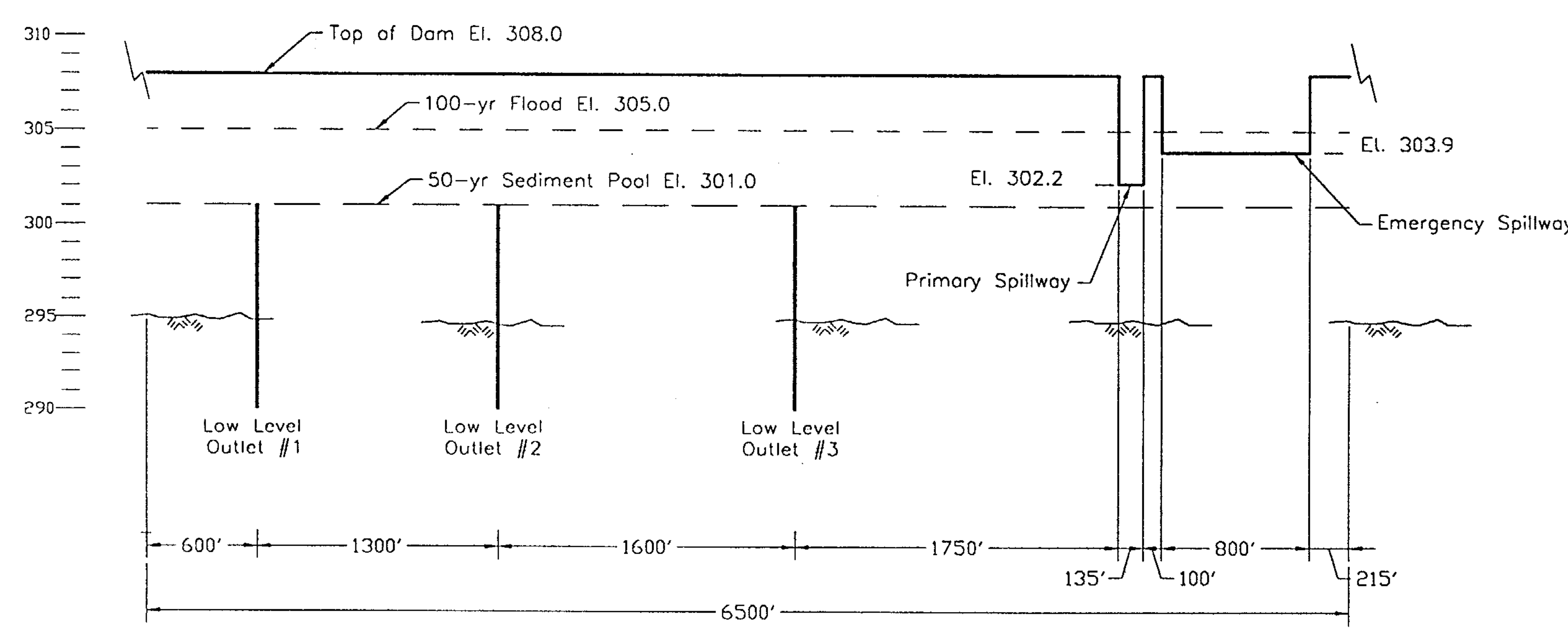
REELFOOT LAKE, TENNESSEE & KENTUCKY

SEDIMENT RETENTION BASIN

US ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
MEMPHIS, TENNESSEE

US Army Corps of Engineers
Memphis District

REEELFOOT LAKE SEDIMENT RETENTION STRUCTURE
PROFILE OF UPSTREAM FACE OF DAM



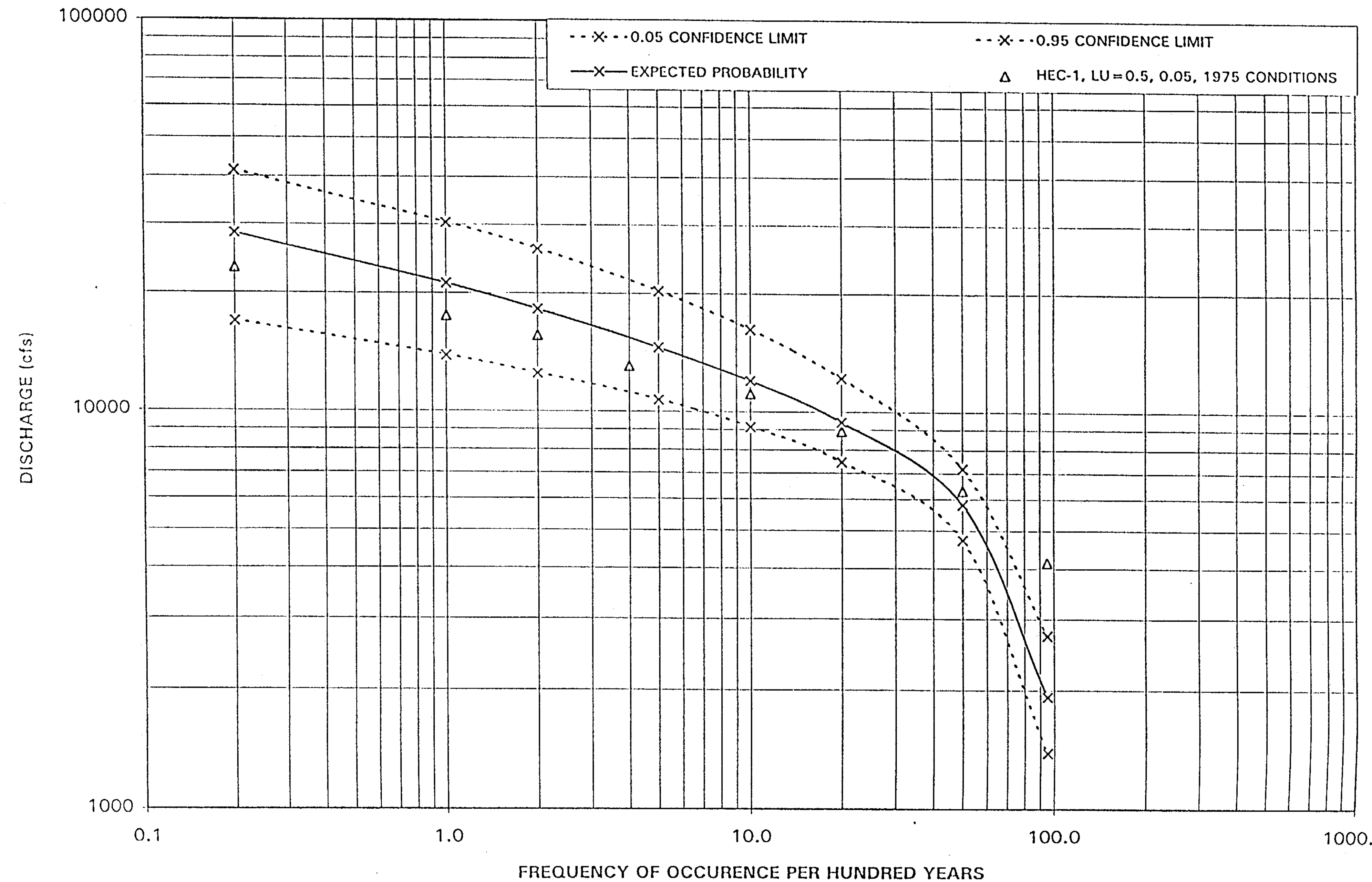
Typical Low Level Outlet:
2-72" Culverts (invert at El. 290.0)
w/ inlet structure at El. 301.0

Scale

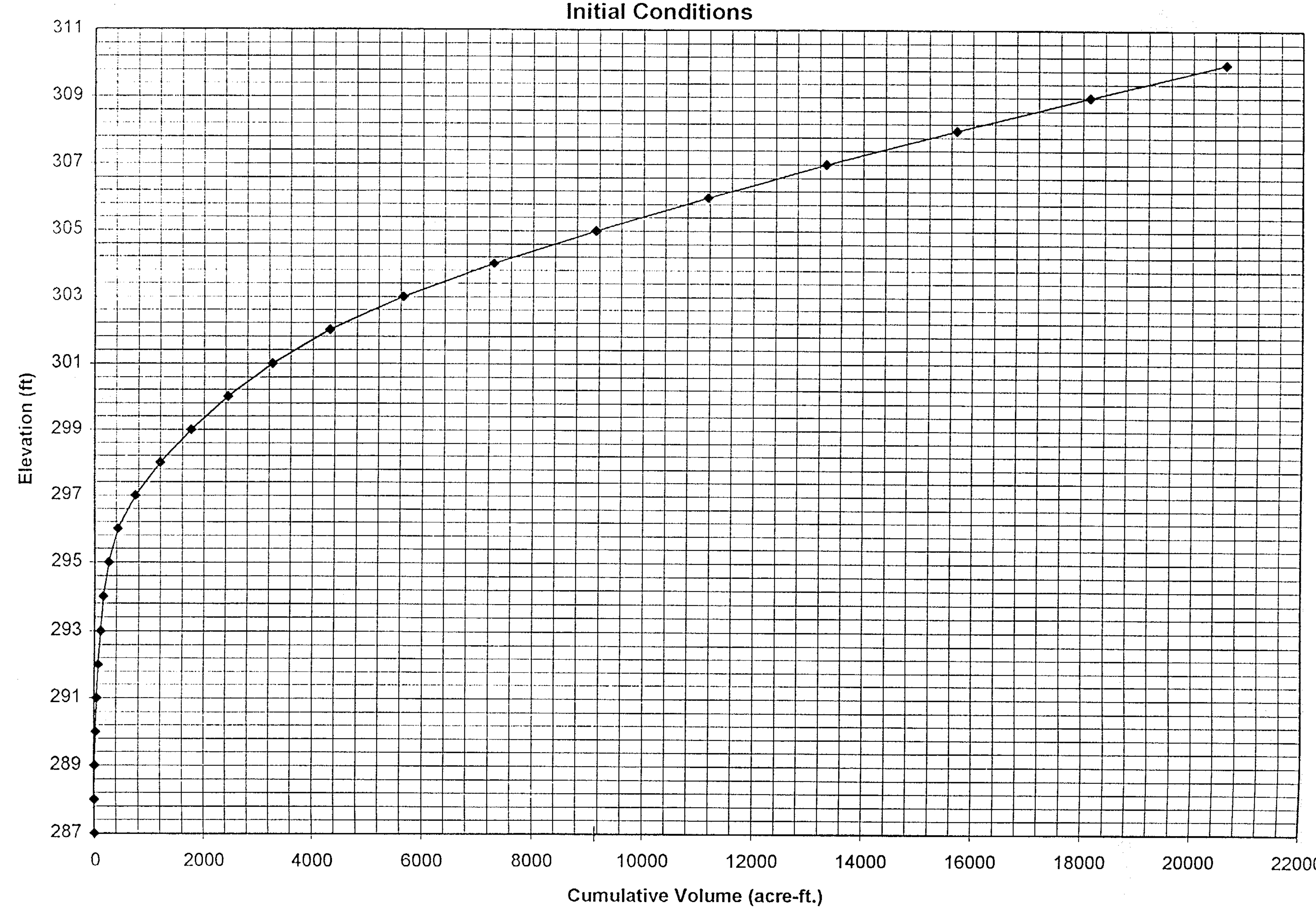
Horiz 1"=1,000'
Vert 1"=10'

WEST Consultants, Inc.		
Reelfoot Lake TN & KY Feasibility Study		
May 19, 1997	Sheet 1 of 1	RETENSTR.DWG

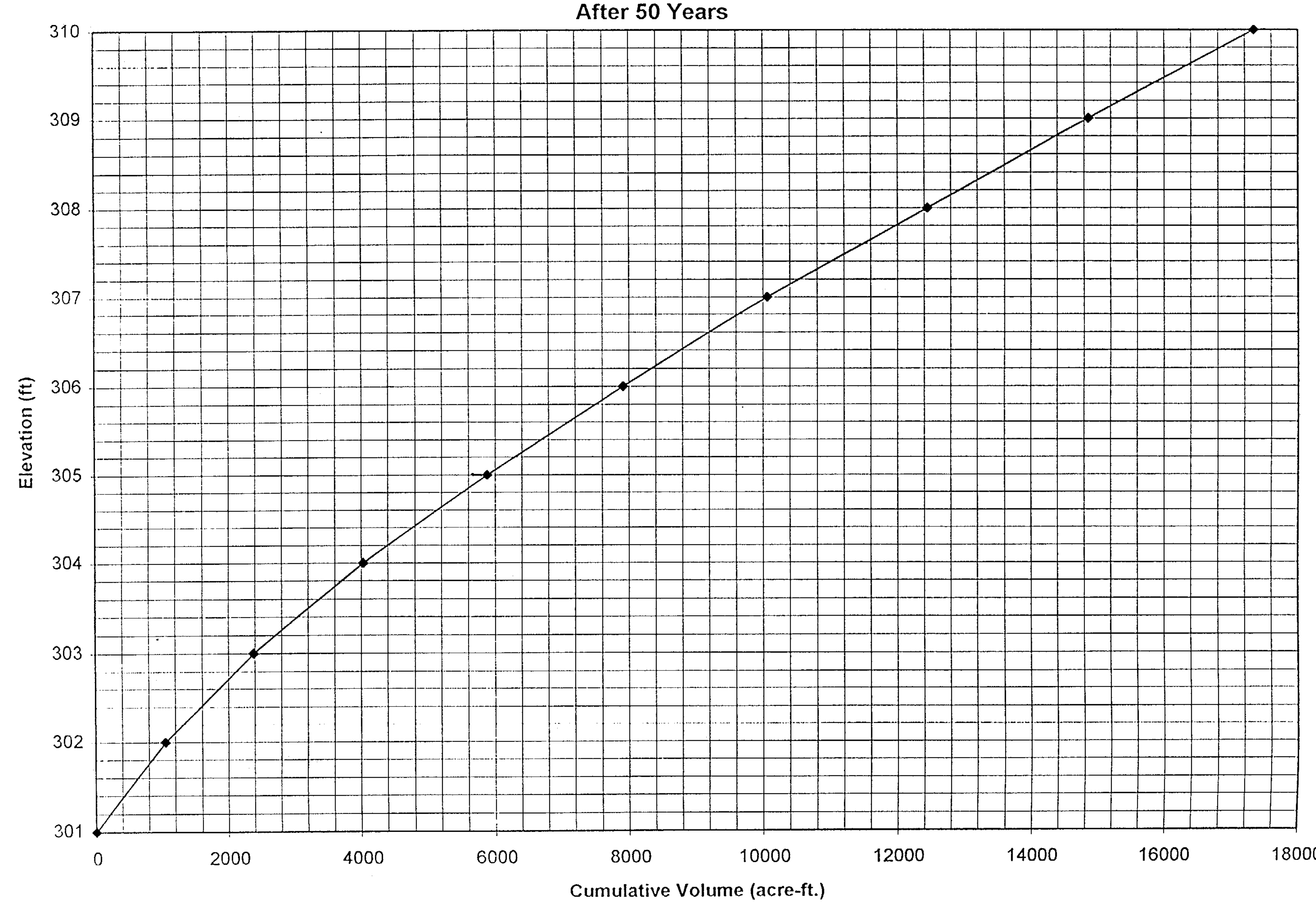
REELFOOT LAKE ASIBILITY STUDY
REELFOOT CREEK NEAR SAMBURG
HEC-1 FREQUENCY CALIBRATION



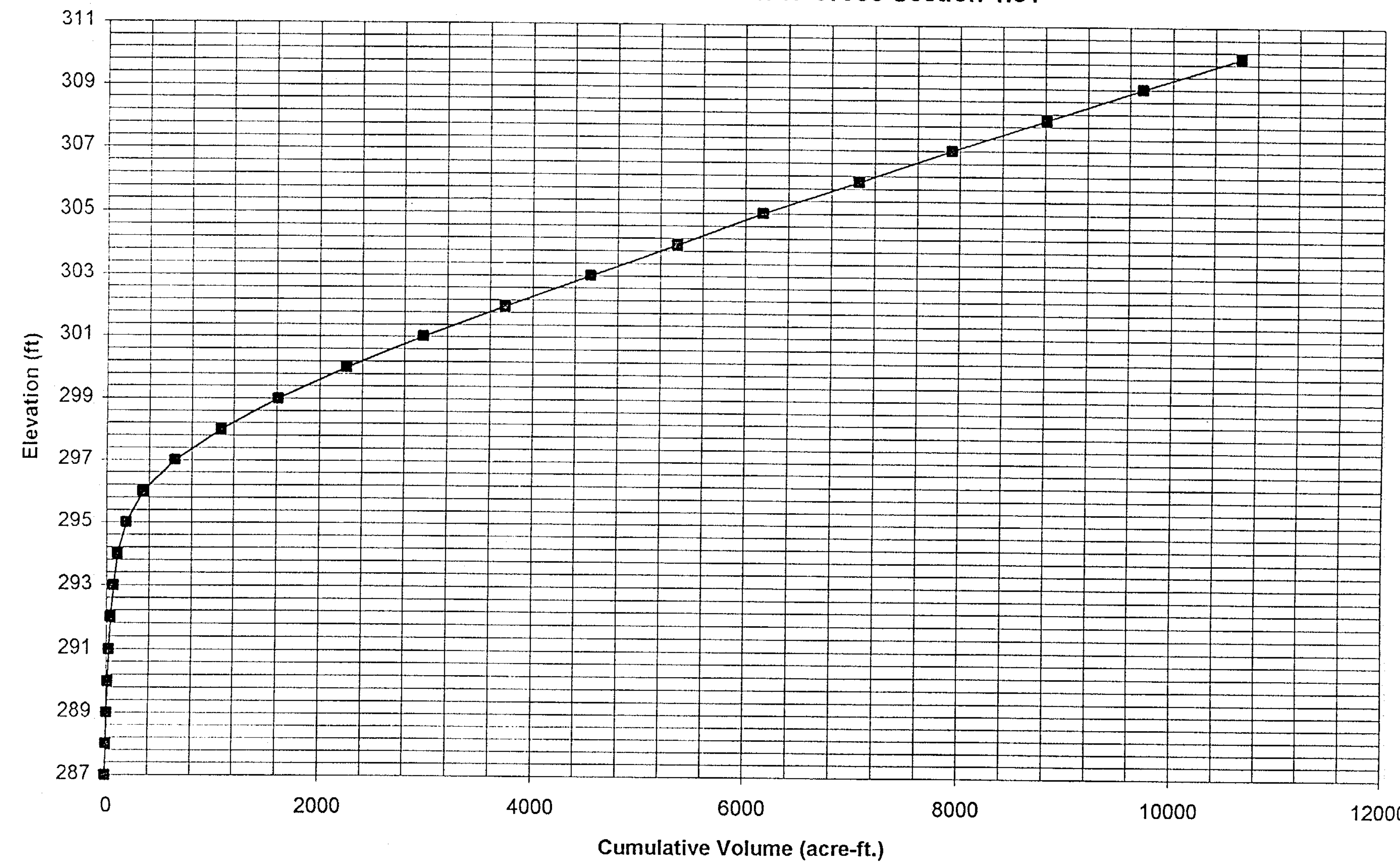
REELFOOT LAKE SEDIMENT RETENTION STRUCTURE
Cumulative Volume vs. Elevation
Initial Conditions



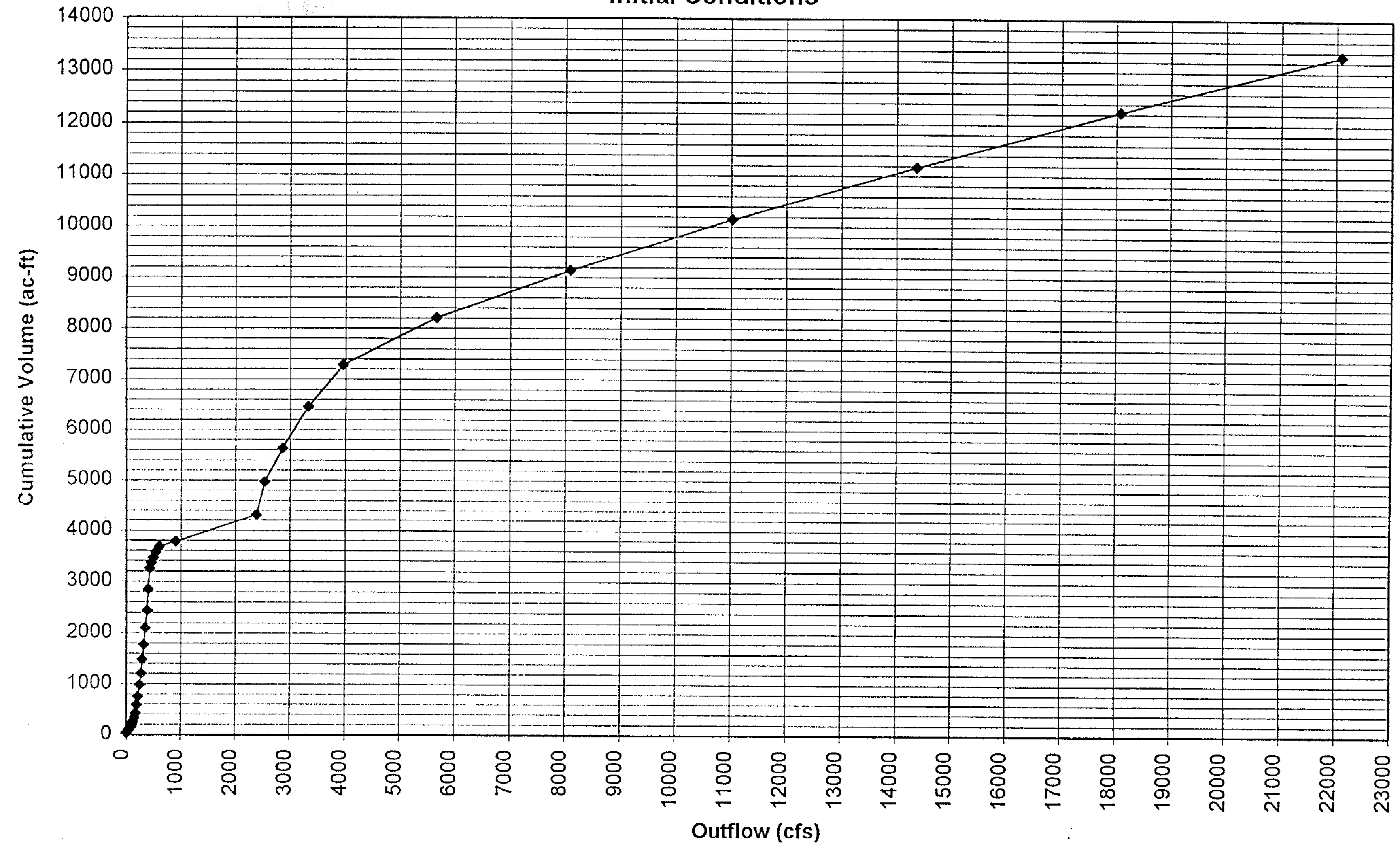
REELFOOT LAKE SEDIMENT RETENTION STRUCTURE
Cumulative Volume vs. Elevation
After 50 Years

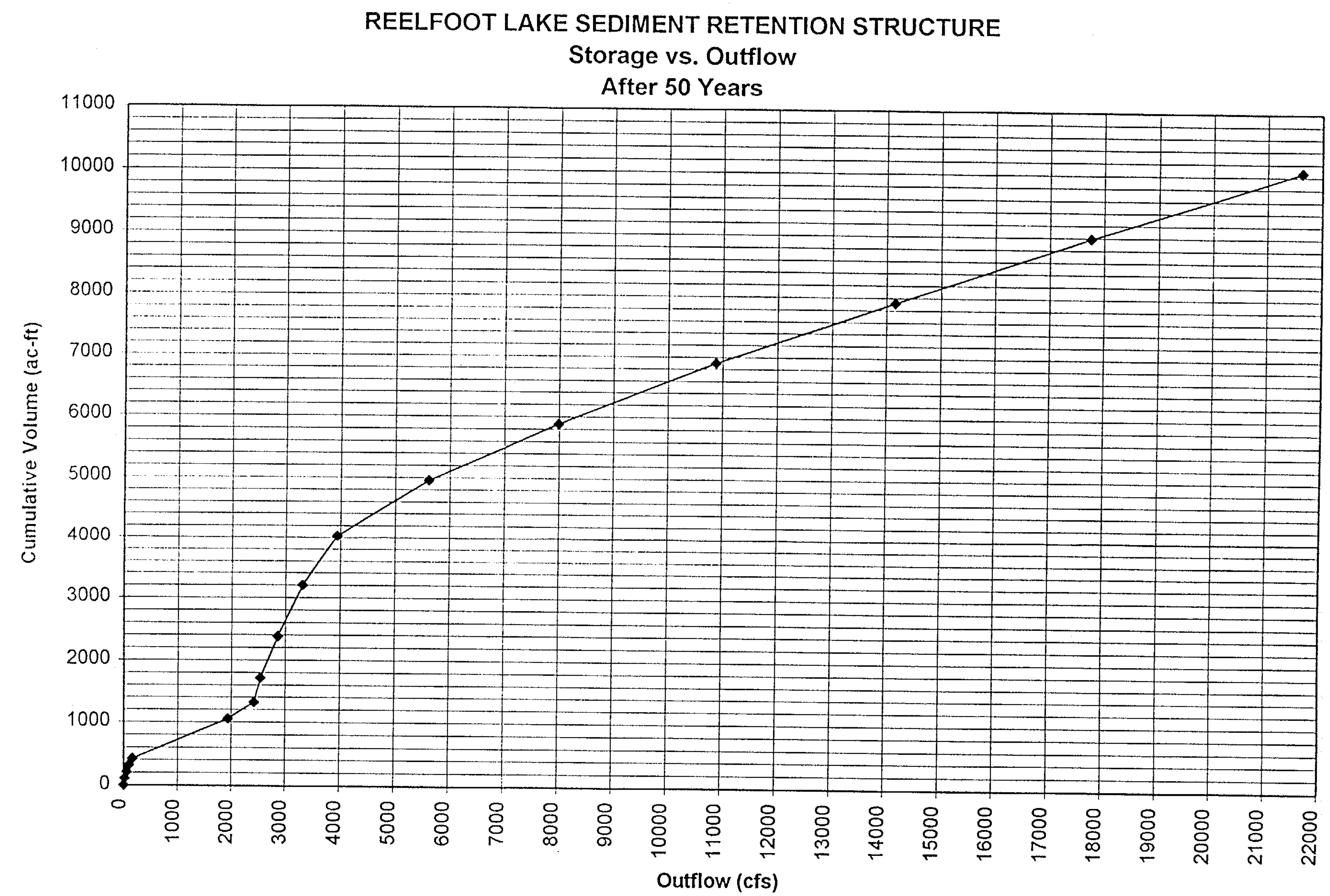


REELFOOT LAKE SEDIMENT RETENTION STRUCTURE
Cumulative Volume vs. Elevation to Cross Section 1.31

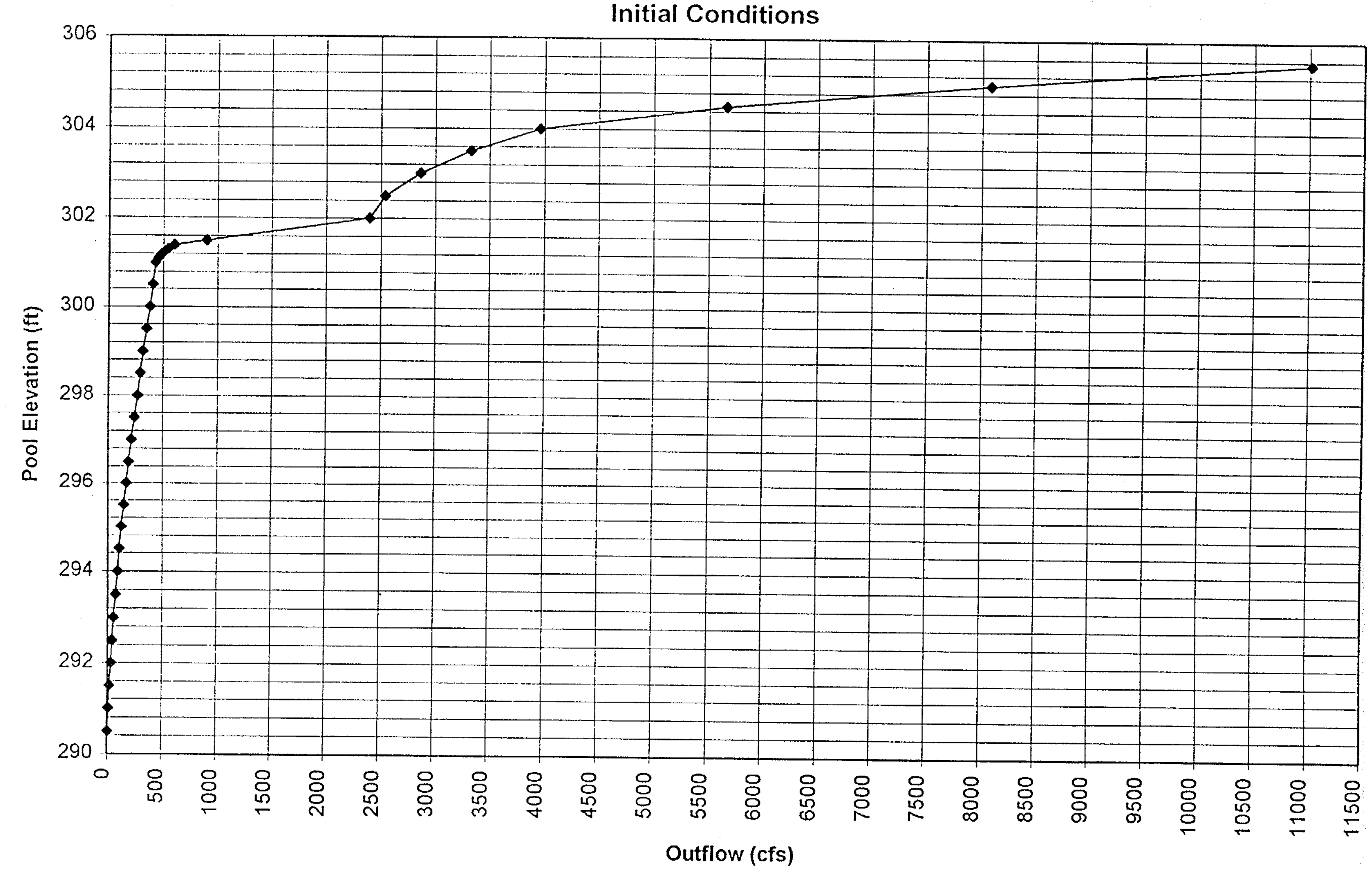


REELFOOT LAKE SEDIMENT RETENTION STRUCTURE
Storage vs. Outflow
Initial Conditions

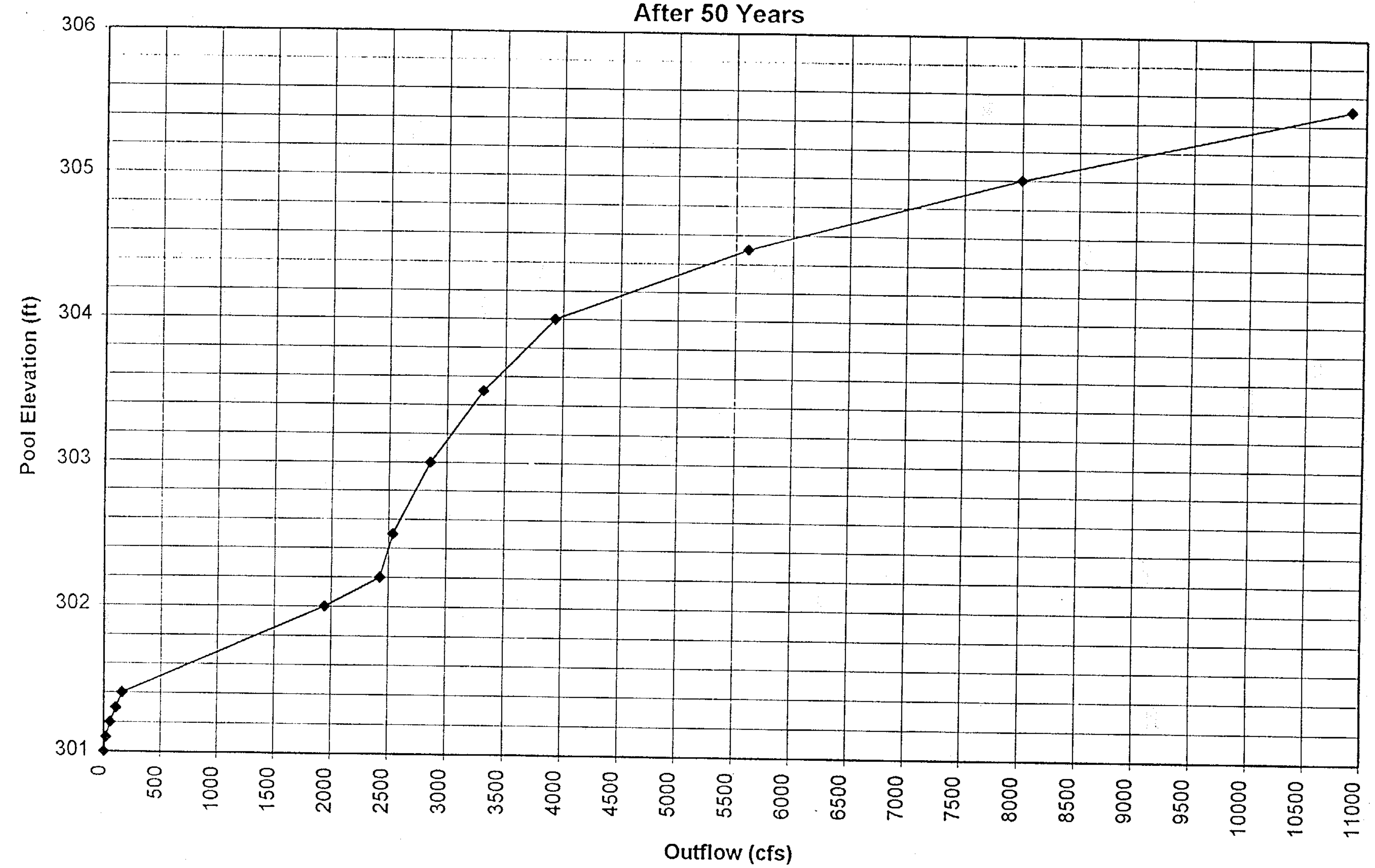



REELFOOT LAKE SEDIMENT RETENTION STRUCTURE
Storage vs. Outflow
After 50 Years
Cumulative Volume (ac-ft)
0
1000
2000
3000
4000
5000
6000
7000
8000
9000
10000
11000
Outflow (cfs)
0
1000
2000
3000
4000
5000
6000
7000
8000
9000
10000
11000
12000
13000
14000
15000
16000
17000
18000
19000
20000
21000
22000

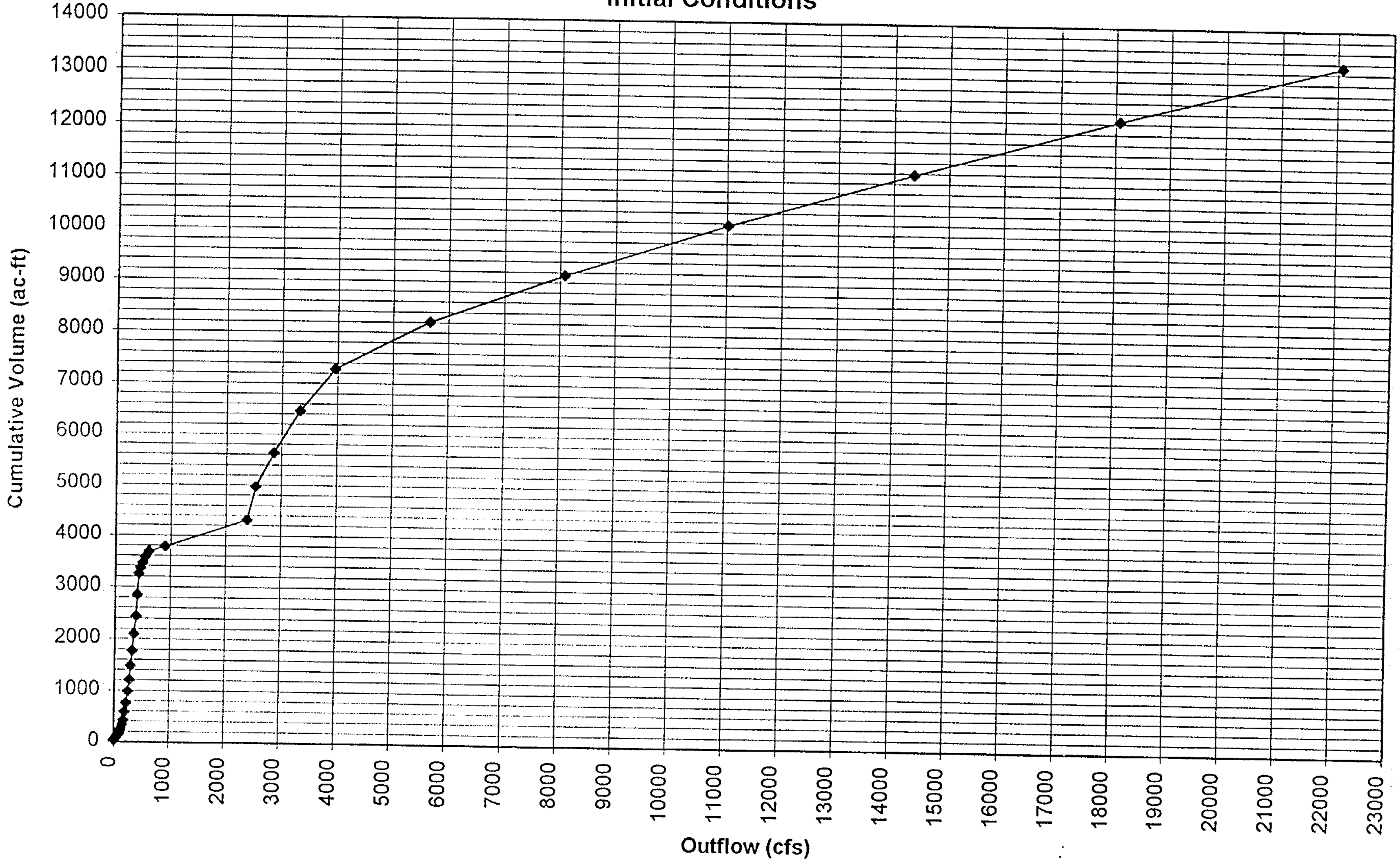
REELFOOT LAKE SEDIMENT RETENTION STRUCTURE
Pool Elevation vs. Outflow
Initial Conditions



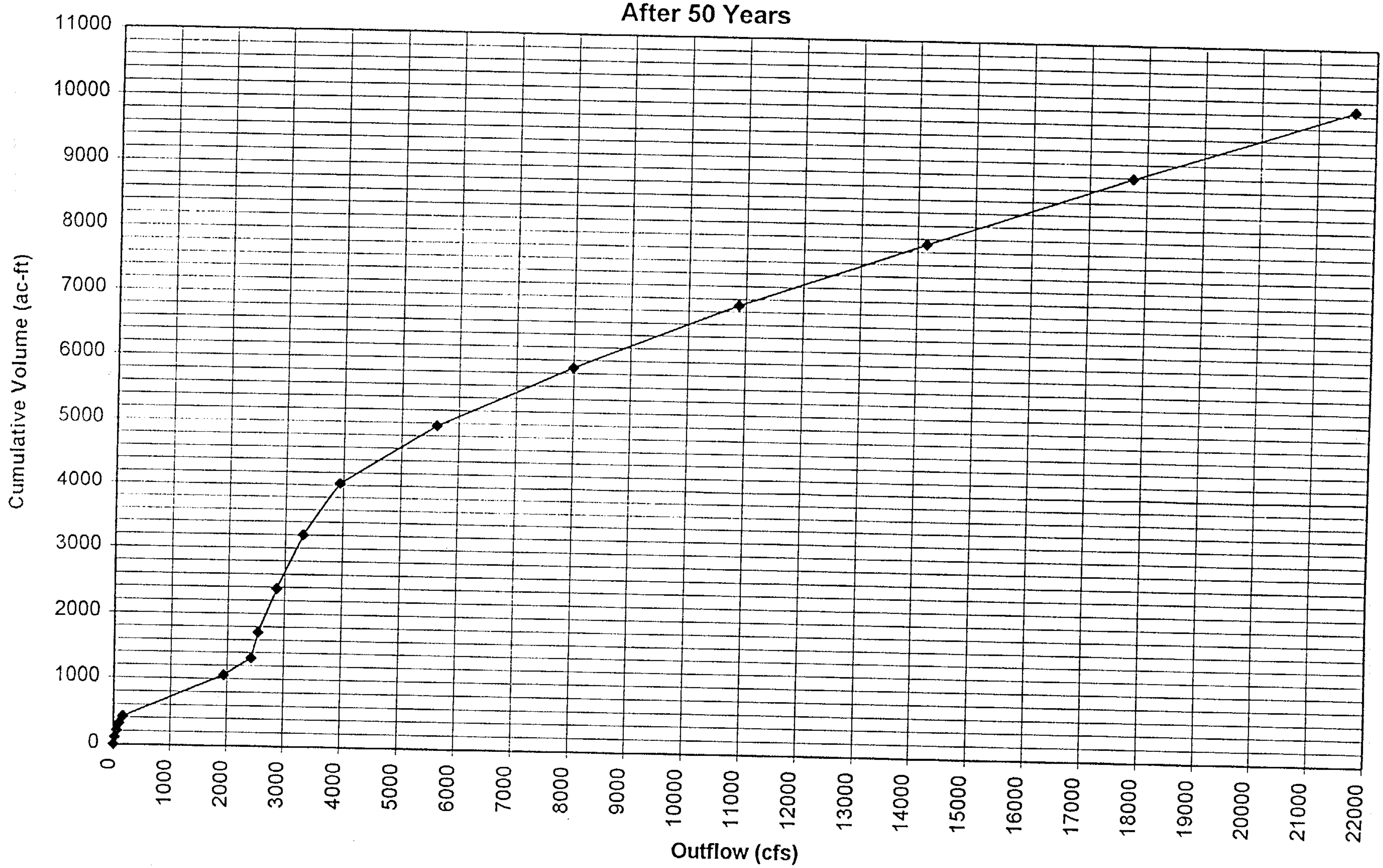
REELFOOT LAKE SEDIMENT RETENTION STRUCTURE
Pool Elevation vs. Outflow
After 50 Years



REELFOOT LAKE SEDIMENT RETENTION STRUCTURE
Storage vs. Outflow
Initial Conditions

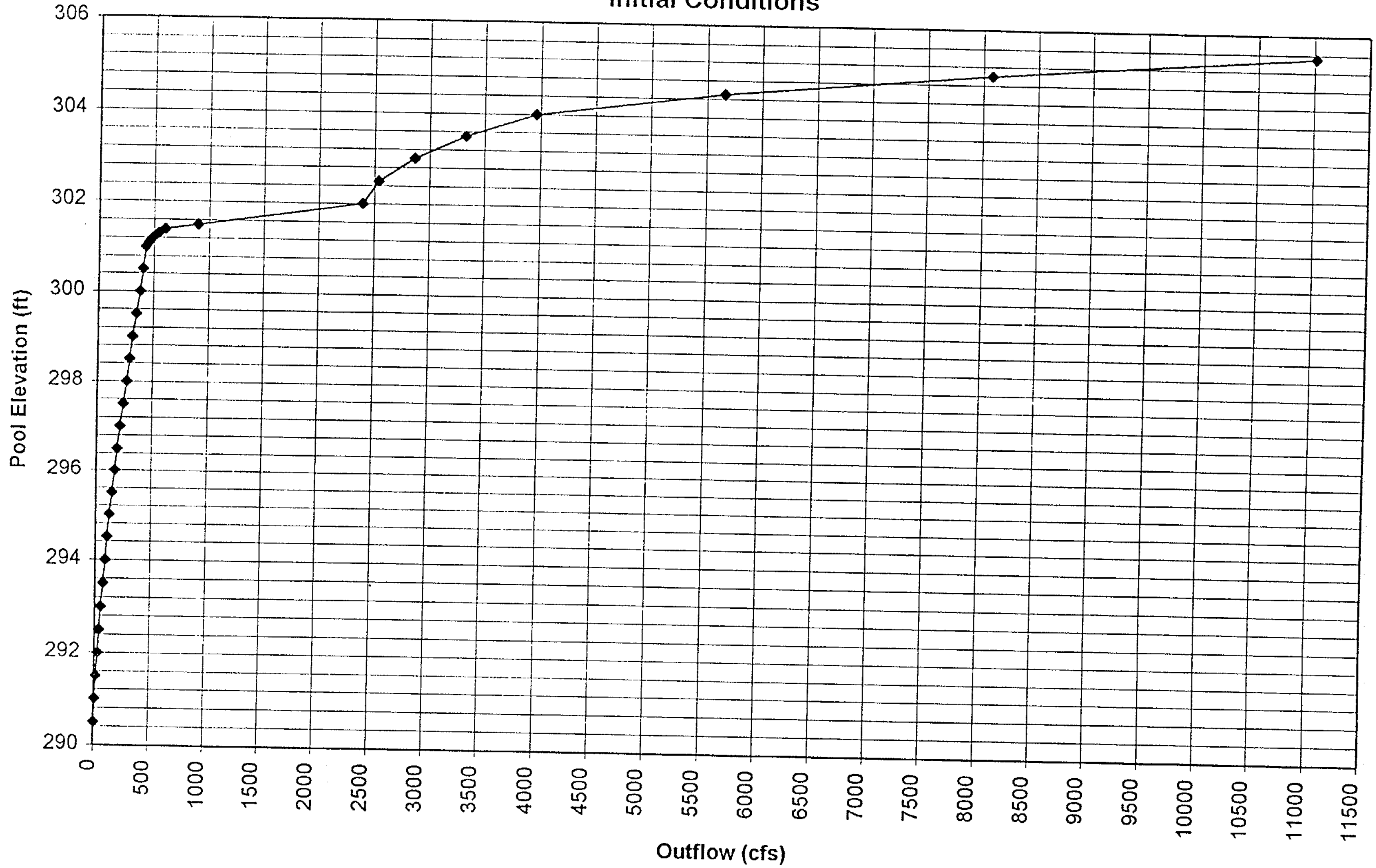


REELFOOT LAKE SEDIMENT RETENTION STRUCTURE
Storage vs. Outflow
After 50 Years



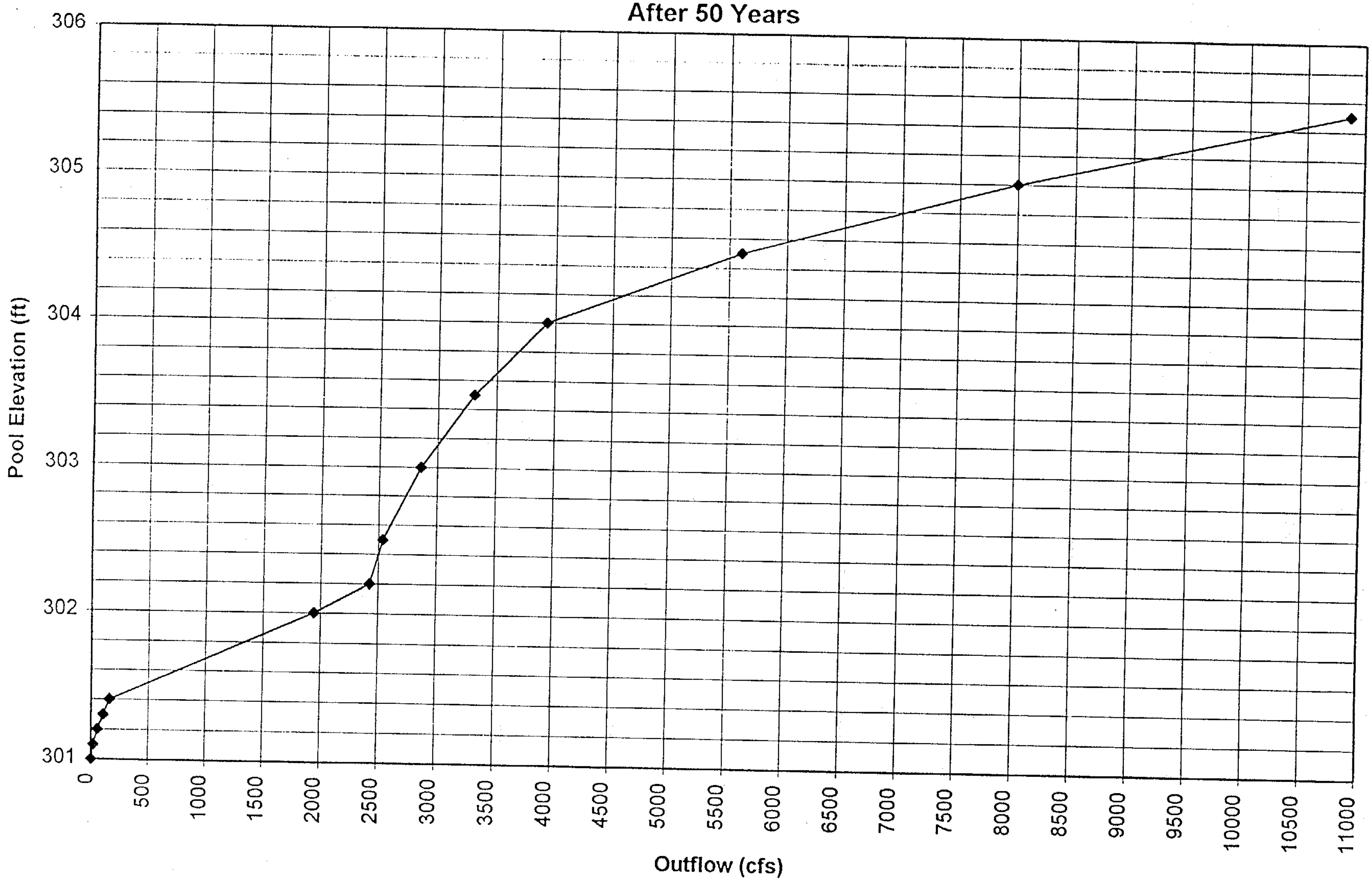
5/18/97

REELFOOT LAKE SEDIMENT RETENTION STRUCTURE
Pool Elevation vs. Outflow
Initial Conditions



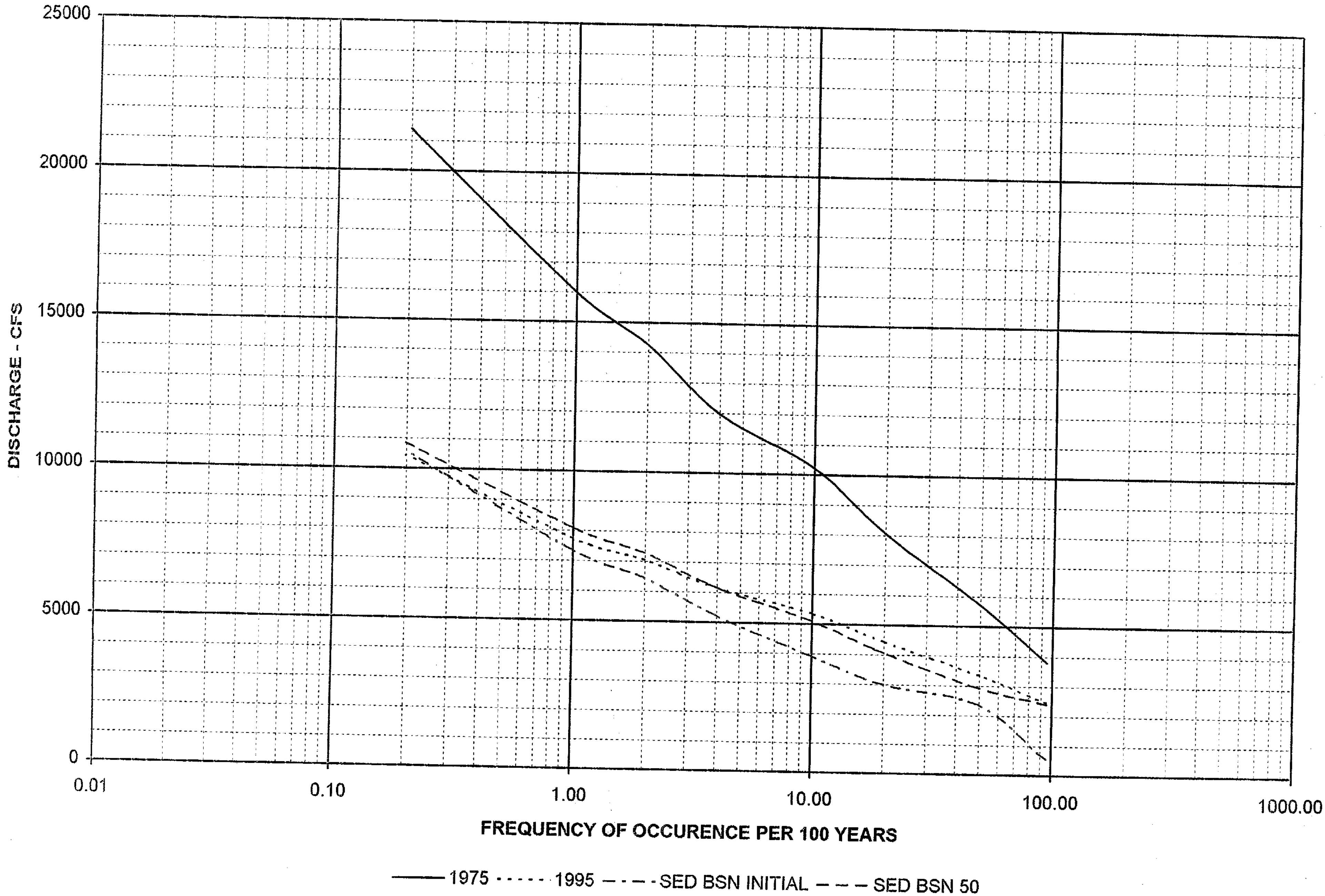
6-72ha1

REELFOOT LAKE SEDIMENT RETENTION STRUCTURE
Pool Elevation vs. Outflow
After 50 Years



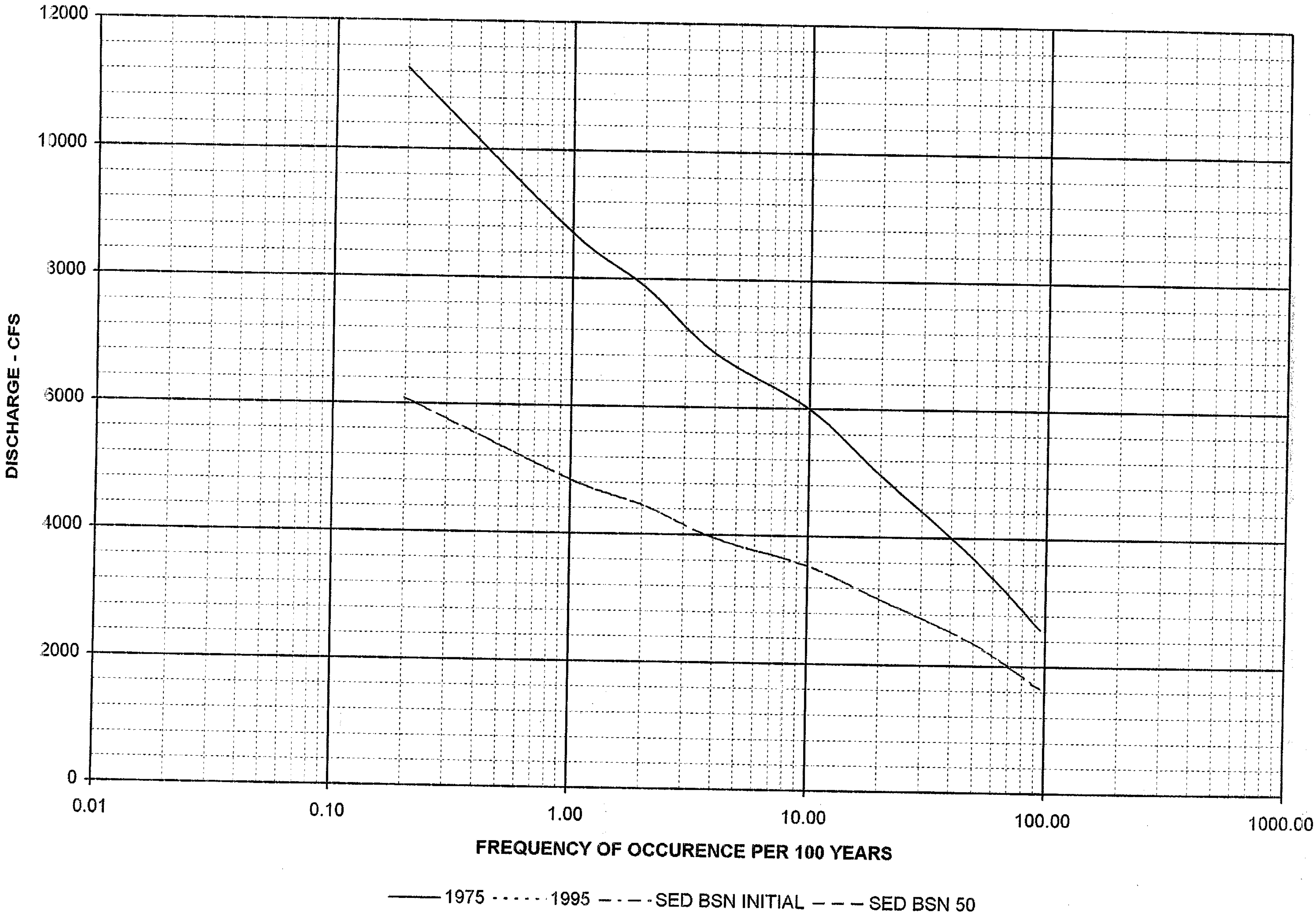
NORTH REELFOOT CREEK

MILE 0.00



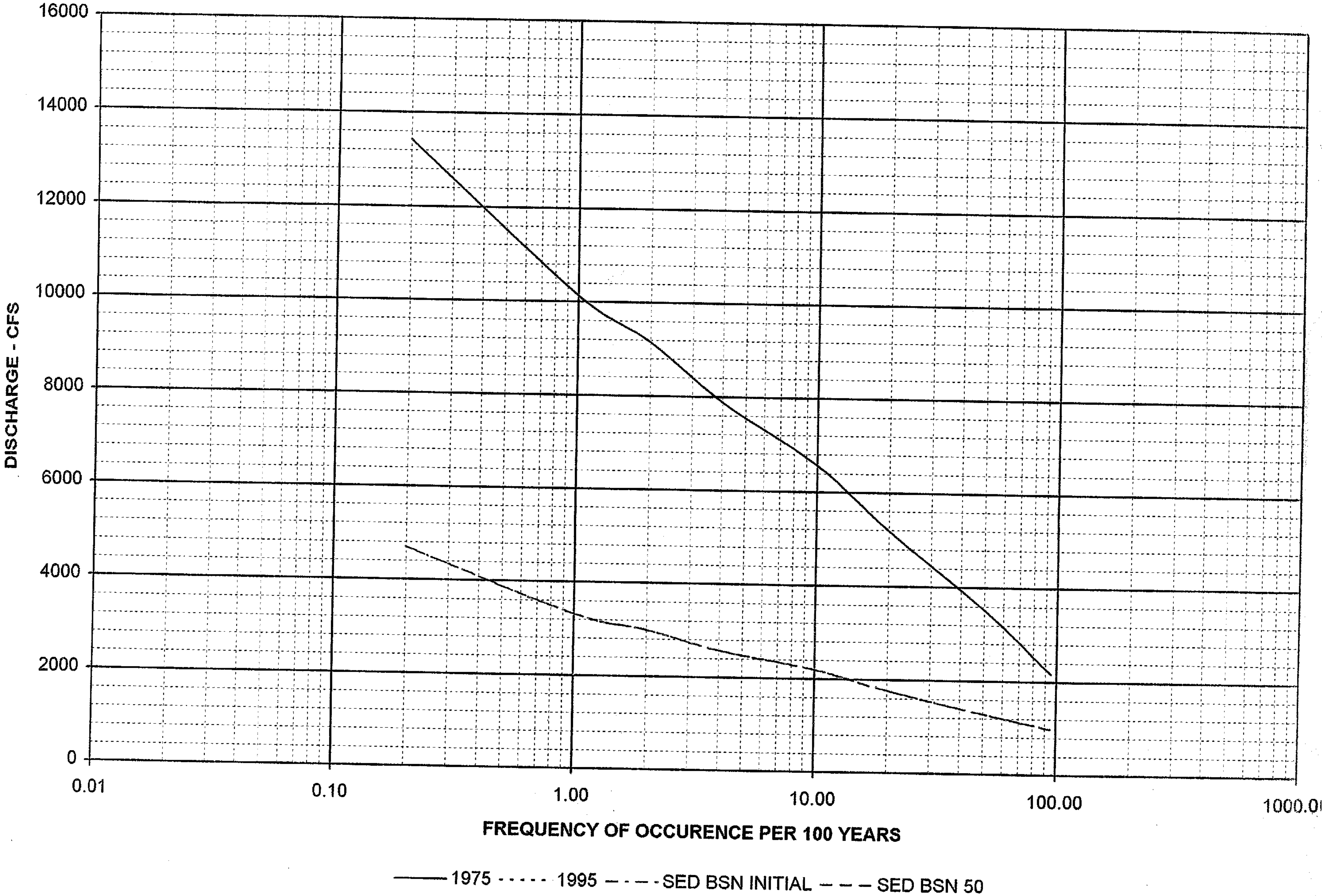
NORTH REELFOOT CREEK

MILE 2.91



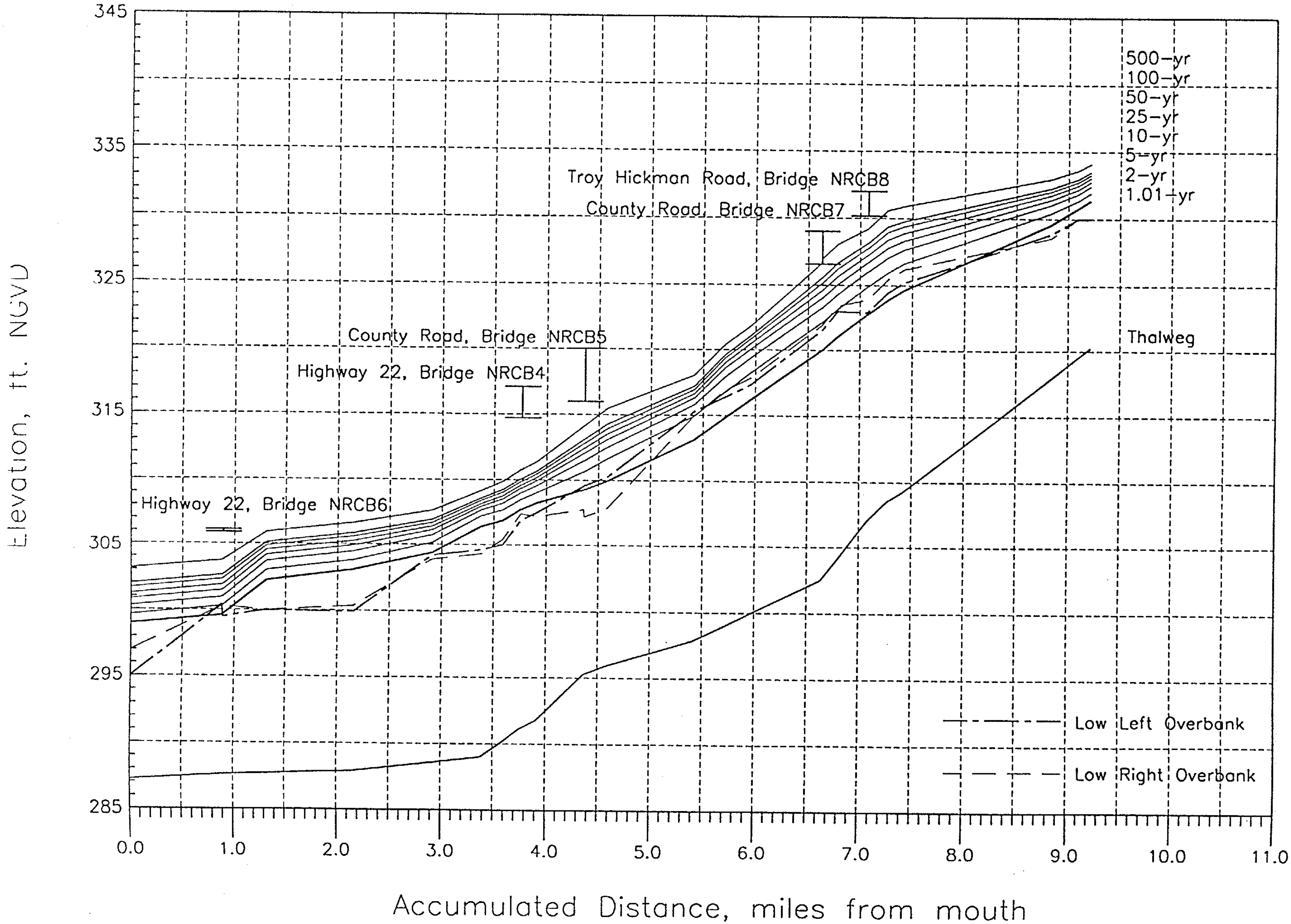
SOUTH REELFOOT CREEK

MILE 0.010



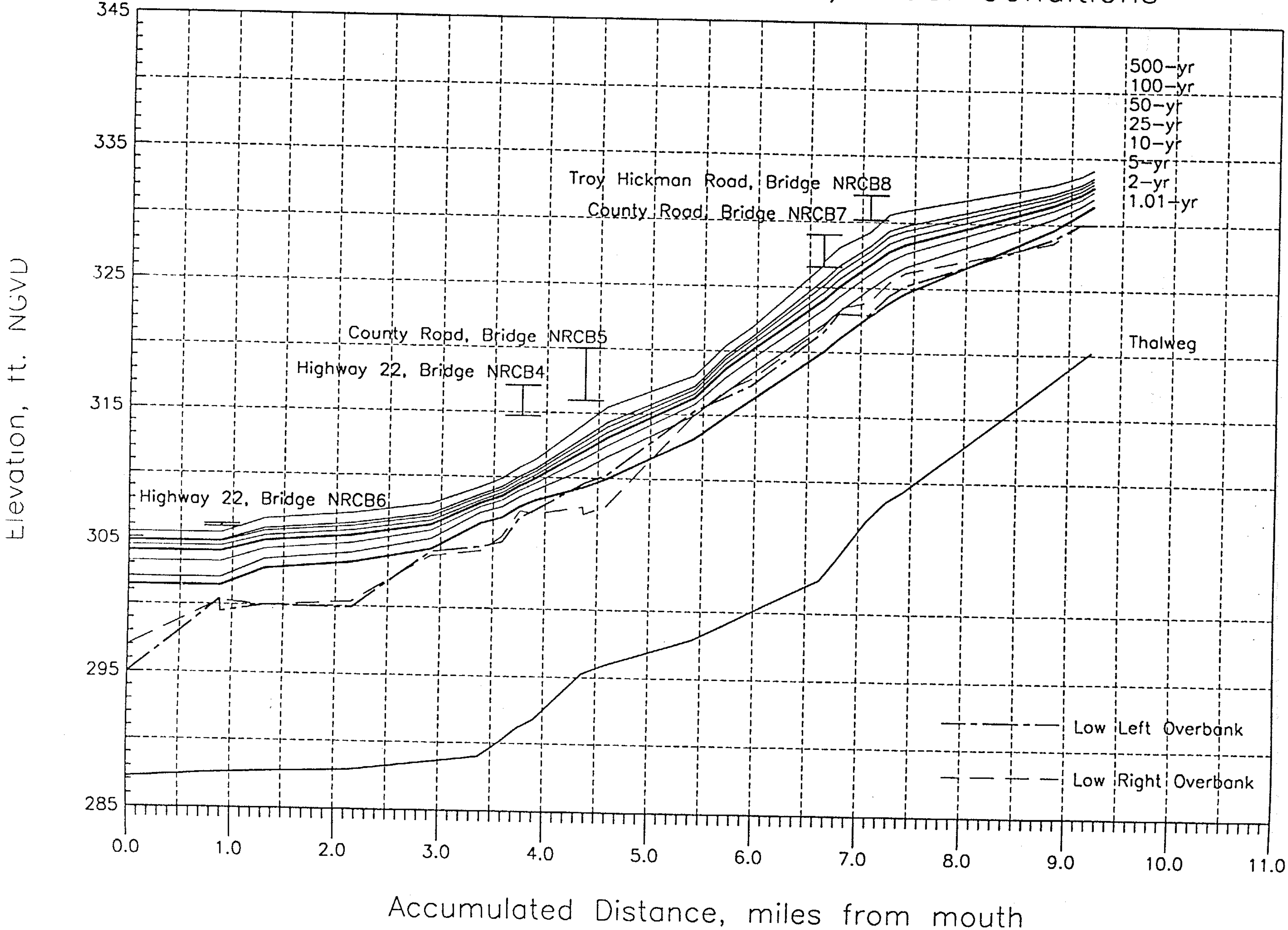
North Reelfoot Creek Summary Profile Plot

Existing Conditions



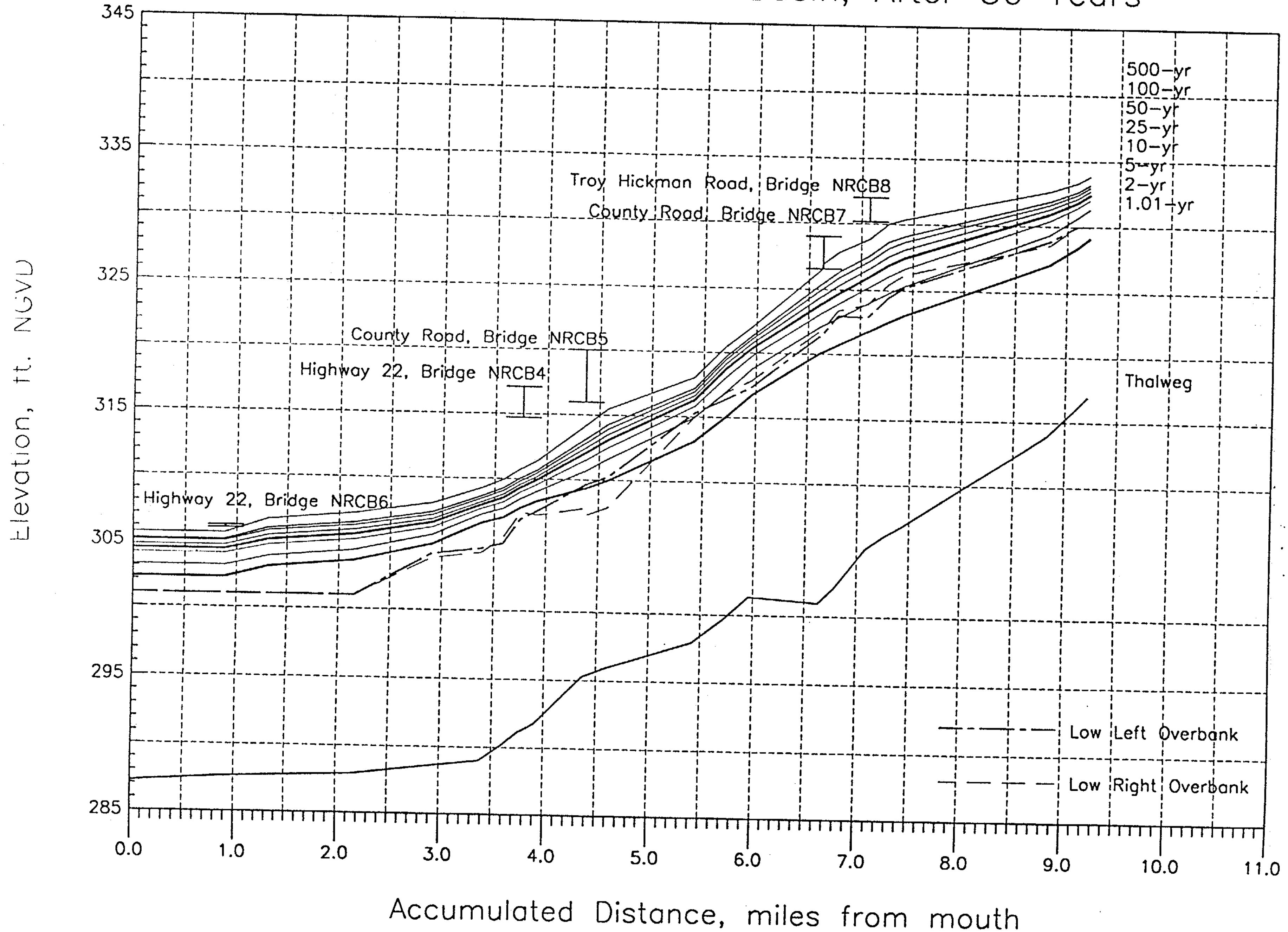
North Reelfoot Creek Summary Profile Plot

With Sediment Retention Basin, Initial Conditions

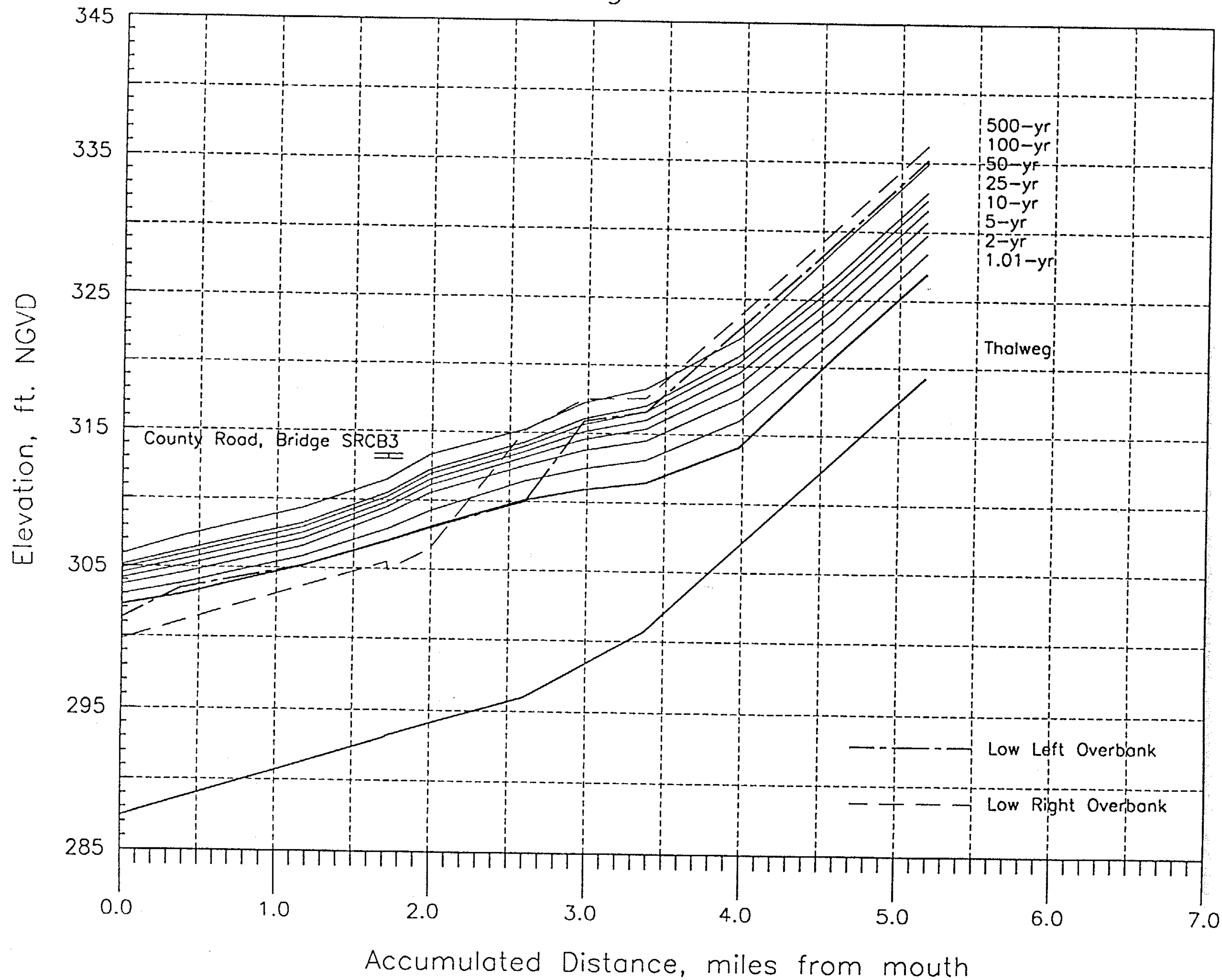


North Reelfoot Creek Summary Profile Plot

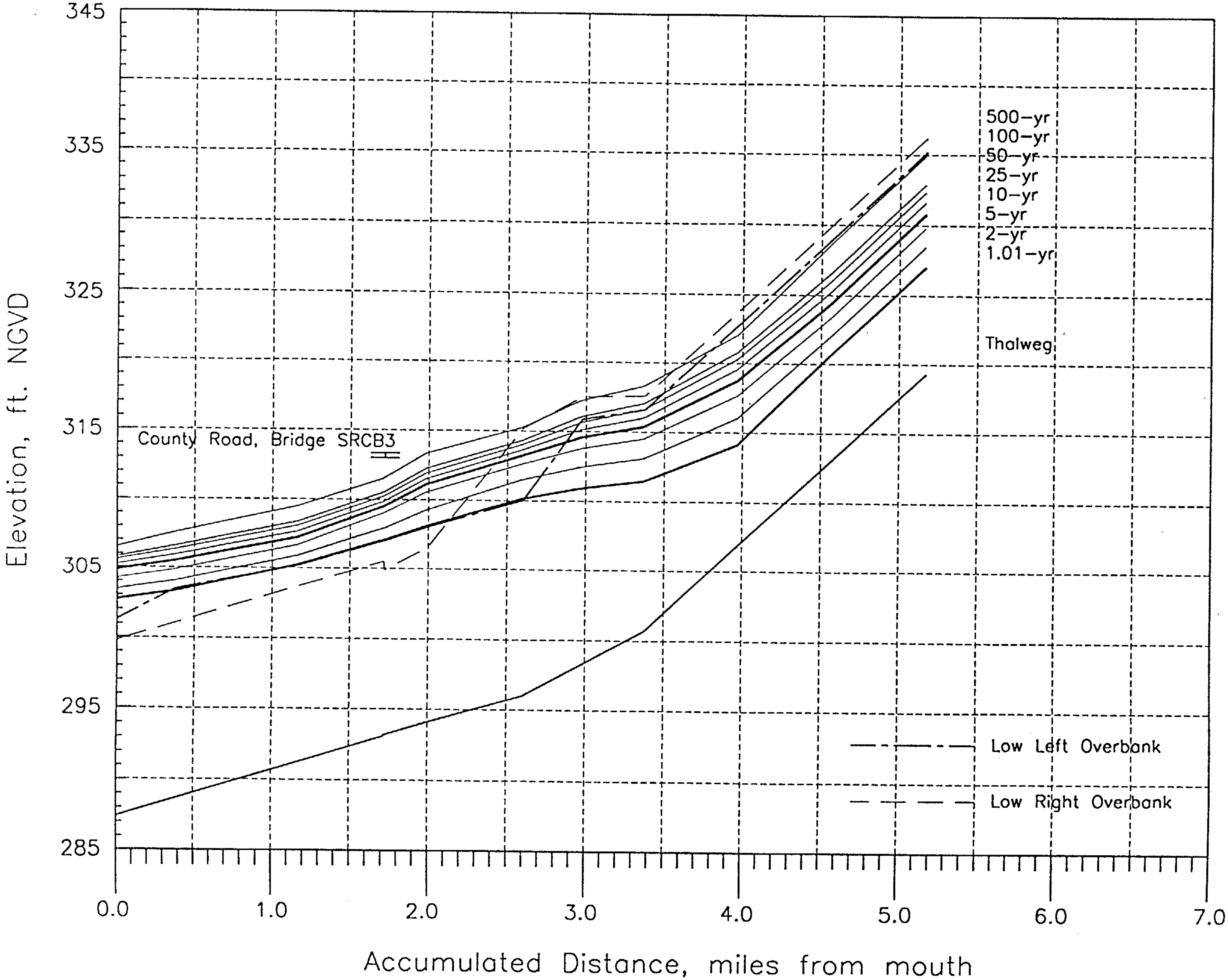
With Sediment Retention Basin, After 50 Years



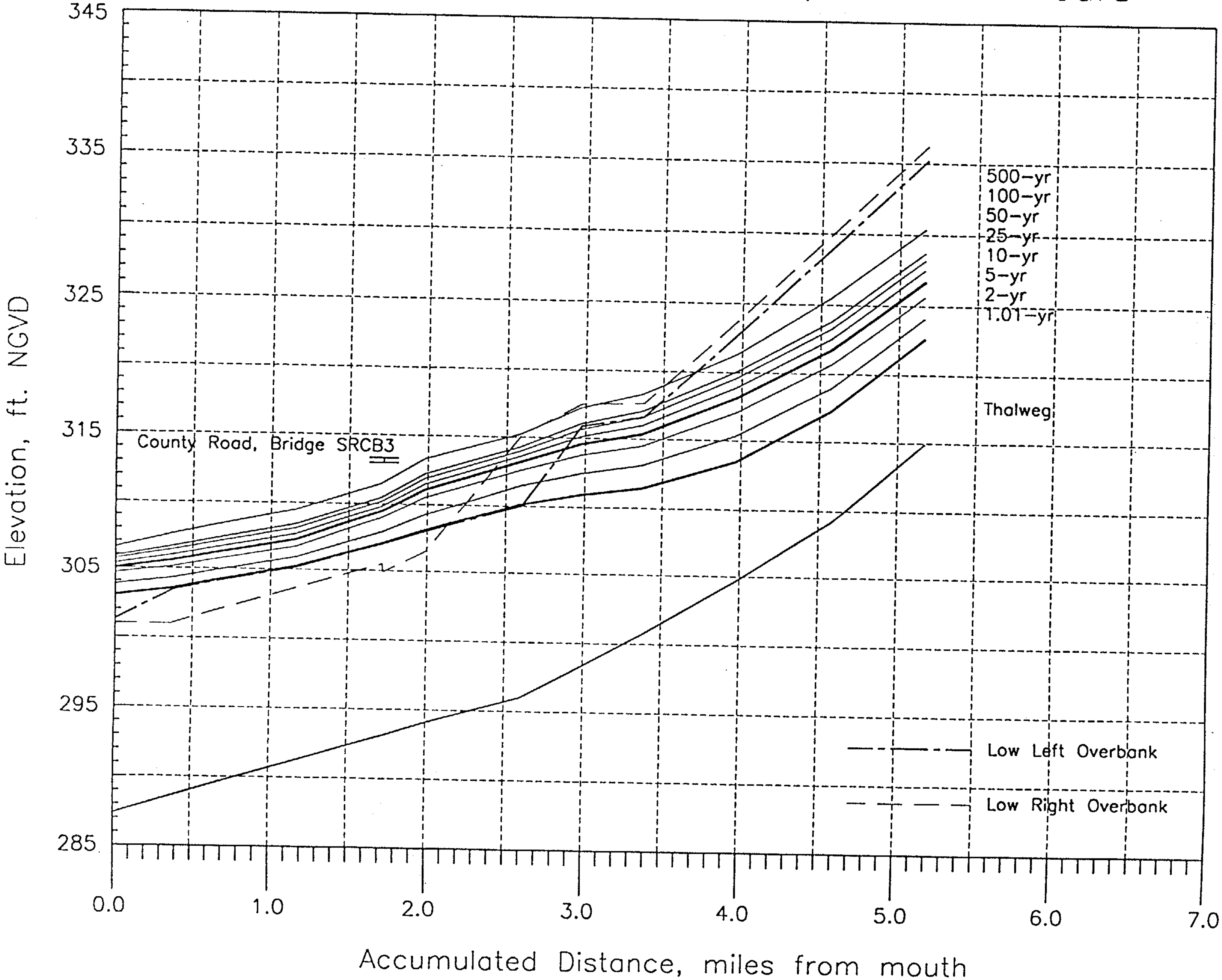
South Reelfoot Creek Summary Profile Plot
Existing Conditions

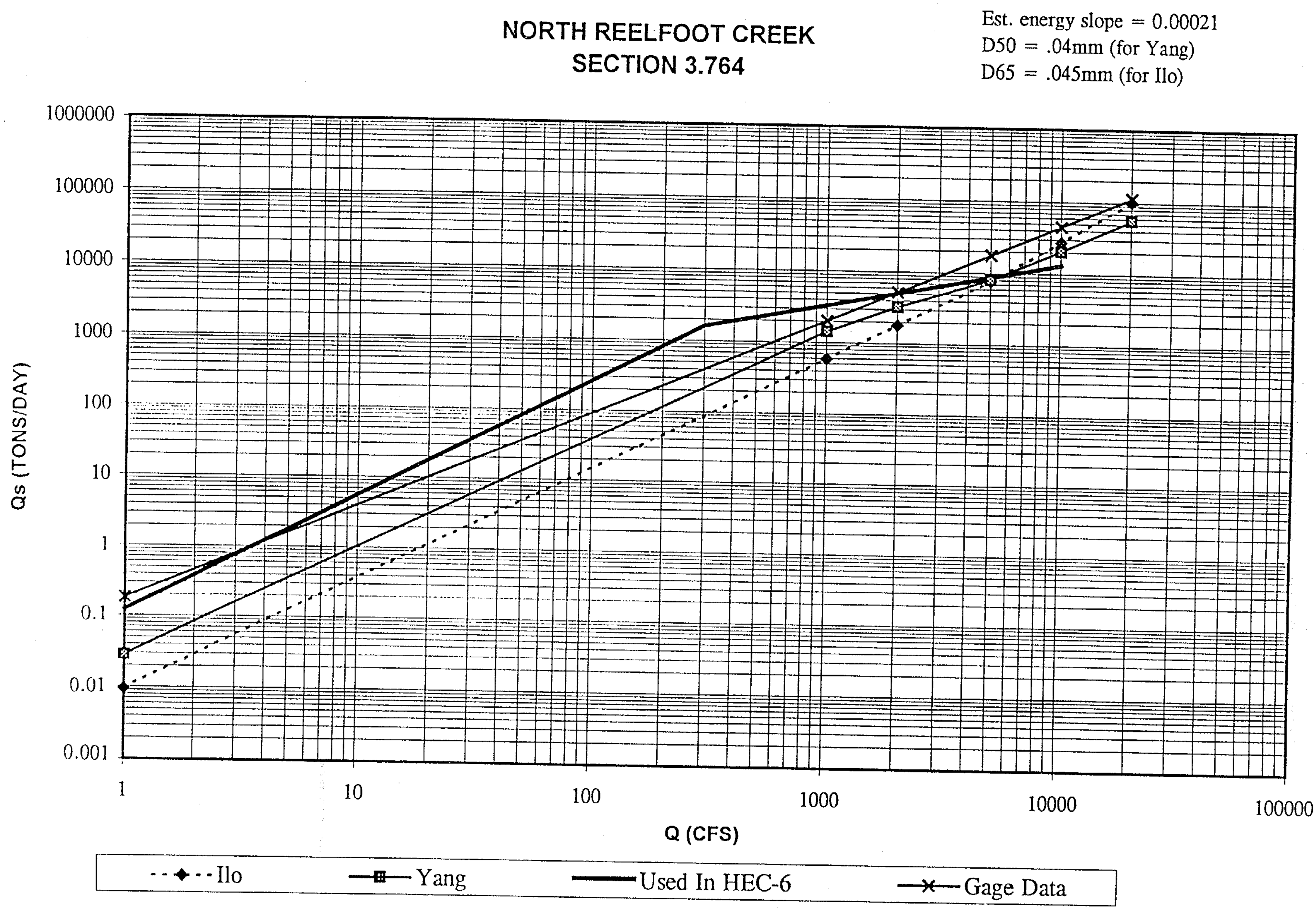


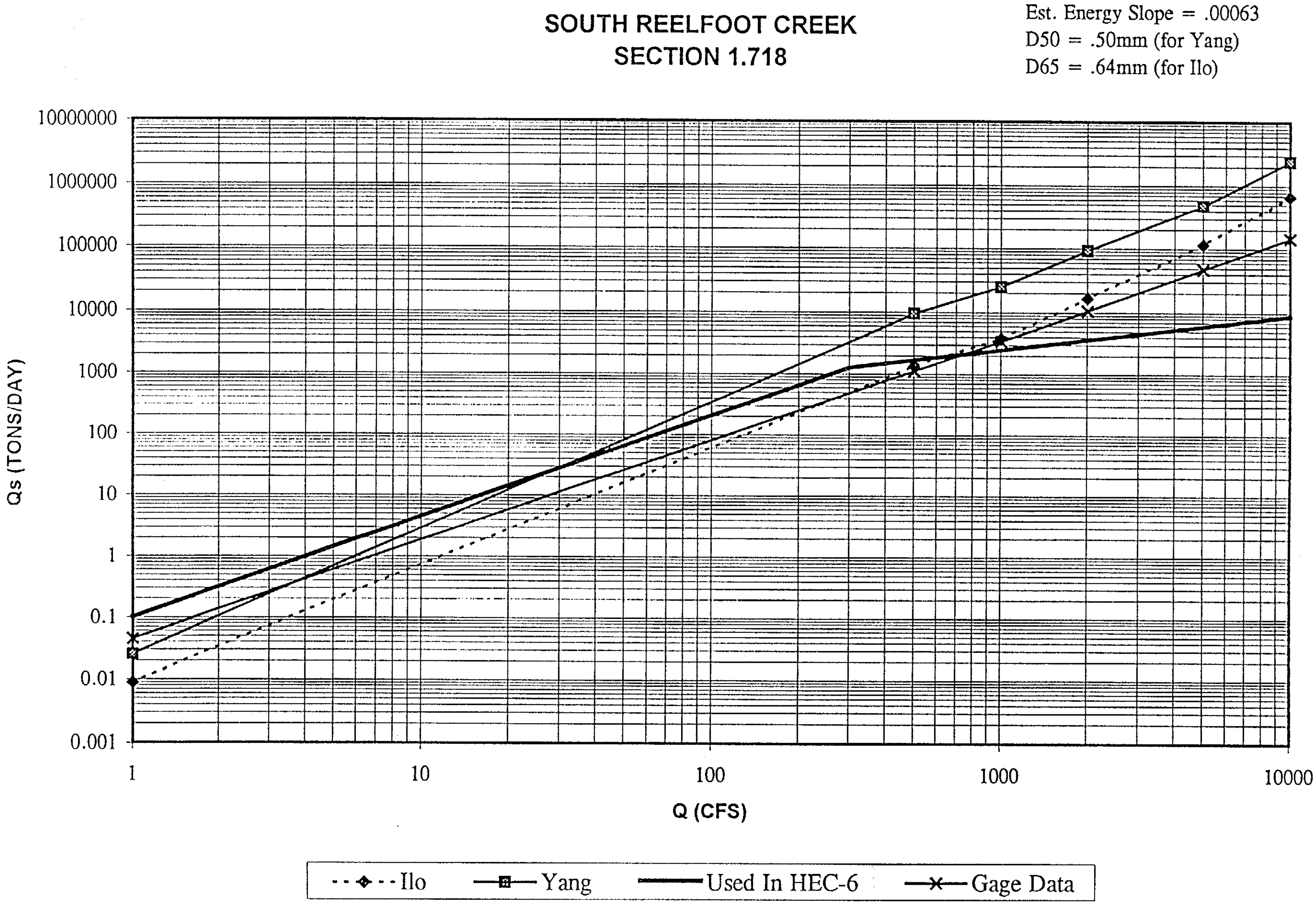
South Reelfoot Creek Summary Profile Plot
With Sediment Retention Basin, Initial Conditions



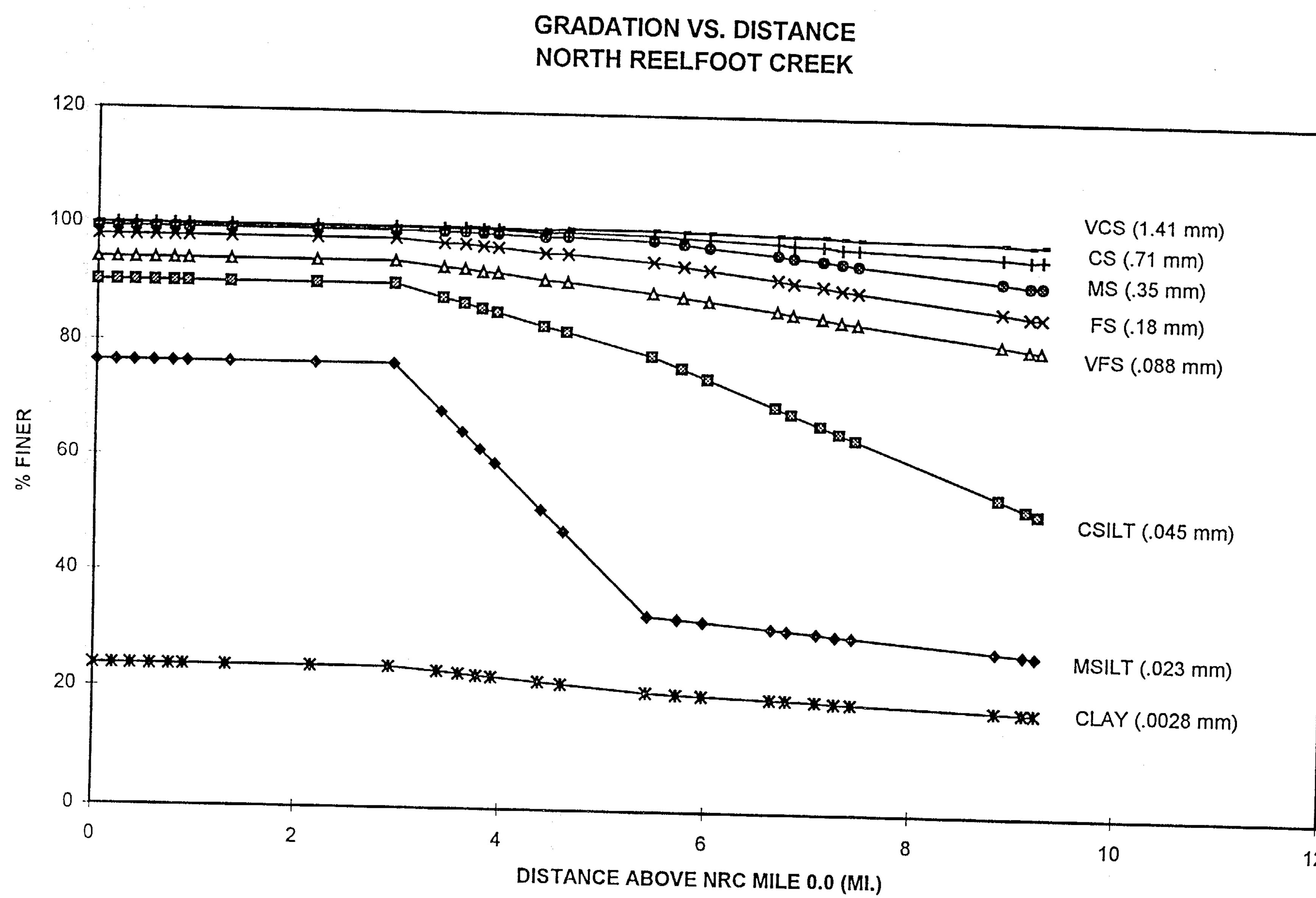
South Reelfoot Creek Summary Profile Plot
With Sediment Retention Basin, After 50 Years



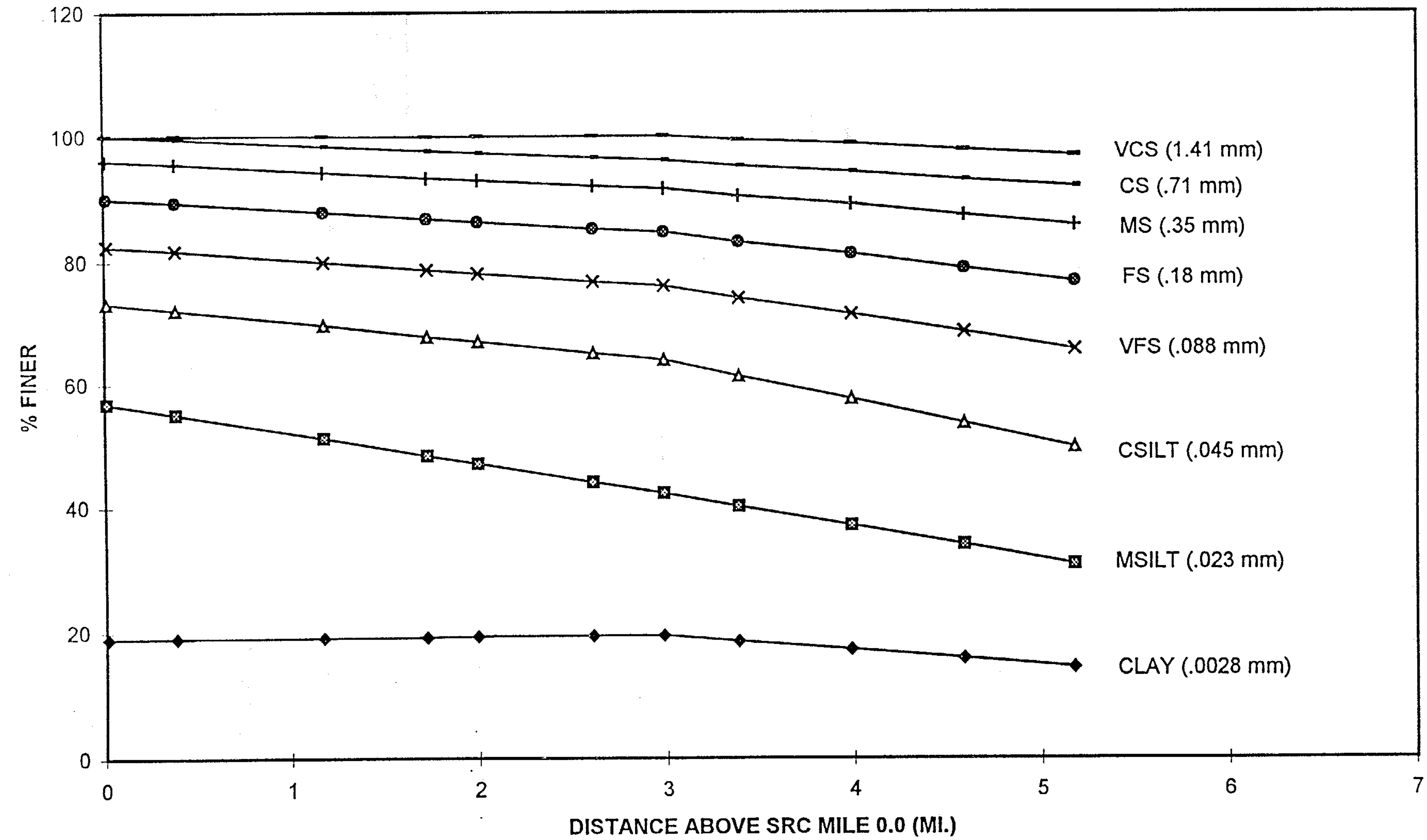

NORTH REELFOOT CREEK
SECTION 3.764
Est. energy slope = 0.00021
D50 = .04mm (for Yang)
D65 = .045mm (for Ilo)
1000000
100000
10000
1000
100
10
1
0.1
0.01
0.001
Qs (TONS/DAY)
1
10
100
1000
10000
100000
Q (CFS)
Ilo
Yang
Used In HEC-6
Gage Data

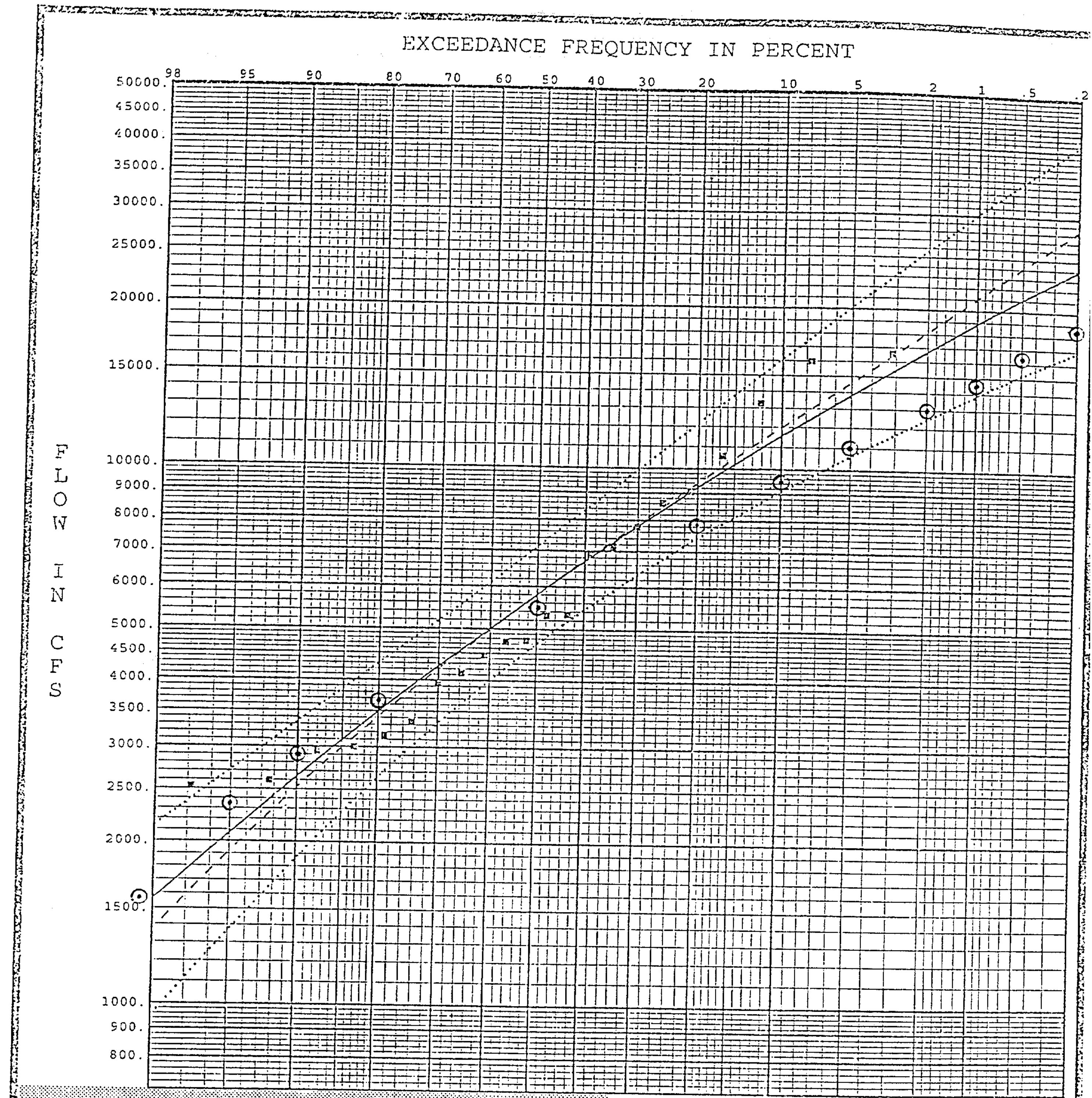


SOUTH REELFOOT CREEK
SECTION 1.718
Est. Energy Slope = .00063
D50 = .50mm (for Yang)
D65 = .64mm (for Ilo)
Qs (TONS/DAY)
10000000
1000000
100000
10000
1000
100
10
1
0.1
0.01
0.001
1
10
100
1000
10000
Q (CFS)
Ilo
Yang
Used In HEC-6
Gage Data


GRADATION VS. DISTANCE
NORTH REELFOOT CREEK
% FINER
120
100
80
60
40
20
0
VCS (1.41 mm)
CS (.71 mm)
MS (.35 mm)
FS (.18 mm)
VFS (.088 mm)
CSILT (.045 mm)
MSILT (.023 mm)
CLAY (.0028 mm)
0
2
4
6
8
10
12
DISTANCE ABOVE NRC MILE 0.0 (MI.)

GRADATION VS. DISTANCE
SOUTH REELFOOT CREEK





—— Flow Frequency (without Exp. Prob.)
- - - Flow Frequency (with Exp. Prob.)
▫ Median Plotting Positions
.... 5% and 95% Confidence Limits

FREQUENCY STATISTICS
LOG TRANSFORM OF FLOW, CFS

		NUMBER OF EVENTS	
MEAN	3.7507	HISTORIC EVENTS	0
STANDARD DEV	.2508	HIGH OUTLIERS	0
SKEW	.4380	LOW OUTLIERS	0
REGIONAL SKEW	-.3100	ZERO OR MISSING	0
ADOPTED SKEW	-.3100	SYSTEMATIC EVENTS	22

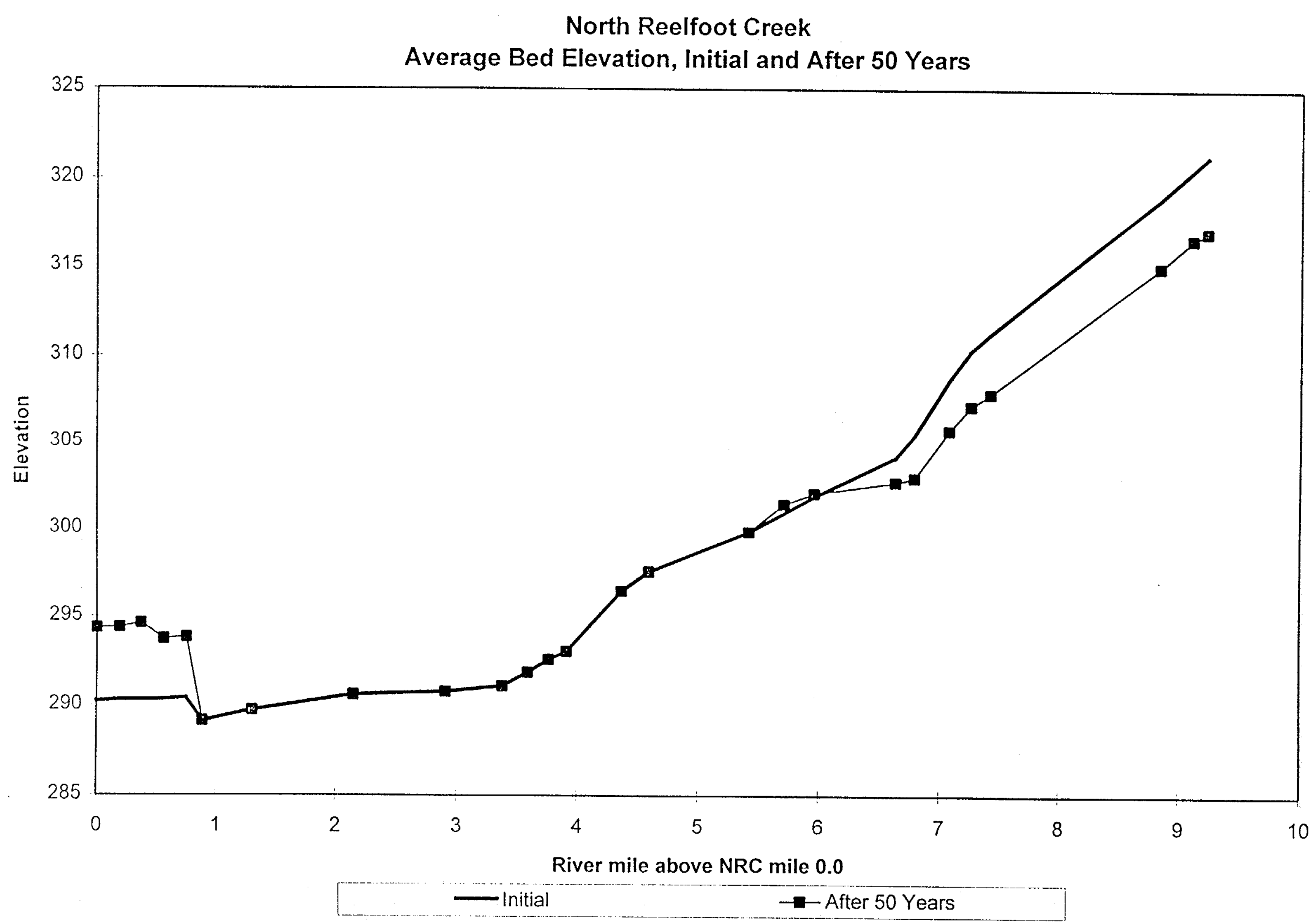
⊙ NODE 332 HUXRAIN
API "A" COEFF. .4875 to .6375

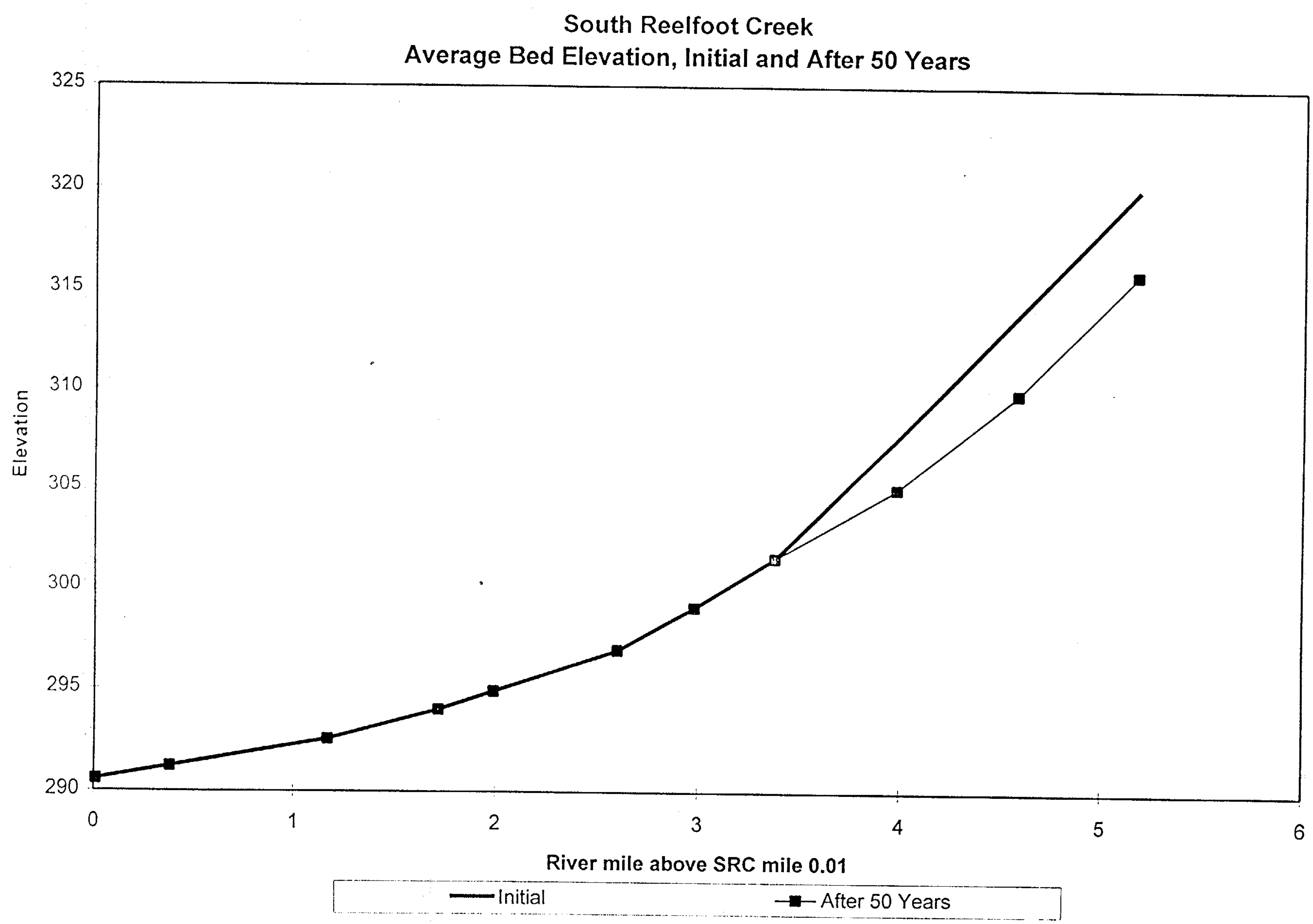
REELFOOT CREEK
NEAR SAMBURG, TENNESSEE
BASIN AREA = 110 SQ MI
WATER YEARS IN RECORD
1951-1972

Reelfoot Lake Feasibility Study

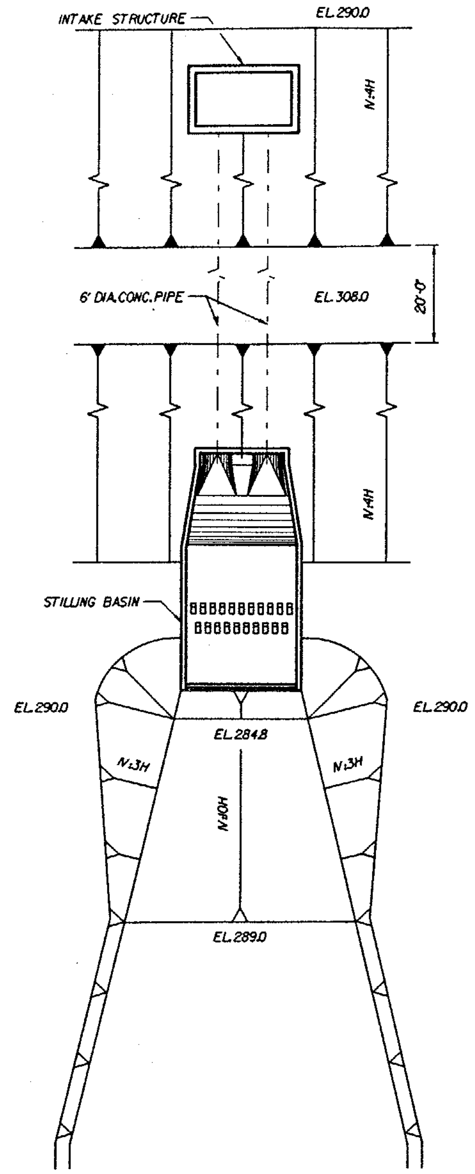
Sediment Retention Structure
50-Year Sediment Routing Results
May 19, 1997

HEC-6 Filename	H6T16.DAT
Sediment inflow /outflow for main stem Reelfoot Creek outlet	
silt inflow (ac ft)	4,455
outflow (ac ft)	1,969
trap efficiency (%)	56
sand inflow (ac ft)	291
outflow (ac ft)	1
trap efficiency (%)	100
Dredging	
totals (cu yd)	464,935
totals (ac ft)	288
annual (cu yd)	9,300
annual (ac ft)	5.8
NRC range (RM)	1.31 - 4.365
SRC range (RM)	.01 - 2.98
Totals	
total trapped (ac ft)	3,064
total trapped below section 1.31 (ac ft)	1,740
elevation of 50-year sediment pool (ft)	299.2

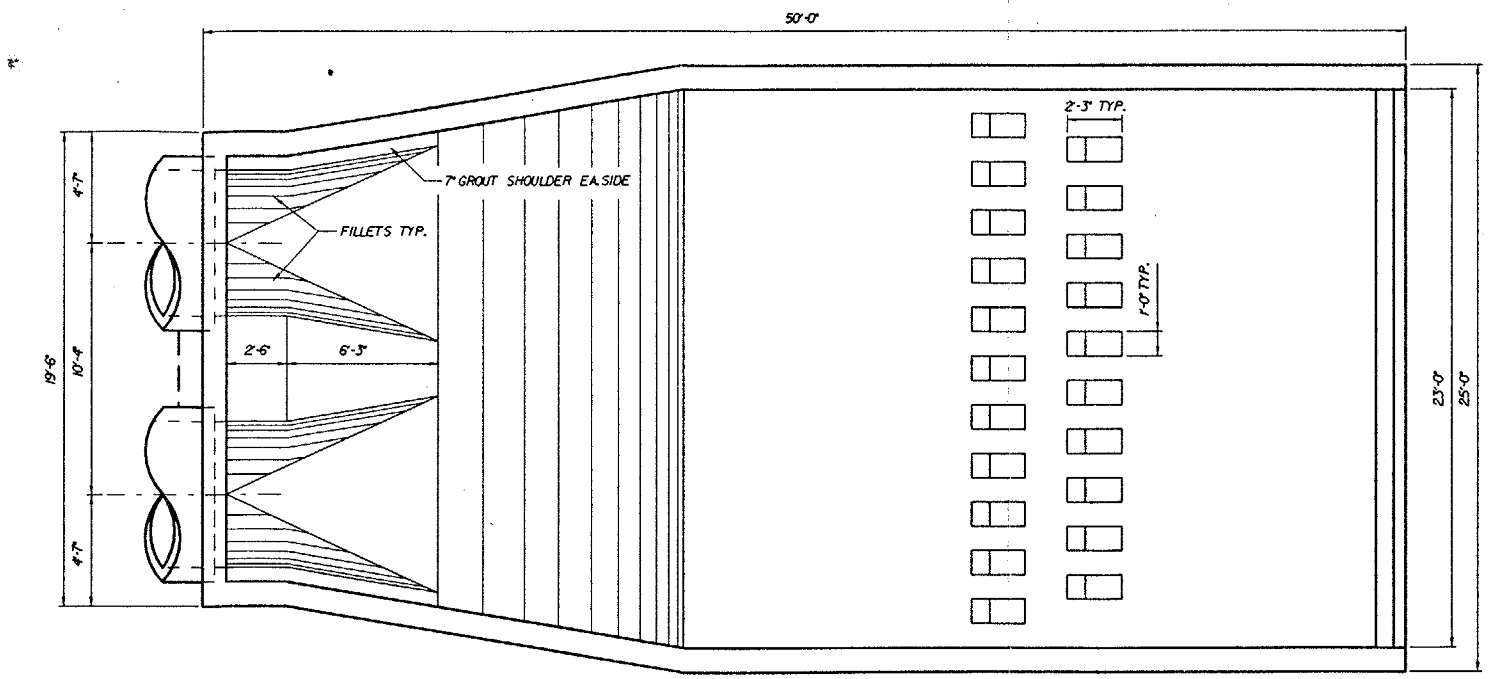

North Reelfoot Creek
Average Bed Elevation, Initial and After 50 Years
Elevation
325
320
315
310
305
300
295
290
285
0
1
2
3
4
5
6
7
8
9
10
River mile above NRC mile 0.0
Initial
After 50 Years



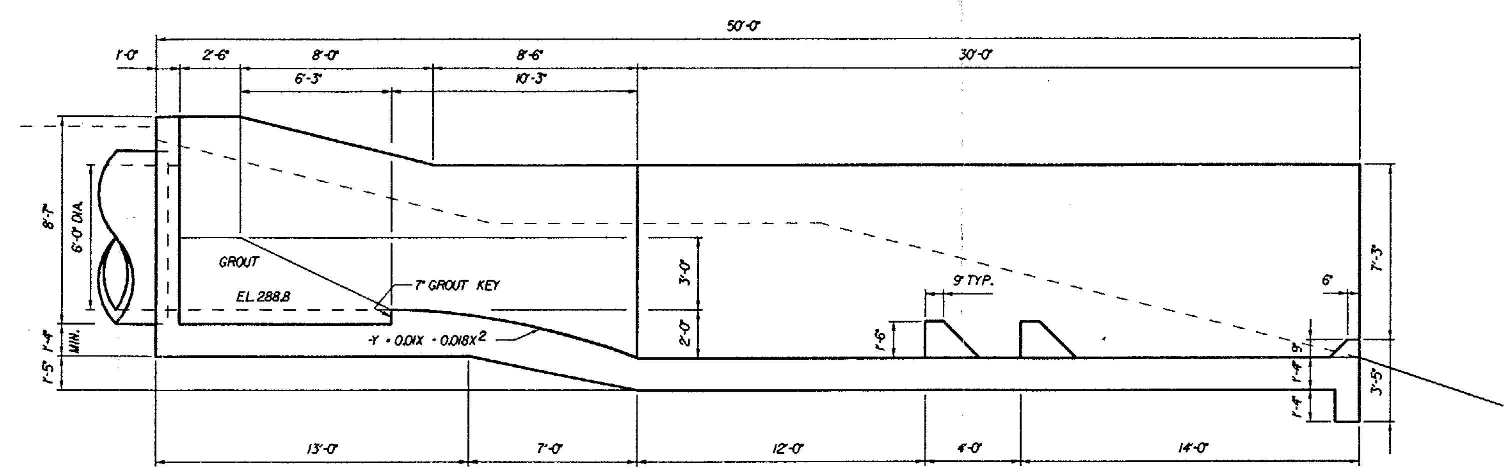
South Reelfoot Creek
Average Bed Elevation, Initial and After 50 Years
Elevation
325
320
315
310
305
300
295
290
0
1
2
3
4
5
6
River mile above SRC mile 0.01
Initial
After 50 Years



SITE PLAN
SCALE 1" = 20'-0"

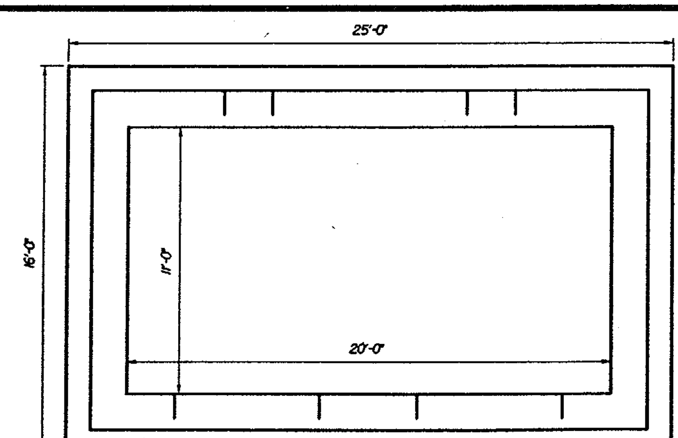


PLAN

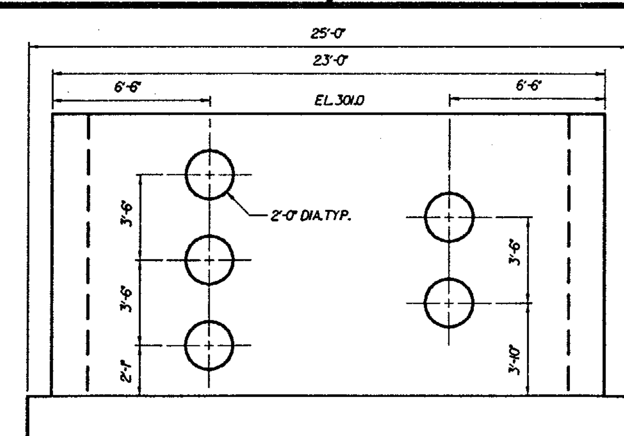


SECTION

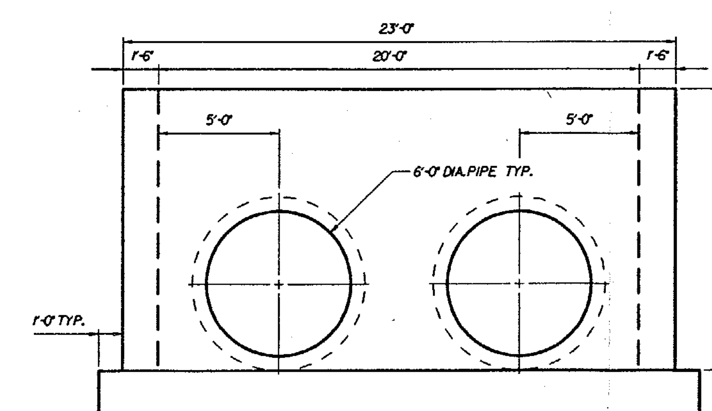
STILL BASIN
SCALE 1/8" = 1'-0"



PLAN



UPSTREAM SIDE



DOWNSTREAM SIDE

INTAKE STRUCTURE
SCALE 1/8" = 1'-0"

US Army Corps of Engineers Memphis District

U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
MEMPHIS, TENNESSEE

ENVIRONMENTAL RESTORATION
NATURAL RESOURCES PROJECT

MISSISSIPPI RIVER AND TRIBUTARIES
CONSTRUCTION

REELFOOT LAKE, TENNESSEE & KENTUCKY
REELFOOT CREEK SEDIMENT RETENTION BASIN
LOW LEVEL SPILLWAY

PLATE
I-61

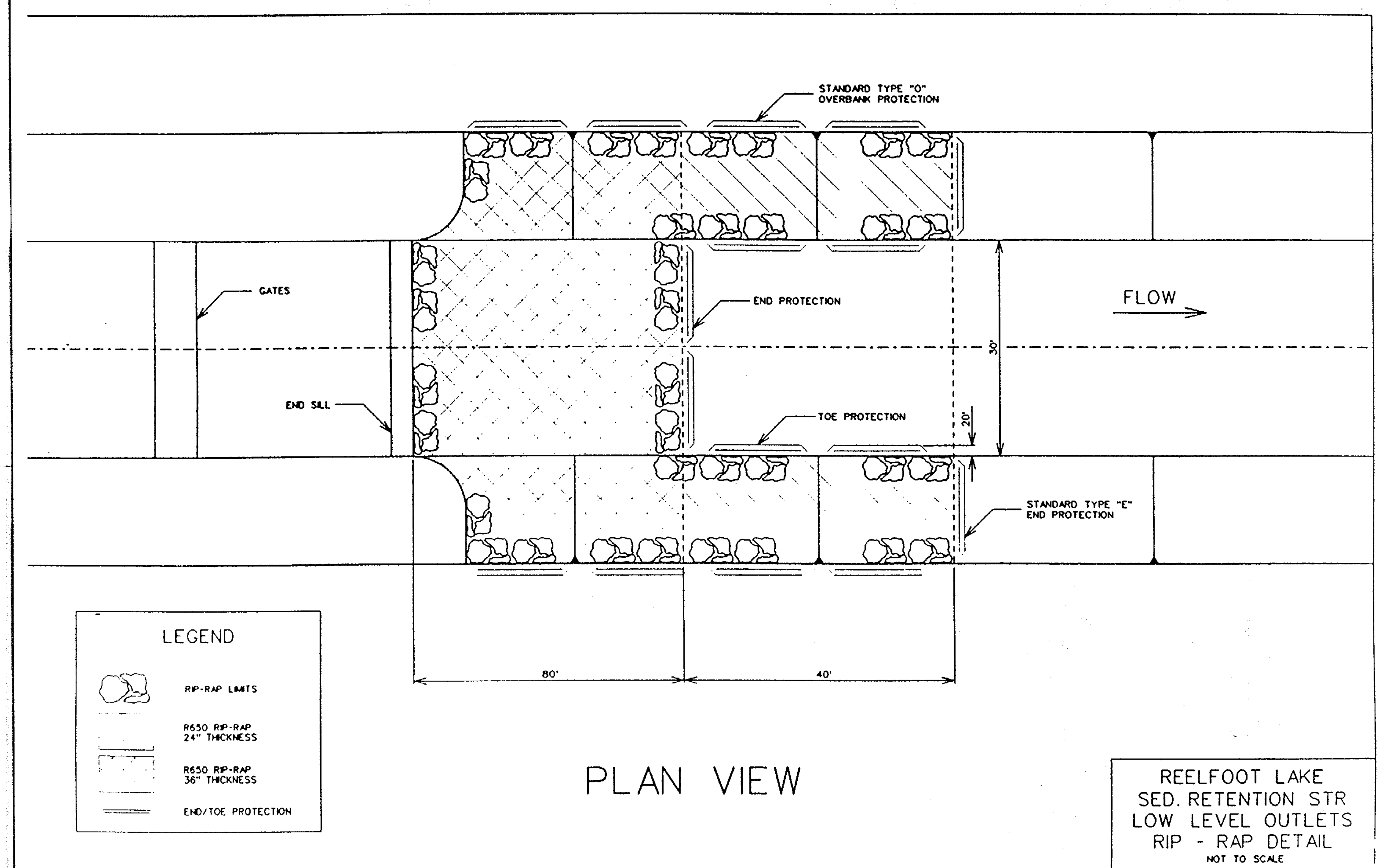

STANDARD TYPE "O"
OVERBANK PROTECTION
GATES
END PROTECTION
FLOW
30'
END SILL
TOE PROTECTION
20'
STANDARD TYPE "E"
END PROTECTION
LEGEND
RIP-RAP LIMITS
R650 RIP-RAP
24" THICKNESS
R650 RIP-RAP
36" THICKNESS
END/TOE PROTECTION
80'
40'
PLAN VIEW
REELFOOT LAKE
SED. RETENTION STR
LOW LEVEL OUTLETS
RIP - RAP DETAIL
NOT TO SCALE

PLATE I-62

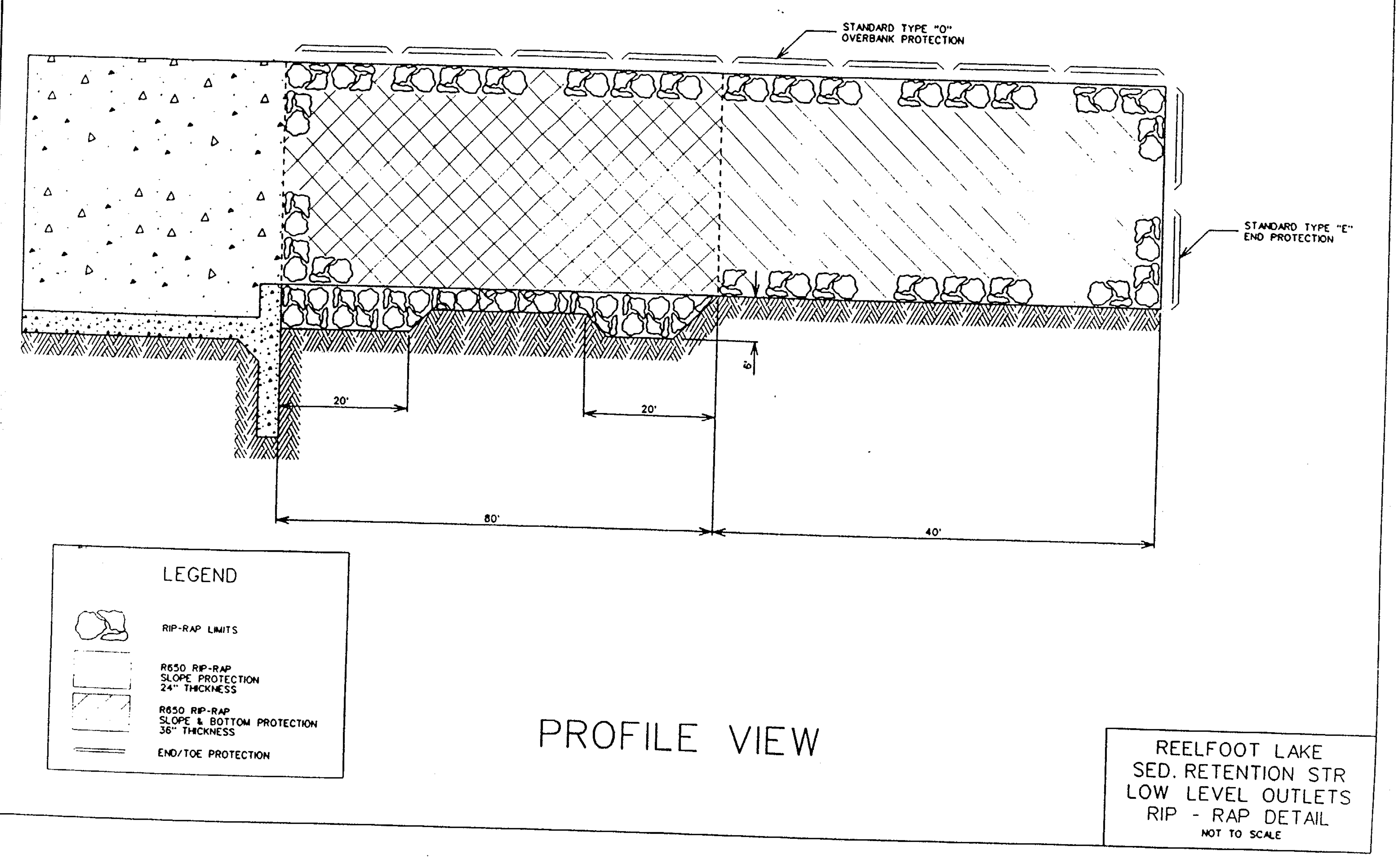
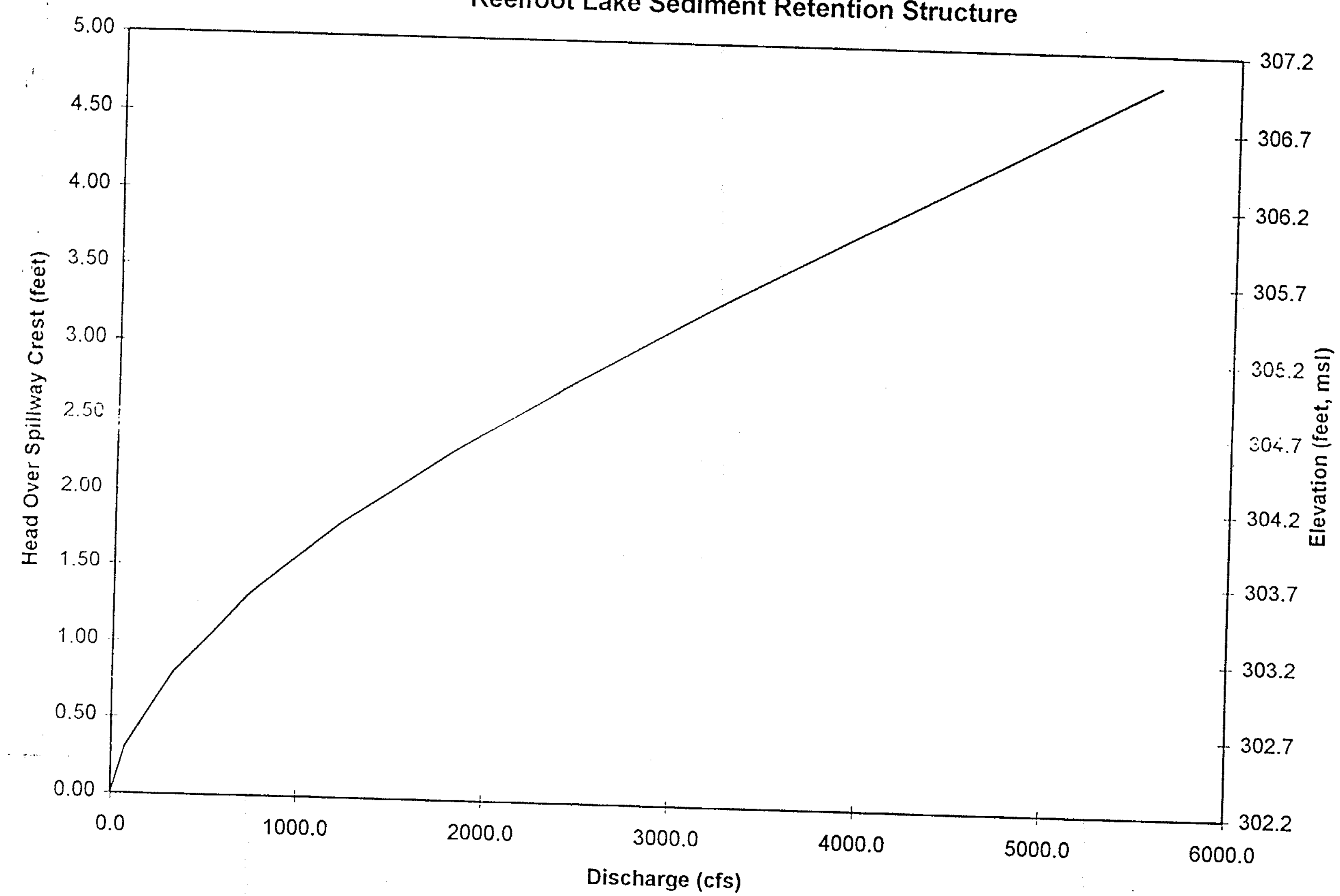
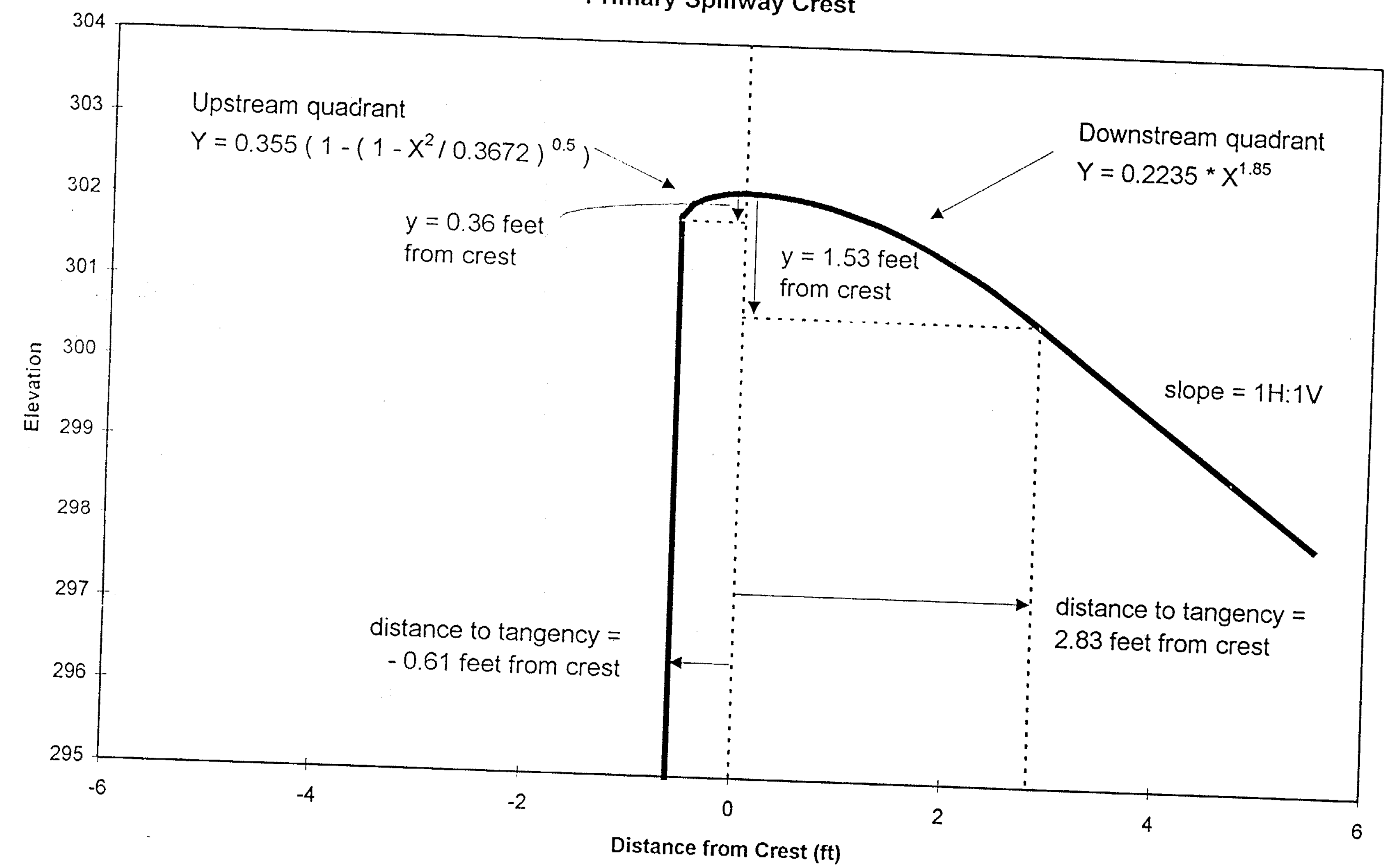

STANDARD TYPE "O"
OVERBANK PROTECTION
STANDARD TYPE "E"
END PROTECTION
6'
20'
20'
80'
40'
LEGEND
RIP-RAP LIMITS
R650 RIP-RAP
SLOPE PROTECTION
24" THICKNESS
R650 RIP-RAP
SLOPE & BOTTOM PROTECTION
36" THICKNESS
END/TOE PROTECTION
PROFILE VIEW
REELFOOT LAKE
SED. RETENTION STR
LOW LEVEL OUTLETS
RIP - RAP DETAIL
NOT TO SCALE

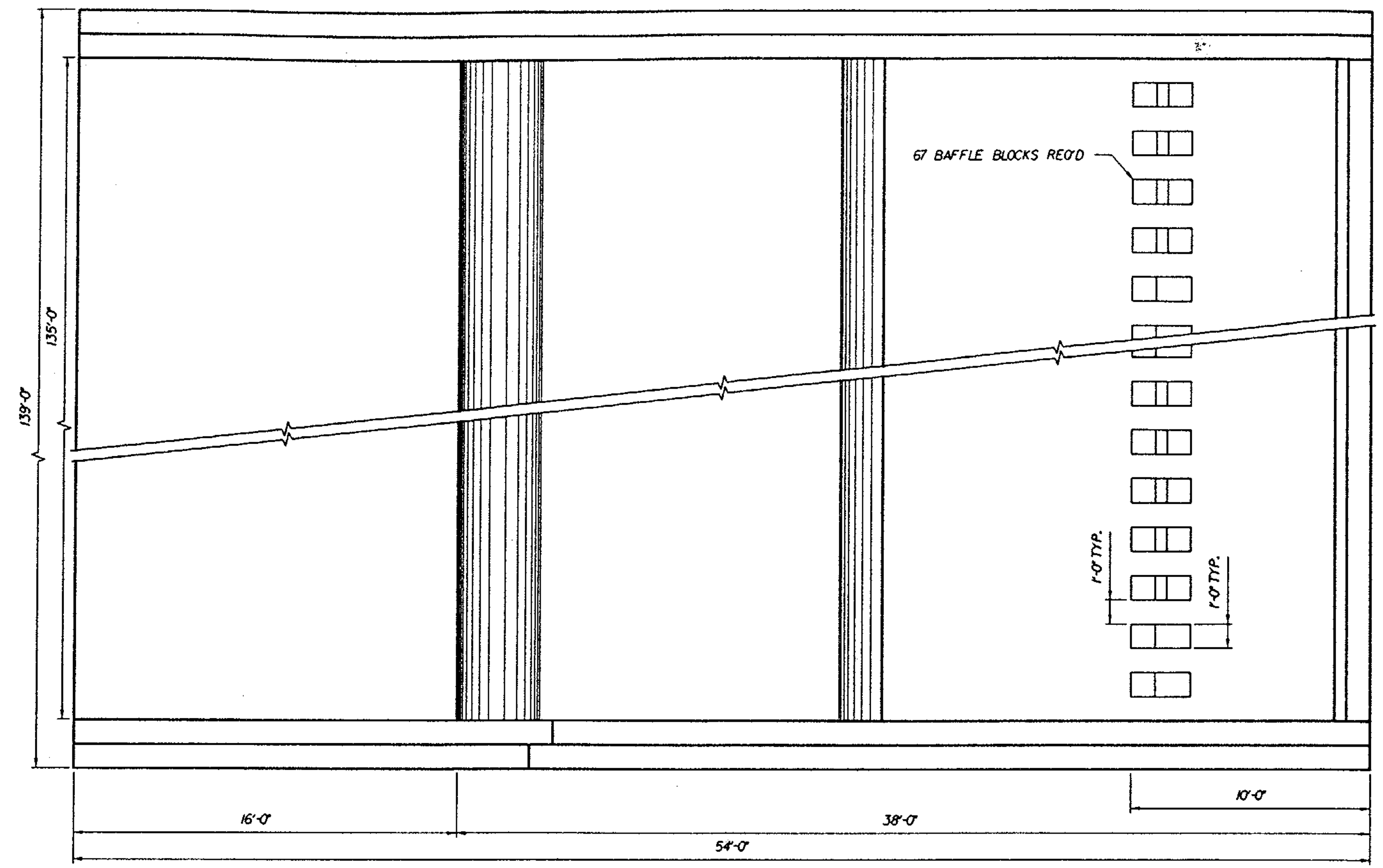
PLATE I-63

Primary Spillway Rating Curve
Reelfoot Lake Sediment Retention Structure

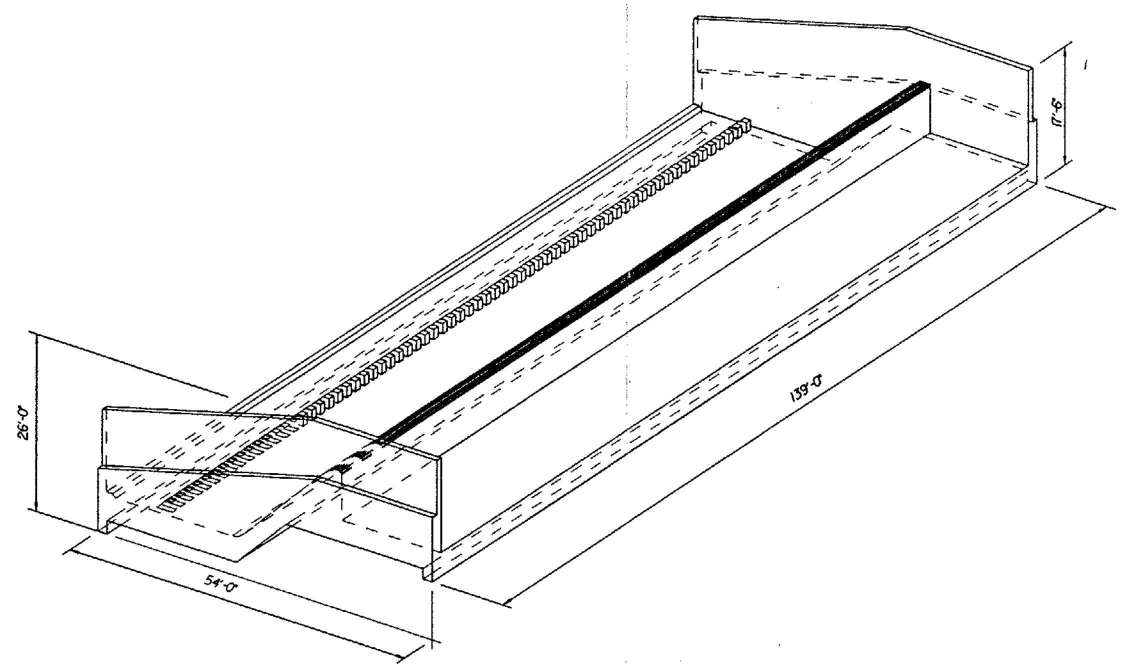


Reelfoot Creek Sediment Retention Structure
Primary Spillway Crest

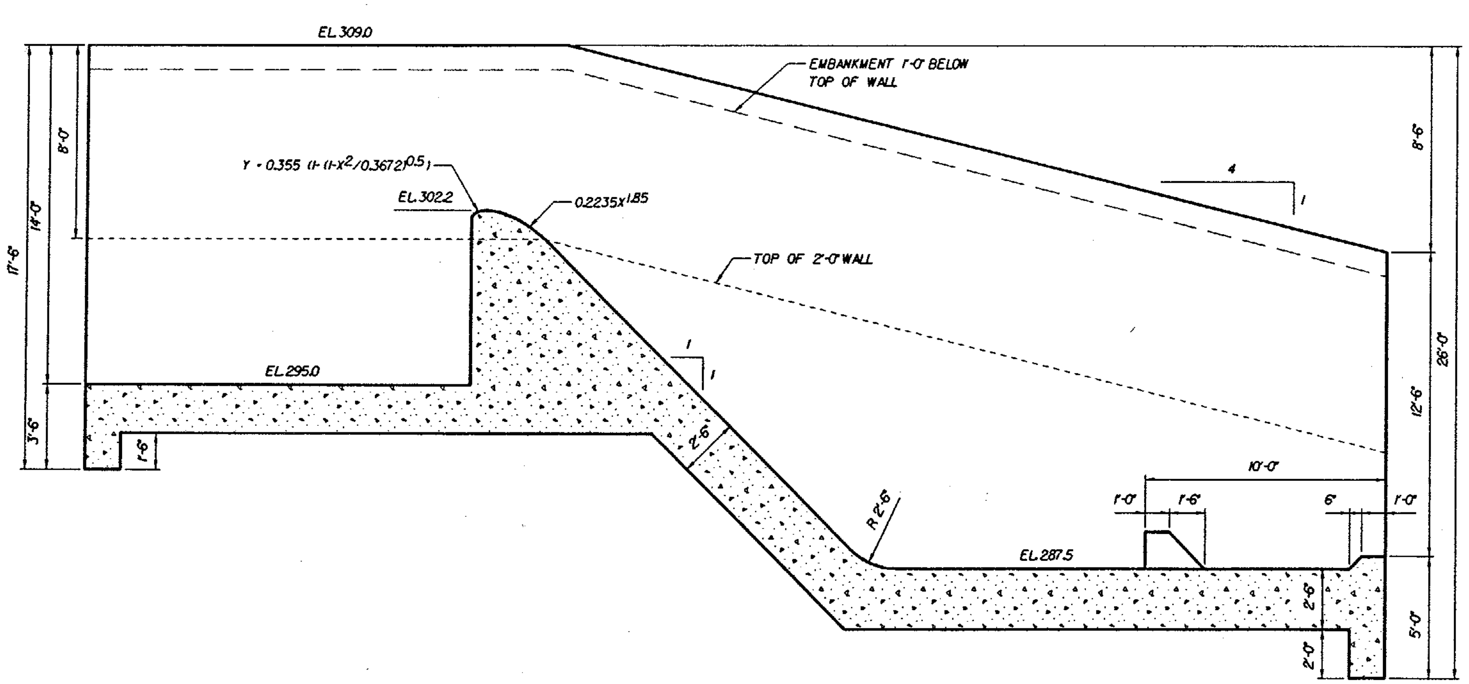




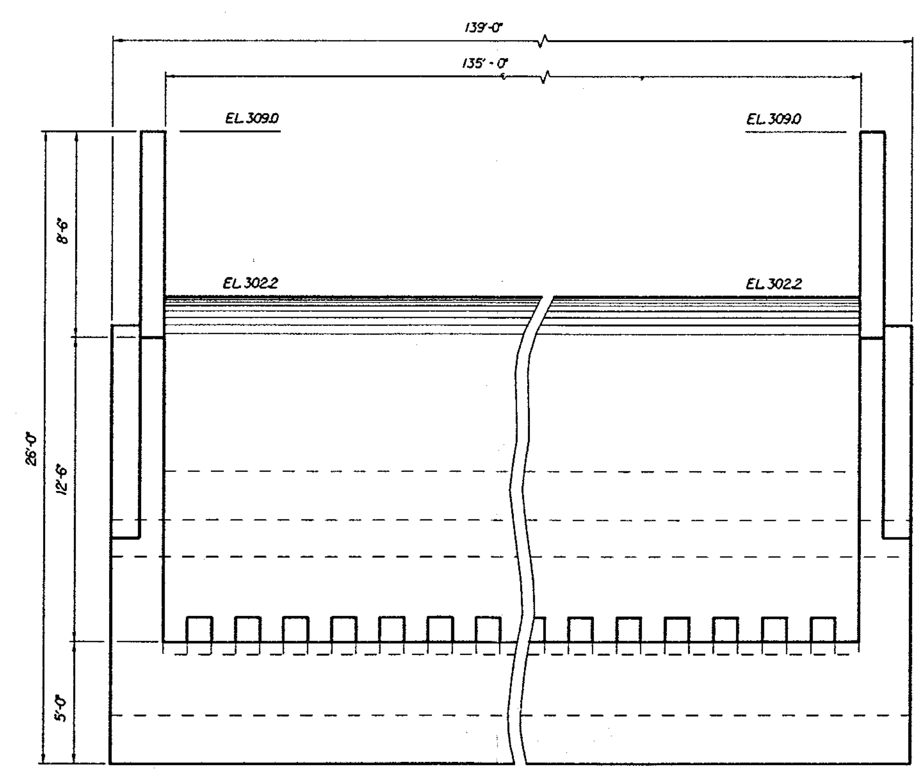
PLAN
SCALE 1/8" = 1'-0"



ISOMETRIC
N.T.S.



SECTION
SCALE 1/8" = 1'-0"



ELEVATION
SCALE 1/8" = 1'-0"

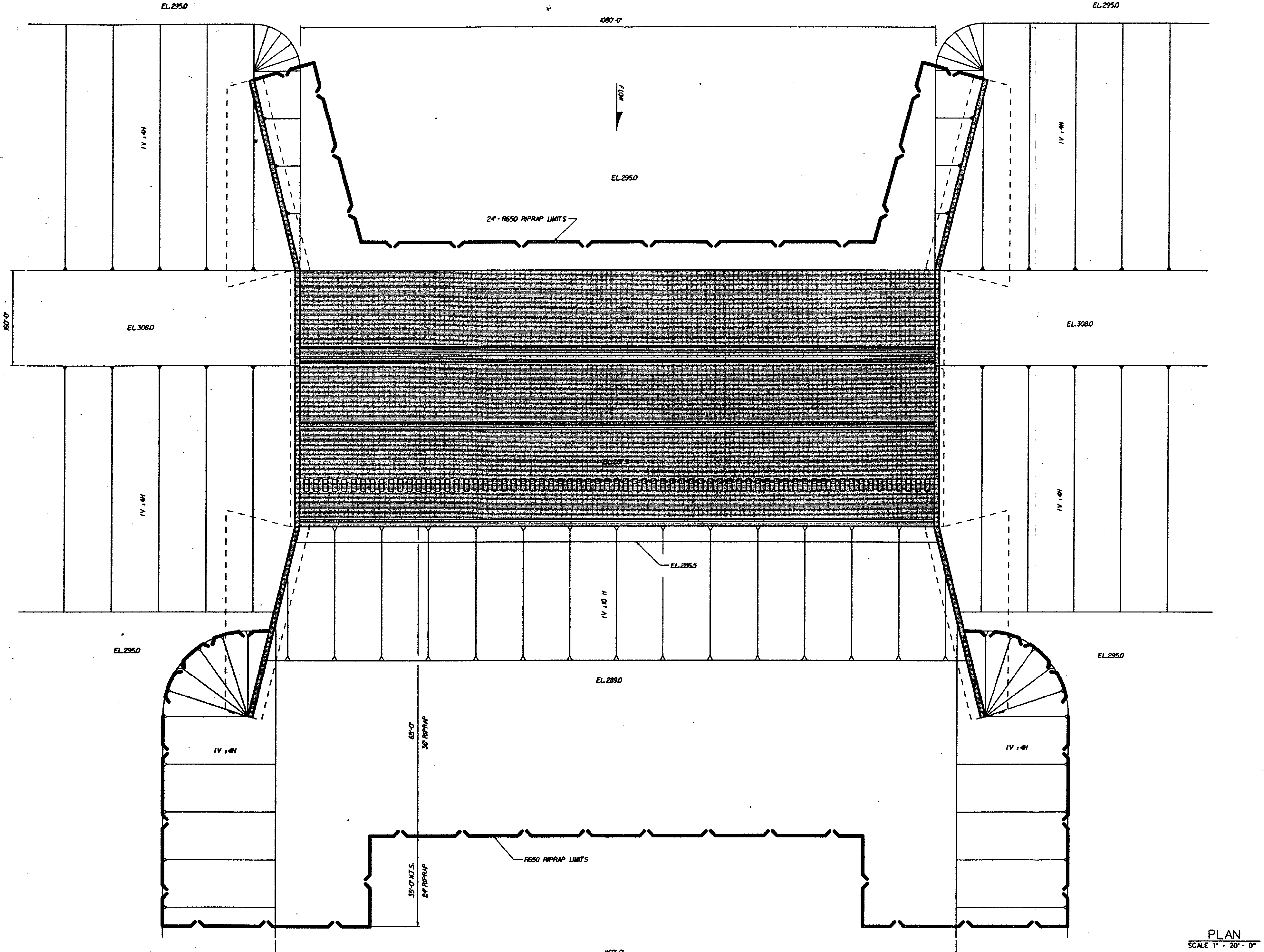
US Army Corps of Engineers Memphis District

U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
MEMPHIS, TENNESSEE

ENVIRONMENTAL RESTORATION NATURAL RESOURCES PROJECT
MISSISSIPPI RIVER AND TRIBUTARIES CONSTRUCTION

REELFOOT LAKE, TENNESSEE & KENTUCKY
REELFOOT CREEK SEDIMENT RETENTION BASIN
PRIMARY SPILLWAY - WEIR MONOLITH

PLATE
I-66



PLAN
SCALE 1" = 20' - 0"

US Army Corps of Engineers
Memphis District

U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
MEMPHIS, TENNESSEE

INTERGOVERNMENTAL RESTORATION NATURAL RESOURCES PROJECT

MISSISSIPPI RIVER AND TRIBUTARIES CONSTRUCTION

REELFOOT LAKE, TENNESSEE & KENTUCKY
REELFOOT CREEK SEDIMENT RETENTION BASIN
PRIMARY SPILLWAY - SITE PLAN

PLATE
I-67

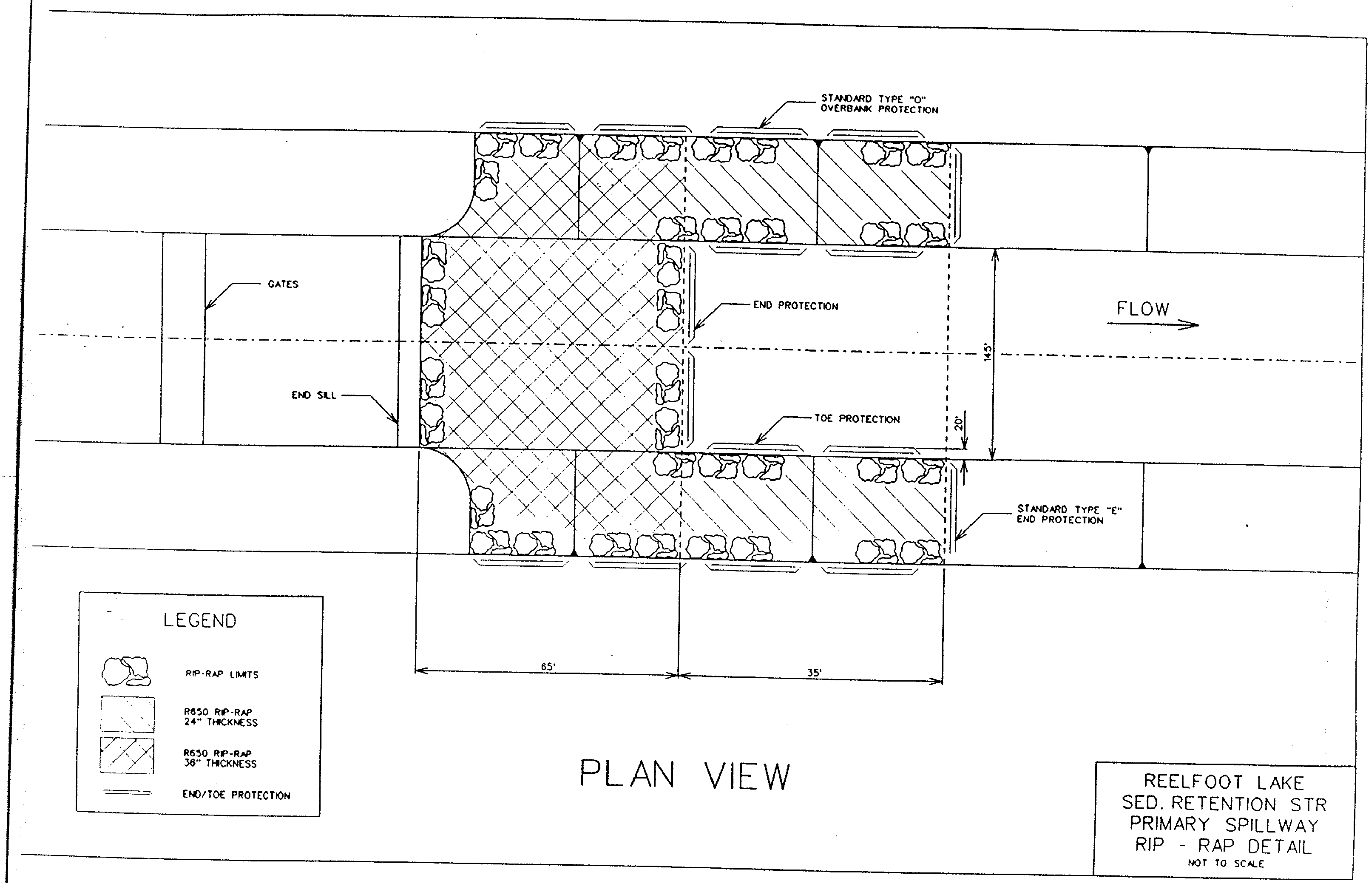
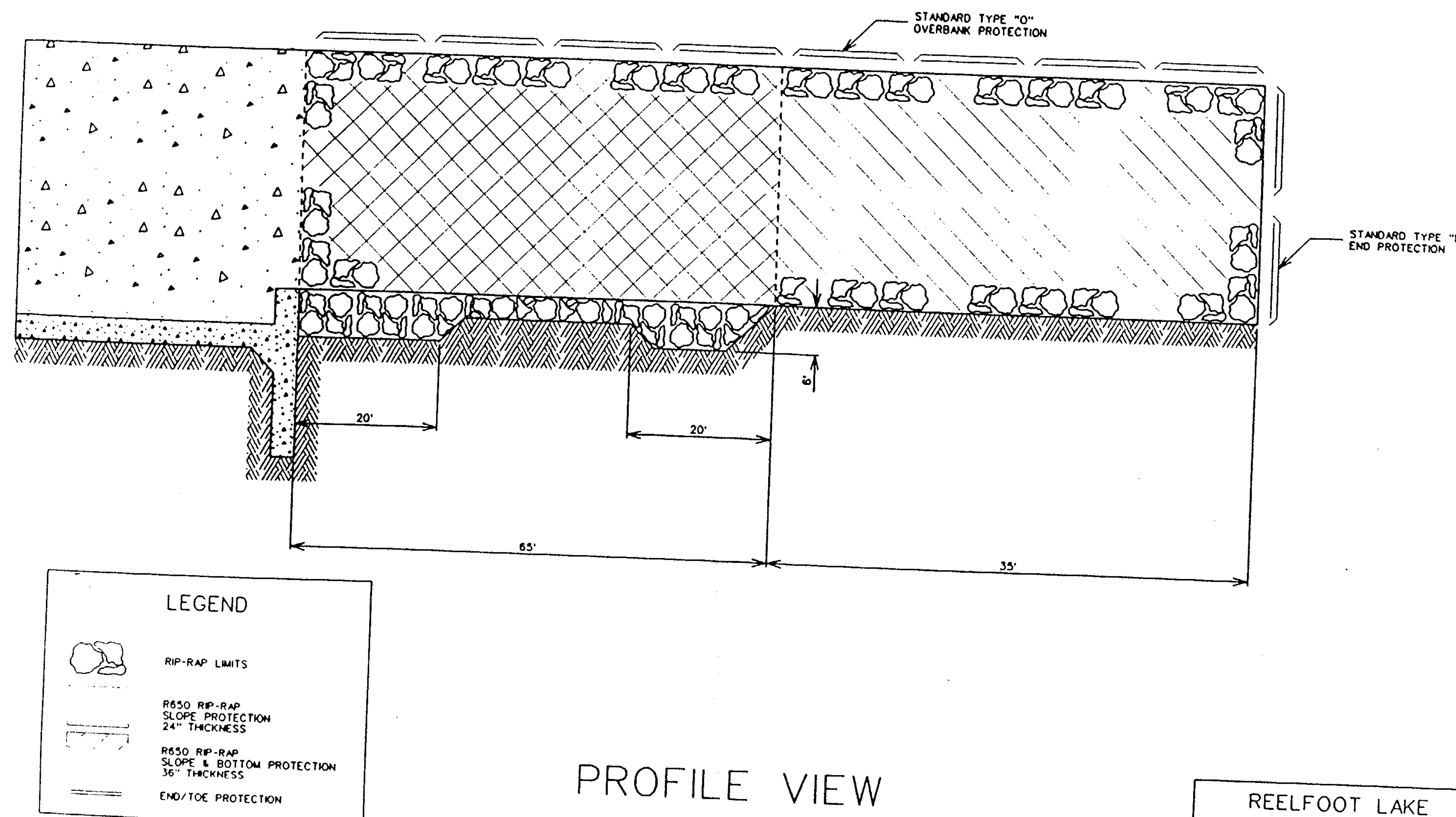

STANDARD TYPE "O" OVERBANK PROTECTION
GATES
END PROTECTION
FLOW
END SILL
TOE PROTECTION
145'
20'
STANDARD TYPE "E" END PROTECTION
LEGEND
RIP-RAP LIMITS
R650 RIP-RAP 24" THICKNESS
R650 RIP-RAP 36" THICKNESS
END/TOE PROTECTION
65'
35'
PLAN VIEW
REELFOOT LAKE
SED. RETENTION STR
PRIMARY SPILLWAY
RIP - RAP DETAIL
NOT TO SCALE

PLATE I-68

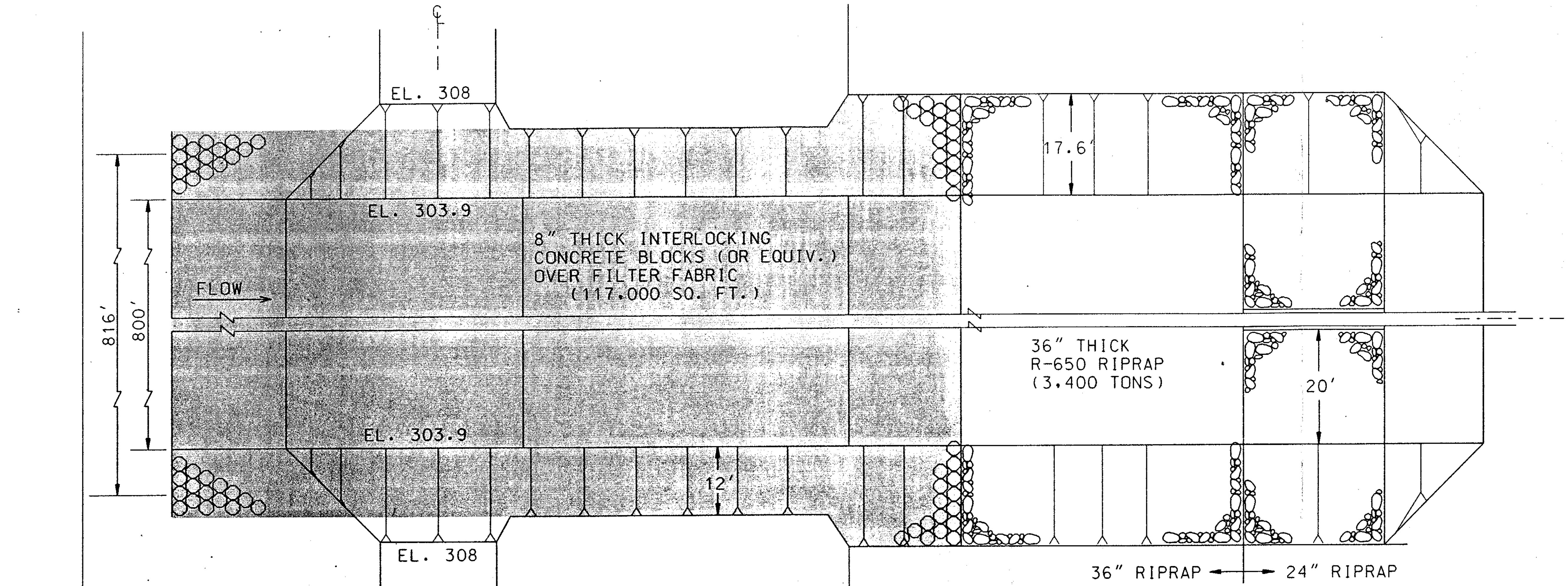


PROFILE VIEW

REELFOOT LAKE
SED. RETENTION STR
PRIMARY SPILLWAY
RIP - RAP DETAIL
NOT TO SCALE

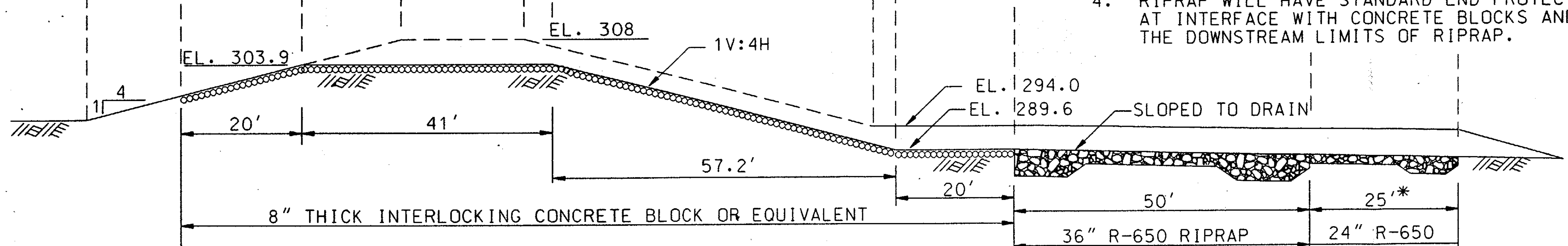
PLATE 1-69

PLAN VIEW
SCALE 1" = 20'



NOTES:

1. ALL SLOPES ARE 1V:4H UNLESS OTHERWISE NOTED
2. 8 INCH THICK INTERLOCKING CONCRETE BLOCKS OR EQUIVALENT TO BE PLACED OVER FILTER FABRIC
3. THE EROSION CONTROL SYSTEM SHALL BE ABLE TO WITHSTAND A VELOCITY OF 27 FEET PER SECOND ON THE DOWNSTREAM SLOPE OF THE EMBANKMENT
4. RIPRAP WILL HAVE STANDARD END PROTECTION AT INTERFACE WITH CONCRETE BLOCKS AND AT THE DOWNSTREAM LIMITS OF RIPRAP.



PROFILE VIEW
SCALE 1" = 20'

*SLOPE AND TOE PROTECTION ONLY

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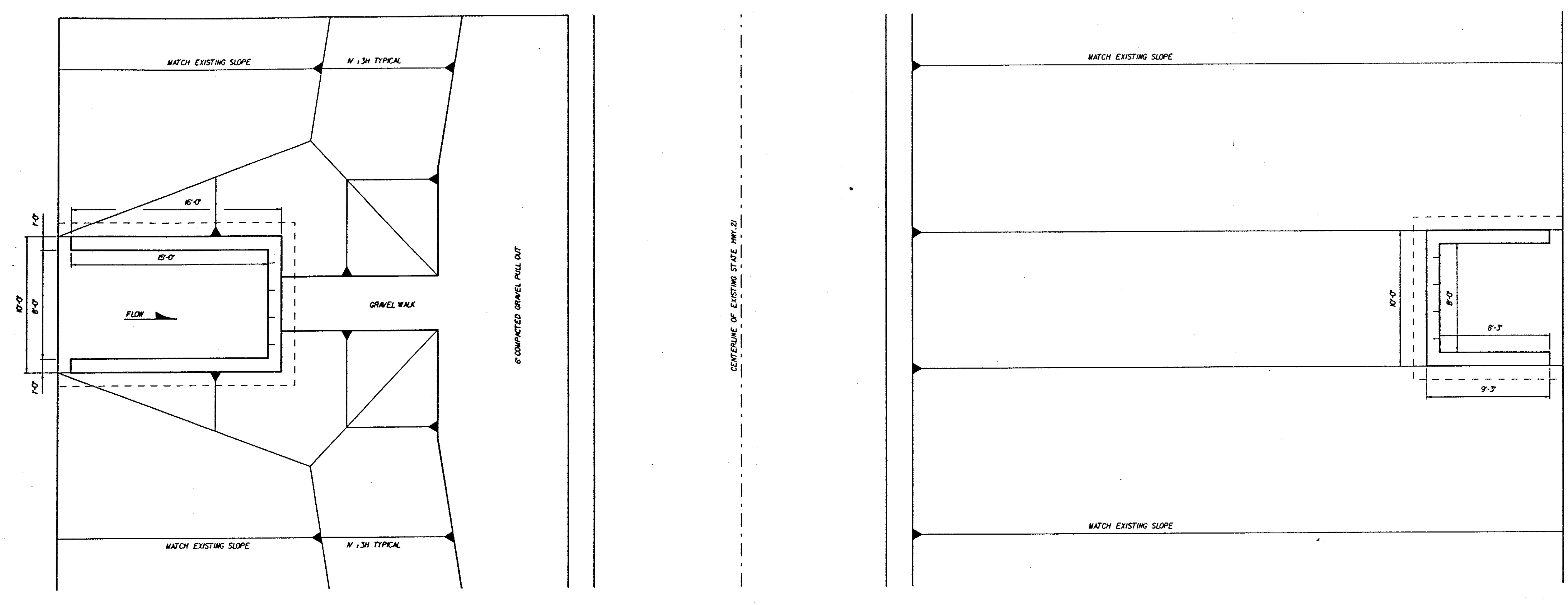
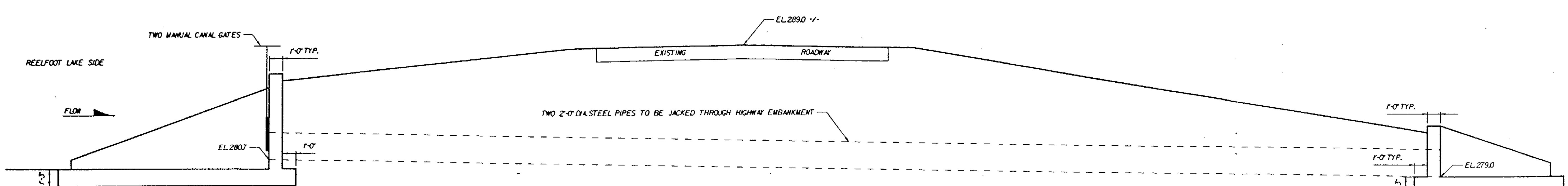
MISSISSIPPI RIVER AND TRIBUTARIES CONSTRUCTION
ENVIRONMENTAL RESTORATION NATURAL RESOURCES PROJECT

REELFOOT LAKE, TENNESSEE & KENTUCKY
SEDIMENTATION BASIN
EMERGENCY SPILLWAY

PLATE
I-70



PLAN
SCALE 1/4" = 1'-0"



SECTION
SCALE 1/4" = 1'-0"

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ENVIRONMENTAL RESTORATION
NATURAL RESOURCES PROJECT
MISSISSIPPI RIVER AND TRIBUTARIES
CONSTRUCTION

REELFOOT LAKE, TENNESSEE & KENTUCKY
WATERFOWL MANAGEMENT UNITS / SHELBY LAKE
REELFOOT LAKE WATER SUPPLY STRUCTURE

PLATE
I-71

UNIT	ACRES	POOL ELEV
A	67	285.0
B	82	284.0
C	91	283.0
D	50	281.5
E	118	283.0
F	75	281.5
FOREST HARDWOODS	342	---
SHELBY LAKE	170	280.0

APPENDIX B

SECTION 2:
STRUCTURAL, ELECTRICAL AND MECHANICAL

APPENDIX B

SECTION 2, Part A: STRUCTURAL

STRUCTURAL

SECTION 2 - STRUCTURAL, ELECTRICAL & MECHANICAL

PART A - STRUCTURAL DESIGN DEVELOPEMENT

2-A-01. STRUCTURAL.

This section presents the basic criteria, assumptions, methods of analysis, and results of the computations for the preliminary design of the structures identified in the Reelfoot Lake Project feasibility study. Sufficient engineering and design was performed to enable refinement of the project features, prepare the baseline cost estimate, develop a design and construction schedule to allow detailed design to begin immediately following receipt of preconstruction engineering and design (PED) funds. The Hydraulics and Hydrology Branch identified the following structural items:

1) Reelfoot Lake Alternate Spillway
   - a) Spillway
   - b) Highway Bridge
2) Reelfoot Creek Sediment Retention Basin
   - a) Primary Spillway
   - b) Low Level Spillway
3) Lake Isom Outlet Structure
4) Shelby Lake Inlet

Functional requirements were defined in the Reelfoot Lake Tennessee & Kentucky, Reconnaissance Report, November 1993. With this information structural configurations were developed for each item. The dimensions for the majority of structures were based on historical data of similarly sized structures and preliminary design calculations. The following unit weights, lateral coefficients, load factors and material properties were used:

<u>Material</u>

Concrete:	150.0 pcf 4000.0 psi
Reinforcement:	60.0 ksi
Earth - Saturated	125.0 pcf
Water:	62.5 pcf
Steel:	36.0 ksi

Load Factors

Dead Load:	1.4
Live Load:	1.7
Hydraulic Factor:	1.3

Soil Condition

At Rest, Level	0.5

Earthquake

Seismic Zone	3

2-A-02. DESIGN CRITERIA.

The structural design criteria are, where applicable, based on appropriate U.S. Army Corps of Engineers design manuals. Where locations, site conditions, types of structures, owner's operational requirements, and safety dictate more stringent requirements, other design criteria from appropriate state or federal agencies were selected. All criteria are currently accepted throughout the engineering community and constitute sound engineering practice. The criteria to be used are contained herein and may be modified as needed to complete final designs.

a) EM 1110-2-1603, Hydraulic Design of Spillways

b) EM 1110-2-2000, Standard Practice for Concrete for Civil Works Structures, Change 1

c) EM 1110-2-2102, Waterstops and Other Preformed Joint Materials for Civil Works Structures

d) EM 1110-2-2104, Strength Design for Reinforced - Concrete Hydraulic Structures

e) EM 1110-2-2105, Design of Hydraulic Steel Structures, Change 1

f) EM 1110-2-2400, Structural Design of Spillways & Outlet Works

g) EM 1110-2-2502, Retaining and Flood Walls

h) EM 1110-2-2902, Conduits, Culverts and Pipes CH 1-3

i) ER 1110-2-1806, Earthquake Design & Evaluation of Civil Works Projects

j) ER 1110-2-8157, Responsibility for Hydraulic Steel Structures

k) TM-5-809-1, Structural Design Criteria for Loads

l) ACI Building Code 318

m) AISC Manual of Steel Construction ASD, 9th Edition

n) Standard Building Code

2-A-03. REELFOOT LAKE ALTERNATE SPILLWAY

The Reelfoot Lake alternate spillway is located approximately 1200 feet west of the existing structure. The gate sill is 5'-3" above the inlet channel bottom elevation creating a normal operating head on the structure of 15'-3" with a maximum head of 17'-3". The normal gate-operating head is 9'-0" with a maximum head of 12 ft. The proposed structure has six 20'-0" wide roller gates. The gates will be purchased from a manufacture of this type equipment. The Big Lake Area Arkansas– Ditch-81 control structure was used to develop the preliminary design and subsequent quantity estimates for the Reelfoot Lake alternate spillway. The Ditch 81 structure has tainter gates rather than roller gates and was designed for a maximum head of 22'-0" approximately 5'-0" higher than required for the proposed project. The Ditch-81 control structure has performed well for over 20 years. This structure has also experienced extreme loadings. In December 1994 the structure was operated improperly causing the rising waters to flank the structure. The proposed Reelfoot spillway structural element sizes are equivalent to the Ditch 81. See Section 3, Geology and Soils, of this report for the various load cases analyzed to verify the overall stability of the structure. The proposed site layout and new structure are shown on drawings II-A-01 thru II-A-06.

The existing spillway is essentially a box culvert with gates on the upstream side. The original structure was built in 1930 and consisted of 20-10'-0"W x 7'-3"H barrels with a 4'-0" wide fish ladder on the west end. Two additional 19'-6" wide barrels were added providing a total of 22 barrels. The structure will be abandoned in place with provision to permanently close the hydraulic portion of the structure. This will be done by filling all barrels with a flowable grout and providing a 160'-0" wide embankment on the downstream side of the structure. The unusually wide embankment is being provided because it is a convenient and necessary location to put excess excavation material. The existing spillway is shown on drawings II-A-07 thru II-A-12.

A new highway bridge will be required to cross the proposed channel. The State of Tennessee was contacted to determine the requirements for the new bridge. The bridge was designed according to their requirements. The proposed bridge is shown in drawings II-A-13 thru II-A-15.

During review of the spillway structure several items were noted as areas of potential savings that may be gained during the final design. The first items were the inlet and outlet walls. The walls are tapered on the backside. It has been documented that it can be less expensive to provide more concrete with vertical faces than to use less concrete and taper the walls. The last item that was noted was that the stilling basin retaining walls are independent of the main slab. This required a large footing projecting back into the fill. It may be more cost-effective to tie the walls into the stilling basin slab forming a U-shaped channel. These items will be investigated further in the final design.

2-A-04. REELFOOT CREEK SEDIMENT RETENTION BASIN

The Reelfoot Creek Sediment Retention Basin has a total of five structures in a 17,000-foot embankment. This included an 800-ft. wide earthen emergency spillway that did not require any work by the Structural, Mechanical & Electrical section. The other four structures include a primary spillway (see Drawings II-A-16 thru II-A-18) and three low-level spillways (see drawing II-A-19).

The primary spillway walls were sized using an equivalent fluid pressure of 94pcf and a factor of safety of 2.21. The stilling basin slabs were sized using the wall thickness plus six inches to account for the additional cover required for this type slab. The inlet slab used the same thickness required for the walls, because it will not be subjected to same type of abrasion as the stilling basin slab. The weir portion of the structure size was determined by hydraulic and constructabilty requirements.

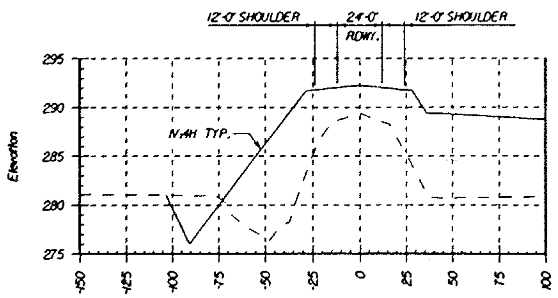
The low-level spillway is shown on drawing II-A-19. This spillway has two distinct parts. The intake structure and the stilling basin. The intake structure was sized assuming up-lift would be the controlling loading. Additional concrete was distributed evenly over the structure to counter act uplift. The sediment retention basin is designed such that the inlet structure will become inundated and fill from the top. Although the inlet structure has five 2'-0" diameter holes in one side, uplift was determine with no interior water and exterior water to the top of the walls (el. 301.0). Sediment will be filled in to an elevation of 301.0 at the end of the project's life. Therefore, the inlet structure's wall reinforcing was designed using an equivalent fluid pressure of 94pcf with a factor of safety of 2.21. The stilling basin (outlet structure) walls were sized using the same equivalent fluid pressure and factor of safety. The slab for the stilling basin was sized using the wall thickness plus four inches to account for additional cover requirements. Concrete pipe meeting ASTM C76 Class IV was use for estimating purposes. The pipe has an overburden of approximately eleven feet at the crown of the embankment.

2-A-05. LAKE ISOM OUTLET STRUCTURE.

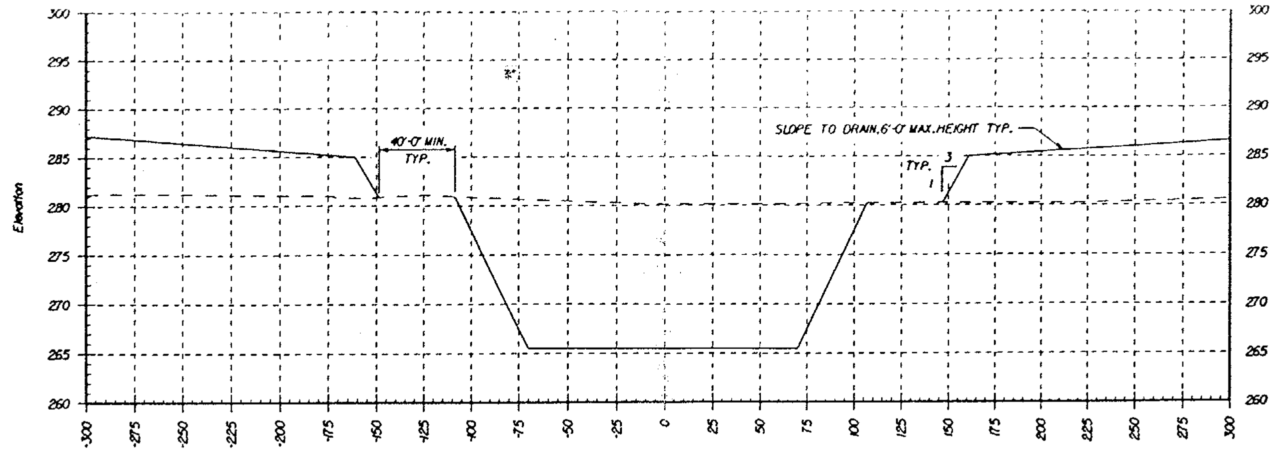
The Lake Isom outlet structure is shown on drawing II-A-20. The proposed Lake Isom outlet is a replacement for an existing structure. The existing structure is a 5'-0"W by 10'-0"H box culvert with wooden 4-by-4 stoplogs on the inlet side. The existing structure has twelve-inch walls and slabs. Lake Isom is a dry lake normally. This project will provide well water in order to maintain lake levels in the future. The proposed structure was designed using an equivalent fluid pressure of 94pcf and a factor of safety of 2.21. The size of the structure was checked against standard State Highway box culverts. It is proposed to use aluminum stoplogs in the new structure to allow for manual placement.

2-A-06. WATERFOWL MANAGEMENT UNITS / SHELBY LAKE.

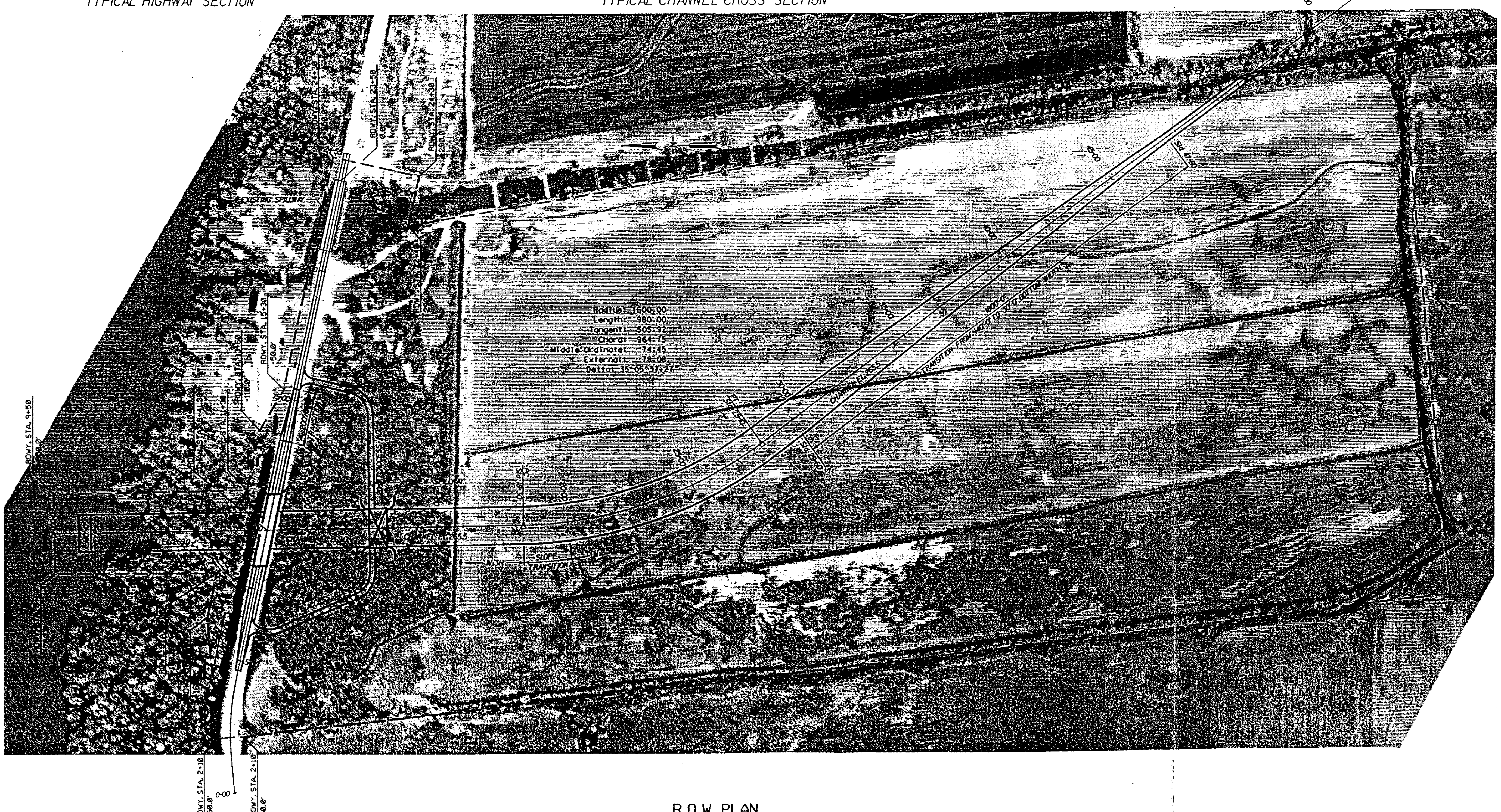
The Waterfowl Management Units and Shelby Lake will be supplied water through the structure shown in drawing II-A-21. This structure consists of two 2'-0" diameter steel pipes that will be jacked under State Highway 21. The pipes will have concrete head walls and a base slab. Two off-the-shelf manual canal gates with a maximum head of 7'-0" will be required on the up-stream end. Drawing II-A-21 assumes that a 250-ft. long ditch with a 10-ft. bottom width will be excavated out to the lake. The need for the downstream headwall should be evaluated during final design.



TYPICAL HIGHWAY SECTION



TYPICAL CHANNEL CROSS SECTION



R.O.W. PLAN

SCALE 1" = 400'

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ENVIRONMENTAL RESTORATION NATURAL RESOURCES PROJECT

MISSISSIPPI RIVER AND TRIBUTARIES CONSTRUCTION

REELFOOT LAKE, TENNESSEE & KENTUCKY
REELFOOT LAKE ALTERNATIVE SPILLWAY
RIGHT-OF WAY & TYPICAL SECTIONS

DRAWING
II-A-1
1 OF 21

CONSTRUCTION EXCAVATION PLAN
SCALE 1" = 200'

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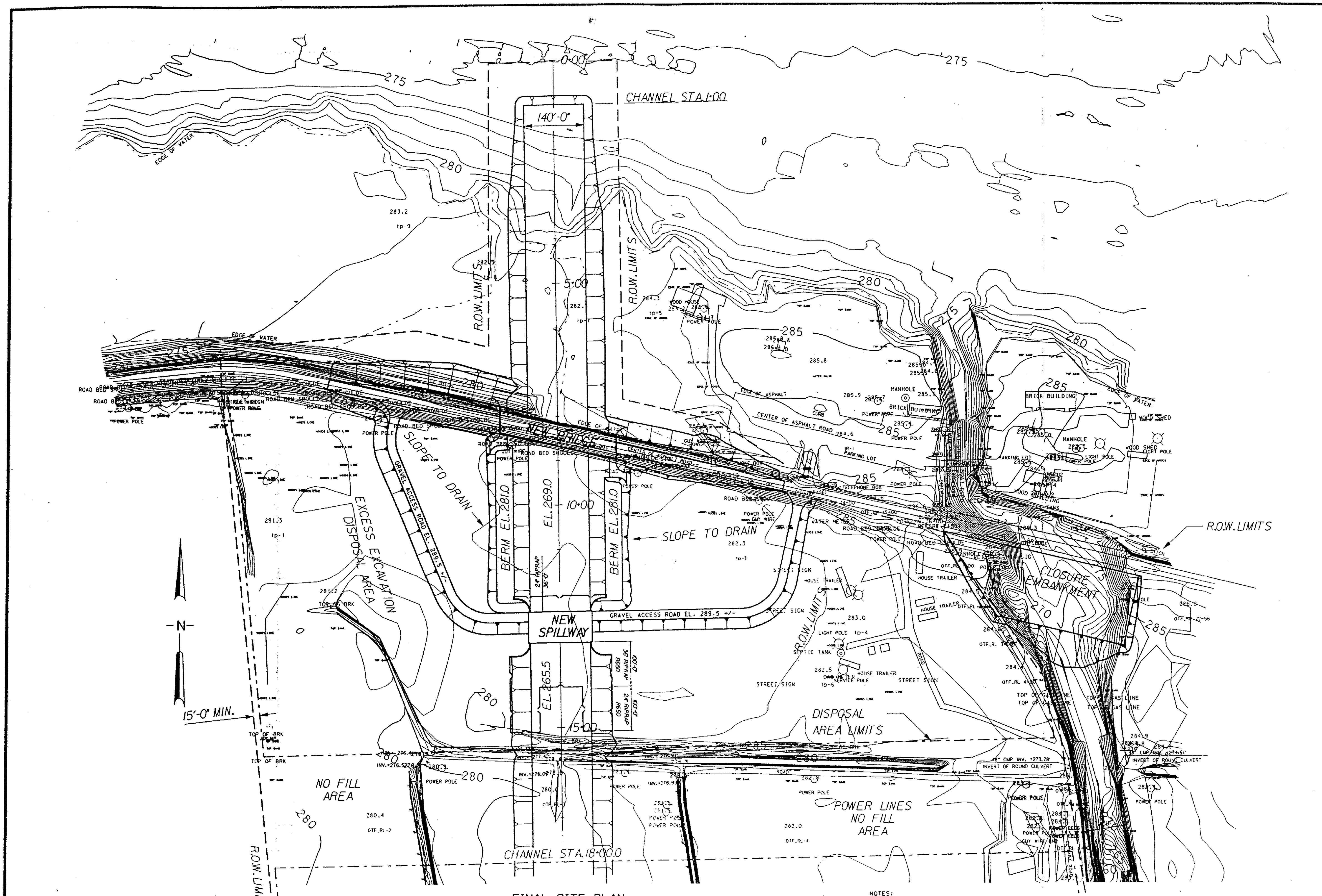
REELFOOT LAKE, TENNESSEE & KENTUCKY
REELFOOT LAKE ALTERNATIVE SPILLWAY
CONSTRUCTION EXCAVATION PLAN

DRAWING
II-A-2
2 OF 21



FINAL SITE PLAN

SCALE 1" = 200'

NOTES:
1. HIGHWAY SLOPES ARE 1V:4H. ALL OTHER SLOPES ARE 1V:3H UNLESS NOTE OTHERWISE.

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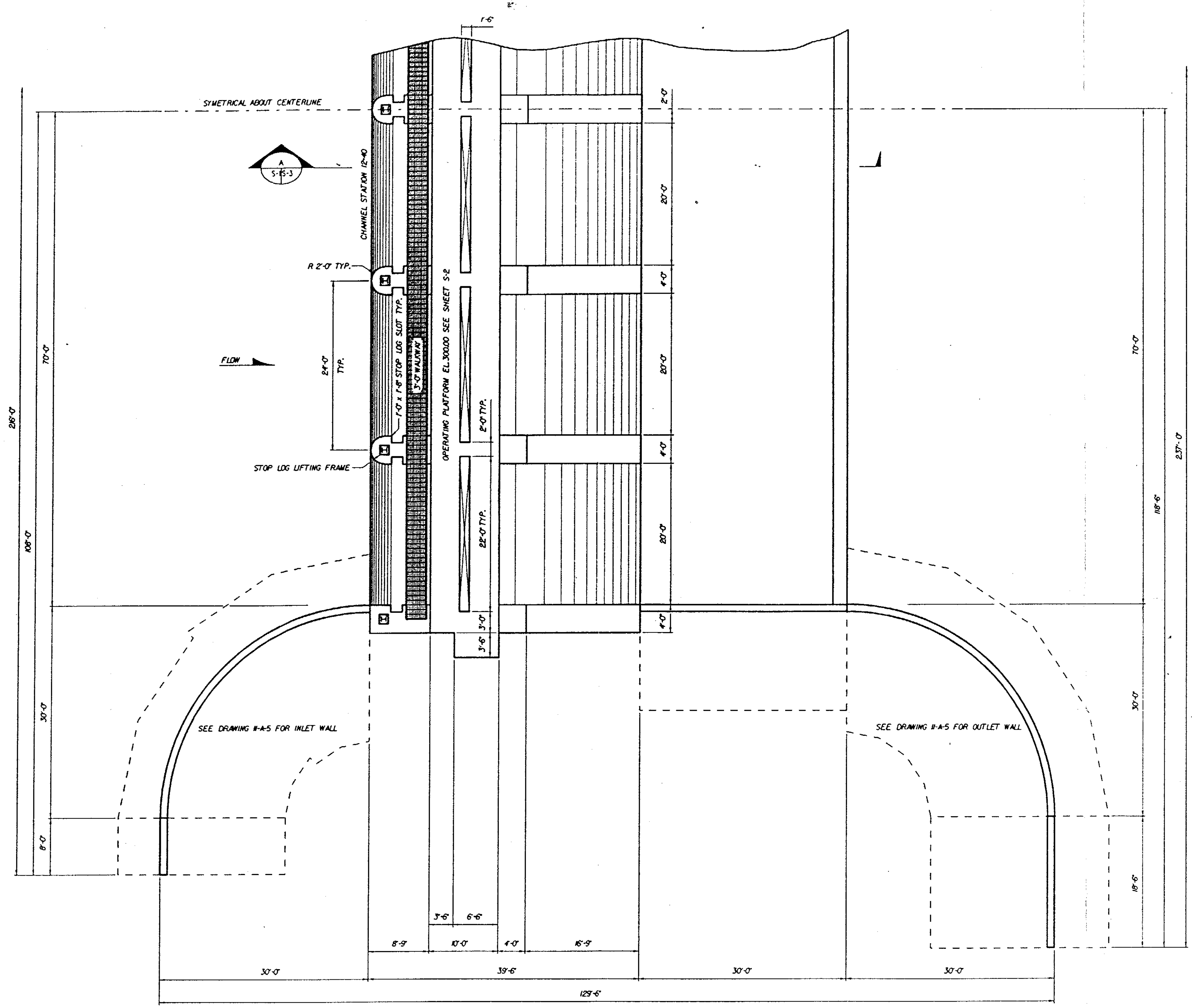
ENVIRONMENTAL RESTORATION
NATURAL RESOURCES PROJECT
MISSISSIPPI RIVER AND TRIBUTARIES
CONSTRUCTION

REELFOOT LAKE, TENNESSEE & KENTUCKY
REELFOOT LAKE ALTERNATIVE SPILLWAY
FINAL SITE PLAN

DRAWING
II-A-3



PLAN
SCALE 1/16" = 1'-0"

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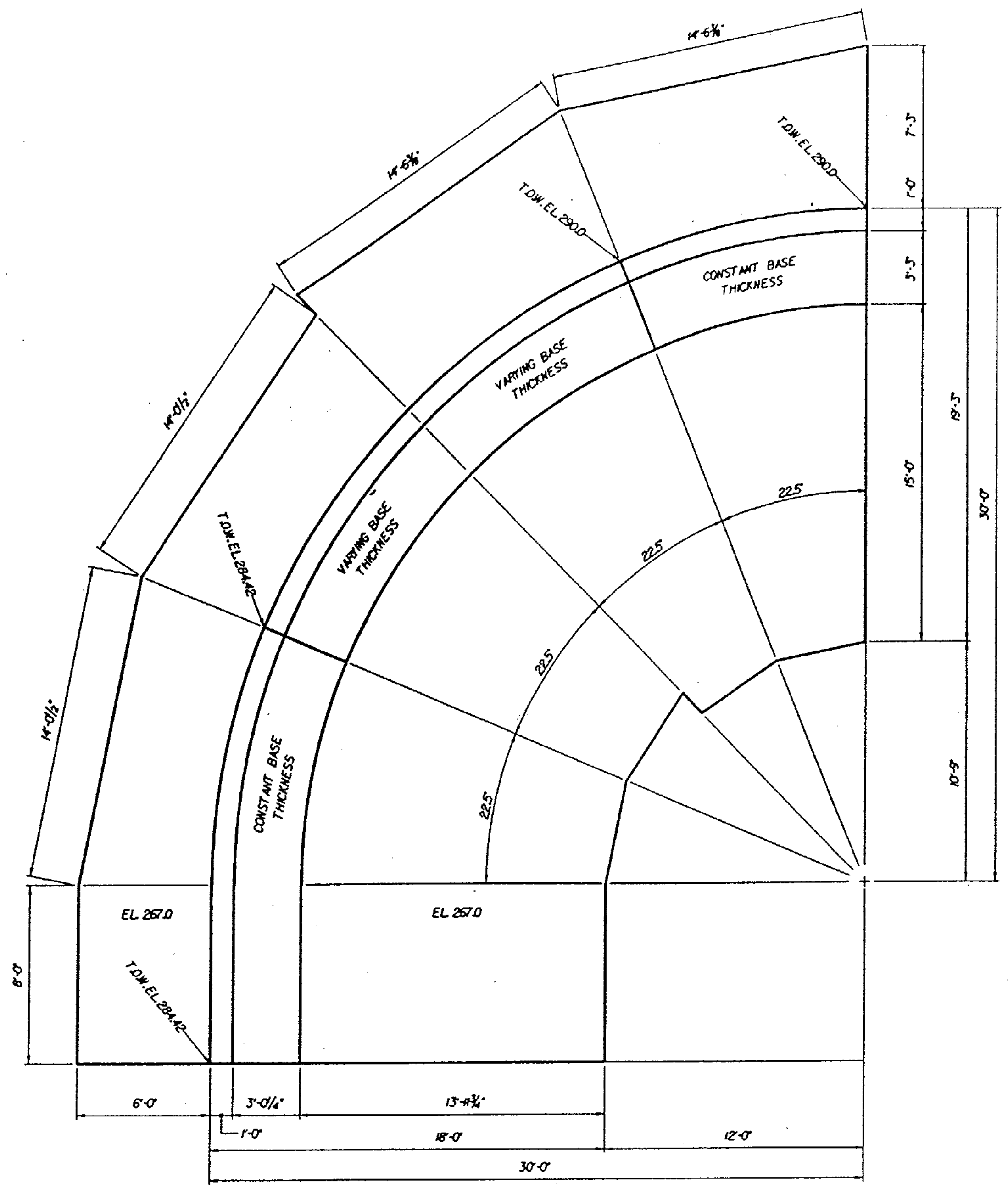
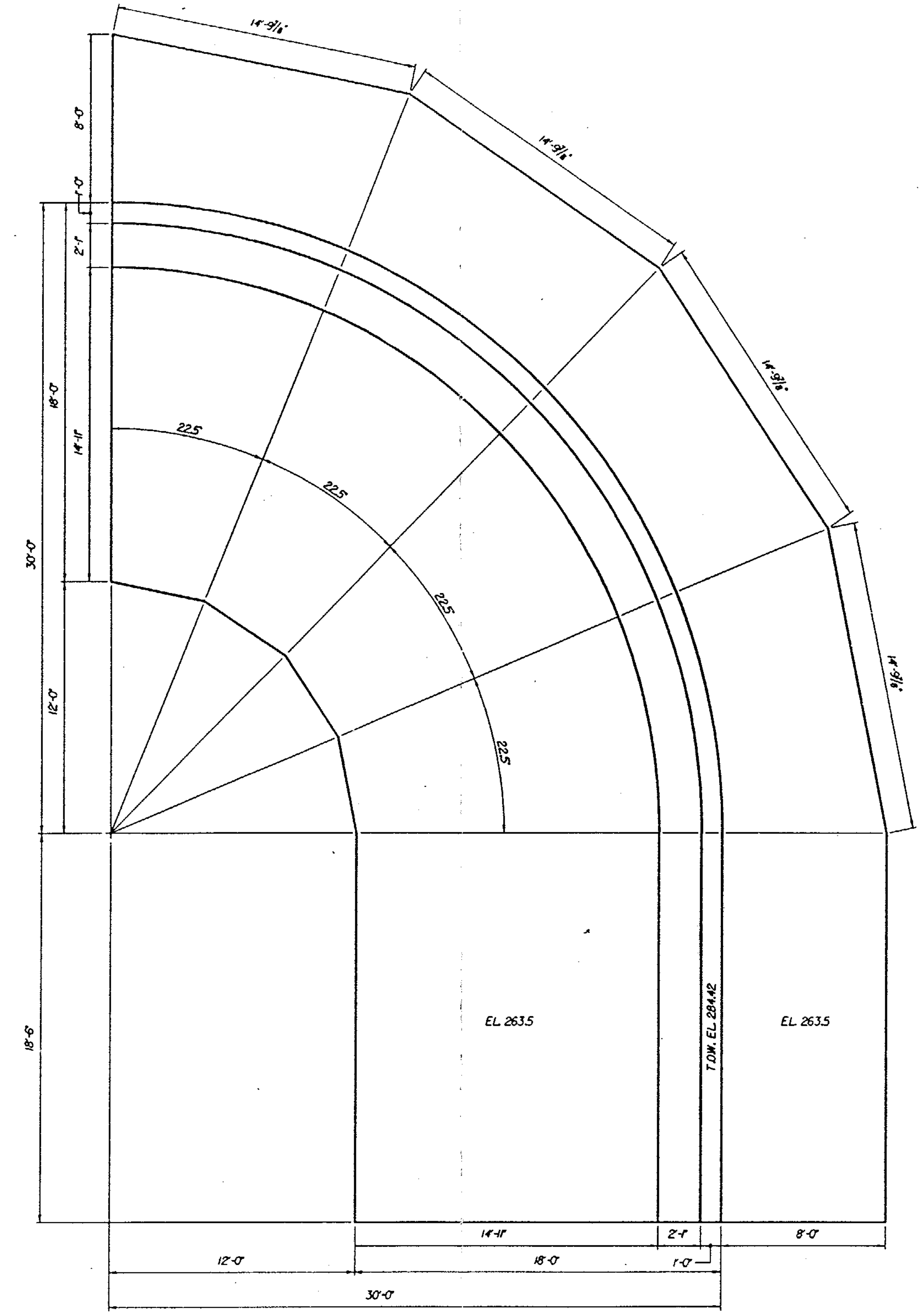
ENVIRONMENTAL RESTORATION NATURAL RESOURCES PROJECT
MISSISSIPPI RIVER AND TRIBUTARIES CONSTRUCTION

REELFOOT LAKE, TENNESSEE & KENTUCKY
REELFOOT LAKE ALTERNATIVE SPILLWAY
SPILLWAY PLAN

DRAWING II-A-4



INLET WING WALL
SCALE ⅛" = 1'-0"



OUTLET WING WALL
SCALE ⅛" = 1'-0"

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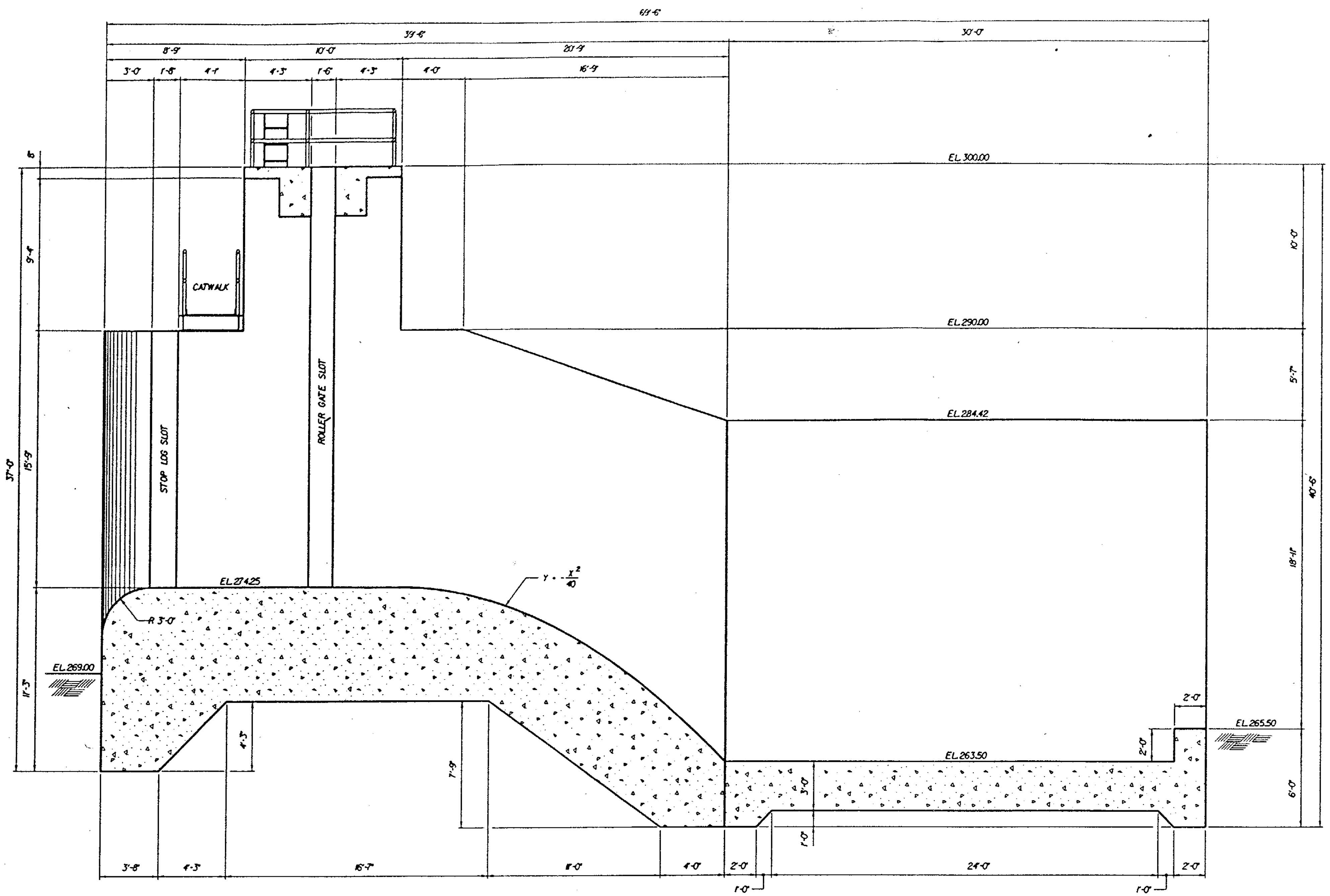
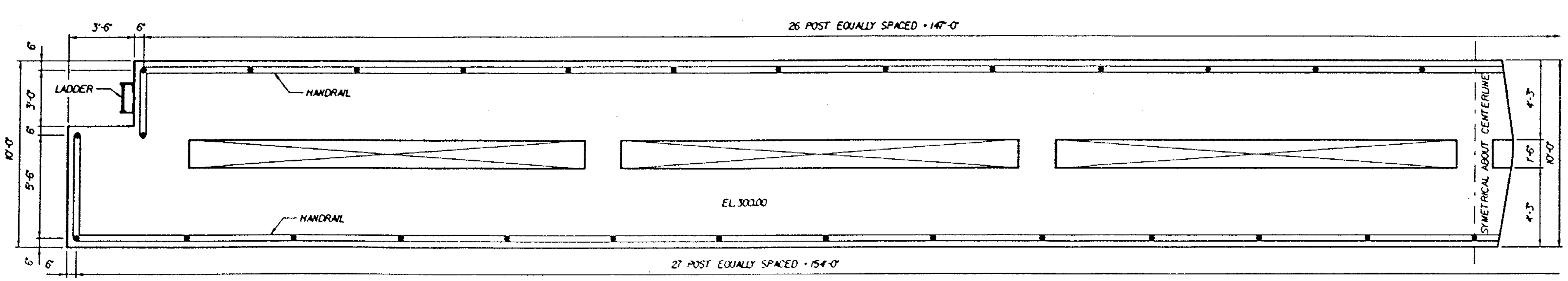
ENVIRONMENTAL RESTORATION NATURAL RESOURCES PROJECT
MISSISSIPPI RIVER AND TRIBUTARIES CONSTRUCTION

REELFOOT LAKE, TENNESSEE & KENTUCKY
REELFOOT LAKE ALTERNATIVE SPILLWAY
WING WALL PLANS

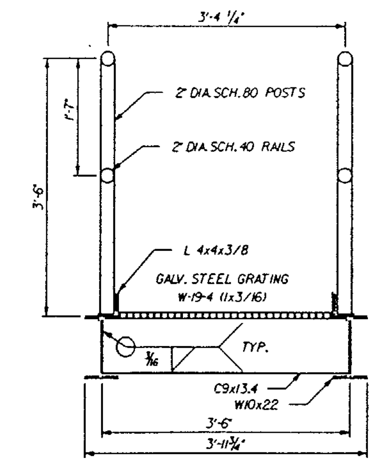
DRAWING
II-A-5
5 OF 21



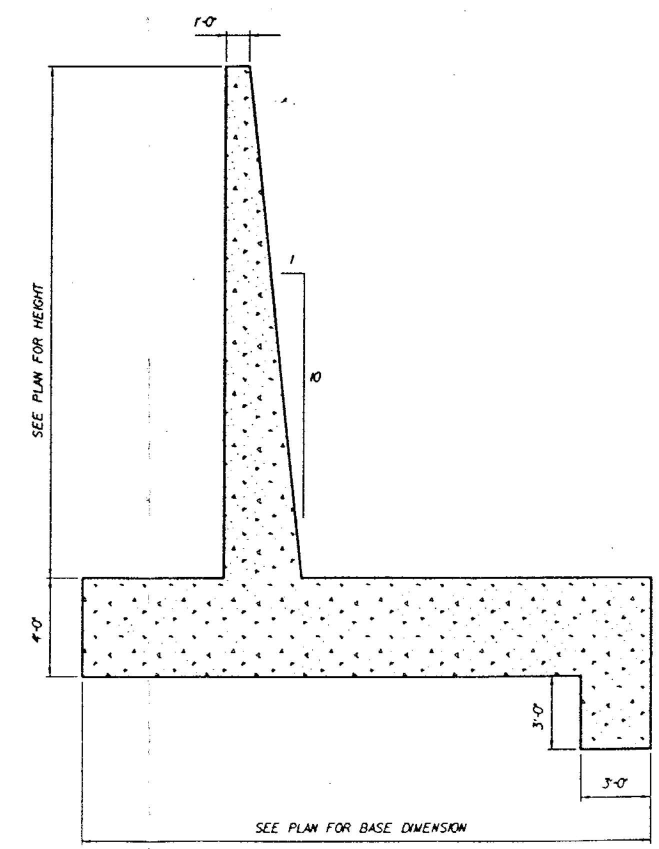
SECTION (A / S-15-2)
SCALE 1/8" = 1'-0"



OPERATING PLATFORM
SCALE 1/8" = 1'-0"



CATWALK SECTION
SCALE 1 1/2" = 1'-0"



TYPICAL WING WALL SECTION
SCALE 1/8" = 1'-0"

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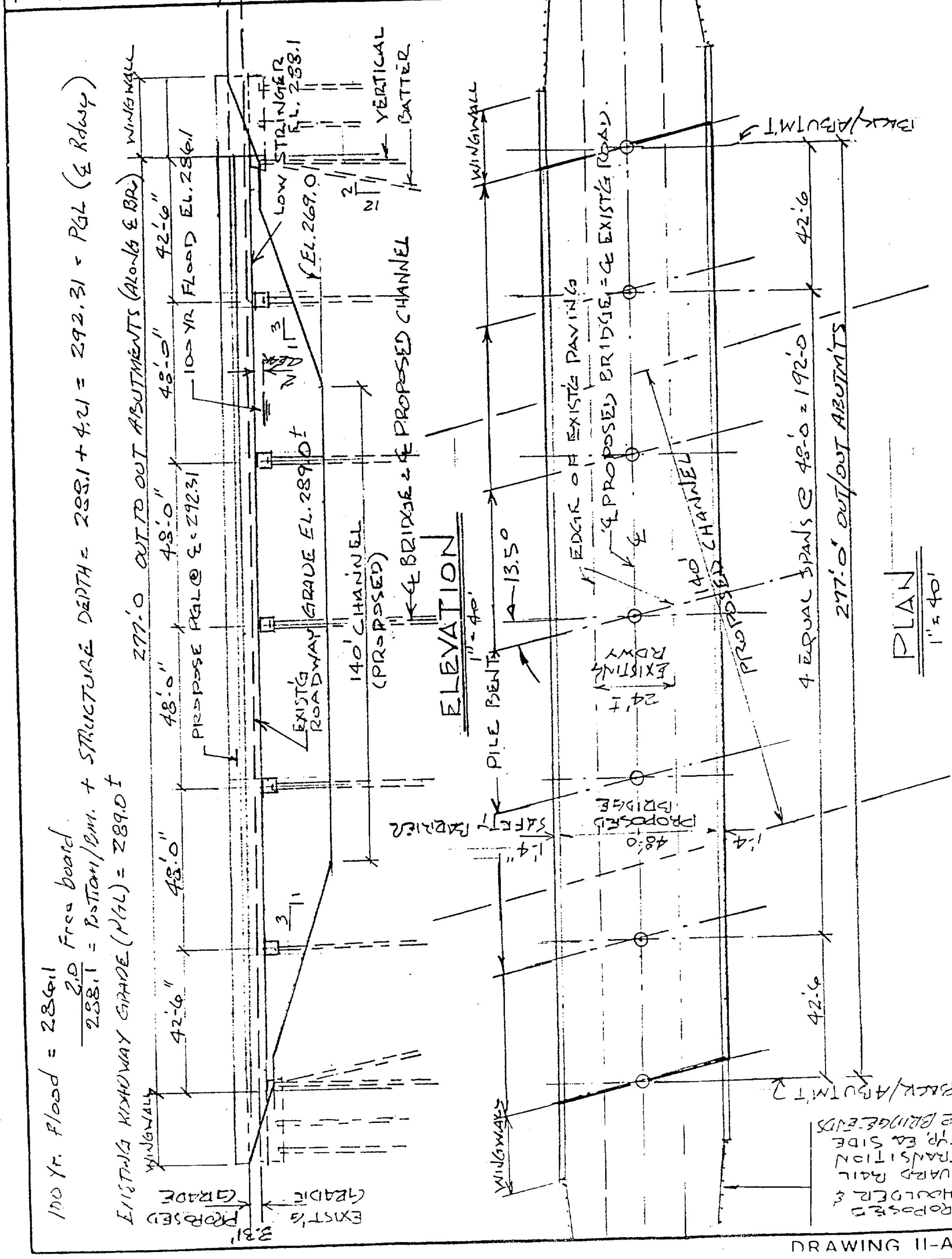
ENVIRONMENTAL RESTORATION
NATURAL RESOURCES PROJECT
MISSISSIPPI RIVER AND TRIBUTARIES
CONSTRUCTION

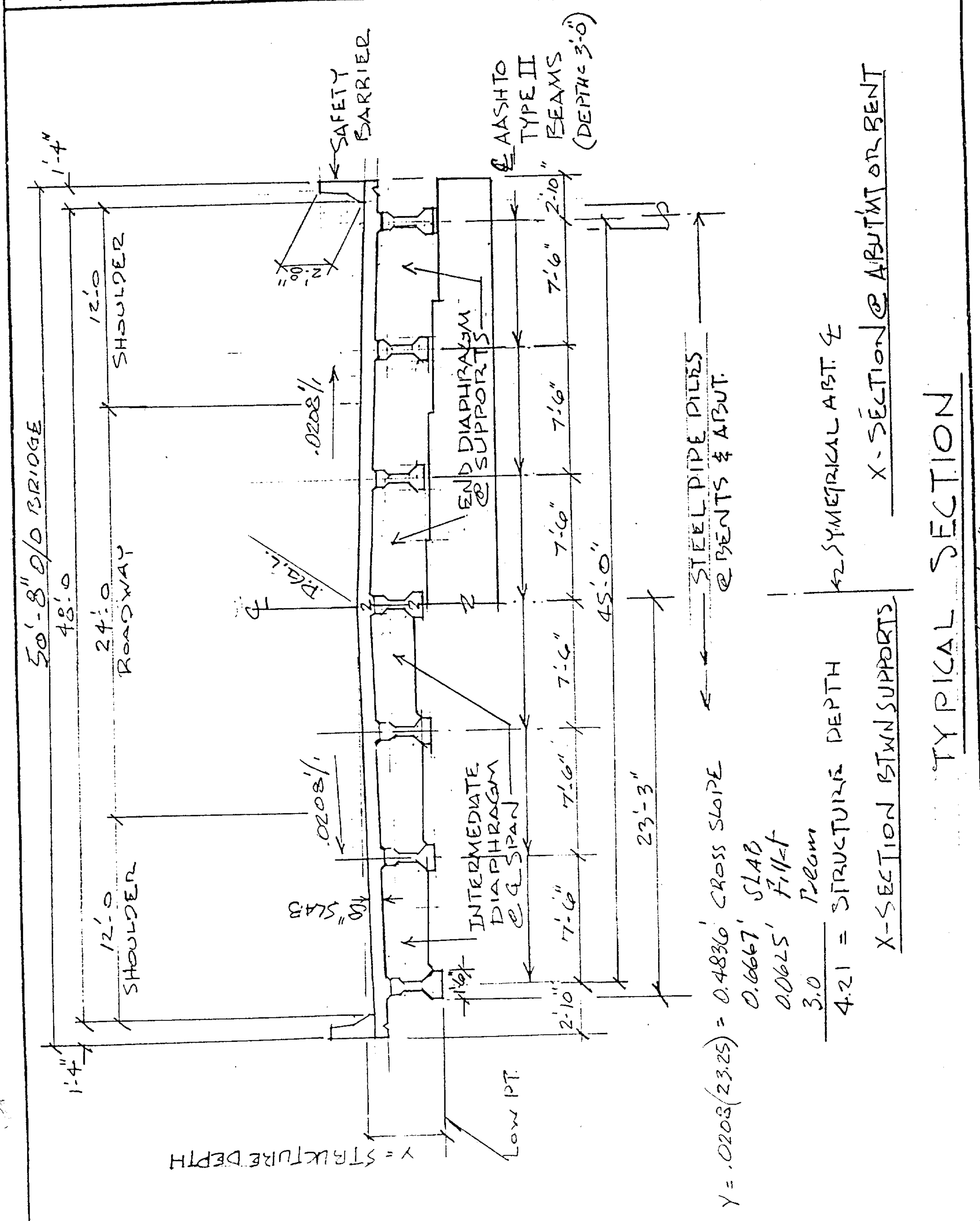
REELFOOT LAKE, TENNESSEE & KENTUCKY
REELFOOT LAKE ALTERNATIVE SPILLWAY
SPILLWAY SECTIONS

DRAWING
II-A-6
6 OF 21

SUBJECT: REELFOOT LAKE FEASIBILITY	COMPUTED BY: WT.	DATE: 13 MAY '97	FILE NO. B-2
HWY 21 BRIDGE/SPILLWAY CHANNEL	CHECKED BY: MSW TN DOT	DATE: 3/12/98	SHEET NO.

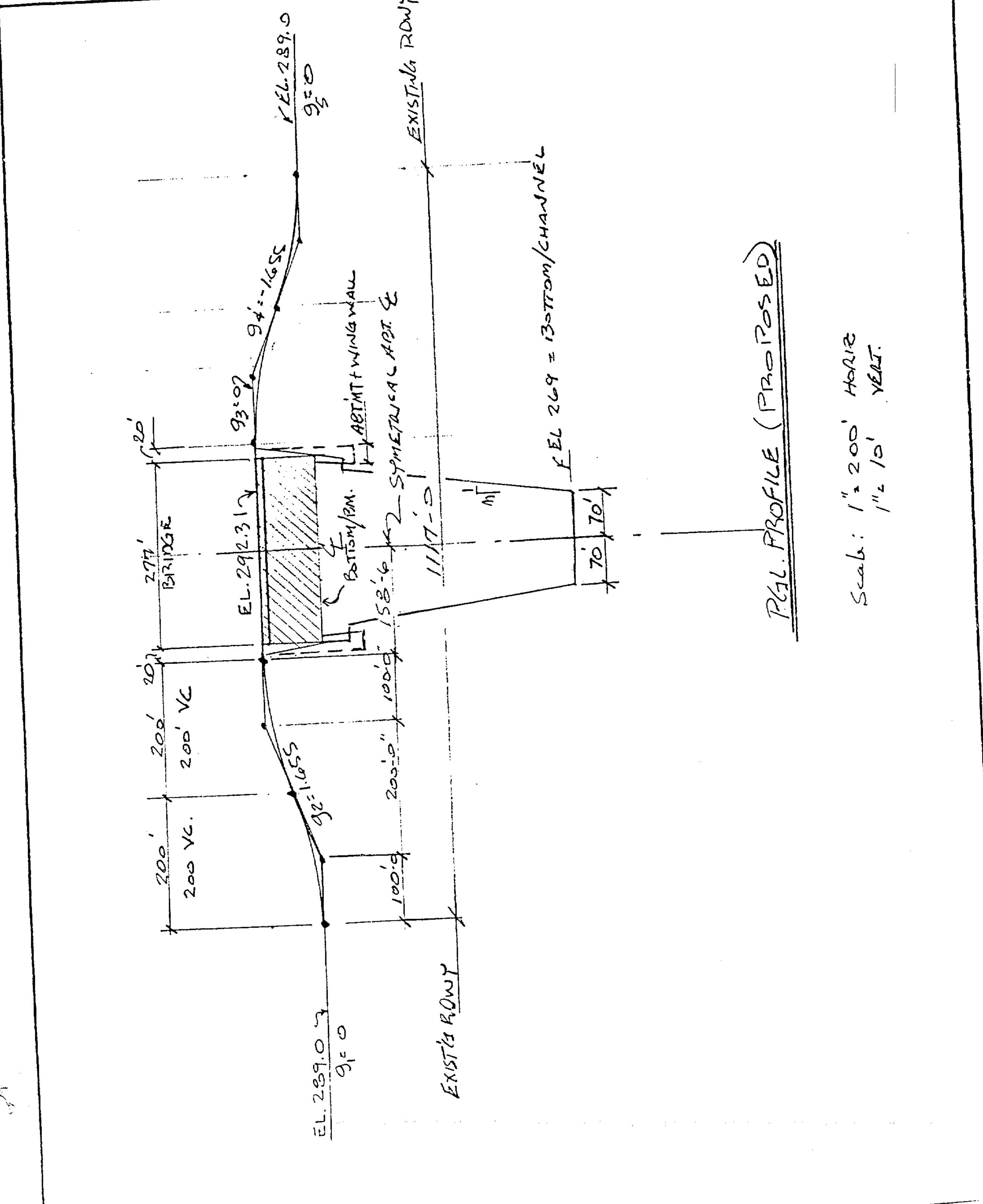


SUBJECT: REELFOOT LAKE FEASIBILITY HWY 21 BRIDGE/SPILLWAY CHANNEL	COMPUTED BY: WT	DATE: 13 MAY 97	FILE NO. B-1
	CHECKED BY: MSW TN DOT	DATE: 3/12/98	SHEET NO.

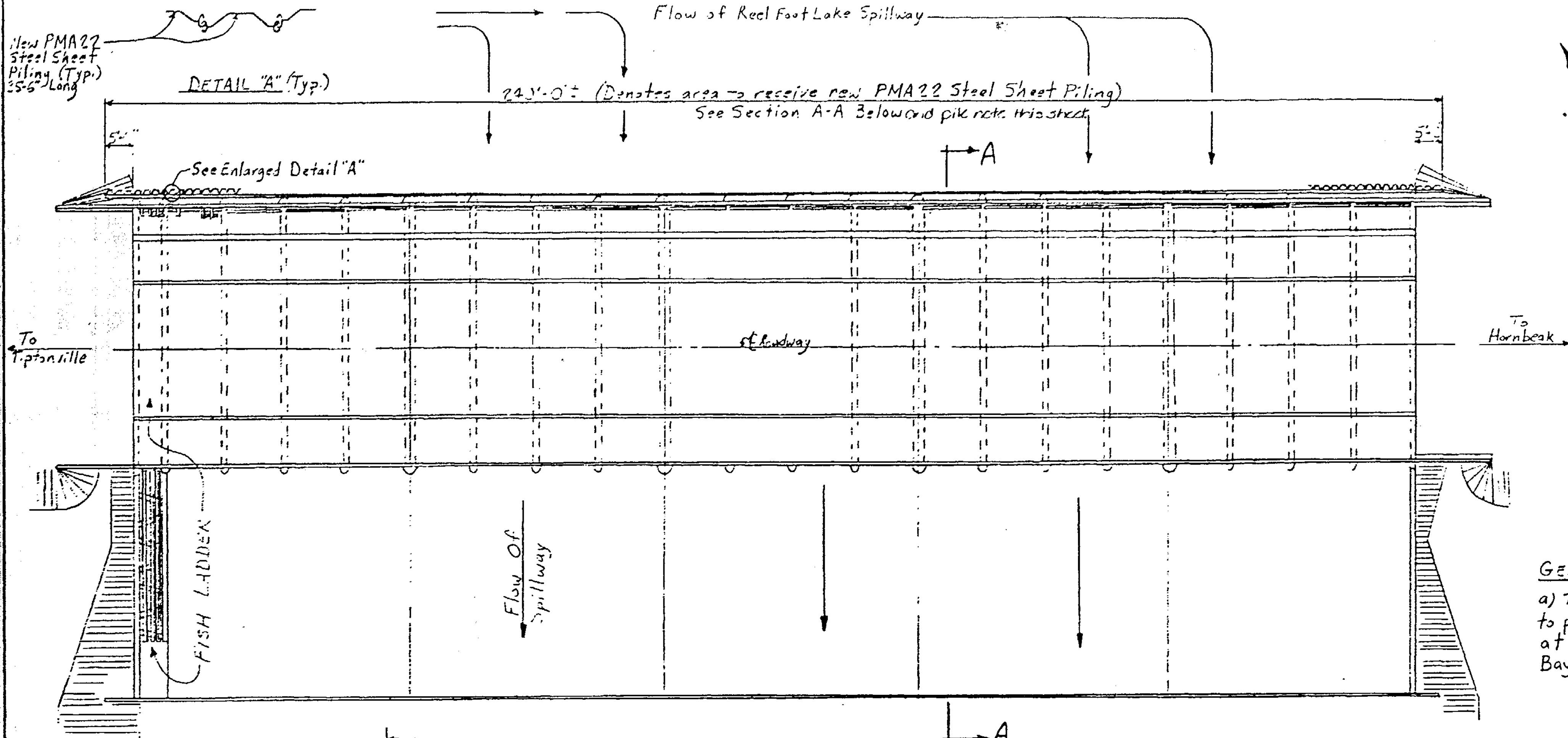


DRAWING II-A-8

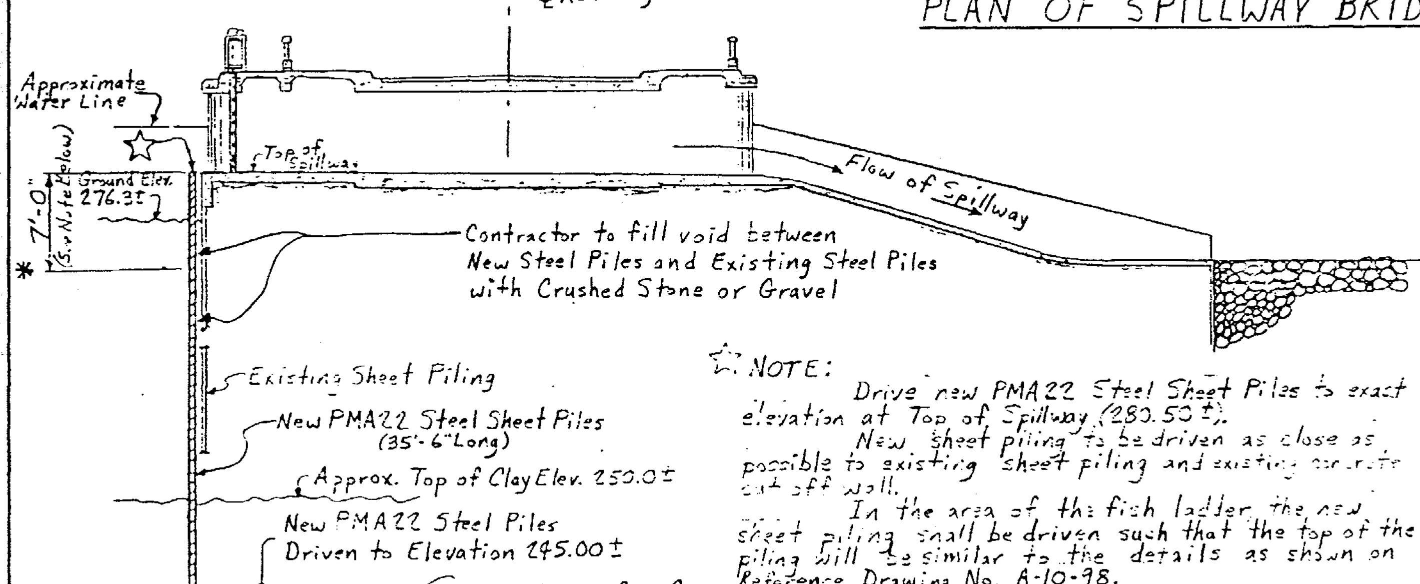
SUBJECT: REELFOOT LAKE FEASIBILITY	COMPUTED BY: W.T.	DATE: 13 MAY 97	FILE NO. B-3
HWY 21 BRIDGE/SPILLWAY CHANNEL	CHECKED BY: MSW TN DOT	DATE: 3/12/98	SHEET NO.



DRAWING II-A-9



PLAN OF SPILLWAY BRIDGE



SECTION A-A

NOTE:
Cost of all new PMA22 Steel Sheet Piles, Pile Driving, Coal Tar Epoxy Coating, Crushed Stone, and any miscellaneous materials and labor necessary to complete all work as shown in these plans, to the full satisfaction of the Engineer, to be included under Item No. 920-01.12, Sheet Piles, S.F.

NOTE:
Drive new PMA22 Steel Sheet Piles to exact elevation at Top of Spillway (280.50±).
New sheet piling to be driven as close as possible to existing sheet piling and existing concrete cut off wall.
In the area of the fish ladder, the new sheet piling shall be driven such that the top of the piling will be similar to the details as shown on Reference Drawing No. A-10-98.

ESTIMATED QUANTITIES

Item No.	Description	Unit	Qty.
①920-01.12	Sheet Piles	S.F.	8,520
712-01	Traffic Control	L.S.	1
713-06	Signs Construction	S.F.	56
717-01	Mobilization	L.S.	1

① Cost of All PMA22 Steel Sheet Piles complete & in place, see Cost Note This Sheet.

GENERAL SCOPE OF WORK:
a) The installation of new steel piling to prevent further undermining of water at Spillway Bridge over Reel Foot Lake Bayou. See details this sheet.

PROJECT NO.	YEAR	SHEET NO.
48002-3(?)-04	1987	2

REVISIONS

NO.	DATE	BY	BRIEF DESCRIPTION
1	[illegible]	[illegible]	[illegible]

GENERAL NOTES

Specifications: Standard Road and Bridge Specifications of the Tennessee Department of Transportation. (March, 1981 Edition)
Design Specifications: AASHTO 1983 Edition with Addenda
Special Note for Utilities: See Special Provision No. 100 regarding the revision of Section 105.07 to the Standard Specifications for Road and Bridge Construction.
Shop Drawings: See Special Provision No. 105A.
Special Note: At least one (1) traffic lane to remain open at all times.

⚠ [illegible]

⚠ Coal Tar Epoxy Paint System

* [illegible]

LIST OF DRAWINGS:

Drawing	Dwg. No.	Last Rev. Date
Layout Bridge Repair Details	A-10-96A	[illegible]

LIST OF REFERENCE DRAWINGS:

Drawing	Dwg. No.
Layout of Spillway	A-10-96
Details of Spillway	A-10-98, 99, & 100

LIST OF SPECIAL PROVISIONS

No.	Regarding	Last Rev. Date
100	Revisions and additions to Standard Specifications for Road and Bridge Construction.	11-3-86
105A	Approval of Shop Drawings	3-25-85

STATE OF TENNESSEE
DEPARTMENT OF TRANSPORTATION
BUREAU OF HIGHWAYS

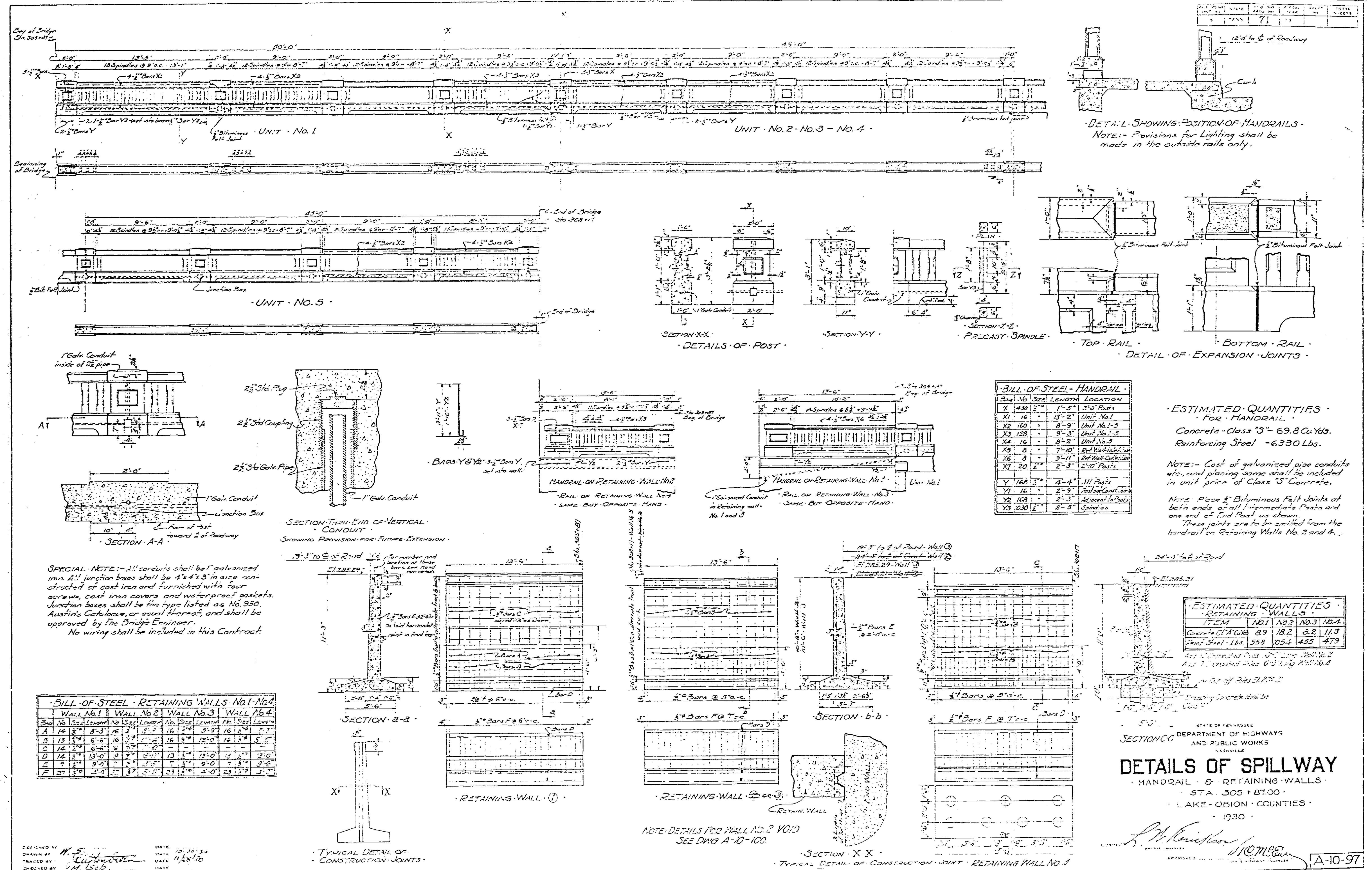
Bridge Repair Details For
State Route 21 Over Reel Foot Lake Bayou
Bridge No. 48-21-7.60
Lake County
1987

CORRECT Edward F. Wasserman, ENGINEER OF STRUCTURES
APPROVED [signature], DIRECTOR OF HIGHWAYS

DESIGNED BY R. Gentry DATE 11-86
DRAWN BY S. Isbell DATE
SUPERVISED BY Gentry & McCormick DATE 11-87
CHECKED BY Slaton & Graves DATE 11-87

A-10-96A

DRAWING II-A-10



SPECIAL · NOTE:- All conduits shall be 1" galvanized iron. All junction boxes shall be 4"x4"x3" in size constructed of cast iron and furnished with four screws, cast iron covers and waterproof gaskets. Junction boxes shall be the type listed as No. 950, Austin's Catalogue, or equal thereof, and shall be approved by the Bridge Engineer.

No wiring shall be included in this Contract.

· ESTIMATED · QUANTITIES ·
· FOR · HANDRAIL ·

Concrete - Class "S" - 69.8 Cu. Yds.
Reinforcing Steel - 6330 Lbs.

NOTE:- Cost of galvanized pipe conduits etc., and placing same shall be included in unit price of Class "S" Concrete.

Note:- Place ½" Bituminous Felt Joints at both ends of all Intermediate Posts and one end of End Post as shown. These joints are to be omitted from the handrail on Retaining Walls No. 2 and 4.

· BILL · OF · STEEL - HANDRAIL ·				
Bar	No	Size	Length	Location
X	430	½"ø	1'-5"	2'-0" Posts
X1	16	"	13'-2"	Unit No. 1
X2	160	"	8'-9"	Unit No. 1-5
X3	128	"	9'-3"	Unit No. 1-5
X4	16	"	8'-2"	Unit No. 5
X5	8	"	7'-10"	Ret. Wall-Inlet
X6	8	"	3'-11"	Ret. Wall-Outlet
X7	20	½"ø	2'-3"	2'-0" Posts
Y	168	½"ø	4'-4"	All Posts
Y1	16	"	2'-9"	Posts at Const. Joints
Y2	168	"	2'-3"	Adjacent to Posts
Y3	2030	½"ø	2'-5"	Spindles

· BILL · OF · STEEL · RETAINING WALLS · No. 1 - No. 4												
	WALL No. 1			WALL No. 2			WALL No. 3			WALL No. 4		
Bar	No	Size	Length	No	Size	Length	No	Size	Length	No	Size	Length
A	14	[illegible]	8'-3"	16	[illegible]	[illegible]	16	[illegible]	5'-3"	16	[illegible]	[illegible]
B	13	[illegible]	6'-6"	16	[illegible]	[illegible]	16	[illegible]	12'-0"	16	[illegible]	5'-[illegible]
C	14	½"ø	6'-6"	[illegible]	[illegible]	[illegible]-0"	—	—	—	—	—	—
D	14	½"ø	13'-0"	[illegible]	[illegible]	[illegible]	13	[illegible]	13'-0"	[illegible]	[illegible]	13'-0"
E	7	[illegible]	9'-0"	[illegible]	[illegible]	[illegible]	7	[illegible]	9'-0"	[illegible]	[illegible]	[illegible]
F	27	[illegible]	4'-0"	[illegible]	[illegible]	[illegible]	23	[illegible]	4'-0"	23	[illegible]	[illegible]

· ESTIMATED · QUANTITIES · RETAINING · WALLS ·				
ITEM	No. 1	No. 2	No. 3	No. 4
Concrete Cl. "A" Cu. Yds.	8.9	18.2	8.2	11.3
Reinf. Steel · Lbs.	558	1054	455	479

STATE OF TENNESSEE
DEPARTMENT OF HIGHWAYS AND PUBLIC WORKS
NASHVILLE

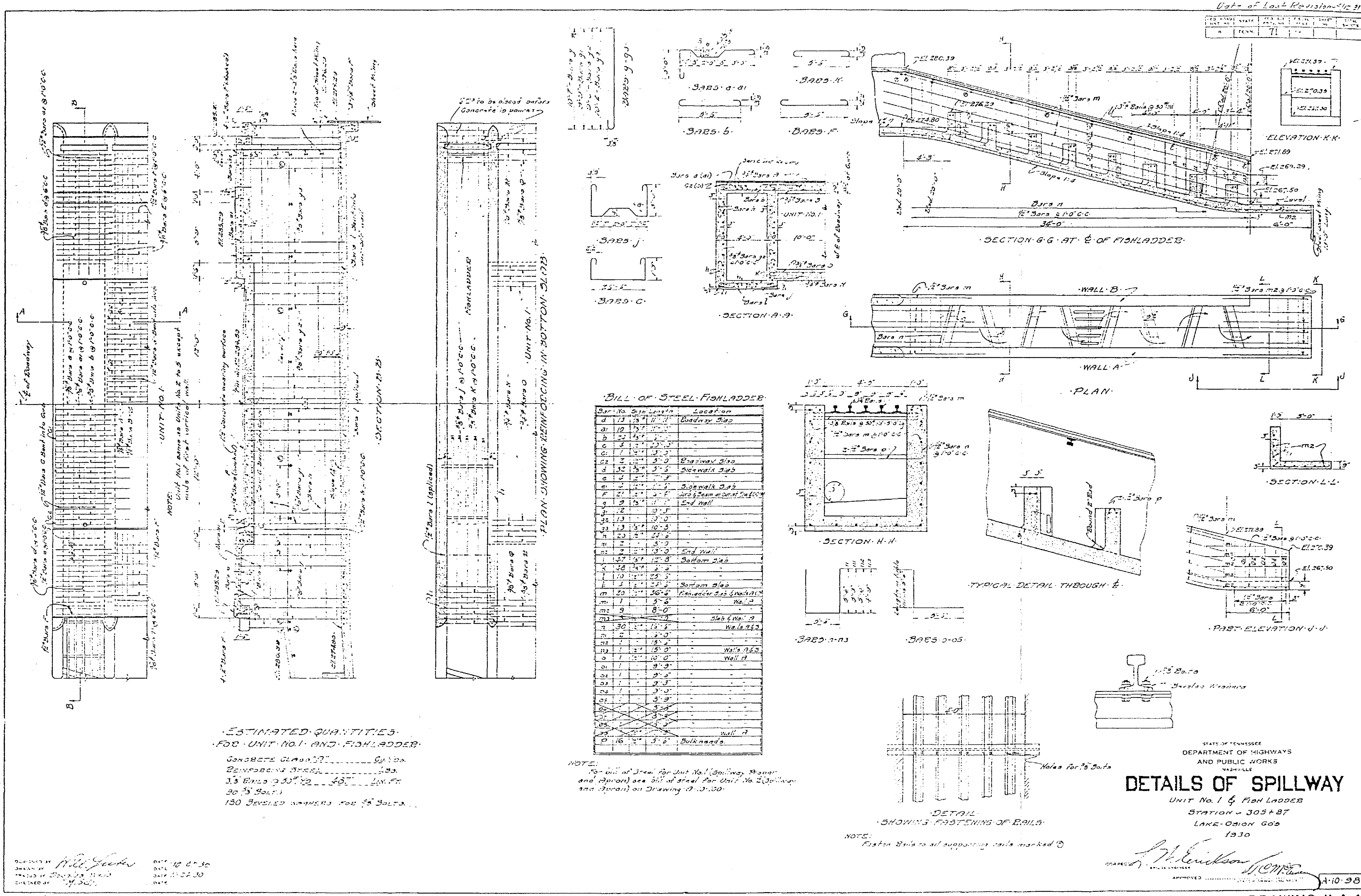
DETAILS OF SPILLWAY

· HANDRAIL · & · RETAINING · WALLS ·
· STA. 305 + 87.00 ·
· LAKE - OBION · COUNTIES ·
· 1930 ·

A-10-97

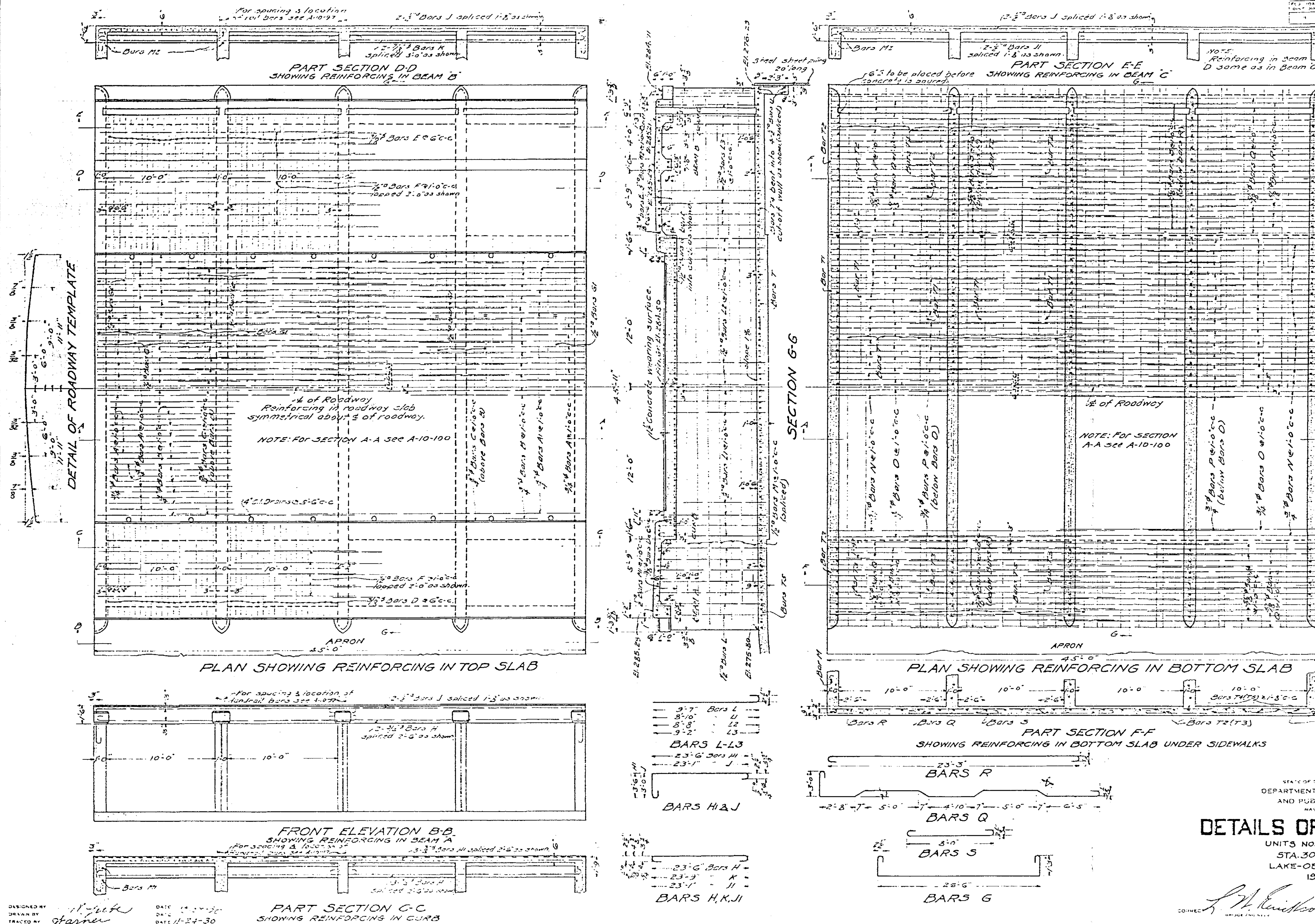
DRAWING II-A-11

Retaining Walls No. 2 & No. 4 Revised for Details & Quantities 5-11-31
Elevation of Inlet raised 1'-0" - 3/12/31



Date of Last Revision
SECTION · G-G · AT · ℄ · OF FISHLADDER
ELEVATION · K-K
WALL · B
WALL · A
PLAN
SECTION · A-A
SECTION · H-H
SECTION · L-L
TYPICAL · DETAIL · THROUGH · ℄
PART · ELEVATION · J-J
SECTION · B-B
PLAN · SHOWING · REINFORCING · IN · BOTTOM · SLAB
UNIT · NO. 1
FISHLADDER
BARS · C
BARS · J
BARS · b
BARS · F
BARS · K
BARS · G-G
BILL · OF · STEEL · FISHLADDER
NOTE: For bill of steel for Unit No. 1 (Spillway Proper and Apron) see bill of steel for Unit No. 2 (Spillway and Apron) on Drawing A-10-100.
ESTIMATED · QUANTITIES · FOR · UNIT · NO. 1 · AND · FISHLADDER
DETAIL · SHOWING · FASTENING · OF · RAILS
NOTE: Fisten Bolts to all supporting rails marked
Holes for ⅝" Bolts
STATE OF TENNESSEE
DEPARTMENT OF HIGHWAYS
AND PUBLIC WORKS
NASHVILLE
DETAILS OF SPILLWAY
UNIT No. 1 & FISH LADDER
STATION - 305+87
LAKE-OBION CO'S
1930
A-10-98
DRAWING II-A-12
Elevation of Inlet raised 1'-0" - 3/12/31

Date of Last Revision - 3/12/31



Elevation of Inlet raised 1'-0" - 3/12/31

STATE OF TENNESSEE
DEPARTMENT OF HIGHWAYS
AND PUBLIC WORKS
NASHVILLE

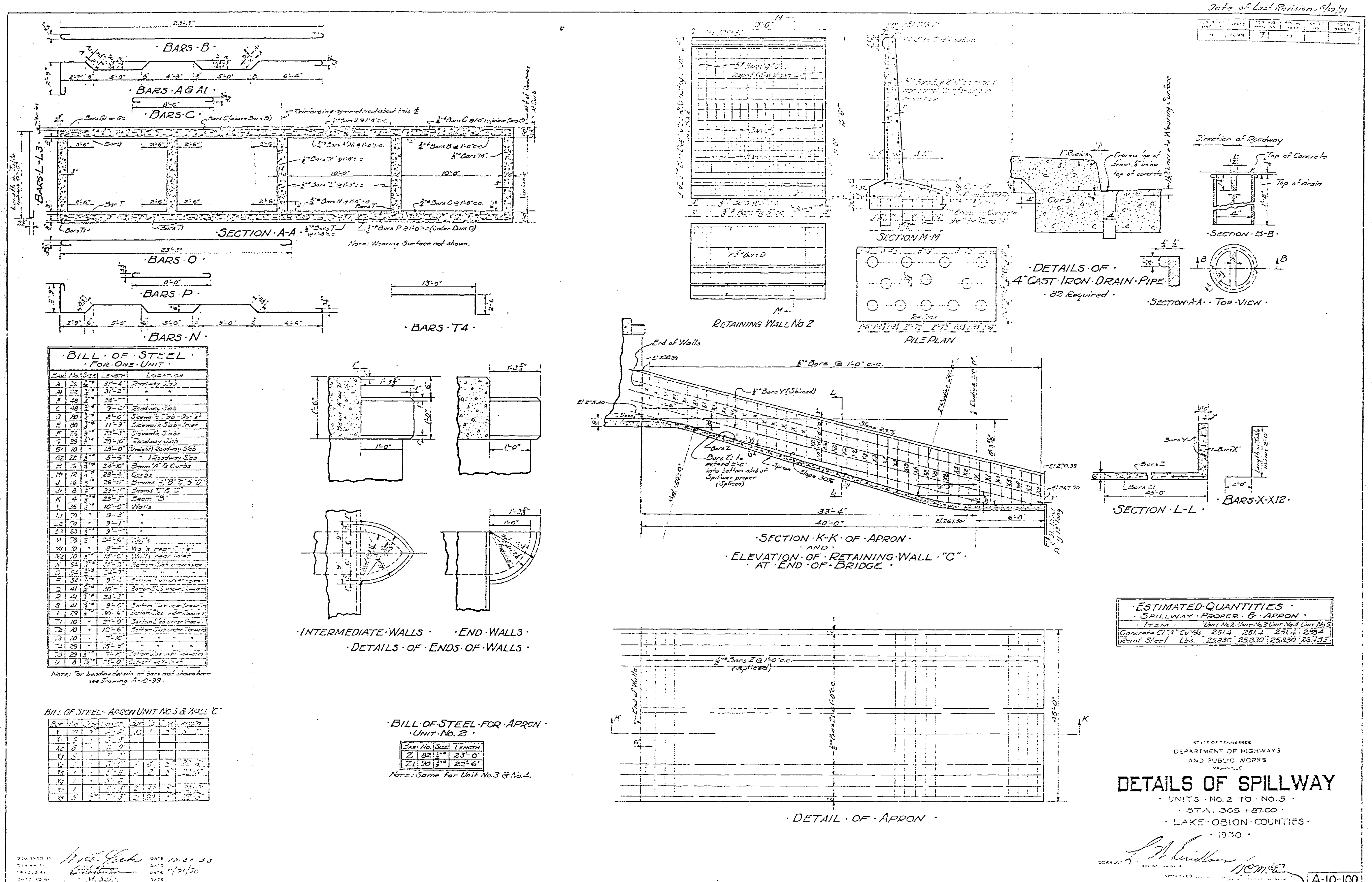
DETAILS OF SPILLWAY

UNITS NO.2 TO NO.5
STA. 305+87
LAKE-OBION CO'S
1930

DESIGNED BY
DRAWN BY
TRACED BY Harner DATE 11-24-30
CHECKED BY

A-10-99

DRAWING 11-A-13



· BILL · OF · STEEL ·
· FOR · ONE · UNIT ·

BAR	No.	SIZE	LENGTH	LOCATION
A	26	[illegible]	31'-4"	Roadway Slab
A1	22	[illegible]	31'-2"	" "
B	48	[illegible]	24'-[illegible]	" "
C	48	[illegible]	9'-4"	Roadway Slab
D	80	[illegible]	8'-0"	Sidewalk Slab - Outlet
E	80	[illegible]	11'-3"	Sidewalk Slab - Inlet
F	26	[illegible]	23'-3"	Sidewalk Slabs
G	29	[illegible]	29'-10"	Roadway Slab
G1	10	"	13'-0"	(Straight) Roadway Slab
G2	20	[illegible]	5'-6"	(") Roadway Slab
H	16	[illegible]	24'-10"	Beam "A" & Curbs
H1	12	[illegible]	28'-4"	Curbs
J	16	[illegible]	26'-11"	Beams "A" "B" "C" & "D"
J1	8	[illegible]	23'-11"	Beams "C" & "D"
K	4	[illegible]	25'-3"	Beam "B"
L	35	[illegible]	10'-0"	Walls
L1	70	"	9'-3"	"
L2	70	"	9'-1"	"
L3	53	[illegible]	9'-[illegible]	"
M	78	[illegible]	24'-6"	Walls
M1	10	"	8'-4"	Walls near Outlet
M2	10	[illegible]	13'-0"	Walls near Inlet
N	54	[illegible]	31'-2"	Bottom Slab - [illegible]
O	54	[illegible]	24'-[illegible]	" " " "
P	54	[illegible]	9'-[illegible]	Bottom Slab under [illegible]
Q	41	[illegible]	30'-[illegible]	Bottom Slab under [illegible]
R	41	[illegible]	24'-3"	"
S	41	[illegible]	9'-0"	Bottom Slab under [illegible]
T	29	[illegible]	30'-6"	Bottom Slab under [illegible]
T1	10	"	27'-0"	Bottom Slab under [illegible]
T2	10	"	12'-6"	Bottom Slab under [illegible]
T3	10	"	7'-10"	" " " "
T4	29	"	15'-5"	" " " "
T5	29	[illegible]	7'-10"	Bottom Slab [illegible]
U	8	[illegible]	23'-0"	Cutoff wall - Inlet

Note: For bending details of bars not shown here see Drawing A-10-99.

BILL OF STEEL - APRON UNIT No. 5 & WALL "C"

· BILL · OF · STEEL · FOR · APRON ·
· UNIT · No. 2 ·

BAR	No.	SIZE	LENGTH
Z	82	$\frac{1}{2}$"	23'-0"
Z1	90	$\frac{1}{2}$"	23'-6"

Note: Same for Unit No. 3 & No. 4.

· ESTIMATED · QUANTITIES ·
· SPILLWAY · PROPER · & · APRON ·

· ITEM ·		Unit No. 2	Unit No. 3	Unit No. 4	Unit No. 5
Concrete Cl. "A"	Cu. Yds.	251.4	251.4	251.4	258.4
Reinf. Steel	Lbs.	25,830	25,830	25,830	26,735

STATE OF TENNESSEE
DEPARTMENT OF HIGHWAYS
AND PUBLIC WORKS
NASHVILLE

DETAILS OF SPILLWAY

· UNITS · No. 2 · TO · No. 5 ·
· STA. 305 + 87.00 ·
· LAKE - OBION · COUNTIES ·
· 1930 ·

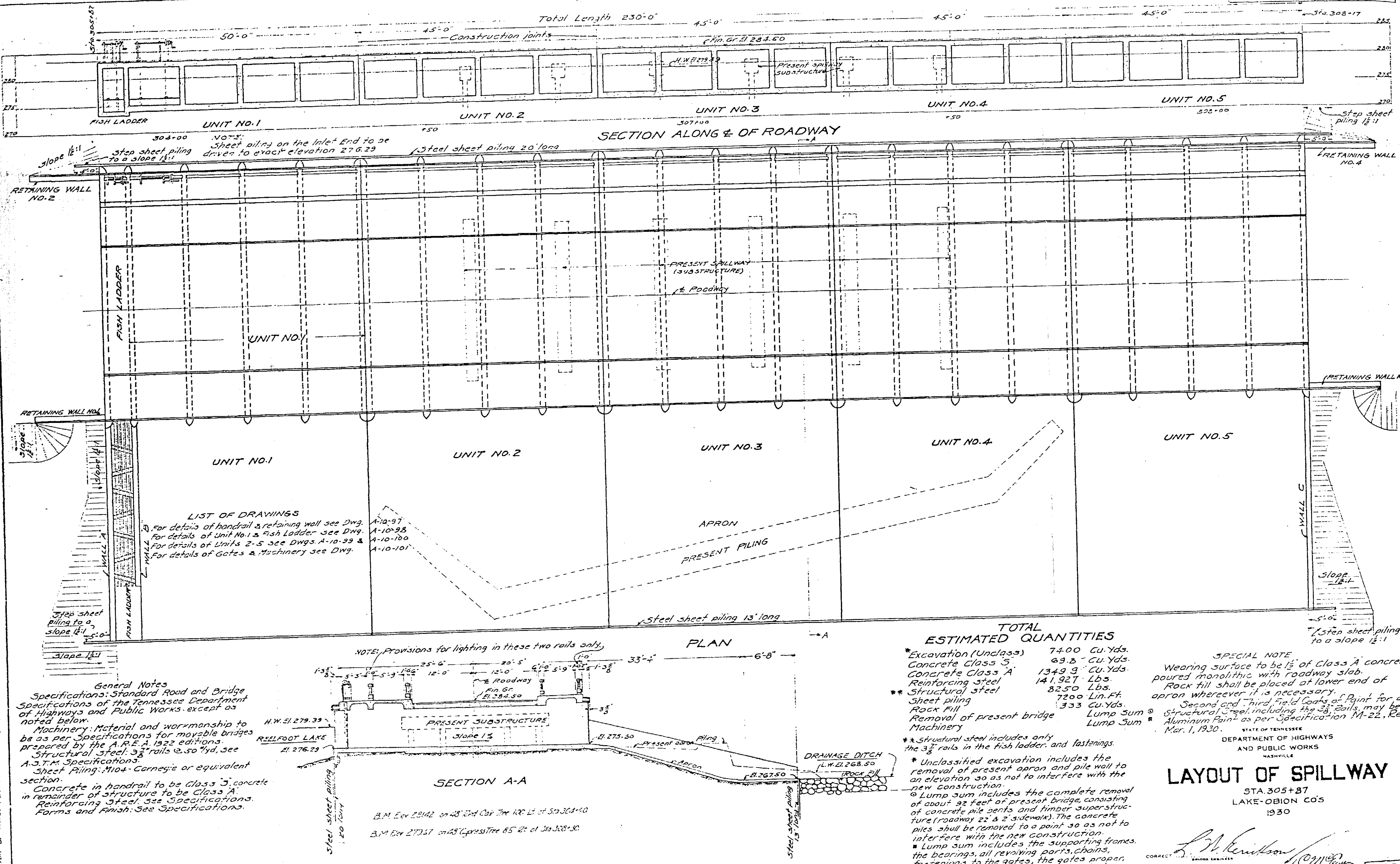
A-10-100

DRAWING II-A-14

Revised Details of Retaining Wall No. 2 Added 5-13-31
Elevation of Inlet raised 1'0" - 3/12/31

Date of Last Revision - 5/11/31

Fed. Road Dist. No.	State	Fed. Aid Proj. No.	Fiscal Year	Sheet No.	Total Sheets
5	Tenn.	71	19		



LIST OF DRAWINGS

For details of handrail & retaining wall see Dwg. A-10-97
For details of Unit No. 1 & Fish Ladder see Dwg. A-10-98
For details of Units 2-5 see Dwgs. A-10-99 & A-10-100
For details of Gates & Machinery see Dwg. A-10-101

General Notes

Specifications: Standard Road and Bridge Specifications of the Tennessee Department of Highways and Public Works, except as noted below.
Machinery: Material and workmanship to be as per Specifications for movable bridges prepared by the A.R.E.A. 1922 edition.
Structural Steel: $3\frac{1}{2}$" rails @ 50 #/yd. see A.S.T.M. Specifications.
Sheet Piling: M104 - Carnegie or equivalent section.
Concrete in handrail to be Class "S", concrete in remainder of structure to be Class "A".
Reinforcing Steel: See Specifications.
Forms and Finish: See Specifications.

TOTAL ESTIMATED QUANTITIES

*Excavation (Unclass.)	74.00	Cu. Yds.
Concrete Class "S"	69.8	Cu. Yds.
Concrete Class "A"	1349.9	Cu. Yds.
Reinforcing Steel	141,927	Lbs.
** Structural Steel	8250	Lbs.
Sheet piling	7200	Lin. Ft.
Rock Fill	333	Cu. Yds.
Removal of present bridge		Lump Sum ⊙
Machinery		Lump Sum ▪

** Structural steel includes only the $3\frac{1}{2}$" rails in the fish ladder, and fastenings.

* Unclassified excavation includes the removal of present apron and pile wall to an elevation so as not to interfere with the new construction.

⊙ Lump sum includes the complete removal of about 92 feet of present bridge, consisting of concrete pile bents and timber superstructure (roadway 22' & 2' sidewalk). The concrete piles shall be removed to a point so as not to interfere with the new construction.

▪ Lump sum includes the supporting frames, the bearings, all revolving parts, chains, fastenings to the gates, the gates proper, the guides for the gates and the placing of the above items.

SPECIAL NOTE

Wearing surface to be 1½" of Class "A" concrete poured monolithic with roadway slab.
Rock fill shall be placed at lower end of apron wherever it is necessary.
Second and Third Field Coats of Paint for all Structural Steel, including the $3\frac{1}{2}$" Rails, may be Aluminum Paint as per Specification M-22, Rev. Mar. 1, 1930.

STATE OF TENNESSEE
DEPARTMENT OF HIGHWAYS AND PUBLIC WORKS
NASHVILLE

LAYOUT OF SPILLWAY

STA. 305+87
LAKE-OBION CO'S
1930

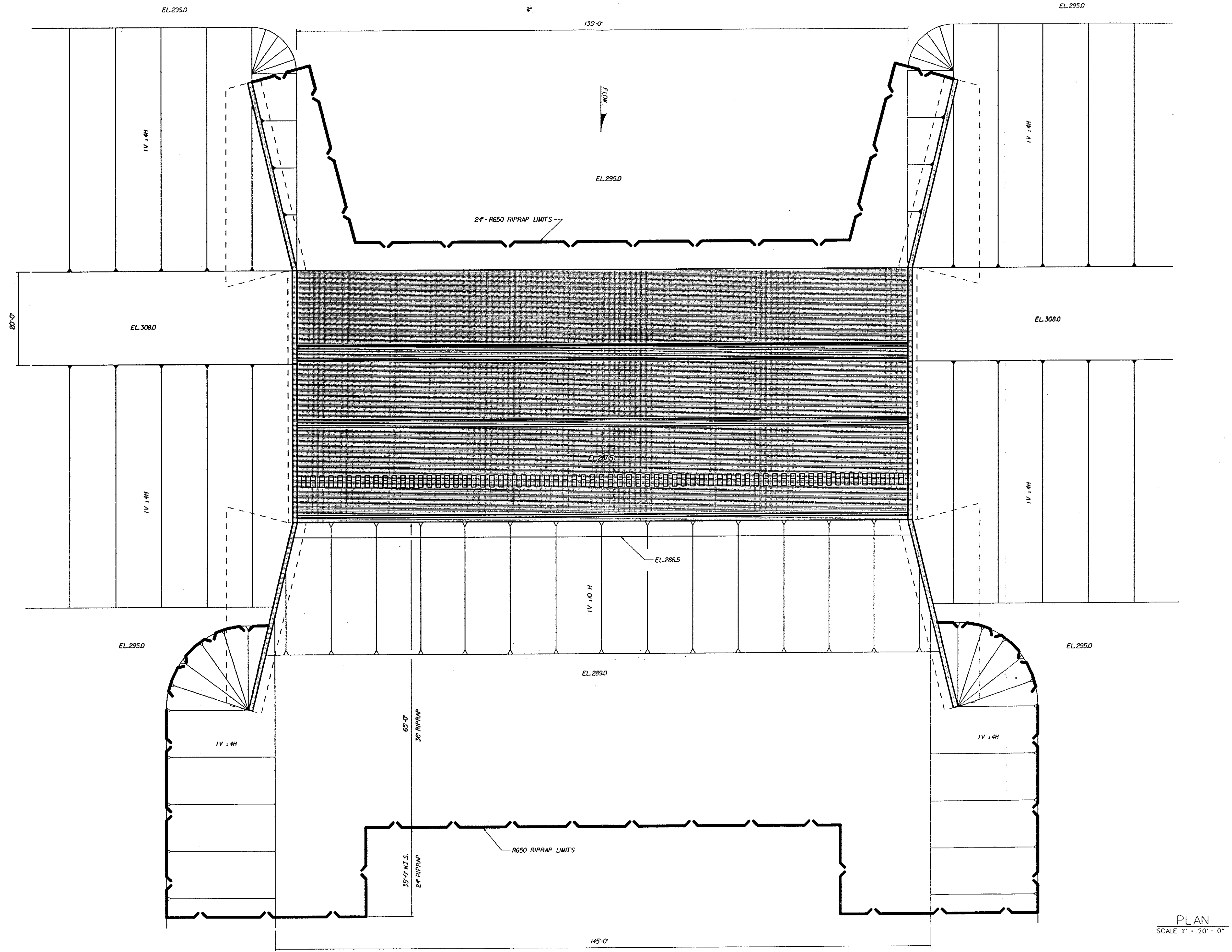
CORRECT: [signature] BRIDGE ENGINEER
APPROVED: [signature] STATE HIGHWAY ENGINEER

A-10-96

DRAWING II-A-15

DESIGNED BY [signature] DATE 10-27-30
DRAWN BY [signature] DATE
TRACED BY [signature] DATE 11-13-30
CHECKED BY H. Sch. DATE

Handrails at 6" from curb 5-11-31
Elevation of inlet raised 1'-0" - 3/12/31



PLAN
SCALE 1" = 20' - 0"

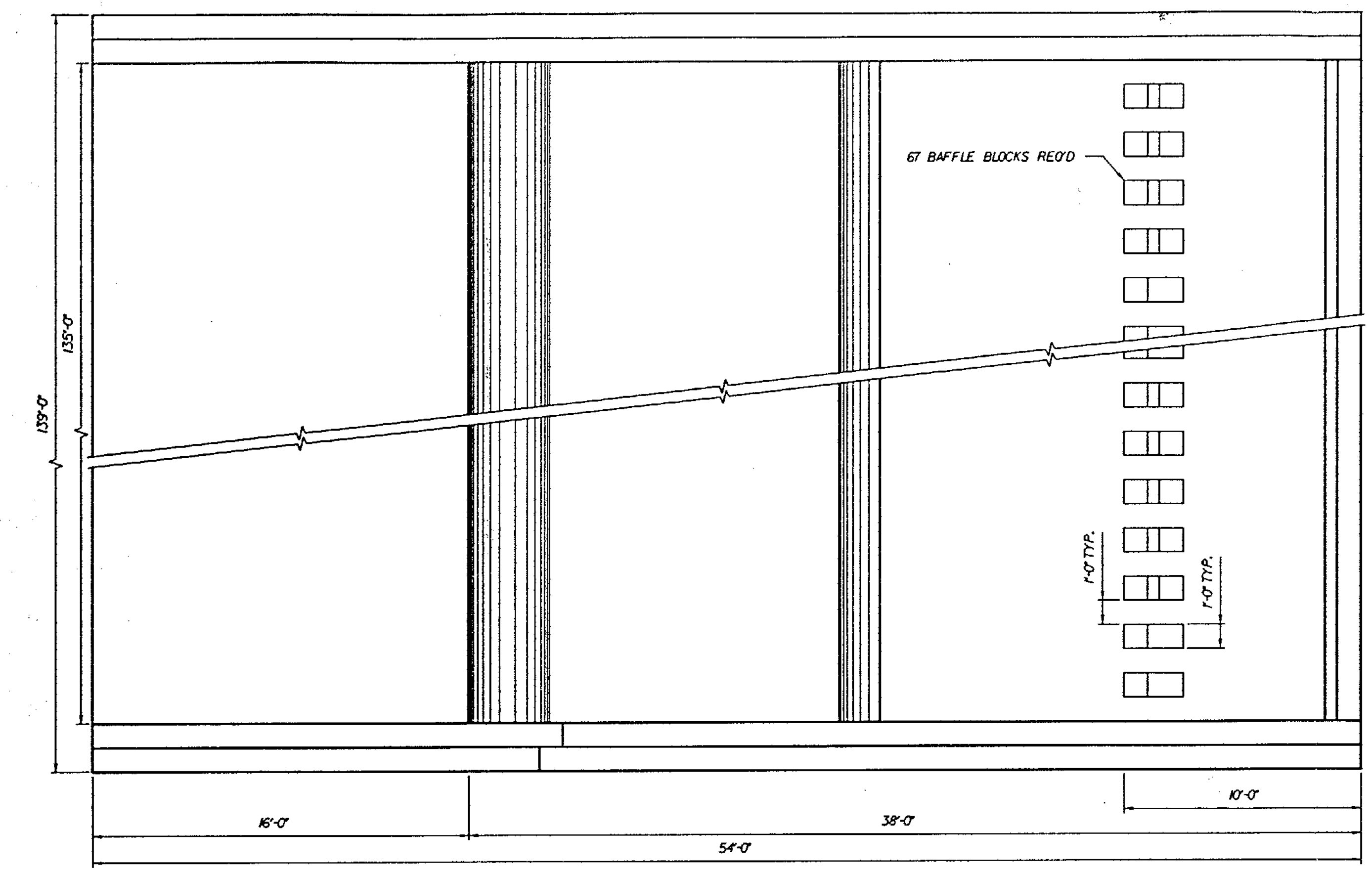
US Army Corps of Engineers Memphis District

U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
MEMPHIS, TENNESSEE

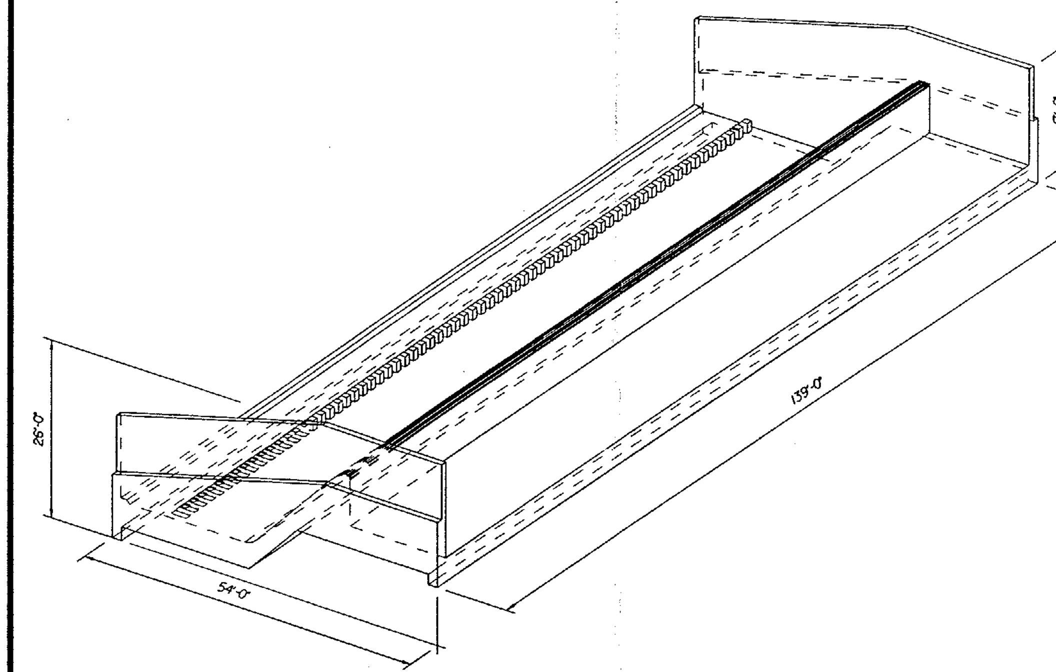
ENVIRONMENTAL RESTORATION
NATURAL RESOURCES PROJECT
MISSISSIPPI RIVER AND TRIBUTARIES
CONSTRUCTION

REELFOOT LAKE, TENNESSEE & KENTUCKY
REELFOOT CREEK SEDIMENT RETENTION BASIN
PRIMARY SPILLWAY - SITE PLAN

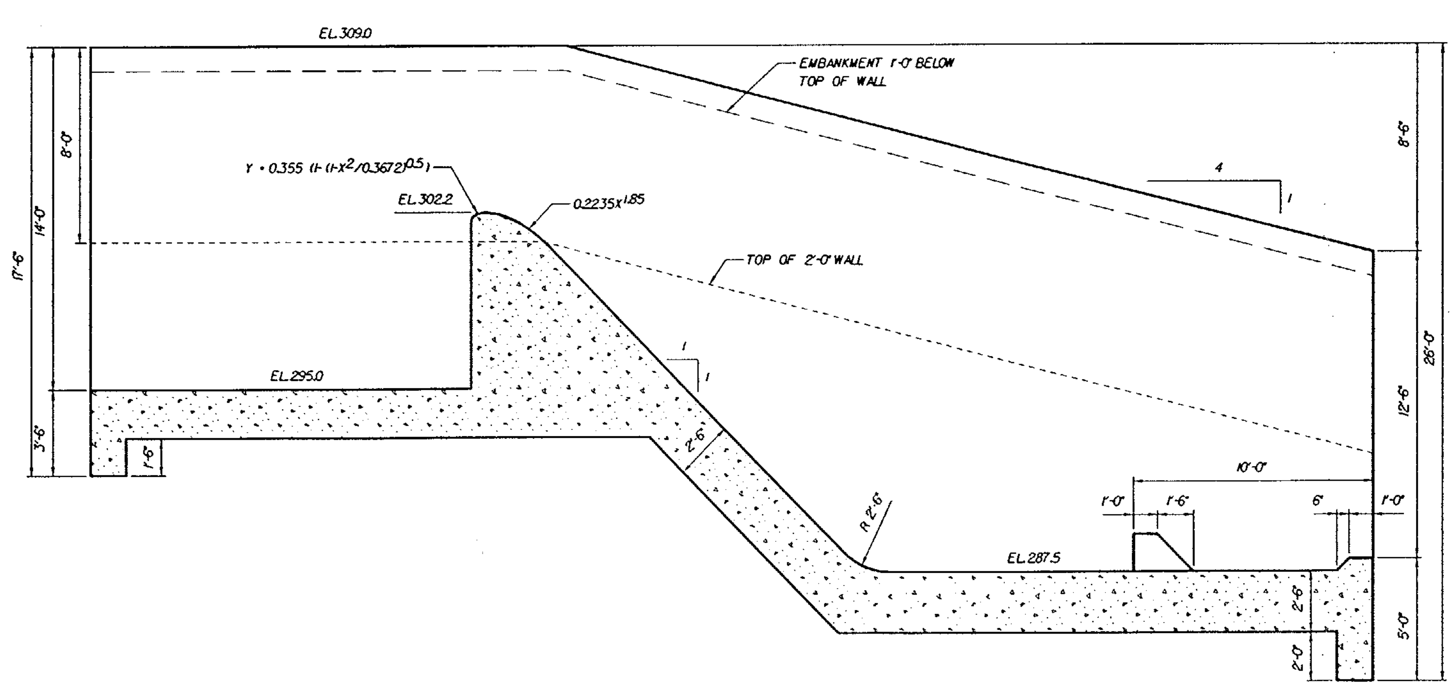
DRAWING
II-A-16
16 OF 21



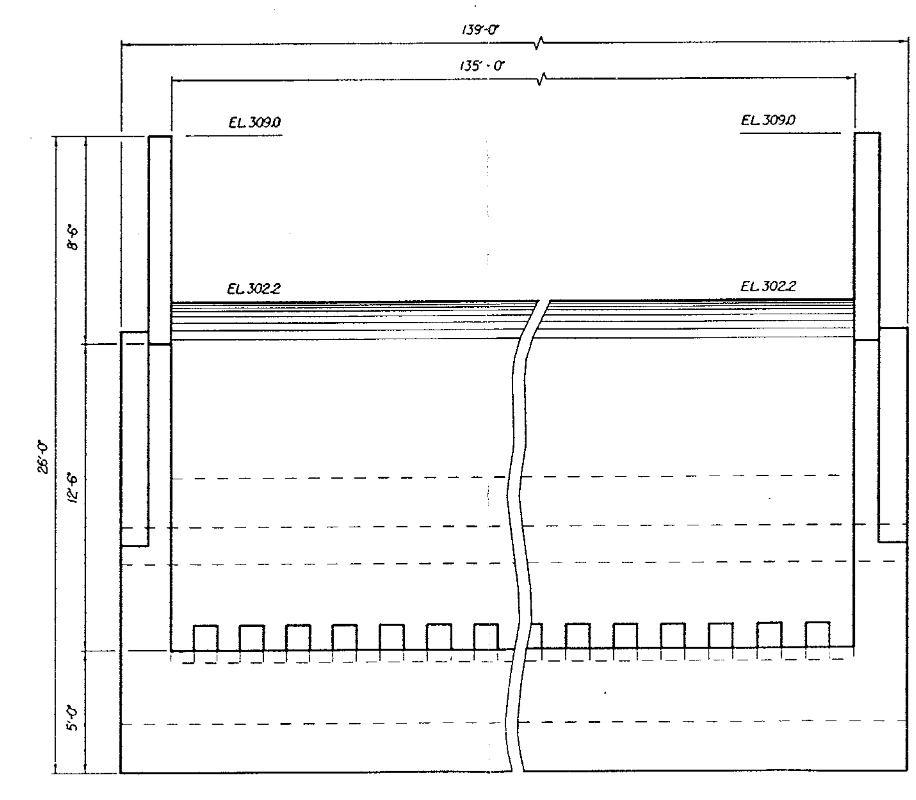
PLAN
SCALE 1/8" = 1'-0"



ISOMETRIC
N.T.S.



SECTION



ELEVATION

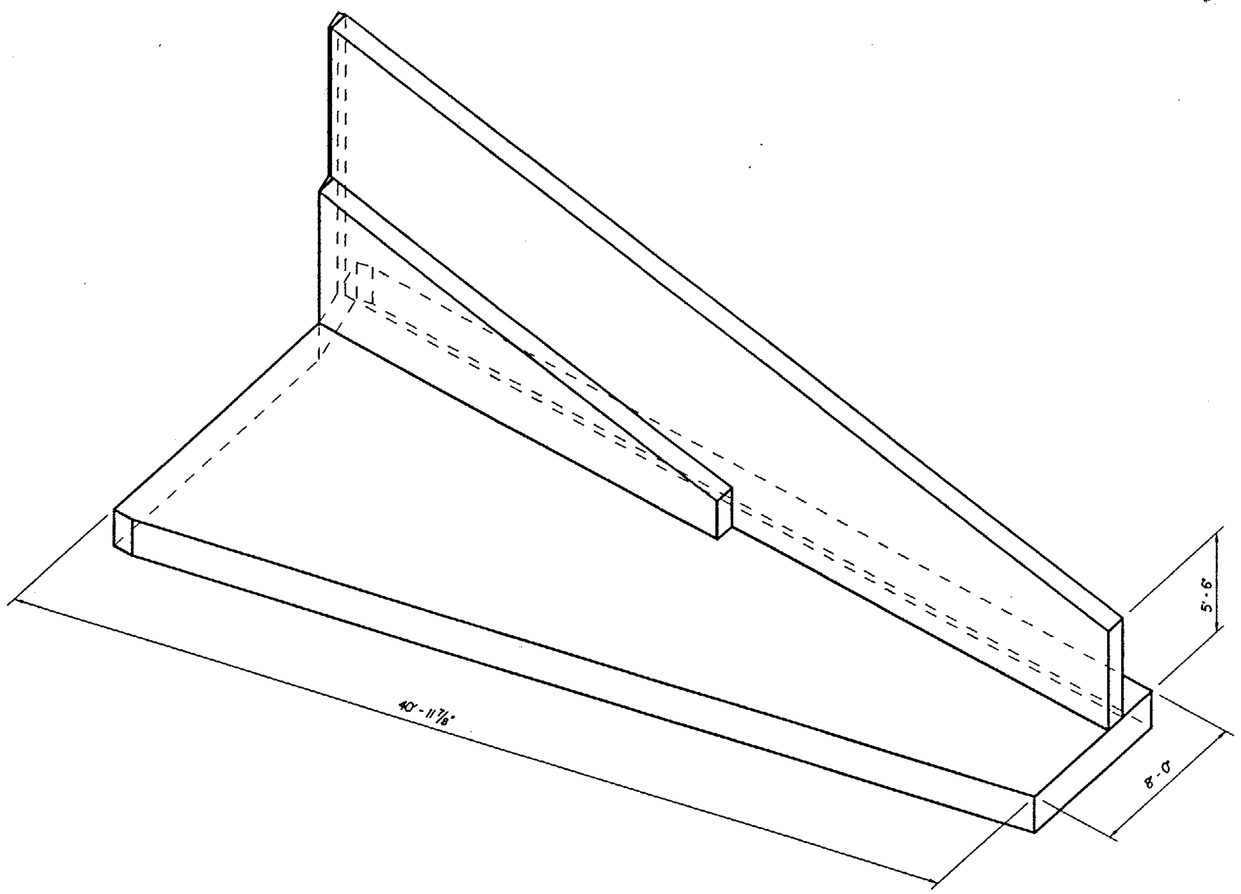
US Army Corps of Engineers
Memphis District

U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
MEMPHIS, TENNESSEE

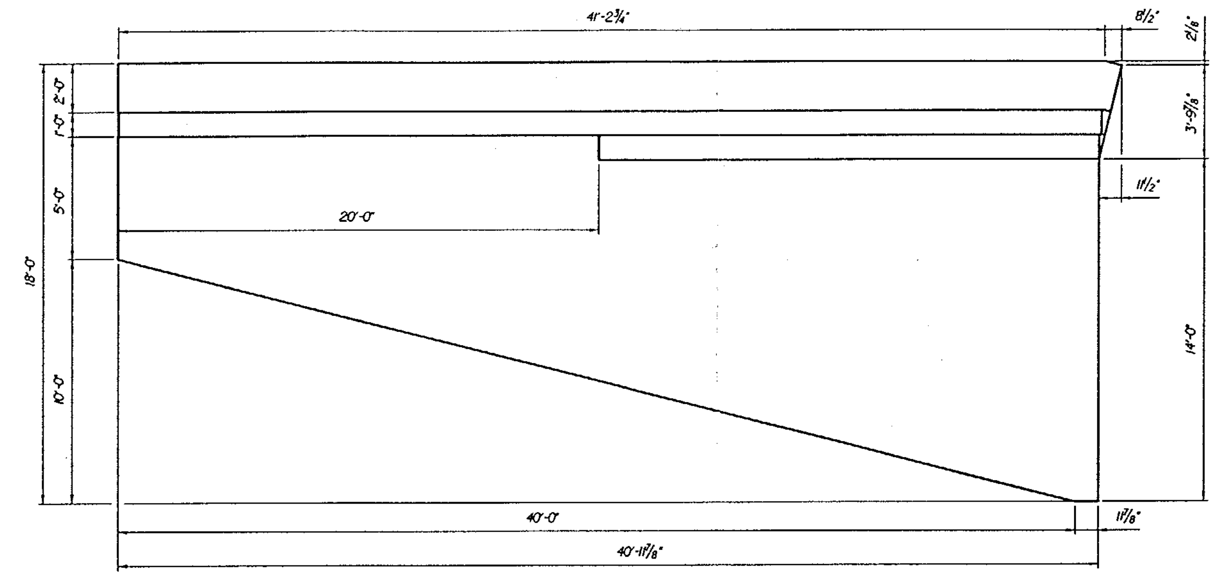
ENVIRONMENTAL RESTORATION
NATURAL RESOURCES PROJECT
CONSTRUCTION
MISSISSIPPI RIVER AND TRIBUTARIES

REELFOOT LAKE, TENNESSEE & KENTUCKY
REELFOOT CREEK SEDIMENT RETENTION BASIN
PRIMARY SPILLWAY - WEIR MONOLITH

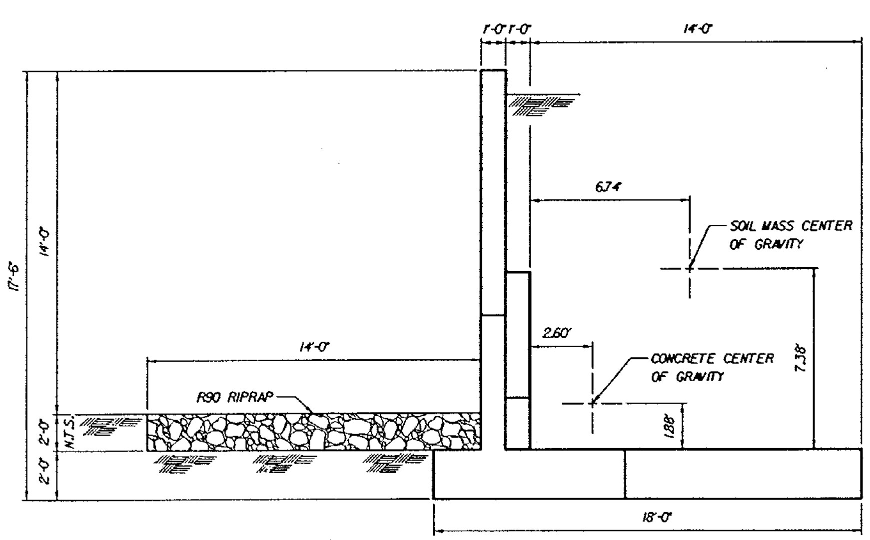
DRAWING
II-A-17
17 OF 21



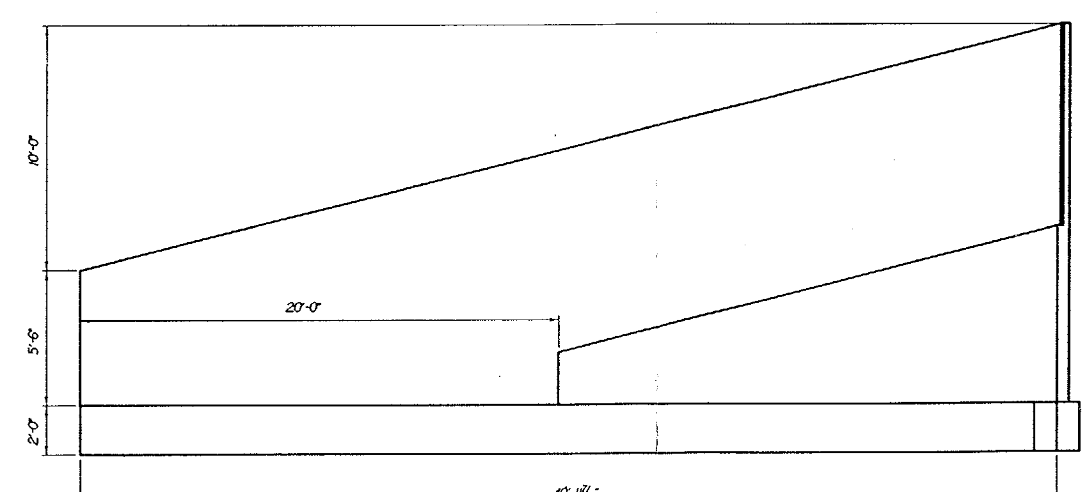
ISOMETRIC
N.T.S.



PLAN
SCALE 1/8" = 1'-0"



LEFT VIEW



ELEVATION

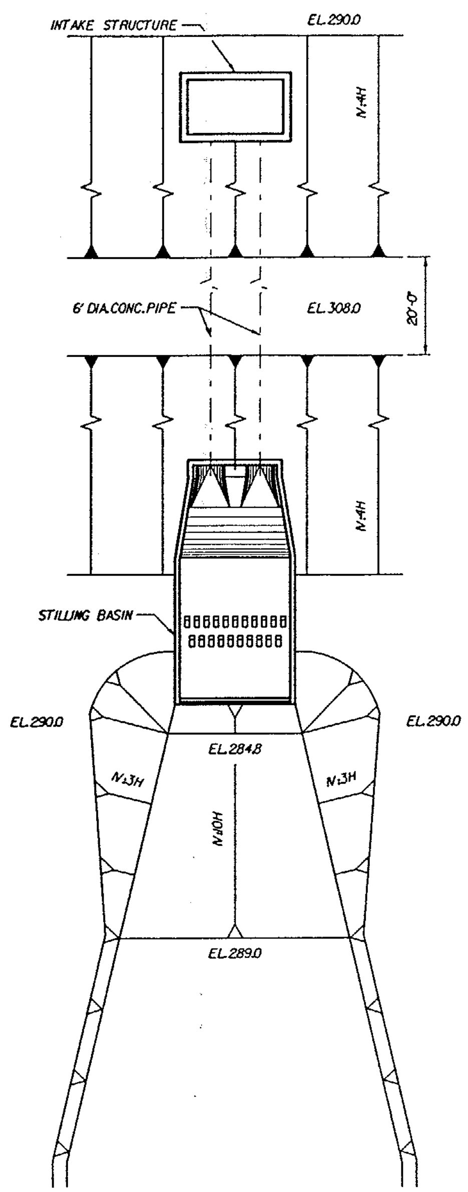
US Army Corps of Engineers Memphis District

U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
MEMPHIS, TENNESSEE

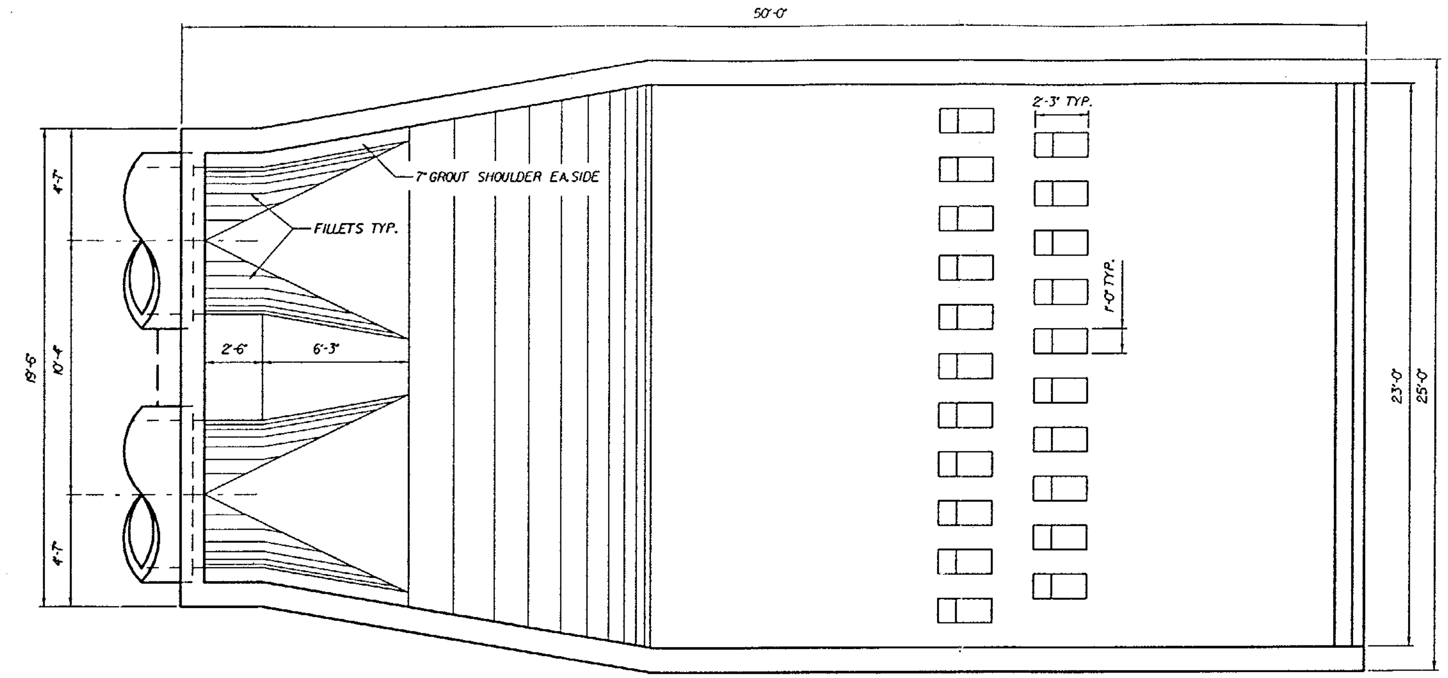
ENVIRONMENTAL RESTORATION
NATURAL RESOURCES PROJECT
MISSISSIPPI RIVER AND TRIBUTARIES
CONSTRUCTION

REELFOOT LAKE, TENNESSEE & KENTUCKY
REELFOOT CREEK SEDIMENT RETENTION BASIN
PRIMARY SPILLWAY - WING WALL

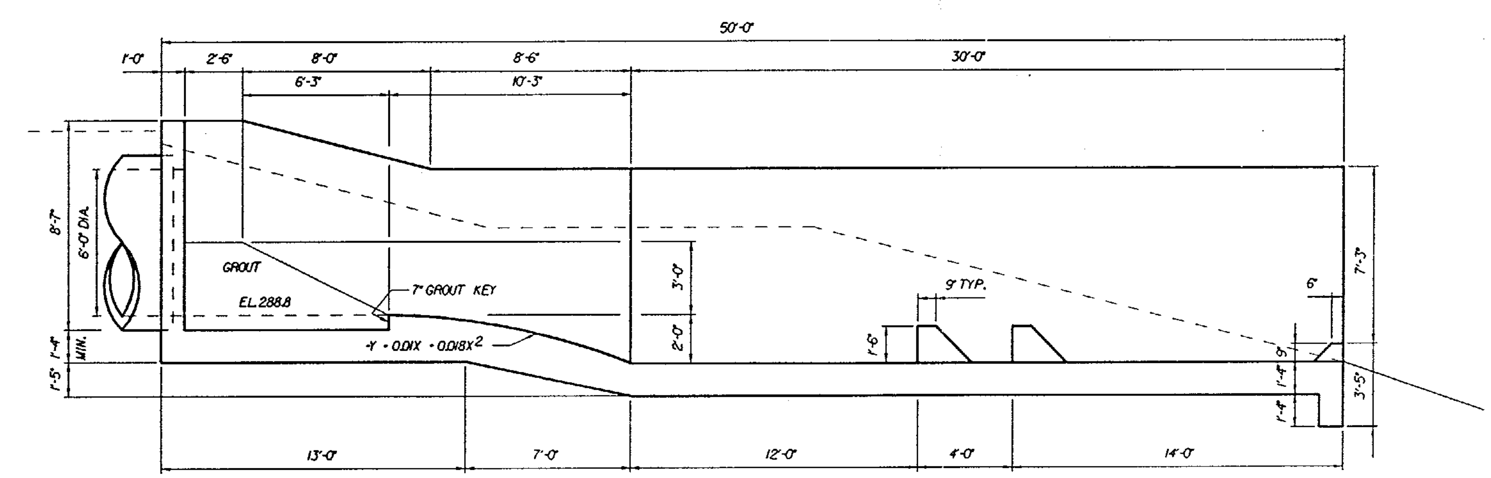
DRAWING
II-A-18
18 OF 21



SITE PLAN
SCALE 1" = 20'-0"

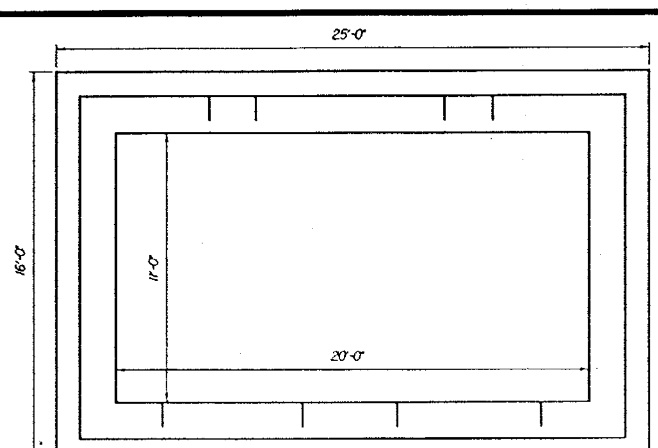


PLAN



SECTION

STILL BASIN
SCALE 1/8" = 1'-0"



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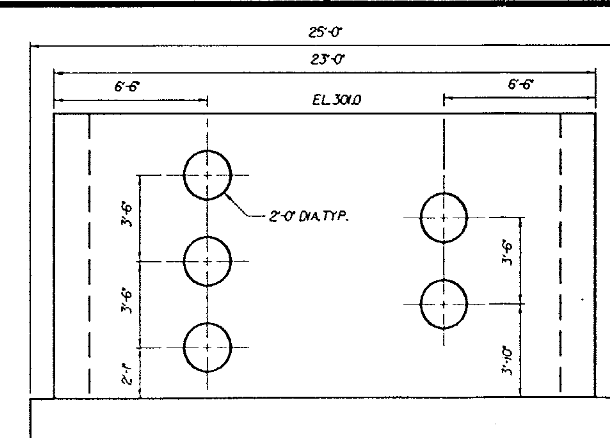
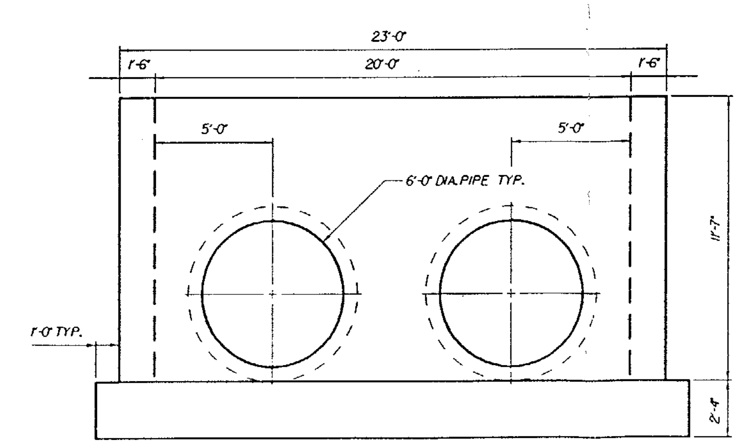


UPSTREAM SIDE



DOWNSTREAM SIDE

INTAKE STRUCTURE
SCALE 1/8" = 1'-0"

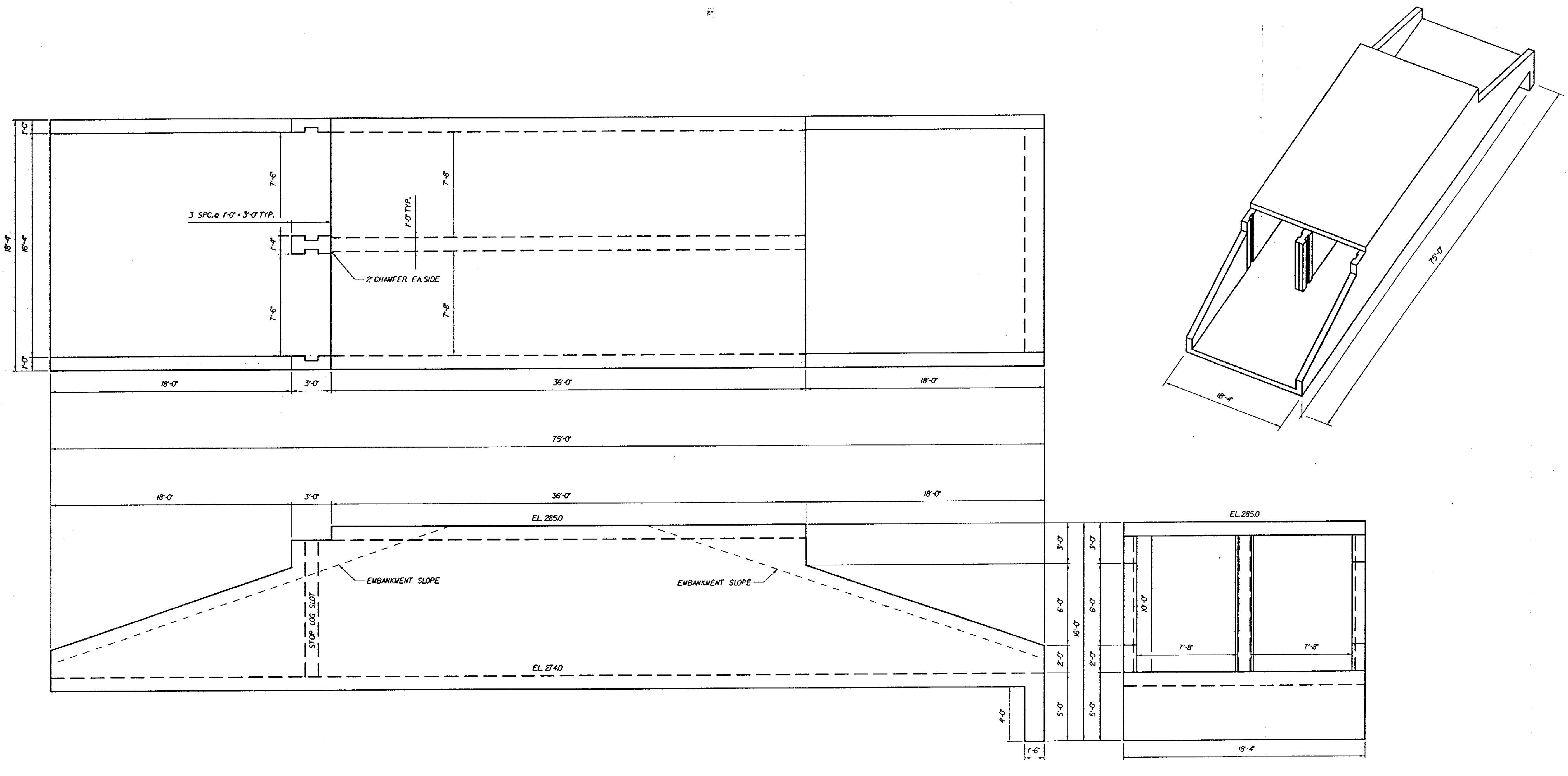
US Army Corps of Engineers Memphis District

U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
MEMPHIS, TENNESSEE

ENVIRONMENTAL RESTORATION NATURAL RESOURCES PROJECT
MISSISSIPPI RIVER AND TRIBUTARIES CONSTRUCTION

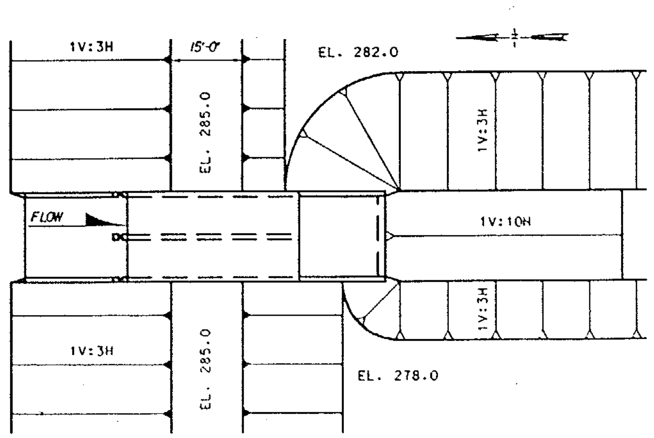
REELFOOT LAKE, TENNESSEE & KENTUCKY
REELFOOT CREEK SEDIMENT RETENTION BASIN
LOW LEVEL SPILLWAY

DRAWING
II-A-19
19 OF 21



OUTLET STRUCTURE

SCALE 1/8" = 1'-0"



SITE PLAN

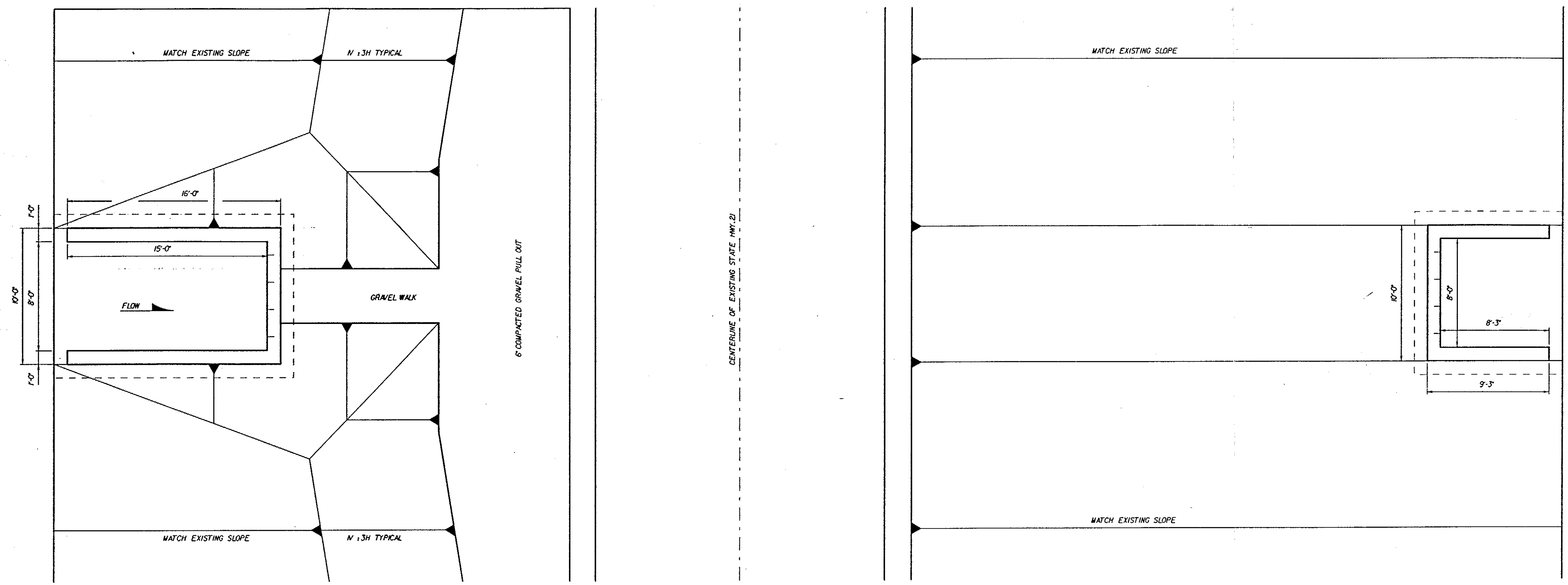
US Army Corps of Engineers Memphis District

U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
MEMPHIS, TENNESSEE

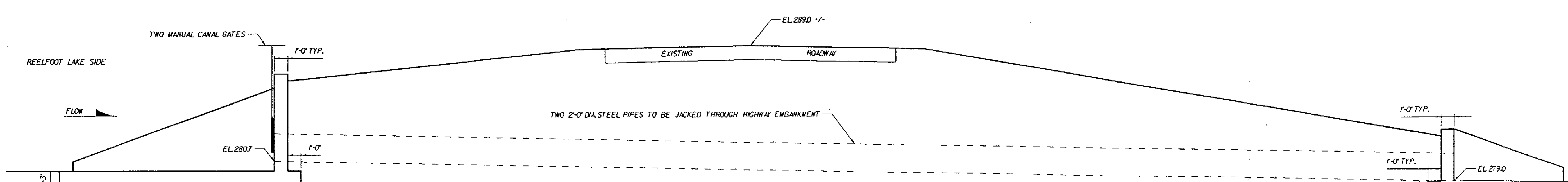
ENVIRONMENTAL RESTORATION NATURAL RESOURCES PROJECT — MISSISSIPPI RIVER AND TRIBUTARIES CONSTRUCTION

REELFOOT LAKE, TENNESSEE & KENTUCKY
LAKE ISOM RESTORATION
OUTLET STRUCTURE

DRAWING
II-A-20
20 OF 21



PLAN
SCALE 1/4" = 1'-0"



SECTION
SCALE 1/4" = 1'-0"

US Army Corps of Engineers Memphis District

U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
MEMPHIS, TENNESSEE

ENVIRONMENTAL RESTORATION
NATURAL RESOURCES PROJECT

MISSISSIPPI RIVER AND TRIBUTARIES
CONSTRUCTION

REELFOOT LAKE, TENNESSEE & KENTUCKY
WATERFOWL MANAGEMENT UNITS / SHELBY LAKE
REELFOOT LAKE WATER SUPPLY STRUCTURE

DRAWING
II-A-21
21 OF 21

APPENDIX B

SECTION 2, Part B: ELECTRICAL

APPENDIX B

SECTION II

FEASIBILITY STUDY

REELFOOT LAKE ALTERNATIVE SPILLWAY

PART B - ELECTRICAL

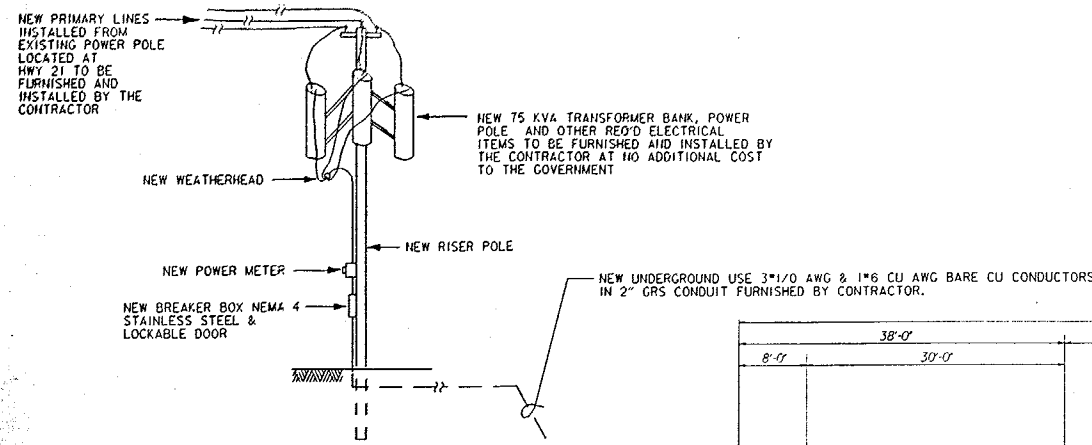
2-B-01 GENERAL

The electrieal design for the proposed Reelfoot Lake Alternative Spillway will include constructing electrical primary and secondary services to the structure. The new electrical primary service will include placement of at least one power pole to support a bank of three 25 kVA transformers, overhead conductors, cut-out fuses, lighting arrestors and grounding. The electrical secondary service will include the installation of a weatherhead, conduit, conductors, metering enclosure and lockable disconnect switch mounted to the new power pole, underground conduit and conductors run to a new power panelboard mounted on the structure and individual conduits and conductors to each of the six electrical gate operators. The center two gates shall have the option to be controlled in manual and/or automatic modes. Each of the other gates shall be controlled manually with electrical push buttons for raise or lower of each hoist gate. There shall be adjustable limit switches and torque limit switches for each hoist operator. Each hoist control compartment shall have one 120 volt AC heater to prevent condensation of moisture.

The design of the various sub-systems will be based on the use of equipment and materials that are available as standard products of the electrical industry. In the selection of materials and equipment, special consideration will be given to ease of operation, reliability and ease of maintenance. The Standards of the National Electrical Manufactures Association (NEMA), the Institute of Electrical and Electronics Engineers (IEEE) and the American Standards Institute (ANSI) will be used as guides in the selection of guides in the selection of motors, panelboard, breakers, breaker boxes, conduit, conductors and other materials. The design of circuits, conduit systems and the grounding system will conform to the National Electrical Code, the National Electrical Safety Code and Corps of Engineers Guide specifications.

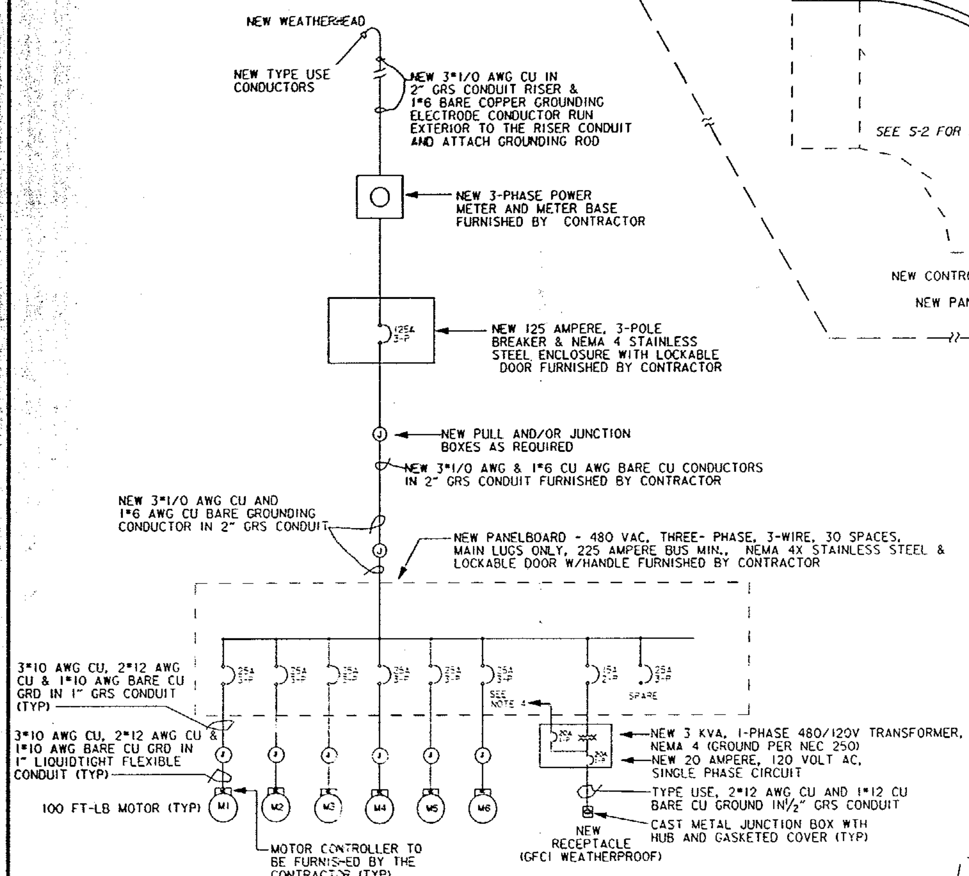
2-B-02 POWER SOURCE.

All commercial electrical power for the gated structure will be supplied by the Gibson Electric Membership Corporation with headquarters in Trenton, Tennessee. The electrical power needed shall be 125 ampere service at 480 volts AC, three (3) phase for 6-10 horsepower motors and 1-3kVA single phase transformer. The operating costs will depend on the amount of usage and shall be paid by local interest.



NEW POWER POLE

SCALE: NONE

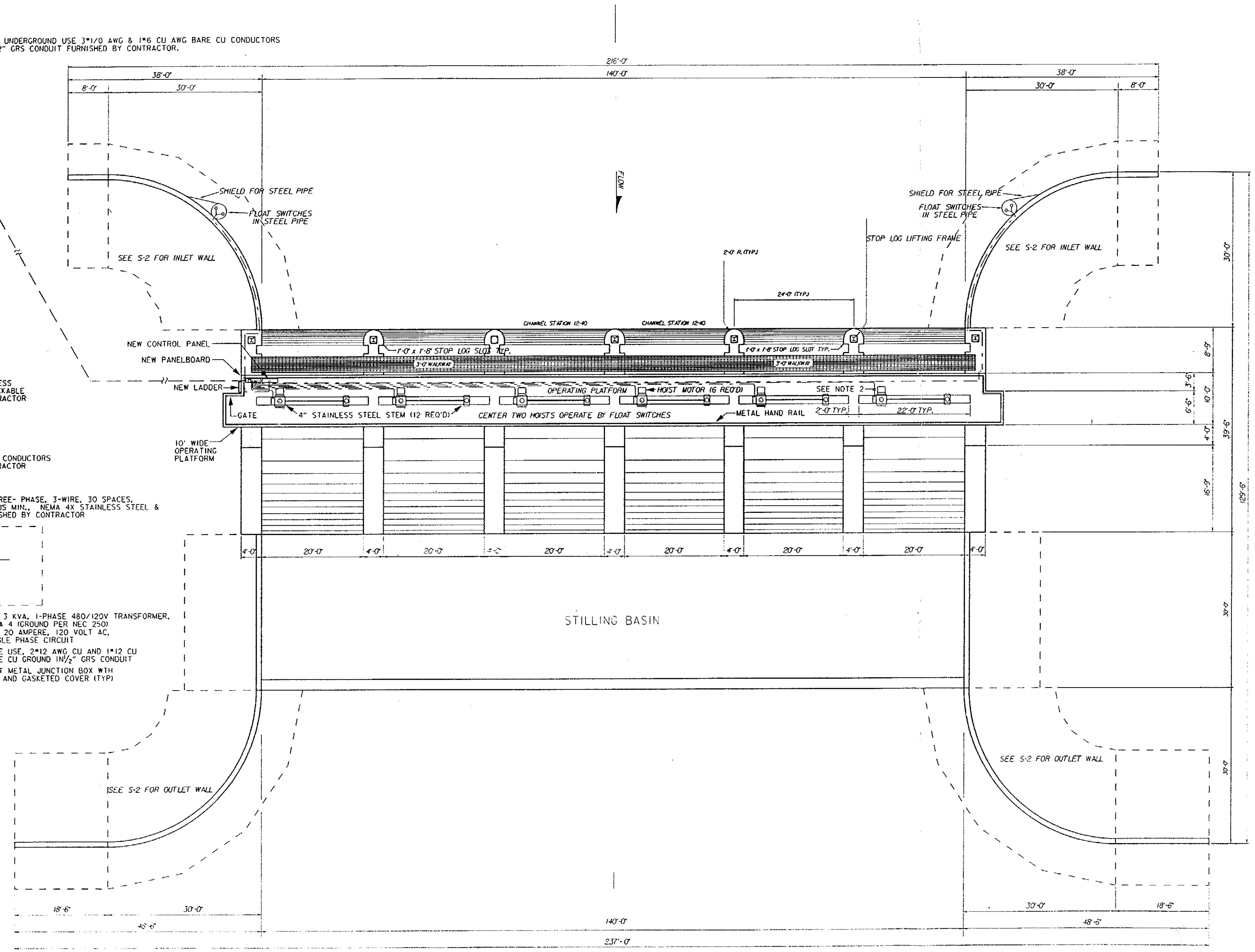


ONE-LINE ELECTRICAL DIAGRAM

SCALE: NONE

NOTES:

1. GRS DENOTES GALVANIZED RIGID STEEL CONDUIT.
2. 3/4" GRS CONDUIT IMBEDDED IN CONCRETE WITH TYPE USE 3#10 AWG CU AND BARE 1#10 AWG CU BARE COPPER GROUND. (TYP.)
3. CONDUCTORS TO BE COPPER (CU). ALUMINUM CONDUCTORS ARE NOT ALLOWED.
4. 2#12 AWG CU CONDUCTORS SHALL BE PROVIDED FOR THE HEATER IN EACH HOIST CONTROL COMPARTMENT.



PLAN VIEW OF GATED SPILLWAY

SCALE 1/32" = 1'-0"

US Army Corps of Engineers Memphis District

U.S. ARMY ENGINEER DISTRICT CORPS OF ENGINEERS MEMPHIS, TENNESSEE

MISSISSIPPI RIVER AND TRIBUTARIES
ENVIRONMENTAL RESTORATION NATURAL RESOURCES PROJECT CONSTRUCTION

REELFOOT LAKE, TENNESSEE & KENTUCKY
REELFOOT LAKE ALTERNATIVE SPILLWAY
ONE LINE ELECTRICAL DIAGRAM

DRAWING
II-B-1
1 OF 1

APPENDIX B

SECTION 2, Part C: MECHANICAL

APPENDIX B
SECTION II
FEASIBILITY REPORT
REELFOOT LAKE SPILLWAY DESIGN

PART C - MECHANICAL

2-C-01 GENERAL

The proposed Reelfoot Lake Control Structure will consist of six bays, and a 40 foot high by 129.5 foot wide reinforced concrete structure . Each bay will have a 12 foot high by 20 foot wide roller gate located on the inlet side of the structure. There will be two 27 foot high by 38 foot wide curved wing walls located on the inlet side of the structure and two 19 foot high by 48.5 foot wide curved wing walls located on the outlet side of the control structure.

2-C-02 ROLLER GATES

The roller gates (six total) will be installed at the entrance of each of the inlet bays. Each roller gate will be 12'-0" high and 20'-0" wide and will be raised by a tandem, electric motor operated gate hoist unit. The roller gates must be capable of withstanding a minimum 25 foot seating head and a minimum 15 foot unseating head. The seating/unseating heads are measured from the water surface to the middle of the gate when the gate is in the closed position. The gate will be manufactured by using horizontal hot-rolled steel channels or wide flanged beams spaced equally so that each channel/beam carries its portion of the hydraulic load. Vertical spacers between the horizontal channel/beams will be of the same size as the channel/beam. These horizontal and vertical members will be arranged in a box configuration with a face plate welded to the members.

The gate will operate on steel rails located in the roller slots in the side of the gate opening. There will be two rails in each roller slot and the roller slots will be fabricated steel plates and/or channels. The rollers will be machined cast iron with a side flange and a self-lubricating bronze bearing. The rollers will be adjustable and bolted on to the side of the gate slide to aid in alignment and seating. There will be a minimum of two rollers per slide.

Seating of the gate will be accomplished, on the upstream side, by rubber seals mating against machined corrosion resistant steel contact surfaces. The side seals will be hollow "J" bulb seals and the bottom seal will be a flat rubber section. The bottom seal will be flat to prevent the accumulation of silt and debris around the bottom of the gate which could adversely affect the gate's operation. The gate will rest on an adjustable bottom sill which will have a machined corrosion resistant contact surface. The adjustable bottom sill shall be grouted in place after alignment and proper seating between contact faces has been checked. The gate and rails will be painted with a coal tar epoxy paint to prevent corrosion. There will be two stainless steel gate stems per gate with bronze bushings in the stem guides for raising and lowering the gate.

The gate will be raised and lowered by an electric motor operated gate hoist with torque limit switches to prevent an overload on the gate and gate stem. The force required to prevent to lift the gate is calculated from the following formula:

$$F=62.4APf+W+w$$

Where:

F= lifting force required

A=area of gate opening in square feet

P=effective head of water in feet

f=coefficient of friction

W=weight of gate slide

w=weight of stem

The area of the gate opening is 240 square feet and effective head as measured from the water surface to the middle of the closed gate is 17 feet, based on the 100 year record high water elevation of 286.10 when the gate is in the closed position. The coefficient of friction is 0.2, the weight of the gate and stem is approximately 37,000 1bs. The lifting force required is approximately 56,000 1bs.

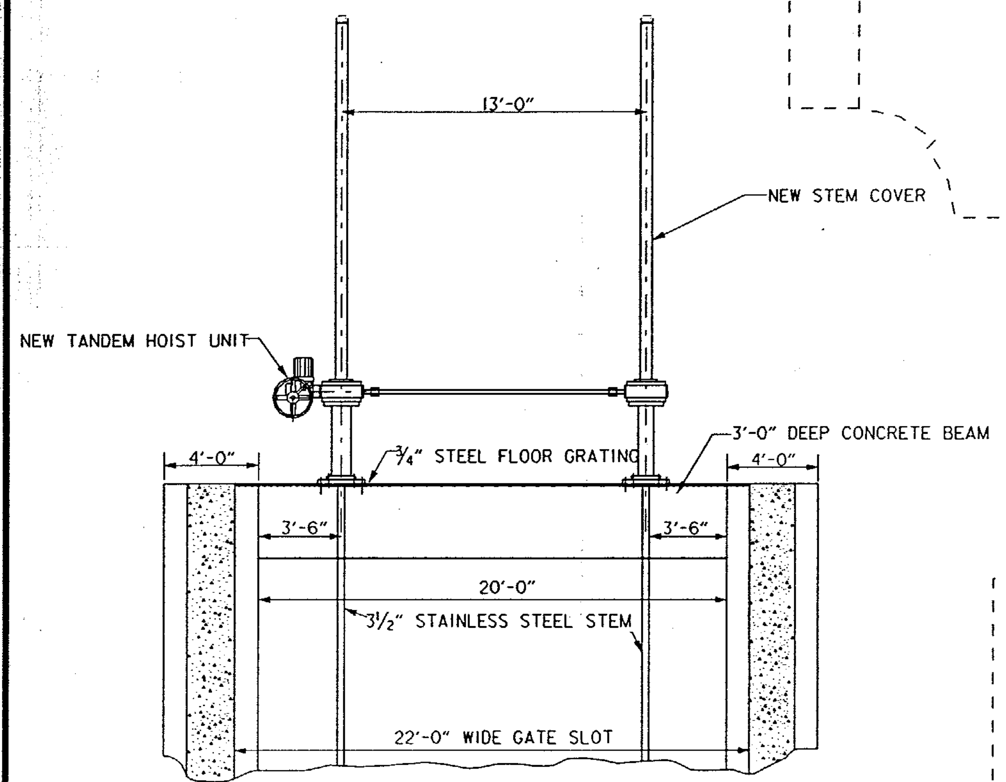
2-C-03 MOTORIZED GATE OPERATOR

Each of the tandem gate lifts shall be driven by a motorized gate operator mounted on one of the tandem gate lifts and as shown on the contract drawings. The electric motorized gate operator shall be similar to a Type EIM Series 5000 motorized control operator as manufactured by EIM Corporation or a L120 motorized "Limitorque" control operator as manufactured by the Limitorque Corporation Inc., or an approved equal. Each unit shall be furnished with integral controls and weatherproof push-button station. Each control package shall be delivered completely wired, assembled and ready for installation and shall include, but not limited to, a mechanically interlocked reversing controller having a continuous current rating not less than required by the motor manufacturer, a space heater, a 120-volt control transformer, overload relays, a sufficient number of contacts to operate the motor and auxiliaries (indicating lights, etc.), one spare normally open contact, one spare normally closed contact, and sufficient terminal strips to make all necessary wiring connections. The terminal strip for the input supply lines shall be capable of accepting copper conductors up to AWG 10. The push-button stations shall be mounted on the side of each hoist unit. Each push-button station shall consist of three pushbuttons: "OPEN", "STOP", and "CLOSE", and two indicating lights, red and green. The push-button stations shall be enclosed in a weatherproof NEMA 6 enclosure. The controls shall conform to NEMA ICS and UL 508 standards.

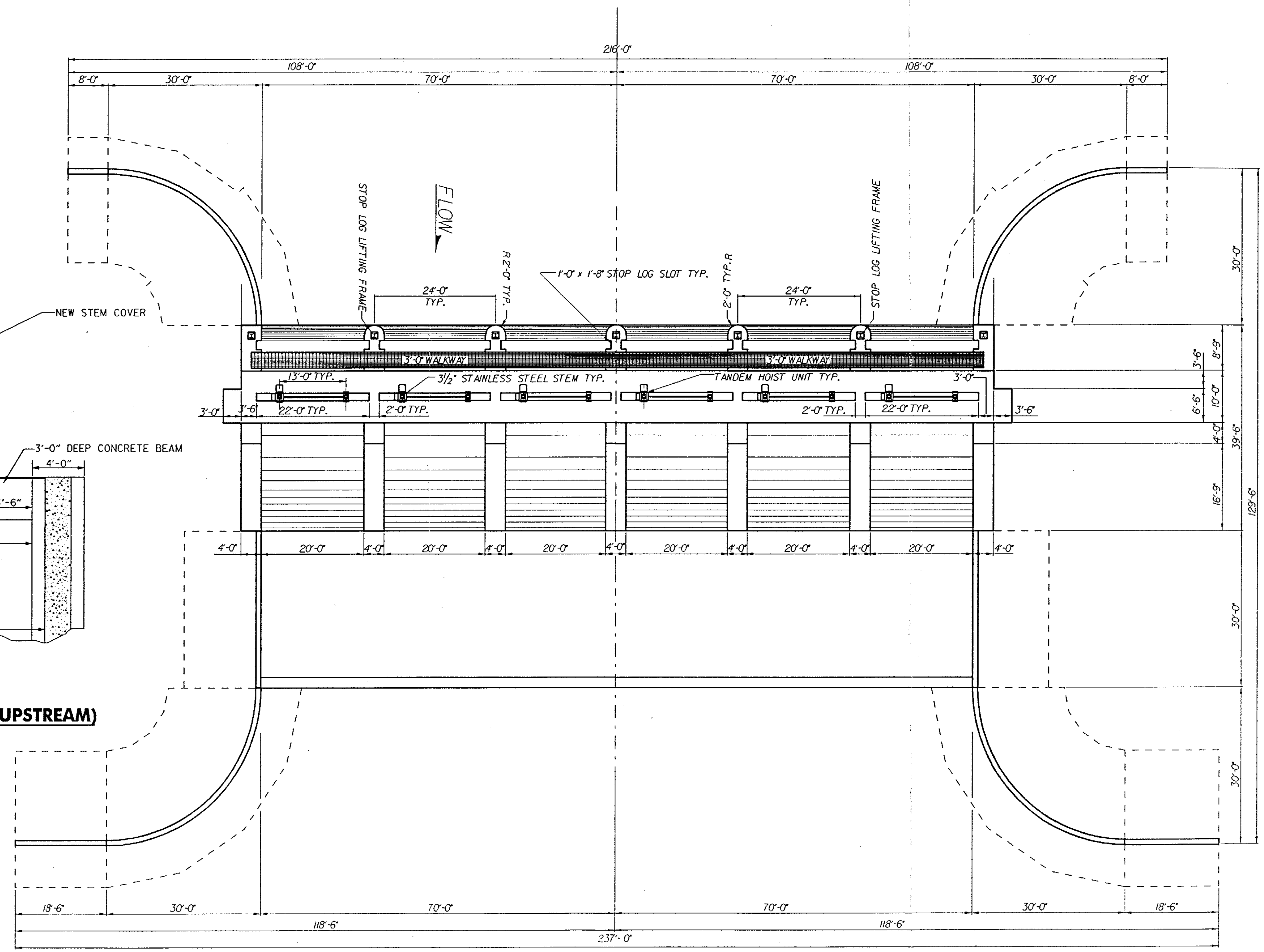
The electric motor for each hoist unit shall have a minimum 80 ft-lbs starting torque rating with a 30 minute duty cycle, 1800 r.p.m. operating speed, be suitable for operation on 480-volts A.C., three phase, 60-Hz power supply, have sufficient horsepower to operate the hoist unit through the full gate travel in both directions without exceeding the full-load ampere rating and conform to applicable requirements of NEMA Publication MG1. All bearings shall be anti-friction type. Each motorized gate operator, including motor, shall be enclosed in weatherproof type enclosures conforming to NEMA 6 specifications. Reduction gearing shall consist of generated helical gears of heat treated steel. Worms shall be of hardened alloy steel with threads ground and polished. The worm gear shall be one piece of chilled nickel bronze accurately cut. All reduction gearing shall run in grease lubricant. All gears shall be carried on heavy-duty ball or roller bearings adequate for all torque and thrust loads imposed by operation of the gate at the specified conditions. Suitable seals shall be provided at all points as required to retain the grease. Rubber for seals shall meet the applicable requirements of ASTM D 2000 for "Grade R 625." Each motorized gate operator shall be of such design as to permit manual operation of the unit in event of power failure or as necessary during servicing. A handwheel for each hoist unit shall be provided and an arrow with the word "open" shall be cast on the rim of the wheel, indicating the direction of opening. Effort required to operate the hoist manually with the gate in motion shall not exceed 40 pounds at the wheel rim. The motor operator shall include a built-in clutch mechanism so that the handwheel will not rotate during motor operation nor shall the motor turn during manual operation. A dial-type gate position indicator shall be located on each of the motor operators to show gate position during both hand and motor operation. Each indicator shall be graduated to show "full open" when the bottom of the gate is in the fully raised position at elevation shown on the drawings. Each of the indicators shall be housed in water tight enclosures. The internal arrangement of each hoist unit and of each gate stem shall be such that all moving parts run in grease lubricant with adequate grease fittings provided to lubricate the gate stems and stem nuts. Each lift nut shall be a one piece bronze nut and shall meet the applicable requirements of ASTM B 584.

2-C-04 LIMIT SWITCHES

During closing and opening travel of each gate, the gate and gate stems shall be protected against overload by a torque responsive mechanical switch. There shall also be provided geared limit switches for stopping the gates at both the fully closed and the fully open positions. The torque switches shall be operative during the entire travel of the gates to protect the stems and gates against possible damage in the event an obstruction is met. The torque switches shall function without auxiliary relays or other devices and shall be field adjustable to ensure (1) stopping the lowering operation of the gates should the stem load, for any reason, become compressive to a degree greater than the normal seating requirements and (2) stopping the raising operation should a 100% overload develop and the stem tension becomes greater than the normal unseating requirements. The position limit switch shall be adjustable and of the intermediate gear type, governed by rotation of the motor driving mechanism. Internal motor control wiring for 120-volts A.C., single-phase, 60 Hz operation and motor wiring for the 480-volts A.C., 3-phase, 60 Hz power. The unit shall be provided with suitable terminal blocks in the limit switch compartment, and terminal blocks shall be clearly marked in a suitable manner to facilitate external control and power connections. All internal wiring shall not be smaller than No. 12 AWG stranded copper conductor with not less than 3/64-inch thick "Neoprene" or approved insulation. The center two gates will operate automatically in fully open and closed positions to maintain the pool elevation between 282.0 to 284.0 with approximately tenth of a foot increment.



HOIST UNIT DETAIL (LOOKING UPSTREAM)
SCALE 1/4" = 1'-0"



PLAN VIEW OF GATED STRUCTURE
SCALE 1/32" = 1'-0"

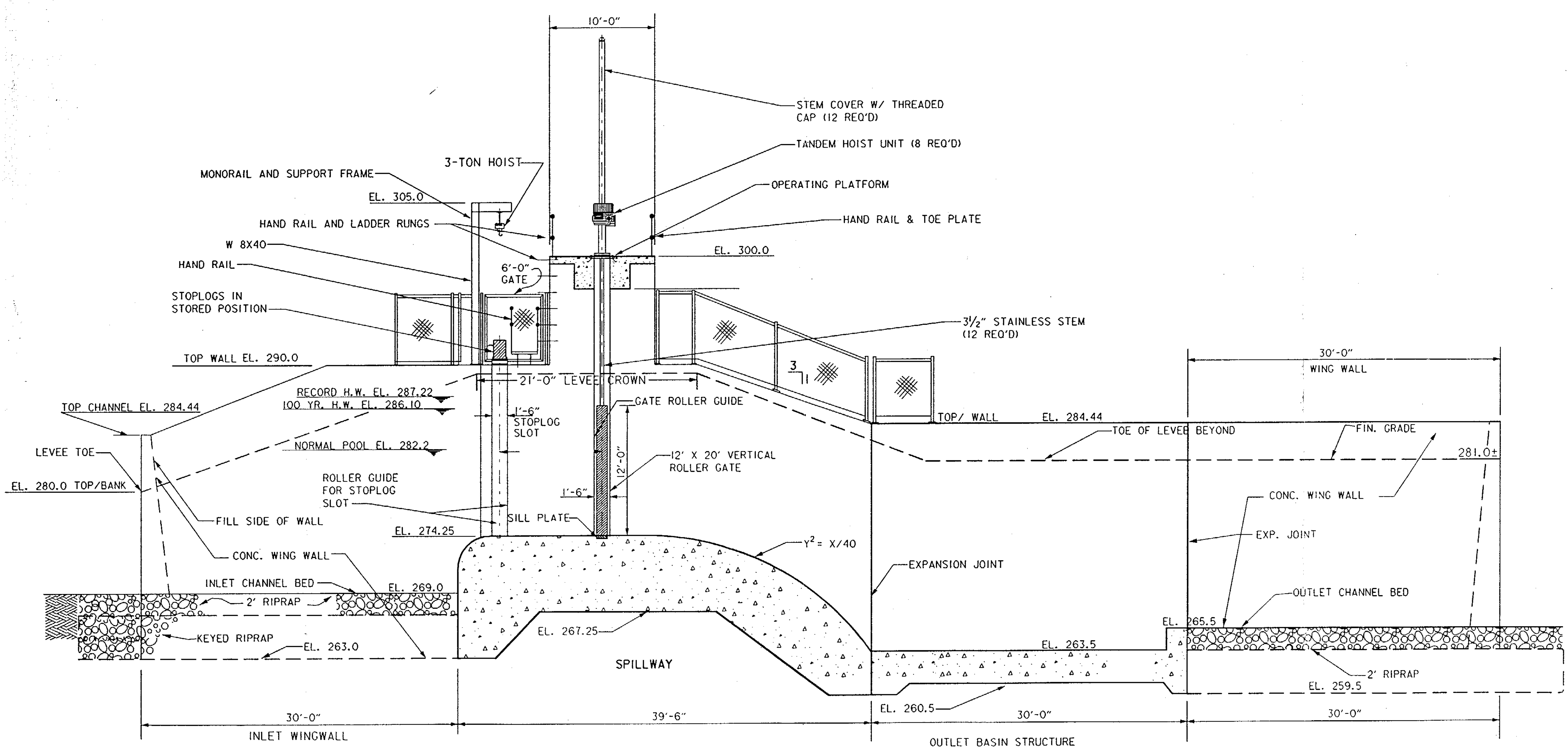
US Army Corps of Engineers Memphis District

U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
MEMPHIS, TENNESSEE

ENVIRONMENTAL RESTORATION NATURAL RESOURCES PROJECT
MISSISSIPPI RIVER AND TRIBUTARIES CONSTRUCTION

REELFOOT LAKE, TENNESSEE & KENTUCKY
REELFOOT LAKE ALTERNATIVE SPILLWAY
SPILLWAY PLAN - MECHANICAL

DRAWING
II-C-1
1 OF 2



GATED SPILLWAY
SCALE 3/16" = 1'-0"

OUTLET BASIN & WALL
SCALE 3/16" = 1'-0"

US Army Corps of Engineers Memphis District

U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
MEMPHIS, TENNESSEE

ENVIRONMENTAL RESTORATION NATURAL RESOURCES PROJECT — MISSISSIPPI RIVER AND TRIBUTARIES CONSTRUCTION

REELFOOT LAKE, TENNESSEE & KENTUCKY
REELFOOT LAKE ALTERNATIVE SPILLWAY
SPILLWAY ELEVATION - MECHANICAL

DRAWING
II-C-2
2 OF 2

APPENDIX B

SECTION 3:
GEOLOGY AND SOILS

SECTION III – GEOLOGY AND SOILS

3-01 General

The purpose of this section is to present the results of the geotechnical design for the Reelfoot Lake feasibility report. This project consists of a large geographical area with several proposed features to improve the environmental aspects of the study area including restoration of fish and wildlife habitat to recent historic levels. The Reelfoot Lake area is located in northwest Tennessee and southwest Kentucky and comprises an area of approximately 240 square miles. The basin includes portions of Lake and Obion Counties in Tennessee and part of Fulton County in Kentucky. See Plate III-1 for a location map. The proposed plan for the Reelfoot Lake project consists of five general areas of improvement. These five areas are as follows: an alternate spillway for the main lake, improvements at Lake Isom, new construction of a sediment retention basin, excavation of circulation channels connecting the three distinguishable basins of the main lake and improvements at Shelby Lake. The locations of these features are shown on the general vicinity map, Plate III-1, except for the layout of the circulation channels. A layout of the circulation channels can be found on Plate III-70.

Each of the proposed Reelfoot Lake improvement items consists of multiple features. This report will address the geotechnical design and considerations of these main features. The alternate spillway will consist of construction of a new spillway, inlet and outlet channels, embankments, closure of the existing spillway, and a new bridge. The sediment retention basin will consist of construction of an embankment, three large drainage culverts, a primary spillway and an emergency spillway. The Lake Isom project will consist of an embankment enlargement including the earthen emergency spillway and replacement of the existing control structure. The Shelby Lake area will consist of a lake enlargement, construction of waterfowl management units around the lake, and mechanical means of providing water to each of the sites. The circulation channels will consist of removing sediments to a specified elevation around the main lake. These channels will especially aid in ability to move around the lake during lake drawdowns. Corps of Engineers design criteria and standards were used for the geotechnical design presented in this feasibility study.

3-02 Geology

The Reelfoot Lake watershed is located within the Mississippi Embayment section of the Gulf Coastal Plain. The Reelfoot Lake area is characterized by several physiographic features – the lake itself; the Mississippi River alluvial floodplain; Tiptonville Dome, an elliptical-shaped rise extending from Proctor City, Tennessee, south to Tiptonville, Tennessee; a bluff line east of the lake that crosses the watershed along a northeast-southwest axis; and uplands east of the bluff.

The western half of the study area is situated on the alluvial floodplain and the eastern half is situated on loess covered uplands. The alluvium consists of point-bar, natural levee, back-swamp and abandoned channel deposits. The deposits are composed of 10-50 feet of silts and clays which are underlain by 40-150 feet of sands and gravels. The alluvium lies on the Tertiary which is the Jackson Formation. The silty loess uplands are underlain by the Continental gravel deposits.

The project area is located within the Seismic Risk Zone 3 and is in the immediate vicinity of the epicenter of the New Madrid earthquakes of 1811-12. The most severe earthquake during this period had an estimated body-wave magnitude of $M_b = 7.5$ or a Modified Mercalli Intensity of XI-XII. During these earthquakes, Reelfoot Lake, as we know it today was formed. A north-northwestern striking fault near Reelfoot Lake vertically displaces post-Paleozoic sediments a minimum of about 53 meters.

3-03 Subsurface Investigation

The level of the subsurface investigation performed for this report was selected in an attempt to provide an accurate yet cost efficient characterization of the subsurface. The investigation consisted of obtaining new data to supplement previous boring information taken for the November 1993 Reelfoot Lake Reconnaissance Report and existing historic data. The historic data came from past projects within the study area and consisted of seepage studies along the Mississippi River system and Lake No. 9 Pumping Station project located near the Mississippi River Levee northwest of Reelfoot Lake. None of the historic boring information is presented in this report. The previous recon subsurface investigation consisted of eight borings taken throughout the main lake (1986 and 1993). Six of these borings were taken on the lake from a raft mounted, portable drilling rig. The other two borings were taken on land within 150 feet of the shoreline with a truck-mounted drilling rig. Eleven additional borings (1995 and 1997) were taken for this study using a truck-mounted drill rig or hand auger as follows: five borings were taken for the alternate spillway and inlet and outlet channels, two borings at Shelby Lake, one boring at Lake Isom, and three borings at the sediment retention basin. All nineteen borings presented in this report were made with an auger, split-spoon sampler and/or 3-inch and 5-inch thin walled Shelby tubes. The boring locations for each boring are shown on Plate III-2 with the boring logs presented as Plates III-4 through III-26. The standard boring legend is presented as Plate III-3. The general stratification of the Reelfoot area indicated by the borings is a varying overburden layer of 25 to 45 feet in thickness consisting of silts and clays intermixed with sandy strata underlain by fine to medium sands.

3-04 Laboratory Testing Laboratory testing of the boring samples was performed by the Waterways Experiment Station (WES) and the Memphis District laboratories. Unconsolidated-undrained (Q), consolidated-drained (S) shear tests, and consolidation (C) tests were performed by WES. Classification, natural moisture contents, natural densities, and Atterberg limits were performed on representative samples by both laboratories. The Memphis District Laboratory performed unconfined compression tests and mechanical analyses on course-grained samples. WES test results for this project are located at the end of the Plates in Appendix III-A. The specific strata where WES tests were conducted are also shown on the boring logs.

3-05 Soils and Foundation Analysis

a. General. The boring logs along with the corresponding test results were examined to determine appropriate soil stratification and shear strength parameters for the major features of the foundation design. Once stratifications and shear strength values were assigned, a variety of foundation analyses were performed as applicable to determine liquefaction potential, channel slope stability, structural excavation slope stability, structural sliding and overturning stability, bearing capacity, settlement, uplift analyses, and dewatering requirements.

b. Design Shear Strengths. (1). Clays. The selection of the design values for the Q and R conditions was based on consistencies indicated by the boring logs, natural densities, moisture contents, Atterberg limits, unconfined compression tests and Q triaxial tests. Design values for the S condition were based on S (direct shear) tests by WES and the Plasticity Index (PI) verses ϕ' relationship developed for LMVD and contained in WES Technical Report No. 31-6-4, June 1962.

(2). Silts. The design shear strengths for silts were selected conservatively using consistencies indicated by the boring logs, natural moisture contents, past experience with similar soils, and values suggested in DIVR 1110-1-400, Section 5, Part 4, Item 1, dated March 1973. Design values of ϕ=20°, c=300 psf were selected for the Q and R conditions. Design values of ϕ=28°, c=0 psf were chosen for the S case strengths.

(3). Sands. Design values for the coarse grained soils were selected based on past experience with similar soils, suggested values in the above DIVR, and correlation's between standard penetration tests (N-values) and the angle of internal friction (ϕ). Cohesive values were taken as zero for all loading conditions.

3-06 Stability Analyses.

a. Slope Stability. Slope stability analyses were performed to determine the required slopes for the features of this project including embankment slopes, excavation slopes, cofferdam slopes, and inlet and outlet slopes. Slope stability analyses were conducted in accordance with guidelines and criteria presented in DIVR 1110-1-400, Section 5, Part 4, Item 1, dated March 1973, for Type A projects. Long term stability analyses for the alternate spillway outlet channel below station 20+00 used Type B criteria or a factor of safety greater than 1.0. All other stability analyses were performed for the following loading cases with respective minimum allowable factors of safety for Type A projects:

Loading Case	Minimum Factor of Safety
After Construction (AC)	1.30
Long – Term (LT)	1.25
Sudden Drawdown (SD)	1.20
Partial Pool (PP)	1.30

Stability analyses for the appropriate loading conditions were performed using the micro computer software package GEOSLOPE which is available through the GEOCOMP Corporation. The program's capability to analyze both circular and non circular shaped failure surfaces was used in the analysis of all slopes. The analysis of circular failure surfaces was performed using the Modified Bishop Method of Slices. For non-circular failure surfaces or wedge failures, the Janbu Method of Generalized Slices was utilized. Manual computations of the controlling case were also performed as a check to the program. A legend for the slope stability analyses is presented as Plate III-3.

b. Structural Stability Analyses. (1). (Sliding). Sliding stability analyses were performed for the major features of the proposed project, including the alternate spillway and spillway retaining walls, sediment retention basin primary spillway retaining walls, and the Lake Isom replacement control structure. All analyses were performed in accordance with the procedures presented in ETL 1110-2-256 and EM 1110-2-2502. These analyses were performed to ensure the stability against sliding at the base of the structure or through any soil layer below the base. The Limit Equilibrium Method was utilized for determining the factor of safety against sliding. The Rankine method for determining forces on the structures was used in the analyses. Program CSLIDE from the WES library was also used for some of the analyses and as a check of the safety factors. A minimum factor of safety of 1.50 was used for normal loading conditions and 1.33 for unusual loading conditions. Since the project is located within Seismic Risk Zone 3, stability of each structure was analyzed for earthquake induced loading. A horizontal acceleration coefficient of 0.15 g was used to determine the lateral earthquake forces acting on the structure. The vertical earthquake acceleration was neglected. Criteria were based on guidance presented in ER 1110-2-1806. The minimum allowable factor of safety for sliding for earthquake loading was 1.10.

(2). Overturning Analyses. Overturning stability analyses were performed for the major features of the Reelfoot Lake Project. These analyses were conducted to ensure the location of the resultant force occurred within the middle third of the base for the after construction (AC) loading condition and the middle half of the base for the sudden drawdown (SD) loading condition. The method of analysis used for overturning along with the required minimum resultant locations are presented in EM 1110-2-2502. The active soil and water pressures were applied to the structure using at rest earth pressure coefficients for the driving side forces. The resisting side forces varied from zero (ignoring passive resistance) to a maximum conservative force calculated using up to the active earth pressure coefficient . An additional force to balance the horizontal loading conditions was also included in the overturning analyses. Overturning analyses were also performed for earthquake loading to determine the resultant location with seismic forces applied. A horizontal acceleration coefficient of 0.15 g was used to determine the lateral earthquake forces. Guidance presented in EM 1110-2-2502 required only that the resultant force be located within the base for earthquake conditions.

(3). Bearing Capacity. Bearing capacity computations were based on principles and methods presented in EM 1110-1-1905. Bearing capacity was determined by Hansen's Equation, which is a modification of the general bearing capacity equation to account for effects of embedment, overburden pressure, foundation shape, and inclination of loading. Earthquake loading using a horizontal acceleration coefficient of 0.15 g was considered a viable case. Analyses included determining a factor of safety by dividing the ultimate bearing capacity of the foundation soil by the soil pressure determined from the net vertical load and the eccentricity loading on each structure. The eccentric loading was determined by summing unbalanced forces on the structure. A minimum allowable factor of safety of 3.0 is required for normal loading conditions, 2.0 for unusual loading conditions and 1.0 for earthquake loading as presented in EM 1110-1-1905 and EM 1110-2-2502.

(4). Settlement. Since the alternate spillway and retaining walls will be founded on sand and compacted clay gravel, settlement was considered negligible and no analyses were performed. However, settlement was a concern for most of the other features of the project. These features include the sediment retention basin embankment, culverts and spillways to be constructed in the embankment and settlement of shore line structures around Reelfoot Lake during a reservoir drawdown. Settlement analyses were conducted to estimate the magnitude of settlement. These estimates were based on consolidation test data taken for this project, empirical relationships developed for clay strata, and experience from previous projects.

(5). Seepage and Pressure Relief Analysis. A seepage and pressure relief analysis was performed for the alternate spillway outlet channel. The computations were based on principles and methods as presented in Technical Memorandum No. 3-424, Investigation of Underseepage and Its Control – Lower Mississippi River Levees.

3-07 Alternate Spillway

a. Introduction. The proposed alternate spillway consists of a six bay reinforced concrete spillway. Each of the six bays is 20 feet in width and contain vertical roller gates 12 feet in height. The structure will control Reelfoot Lake levels by adjusting the gate height and controlling the amount of water flowing through the structure. The current normal lake pool is elevation 282.2 feet NGVD. The new structure will fluctuate water levels annually between elevations 284 and 280 feet NGVD. The spillway will also have capability to draw the lake level down to elevation 274.2 feet NGVD or the top of the spillway weir. The proposed alternate spillway location is shown on Plates III-27 and III-28 and will be located approximately 1000 feet west of the existing spillway. The structure was moved from the proposed location 1500 feet east of the existing spillway as recommended in the November 1993 Reelfoot Lake Reconnaissance Report. This new location allowed for a straight bridge and highway alignment for the new spillway inlet channel instead of a curved bridge alignment as would have been the case in the 1993 report location. Five borings were taken along the proposed inlet and outlet channels with one of the five borings taken at the new structure location. Tertiary Boring 2-RLST-95 was taken at the spillway site and was used for most of the foundation design for the replacement structure. The boring locations are shown on Plate III-27 with the boring logs presented on Plates III-4 through III-10.

The alternate spillway design included both seismic risk and foundation analyses to determine if the proposed site would be suitable to support the structure. After evaluating the results of these analyses, it was determined that a foundation on grade would not meet present design criteria for a stable structure. A settlement analysis was conducted for the replacement spillway structure on grade or at El. 264.0 and resulted in an estimated excessive settlement of around 6 inches as shown on Plate III-32. Several options were then considered for the foundation of the spillway including a pile foundation under the structure, ground modifications to improve existing conditions, and excavation of the weaker material located below the structure. Based on costs and construction considerations, removal of this weaker material beneath the structure was selected as the best alternative. This included excavation of the existing soil to El. 242 and backfilling with a compacted clay gravel to the foundation grade. The structural backfill plan for the alternate spillway is presented on Plate III-31. Plates III-43 through III-45 and Plate III-47 show the proposed plan and elevation views of the spillway.

Two additional borings are proposed at the alternate spillway site since the structure was moved 100 feet closer to Reelfoot Lake from the site assumed when Boring 2-RLST-95 was taken. The proposed structure location was moved to the site presented in this report because of existing utility considerations and alignment of the inlet channel. (See Plate III-27 for location.) These borings will also verify soil conditions assumed for the analyses in this report and provide additional information before finalizing plans and specifications. A minimum of four piezometers will be installed at least six months before construction begins to monitor ground water conditions and fluctuations. This information will aid the contractor in determining a dewatering plan required for the excavation of the site and in selecting parameters for the design of the relief well system required downstream of the spillway structure.

b. Seismic Risk Study. An analysis of the proposed site of the new spillway was undertaken to determine the annual risk factor (R_t) for an earthquake to cause liquefaction of the underlying foundation sands. Based on the logs and standard penetration tests results from Boring 2-RLST-95, the most susceptible sand layers occurred immediately below the topstratum clays between elevations 249 and 245. This factor coupled with other concerns regarding the structural stability of the spillway (settlement and bearing), resulted in over-excavation of the foundation to El. 242 and replacing the material with compacted clay gravel. The next layers of sand that are most susceptible to liquefaction occur at depths between 69 and 79 feet. A risk analyses was performed to determine the cost effectiveness of abating the liquefaction potential in these layers as presented on Plates III-33 and III-34. The analysis resulted in an existing condition risk factor of 0.005360. Assuming the sand layer between 69 and 79 feet was densified to a relative density of 80%, then the risk factor (R_t) would be reduced to 0.001641. Ground densification at this depth would be most cost effective by using vibroflotation techniques. The cost effectiveness for densifying the foundation sands was determined by comparing the present worth value of the annual damages eliminated by densifying the foundation sands to the cost of densifying the foundation sands. The damages from liquefaction were estimated by assuming that the structural damage was equal to the initial cost of the structure minus salvage value or approximately 3.5 million dollars. The benefit to cost ratio to perform ground densification in the deeper sand layer was 0.44. Therefore, densification of the deep sand layers is not an economic solution to mitigate seismic risk due to liquifaction.

c. Excavation and Cofferdam Stability Analysis. The structural excavation will be protected from flood stages at the site by an earthen cofferdam. The cofferdam will consist of a semi-compacted embankment with side slopes of 1V:2.5H minimum, a minimum crown width of 15 feet and a crown elevation of 288 feet NGVD. This crown elevation is based on the 100-year flood plus an additional 2 feet to provide adequate freeboard and account for settlement. A minimum 15-foot berm will also be required between the cofferdam and structural excavation. The excavation slopes will begin at natural ground (El. 281 feet NGVD) and extend to elevation 242 feet NGVD on a minimum slope of 1V:2.5H. The deep excavation will allow removal of the non-suitable material and replacement with compacted clay gravel.

Stability analyses were conducted for the combined cofferdam and excavation together with the above slopes. The results of the stability analyses are presented on Plate III-35. Boring 2-RLST-95 was used for the stratification and shear strengths in the analyses. The analyses conservatively assumed that a dewatering system was in place and maintaining the ground water three feet below the excavation bottom. A steady seepage water profile was assumed for the upper strata and a water profile as shown on Plate III-35 was assumed for the deeper clay strata. A 100-year flood event was assumed against the cofferdam for all analyses. The minimum factor of safety was 1.20 at elevation 255 feet NGVD with a 1.24 at the clay/sand aquifer interface at elevation 249 feet NGVD. A manual check is presented on Plate III-35 of the analysis at elevation 249 feet NGVD with the Geoslope computer printout results shown on Plate III-36. The long term stability analysis resulted in a factor of safety of 1.17. Both minimum factors of safety are slightly below the required minimums for permanent Type A slopes. However, since the excavation will be open for only a short period of time or approximately two months from elevation 242 to 265 feet NGVD, the factors of safety are considered satisfactory. Also for this reason, S-strengths were not considered a necessary loading case and were not utilized in the analyses. Full uplift seepage forces (el. 286 feet NGVD) were also applied under each of the layers between elevations 264 and 273.5 feet NGVD as shown in the table on Plate III-35.

d. Dewatering. A dewatering analysis was performed to determine the required number and layout of wells for the dewatering cost estimate for the deeper aquifer (below el. 249 feet NGVD). Dewatering requirements were based on procedures and guidelines presented in TM 5-818-5. The dewatering system was designed to lower the water table to elevation 239.0 feet NGVD which is 3 feet below the bottom of the deepest excavation. It was assumed that Reelfoot Lake would act as the line source of seepage for the ground water. The lake is located approximately 450 feet from the centerline of the excavation. The analyses was performed assuming a headwater elevation of 286 feet NGVD which is the 100-year flood for the Reelfoot Lake area and assuming artesian flow conditions would prevail. The Tertiary deposits were assumed to be located at elevation 180 feet NGVD based on Boring 1-RLUT-93 and the assumptions shown on the analysis presented as Plate III-37. An average horizontal permeability around 1100 x 10^{-4} cm/sec was selected based on the D_{10} grain size of the foundation sands taken from boring samples in the area. The analysis as presented on Plate III-37 indicates that a dewatering system consisting of seventeen 12-inch-diameter fully penetrating deep wells is required for the 100-year flood. The estimated dewatering well locations are also shown on Plate III-37.

Dewatering will also be required in the shallow aquifer from elevations 264 to 273.5 feet NGVD. The silty material will require either a vacuum wellpoint system or cutoff wall (ex. - bentonite slurry trench) to effectively seal the water from entering the excavation. For the preliminary estimate it was assumed that approximately 400 wellpoints twenty feet in length will be required along with one large pump rated at least 1000 gallons per minute and one standby pump of the same size. The dewatering analysis presented in this report is for cost estimating purposes only since the actual dewatering system will be Contractor designed, installed, and operated. As stated above, additional data will be obtained at the proposed spillway site that will aid in the design and operation of a dewatering system.

e. Seepage and Relief Analysis. An analysis was performed on the outlet channel downstream of the stilling basin to determine if relief measures would be required for the outlet channel. Borings 1-RLSU-95, 2-RLST-95 and 3-RLU-95 were used for the analyses. The results are presented on Plate III-38 and indicate that the outlet channel is susceptible to excessive uplift pressures under the clay layer between elevations 249 and 265.5 feet NGVD. A relief system will be required on both sides of the outlet channel along the stilling basin and downstream riprap protection below the alternate spillway. Preliminary analyses indicates approximately twelve 8-inch diameter relief wells approximately 85 feet in length will be required.

f. Inlet Channel Stability. The inlet channel for the alternate spillway consists of a proposed 1200 foot long, 140 foot bottom width channel with an inlet invert elevation of 269 feet NGVD. A new highway bridge will be constructed where the inlet channel crosses the existing highway embankment as an integral part of the alternate spillway replacement project. The Tennessee Department of Transportation will design and construct this bridge facility. Borings 1-RLBU-95 and 2-RLST-95 were used to analyze the inlet channel for stability utilizing Type A design criteria. See Plates III-27 and III-28 for the inlet and outlet channel layout and location of the borings. A 1V:3H slope is required to meet design criteria for the inlet channel. Since there will be no excavated material placed north of the highway, the controlling analyses for the upper reach included stratification from Boring 1-RLSU-95 and a section through the proposed new bridge site embankment. The analysis is presented on Plate III-39 with the computer printout from GEOSLOPE for the controlling case on Plate III-40. The sudden drawdown (SD) case resulted in the lowest factor of safety using conservative water limits and a wedge type failure through the clay layer at elevation 260 feet NGVD. A shear strength of ϕ=0, c=400 psf was used for the failure layer. Both Borings 1-RLSU-95 and 2-RLST-95 were analyzed with a 40-foot berm and 20 foot high excavated material embankment for fill placed between the highway and alternative spillway site. A slope of 1V:3H is required from the analyses using Boring 1-RLSU-95 with the SD case controlling with a factor of safety of 1.18. Analyses with Boring 2-RLST-95 resulted in a slope of 1V:2.5H meeting criteria using an outlet channel grade of elevation 265.5 feet NGVD. The lower channel grade was conservatively used so that it would apply to both upstream and downstream conditions. As shown on Plate III-41, the long-term stability analysis controlled for this reach with a factor of safety of 1.23.

g. Outlet Channel Stability. The outlet channel consists of a 4000-foot long channel with a 140-foot bottom width at the structure that transitions to a 30-foot bottom width 400 feet from where the channel enters into existing Running Reelfoot Bayou. Borings 2-RLST-95, 3-RLU-95, 4-RLU-95, and 5-RLU-95 were analyzed to determine the minimum required slopes to meet both Type A criteria (from the proposed spillway to station 20+00) and Type B criteria (station 20+00 to the confluence with Running Reelfoot Bayou). See Plate III-27 for the boring locations and proposed channel layout. The analyses using Boring 2-RLST-95 is presented as Plate III-41 and discussed in the paragraph above resulting in a minimum slope of 1V:2.5H. Boring 3-RLU-95 was also analyzed using Type A criteria resulting in a minimum slope of 1V:2.75H with the long term (LT) stability analysis being the controlling case (FS=1.31). The AC and SD cases resulted in higher factors of safety greater than 2.0. Since the LT case controlled, a plate representing the analyses with Boring 3-RLU-95 is not presented in this report. Borings 4-RLU-95 and 5-RLU-95 along with pertinent survey information were analyzed using Type B criteria. The analysis with Boring 4-RLU-95 was the controlling case and is presented as a typical analysis on Plate III-42. A 40-foot berm, 20 foot maximum excavated material embankment (EME), channel grade of elevation 265.3 feet NGVD, and EME slopes of 1V:2H were used in the analyses. The AC case controlled with a factor of safety of 1.24. The analyses with Boring 5-RLU-95 resulted in a minimum channel slope of 1V:2H with a LT factor of safety of 1.05 and an AC factor of safety of 1.38.

h. Construction Considerations and Recommendations for Inlet and Outlet Channel Slopes. A 1V:3H channel slope and excavated material embankment slope with a minimum 40 foot berm is recommended for the inlet channel and outlet channel to station 20+00. This slope is recommended because of stability analyses presented in this report and maintenance considerations, aesthetics, and input from the local sponsors (TWRA).

The inlet channel excavation will proceed from within the top bank slope intersections with no clearing beyond these limits. Either a platform embankment for equipment to operate from will be constructed in the channel limits to accommodate the channel cut or a dredge from the lake side or both will be required to complete the excavation. The recommended slope for the outlet channel below station 20+00 to the confluence with Reelfoot Running Bayou is a 1V:2.5H. The EME embankment can be placed to a maximum height of 20 feet above natural ground with a 40-foot berm (no higher than elevation 300.5 feet NGVD). However, feedback from the local sponsor (TWRA) indicates that a 1V:3H slope would be more favorable with the EME material placed over a larger area to minimize the EME height. Possible additional right-of-way and slightly higher excavation costs most likely would be required to accommodate this request.

i. Riprap Protection. Riprap protection will be required to protect the outlet channel downstream of the alternate spillway. The protection will extend 200 linear feet along the outlet channel beginning at the end of the concrete stilling basin. The first 100 linear feet will be protected with a 36-inch thick R650 riprap blanket from top bank to top bank. The next 100 linear feet will be protected with a 24-inch thick R650 riprap blanket along both side slopes, extending 20 feet from the toe into the outlet channel. Standard end, toe and overbank protection details will be incorporated into the riprap design. The riprap will be placed on 6-inches of bedding material and will be constructed in the dry.

3-08 Alternate Spillway Structural Stability

a. General. The alternate or replacement spillway was analyzed for structural stability. Plates III-43 through III-45 show plan and elevation (section) views for the proposed spillway. Six loading cases were assumed applicable for the structure to insure structural stability for the life of the structure. These loading cases are presented on Plate III-46 along with the results of the overturning, sliding and bearing stability analyses. Two typical loading cases were chosen as examples to present assumptions, loading diagrams, determination of horizontal and vertical forces including uplift pressures if applicable and the actual analyses to determine the factors of safety. Loading Case 3 or high normal pool with earthquake forces and Loading Case 4 or unusual loading (100 year flow line) were selected for presentation in this report as Plates III-47 through III-50.

b. Overturning Analyses. Overturning stability analyses were performed for the alternate spillway for the six cases as stated above and presented on Plate III-44. Both balanced and unbalanced horizontal forces were used to determine the location of the resultant force, although the balanced force method is recommended by the design manual. A lateral at-rest earth pressure coefficient of K_o=0.5 was selected based on an average value of ϕ=30° for a horizontal backfill (combination of riprap and impervious backfill as shown on Plate III-31). Results of the analyses indicate a resultant force location within the allowed structural base to meet the minimum requirements for all cases analyzed as shown on Plate III-46 and as presented for cases 3 and 4 on Plates III-48 and III-50.

c. Sliding Stability Analyses. Sliding stability analyses were performed for the alternate spillway for the six cases presented on Plate III-46 with the calculated factor of safety shown in the next to last column. A lateral at-rest earth pressure coefficient of K_o=0.5 was also selected based on an average value of ϕ=30° as stated above. Passive resistance of the concrete stilling basin was conservatively ignored. After determining the forces on the structure for each of the loading cases, an overturning analysis was computed to determine the resultant tangential (parallel) and normal forces on the spillway. Unbalanced horizontal forces were used in the overturning analyses. The factors of safety were calculated by dividing the normal force times the phi angle of the foundation material by the tangential resultant (driving) force. A shear strength of ϕ=33° was used for the structural backfill and based on conservative shear strengths for compacted clay gravel. The results of the analyses indicate that sliding stability for the alternate spillway is adequate for all loading cases. Although Case 3U is less than the minimum required for earthquake loading (FS>1.1), the analysis is considered safe because of the conservative assumptions of not utilizing the passive resistance of the stilling basin slab and ignoring the soil resistance at each end of the spillway structure. Both cases 3 and 4 are presented on Plates III-49 and III-50.

d. Bearing Capacity. Using the resultant forces from the sliding analyses, bearing capacity analyses were then performed to determine the maximum foundation pressures and the corresponding factors of safety for bearing. The six cases analyzed are shown on Plate III-46 with the results shown in the last column. All six cases analyzed resulted in favorable factors of safety for bearing capacity failure. Cases 3 and 4 are presented in detail on Plates III-49 and III-50.

e. Settlement. Since the alternate spillway will be placed on compacted clay gravel fill underlain by a dense sand, settlement of the spillway was considered negligible and no other additional analyses were performed. However, approximately 8 feet of fill will be placed around the structure and blanket filled to the highway north of the structure. This fill will include the permanent access roads to the structure and also serve as an embankment to withhold Reelfoot Lake floodwaters during significant flood events. Typical settlement analyses of Borings 1-RLU-95 and 2-RLST-95 indicate that the fill will settle in the range of 1.0 feet. Most of this settlement will take place in the first six months after fill placement. The permanent access roads shall be overbuilt to accommodate this settlement

3-09 Alternate Spillway Inlet and Outlet Wingwalls

a. General. The proposed inlet and outlet wingwalls for the alternate spillway will consist of four separate conventional reinforced concrete walls constructed on a 30 foot radius curvature as shown on the plan view on Plate III-44. Each of the four wingwall sections will consist of individual monoliths with a base footing that extends from the center of the curve through the wall stem on a 22.5 degrees angle. The wingwalls will be founded on the clay gravel backfill supporting the spillway structure site. The clay gravel backfill is required to meet current foundation design criteria. The compacted clay gravel will be placed from elevations 242 to the bottom of the wingwall footings similar to the section shown on Plate III-31. Six loading cases were considered for the design of the inlet wingwalls as shown on Plate III-51 and four loading cases were considered for the outlet wingwalls as shown on Plate III-54. Due to the geometry of the individual monolith sections, standard two dimensional analyses would not accurately represent actual loading on the structure as soil and wall weight, uplift pressures, internal and external forces varied. Therefore, a three dimensional analysis was performed for the appropriate loading conditions to insure stability of the wingwalls. These analyses are considered conservative in that the monolithic sections analyzed utilized minimum resisting forces with full backfill loading and the fact that the individual wall monoliths as shown in this report will most likely be constructed as one unit.

b. Inlet Wingwall Stability. The inlet wingwalls were analyzed for the loading cases presented on Plate III-51. Overturning, sliding and bearing analyses were performed for each case (Cases A-F) with the results and factors of safety presented in the Summary Table on Plate III-51. All cases analyzed exceeded the minimum required by the design manuals for a stable wingwall. Case A or normal lake level in the backfill with a four foot drawdown is presented in detail on Plates III-51 through III-53. Since the heel for the wingwall base footing only extends across the front of the base as shown on the plan view on Plate III-51, the center portion of the wall has different loading as opposed to the sides as represented by the force loading diagrams. The three dimensional analyses for each case analyzed assumed balanced horizontal forces for overturning and unbalanced horizontal forces for both sliding and bearing. A lateral at-rest earth pressure coefficient of K_o=0.5 (sand backfill with ϕ=30°) was selected for determining both the driving and resisting horizontal forces for the analyses. Cases C through E represent loading conditions if the structure was completely built and subjected to normal ground water conditions because the bridge and inlet channel were not constructed or operational for uncontrollable reasons since another agency is designing and constructing the bridge.

c. Outlet Wingwall Stability. The outlet wingwalls were also analyzed for the loading cases as shown on Plate III-54. Overturning, sliding and bearing analyses were performed for each case and the results are presented in the Summary Table on Plate III-54. Case A is presented in detail with the force diagrams presented on Plate III-54 and the analyses presented on Plates III-54 and III-55. The results and factors of safety for all cases analyzed exceeded the minimum required by the Corps. The same assumptions used for the inlet wingwall analyses were used for the outlet wingwall. An average ground water elevation of elevation 276 feet NGVD was utilized in the backfill with a minimum tailwater elevation of 263.5 below the outlet channel.

3-10 Settlement During Drawdown of Reelfoot Lake Levels

a. General. The Tennessee Wildlife Resources Agency (TWRA) proposes to periodically drawdown Reelfoot Lake from normal pools between elevations 280 and 284 feet NGVD to a minimum pool elevation of 274.2 feet NGVD. As stated in the 1993 Recon Report, the lake bottom temporarily exposed during drawdown will experience settlement due to evaporation and the temporary increase in effective stress. The amount of settlement is dependent on the following:

(1). Soil stratigraphy
(2). Evaporation rate during the drawdown period
(3). Water contents of soils
(4). Liquid and plastic limits of the soils
(5). Depth to the water
(6). Specific gravity of the soils
(7). Unit weight of the soils
(8). Compression index for the soils
(9). Recompression index for the soils

b. Settlement Analyses. Borings 1-RLUT-93 and 1-RU-86 through 7-RU-86 were analyzed at different drawdown limits to estimate settlement due to the temporary increase in effective stress from lowering the lake levels. The boring logs are presented on Plates III-18 through III-26. The table below summarizes the estimated settlement for a full lake drawdown from elevation 282.2 to elevation 274.2 feet NGVD for each of the borings. Data from nine consolidation tests performed for selected strata were used in the analyses. The results of the consolidation tests are presented in Appendix III-A, Plates A-72 through A-140.

Settlement Summary – Eight Foot Drawdown
(El. 282.2 to 274.2 in inches)

Boring	Existing Condition	Raising Ground Surface to Elevation 282.2
1-RLUT-93	1.97	NA
1-RU-86	5.04	10.67
2-RU-86	0.78	1.33
3-RU-86	1.39	1.99
4-RU-86	0.07	0.07
5-RU-86	0.28	0.46
6-RU-86	0.98	1.57
7-RU-86	2.03	NA

NA – Not applicable since the ground surface from borings exceeds El. 282.2.

b. Estimated Settlement. (1). Lake Bottom. As the table above indicates, approximately 2-inches of settlement will be expected for a full lake drawdown. The analyses indicate (except for Boring 1-RU-86) that the temporary effective stress will be less than the existing preconsolidation stress. When the effective stress is decreased by refilling Reelfoot Lake to normal pool elevations, the rebound for all practical purposes will be equal to the amount of temporary settlement. Therefore, the net settlement of the exposed lake bottom during a drawdown is controlled almost entirely by evaporation. For a full eight foot drawdown, the average amount of net settlement was estimated to be 3.5-inches for the exposed lake bottom. For subsequent drawdowns, the increase in net settlement will be minimum (less than 10 percent).

(2). Shore Structures. Borings 1-RLUT-93 and 7-RU-86 are in areas considered to representative of conditions where houses and similar structures are located. Maximum projected settlement for a full drawdown is estimated to be approximately 2 inches for structures within 1000 feet from the shoreline. Again, the ground surface will rebound to its original elevation when the lake level rises back to normal operating pool elevations. Damage to structures will be limited to buildings and houses with rigid masonry walls and concrete floor slabs. The damage will consist mainly of cracking . Wood structures are more flexible in nature resulting in only minor damage predicted during lake drawdowns.

(3). Conclusion. Although the alternate spillway will have capability of a maximum drawdown to elevation 274 feet NGVD, TWRA has recently stated that the lake's surface area should remain above a minimum 5000 acres. Projected surface lake areas for different drawdown pools indicate that a 278 elevation would reduce the surface area of the lake to approximately 5000 surface acres. A full drawdown to elevation 274 feet NGVD would further reduce the lake level to less than 2000 surface acres. If the 5000 surface acre minimum is adopted, then elevation 278 would be the minimum accepted pool drawdown elevation. The lake was drawn down to approximate elevation 278 feet NGVD in May-July 1985. Cracking of some brick structures and asphalt roads occurred during the drawdown period. Since the lake level has previously been drawn to elevation 278 feet NGVD, future drawdowns would limit consolidation of the lake bottom to mainly organic material and the estimated settlement of shore structures would be in the range of 1 inch.

3-11 Sediment Retention Basin

a. General. The proposed sediment retention basin will be located at the confluence of where Reelfoot Creek flows from the uplands into the Reelfoot Lake area northeast of the existing spillway site as shown on the general vicinity map on Plate III-1. The sediment retention basin will consist of an earthen embankment approximately 11,000 feet in length with a minimum 20 foot crown width and 1V:4H side slopes. The 1V:4H side slopes were chosen to insure adequate slope stability and provide reasonable slopes for maintenance purposes. An aerial view of the proposed embankment along with the pertinent data is presented as Plate III-57. Five separate control features will be constructed into the embankment to control discharge water into Reelfoot Lake after rain events: three low level outlet drainage structures, a primary spillway and an emergency spillway. Due to inability to obtain rights of entry, only three borings were taken at this proposed project site as shown on Plate III-57 (Borings 1-SBU-97, 2-SBU-97 and 3-SBU-97). Boring 1-SBU-97 was taken in reasonable proximity of the proposed Low Level Outlet Structure No. 2 location. The other two borings were not drilled at or near the other proposed drainage sites because of limited accessibility and denial of right of entry. Therefore, the majority of the design for the sediment retention basin presented in this report is based on limited subsurface information. At least five additional borings will be required to check assumed soil conditions and design parameters at both spillway sites and at Low Level Outlet Structure sites No. 1 and No. 3. Sufficient borrow pit borings will also be taken to determine suitability of material when the pit layout has been determined upstream of the proposed embankment. After obtaining the additional subsurface data, additional analyses (if necessary) will be completed before development of plans and specifications prior to construction.

b. Embankment Settlement. The embankment will be constructed of semi-compacted clay material obtained from borrow pits located within the basin area. Based on natural ground elevations and an embankment crown elevation of 308 feet NGVD, most of the embankment fill will range in height from 10 to 21 feet. Settlement analyses were performed using stratification from each of the three borings taken for this project. The analyses resulted in an estimated range of settlement varying from 8-inches to 2 feet depending on fill height and whether C_c or C_r values were used in the analyses. A typical embankment section representing a maximum fill of 21 feet is shown on Plate III-58 along with results from settlement analyses using stratification from Boring 1-SBU-97. The estimated settlement was 15 inches. The maximum height of fill will occur through the reach at the proposed primary and emergency spillway locations. Additional borings will be taken at the spillway sites and at the other proposed outlet structure sites to estimate settlement and time required for settlement completion. The proposed construction sequence will be to construct the primary and emergency spillway embankments as the first order of work to pre load the sites. Based on assumed conditions using Boring 1-SBU-97, pre loading the embankment site for approximately 6 months will result in around 95% completion of the estimated settlement. The next item of work will be construction of the three low level outlet structures and outlet channels. Once the culverts are operational, the contractor can proceed with the construction of the project.

c. Low Level Outlet Structures. (1). General. Each of the three low level outlet structures consist of two six foot diameter concrete pipes that extend through the embankment with inlet and outlet structures as shown on Plate III-59. These structures will drain storm water impeded by the embankment for all events with headwaters less than elevation 302.2 feet NGVD or the crest elevation of the primary spillway. The inlet consists of a box culvert with five two foot diameter holes placed on the side of the box culvert with the top open (11 feet by 20 feet at elevation 301 feet NGVD) as shown on Plate III-59. As the basin fills with sediment, then the next adjacent hole will provide drainage into the box culvert. Eventually the culvert sides (11 feet in height) will be completely filled with sediment with all flow entering through the top of the culvert unless the sediment is periodically removed from around the culvert walls and inlet basin. The outlet structure consists of a preformed concrete stilling basin structure (top of slab elevation of 286.8 feet NGVD) with energy dissipaters and concrete walls as shown on Plate III-59. The outlet channel will have a bottom grade of elevation of 289 feet NGVD and extend approximately 1000 feet downstream from each structure with a 30 foot bottom width channel. Minimum slopes of 1V:3H will be required for the outlet channel slopes. Eighty linear feet of 36-inch R650 riprap will be required downstream of the stilling basin structure from top bank to top bank. An additional 40 feet of 24-inch R200 riprap will be placed downstream of the R650 on both slopes and extend 20 feet into the channel. The riprap will be placed on 6-inches of bedding gravel and be placed in the dry.

(2). Settlement of Culvert Pipes. Natural ground at the proposed low level structure sites vary from elevations 290 to 298 feet NGVD. This will result in a varying embankment fill between 10 to 18 feet for the culverts. Settlement analyses were performed using stratification from borings 1-SBU-97 and 2-SBU-97. The results for the site at Low Level Structure No. 2 are presented on Plate III-60 using the data from Boring 1-SBU-97. Based on the projected settlement, the two concrete pipes will be constructed from the inlet to the center of the embankment with an invert elevation of 290 feet NGVD and sloped from the center of the embankment to elevation of 288.8 feet NGVD at the stilling basin invert. This will allow for positive flow in the pipes after foundation settlement has occurred. Additional borings will be taken at the two other culvert sites (No. 1 and No. 3) to estimate the magnitude of settlement for each site. If excessive settlement is predicted at these sites, then pre loading the sites may be required since the invert elevations have been established for the hydraulic assumptions for the sediment retention embankment performance.

(3). Construction Considerations. Each low level outlet structure will require excavation to a minimum elevation of 285.5 feet NGVD for the stilling basin slab. Natural ground varies at the proposed sites between elevations 290 – 298 feet NGVD. Borings 1-SBU-97 indicates excavation into a soft wet clay material with a water table reading of elevation 282 feet NGVD at the time the boring was taken. Dewatering will not be required for construction, but undercutting the foundation grade and constructing a grout pad may be necessary to meet compaction requirements for placement of material under and around the concrete pipes. Compaction to 95 % standard density will be required for the clay material in the immediate vicinity of the concrete pipes. Unwatering of the site will also be required to control bleeding of water from the excavation slopes. A minimum excavation slope of 1V:2.5H will be required for all excavations. No under drainage system will be necessary for the low level outlet structure stilling basins due to the depth of overburden material above the sand aquifer. Since these culverts are being constructed below natural ground, the tendency will be for the structures to accumulate silt at lower flows. A substantially large sump area at elevation 291 feet shall be excavated with the material used as fill in construction of the sediment retention embankment.

d. Primary Spillway. (1). General. The proposed primary spillway will be located on the northern end of the sediment retention basin embankment as shown on Plate III-57. The spillway will consist of a reinforced concrete structure of 135 feet in width and a crest elevation of 302.2 feet NGVD as shown on Plates III-61 through III-64. The primary spillway will operate initially on a 2-year frequency that will increase to a 1.05-year frequency after the basin has been in operation for 50 years. The spillway is designed as a U frame type structure with a 1V:1H downstream slope with a crest and stilling basin design as shown in plan, section, elevation and isometric views on Plate III-62. Since the massive concrete structure will be withholding only 8 feet of hydrostatic pressure, no overturning, sliding, and bearing capacity analyses were performed for the structure. However, settlement of the structure and embankment is a concern and will need to be held within tolerable design limits when the structure is constructed. Four reinforced concrete walls (two upstream and two downstream) will be attached to the U frame structure as shown on the plates to retain the flatter embankment slopes and channel the water into the stilling basin and outlet channel during periods of flow. These proposed walls are identical in nature with similar loading conditions between the upstream and downstream walls. The outlet channel consists of a 145 foot bottom width channel initially that necks down to an 80 foot bottom width channel 400 feet downstream of the stilling basin and to a 20 foot bottom width channel at 800 feet downstream of the structure. The 20 foot bottom width channel extends to a total length of 1350 feet downstream of the stilling basin.

(2). Settlement. The average fill height of the proposed embankment at the spillway locations varies between 15 and 21 feet from natural ground resulting in an estimated settlement range of 15 to 20 inches. To compensate for this settlement, the embankment for the primary spillway and emergency spillway will be constructed as the first item of work for the sediment retention basin project. Both spillway sites will be surcharged with earth fill to elevation 308 feet NGVD. The fill shall extend from 300 feet south of the primary spillway to 300 feet north of the emergency spillway. After settlement has subsided or approximately 6 months in time, the embankment will be excavated to the required foundation grade for construction of the spillways as shown on the plates. The embankment fill material when placed will be compacted to 95 % standard density from natural ground to a foot above the required foundation grades. The excavated material used for surcharging the site shall be placed either upstream or downstream of the spillway structures into the embankment.

(3). Primary Spillway Retaining Walls. The proposed retaining wall (four separate walls) for the primary spillway is shown in plan, isometric, section and elevation views on Plate III-63. Stability analyses were performed for overturning, sliding and bearing capacity for the applicable loading cases as presented on Plate III-64. The factors of safety are also presented in the summary table. The results of the analyses indicate adequate factors of safety meeting minimum Corps design criteria for earthen retaining walls. Although the base footing dimensions and wall loading decreases from one end of the wall to the other, a two dimensional analysis was performed at the wall centroid (including soil weight) for each of the cases analyzed since the wall will be constructed as one monolithic structure. The detailed analyses for Case 1 or the after construction case without earthquake is presented on Plate III-65.

(4). Riprap Protection. Riprap will be required to protect the outlet channel downstream of the primary spillway. The riprap coverage will begin at the stilling basin and extend 100 feet along the outlet channel. The first 65 linear feet of riprap protection will consist of a 36-inch thick R650 blanket placed from top bank to top bank. The next 35 linear feet of protection will consist of a 24-inch thick R650 blanket placed along both side

slopes, extending 20 feet from the toe into the channel. Standard end, overbank and toe protection details will be incorporated into the design. The riprap will be placed on 6-inches of bedding gravel and will be constructed in the dry.

e. Emergency Spillway. (1). General. The proposed emergency spillway structure will be located 100 feet north of the primary spillway as shown on Plate III-57. The structure will consist of an 800-foot wide opening in the embankment with a crest elevation of 303.9 feet NGVD. The structure will be recessed into the embankment with 1V:4H side slopes to eliminate the need for walls and insure that the flow remains within the structure limits. The downstream spillway slope will be the same as the sediment retention embankment or 1V:4H. A plan and profile view of the structure is presented on Plate III-66. The emergency spillway will operate initially on a 10-year frequency that will reduce to a 5-year frequency after 50 years as the basin fills with sediment. Fifty linear feet of 24-inch riprap will extend downstream of the structure to reduce the velocities and prevent scour adjacent to the embankment. There will not be a defined or improved outlet ditch below the limits of the riprap protection as the flow will spread across natural ground into the Reefoot Lake Basin. The structure will consist of 8-inch thick interlocking concrete blocks or equivalent placed on filter fabric. The block system will be capable of withstanding a maximum velocity of 27 fps for the duration of a project flood event.

(2). Design of the Emergency Spillway. As stated above in paragraph 3.11d(2), the site area will be surcharged with the earthen embankment material for at least 6 months. The embankment surcharge will consist of approximately four feet of fill placed to elevation 308 feet NGVD. All embankment material placed between natural ground to one foot above the crown elevation of the spillway structure will be compacted to 95% standard density and consist of a clay material.

(3). Riprap Protection . Riprap will be required to protect the outlet channel downstream of the emergency spillway. The riprap coverage will begin at the end limits of the interlocking concrete blocks and extend 75 feet along the outlet channel defined by earthen berms along both sides of the channel to contain the basin flow as shown on Plate III-66. The top of the berms will be elevation 294 feet NGVD to contain a predicted maximum water surface elevation 292.6 feet NGVD in the outlet channel. The first 50 linear feet of riprap protection will consist of a 36-inch thick R650 blanket placed from top bank to top bank. The next 25 linear feet of protection will consist of a 24-inch thick R650 blanket placed along both side slopes, extending 20 feet from the toe into the channel. Standard end, overbank and toe protection details will be incorporated into the design. The riprap will be placed on 6-inches of bedding gravel and will be constructed in the dry.

3-12 Lake Isom Refuge

a. General. The southern extreme and existing Lake Isom embankment are located approximately 5 miles south of the existing spillway as shown on the vicinity map on

Plate III-1. Lake Isom consists of an earthen embankment approximately 1400 feet in length with a crown elevation of 283 feet NGVD. Included in this embankment are a 10 foot wide concrete control structure with wood stop logs and a 150 foot wide spillway at elevation 280.5 feet NGVD as shown in aerial view on Plate III-67. The proposed plan of improvement is to increase the water level management capabilities in the refuge including the ability to draw the water surface level to elevation 274 feet NGVD and to raise the lake level to a maximum elevation of 282 feet NGVD. To accommodate these proposed features, the embankment will require enlargement to a height of elevation 285 feet NGVD, the spillway to elevation 282.5 feet NGVD and the control structure will need replacing.

b.Embankment. The proposed embankment will consist of a downstream earthen enlargement with 1V:3H side slopes and 15 foot crown width as shown on Section A-A on Plate III-68. The enlargement material will consist of a semi-compacted clay material from the proposed borrow pit location as shown on Plate III-67. The borrow pit will be non uniform in shape with a maximum cut of four feet below natural ground. A connector ditch to the existing ditch east of the pit location will be included with the project. The proposed spillway will require earthen fill to elevation 282.5 feet NGVD with a gradual slope as shown on Section B-B on Plate III-68. A compacted gravel road of 6 inches in thickness will be constructed along the top of the embankment. The road foundation will also be compacted to two feet below the gravel road to 95 % standard density.

c. Replacement Outlet Structure. The proposed replacement outlet control structure is presented in plan and sectional view on Plate III-69. The structure will consist of a two bay (each bay 7.5 feet in width) reinforced concrete gravity structure with wood stop logs for controlling the lake levels. Overturning, sliding and bearing capacity analyses were performed for the structure with the results presented on Plate III-69. Boring 1-LIT-95 was taken at the existing structure site and the proposed site of the replacement structure and was used for stratification and shear strengths in the analyses. A clay foundation assuming conservative Q and S strengths was chosen for the analyses. A hydrostatic water pressure of elevation 282.5 feet NGVD or the spillway crest elevation was used for the driving force to cause movement. As shown by the results on Plate III-69, the structure exceeds the minimum required factors of safety and resultant force locations for a safe structure. The excavation slopes will be a minimum 1V:2.5H for both removal of the old structure and replacement of the new structure. A small earthen cofferdam constructed to elevation 283 will be required upstream of the replacement structure. The cofferdam will require a 5 foot minimum width and 1V:2.5H side slopes. A riprap stilling basin will be design downstream of the structure to dissipate the energy during flow and prevent scouring of the outlet ditch.

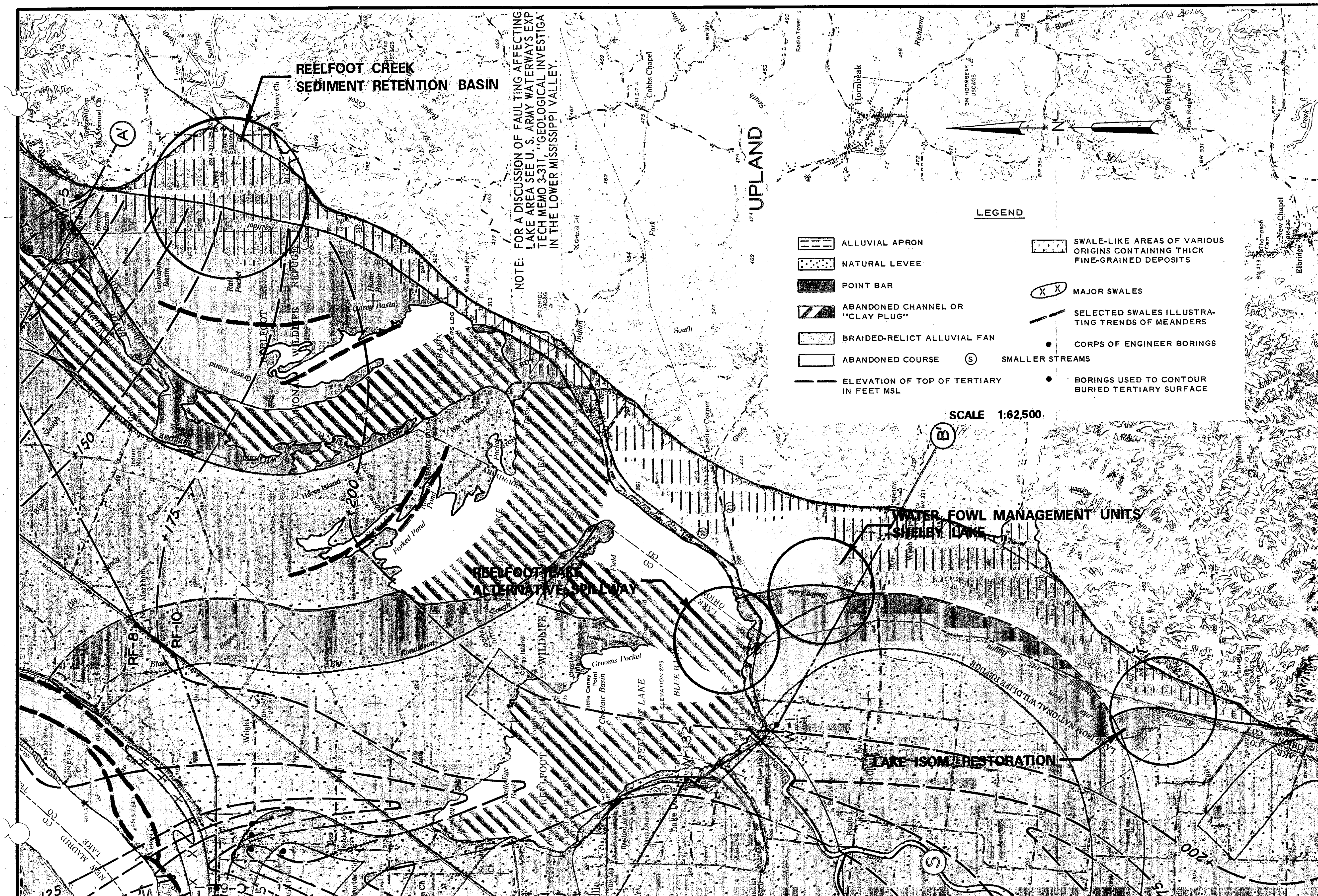
d. Riprap Protection. Riprap will be required to protect the outlet channel downstream of the replacement structure. The riprap coverage will begin at the stilling basin and extend 75 feet along the outlet channel. The first 50 linear feet of riprap protection will consist of a 36-inch thick R650 blanket placed from top bank to top bank. The next 25 linear feet of protection will consist of a 24-inch thick R650 blanket placed along both side slopes, extending 20 feet from the toe into the channel. Standard end, overbank and toe protection details will be incorporated into the design. The riprap will be placed on 6-inches of bedding gravel and will be constructed in the dry.

3-13 Excavation of Circulation Channels

Excavated circulation channels are proposed for connecting the three major basins of Reelfoot Lake. The majority of the proposed channels will be through the areas of land that separate Blue, Buck and Upper Blue Basin. Plate III-70 shows the proposed circulation channel layout. Maximum channel cut will be approximately 3 feet. A channel bottom width of 30 feet with 1V:4H side slopes and a target channel bottom grade of 276 feet NGVD is proposed. Removal of an additional couple of feet of material below the channel grade is recommended as the soft and wet excavated material will tend to slide back into the excavation. The excavated material will be placed or spread along the proposed channel layout. Excavation will most likely be accomplished using an amphibious hydraulic excavator or mechanical dredge.

3-14 Shelby Lake

Shelby Lake and the proposed waterfowl management units are located just south of Reelfoot Lake as shown on the vicinity map of Plate III-1. This feature of the project involves removal of sediment from existing Shelby Lake and construction of six waterfowl management unit areas as shown on the aerial photo on Plate III-70. Two hand auger borings were taken at approximate locations as shown on Plate III-72. The boring logs are presented as Plates III-16 and III-17. The proposed maximum depth of cut in Shelby Lake is around six feet. Minimum side slopes of 1V:3H will be required for the excavation. The water fowl units will be constructed by building levee type embankments of three to four feet in height with a proposed 18 foot crown width and 1V:4H side slopes. The material will come from within the units or from the excavated material from Shelby Lake. Drying of the material may be required since a proposed gravel road will be located on top of the embankments and compaction of the fill will be necessary for the road foundation. Water will be diverted from Reelfoot Lake using two 24-inch diameter pipes at the location as shown on Plates III-71 and III-72. Each culvert will have an outlet sluice gate for controlling the flow. The pipes will be jacked under the highway embankment.

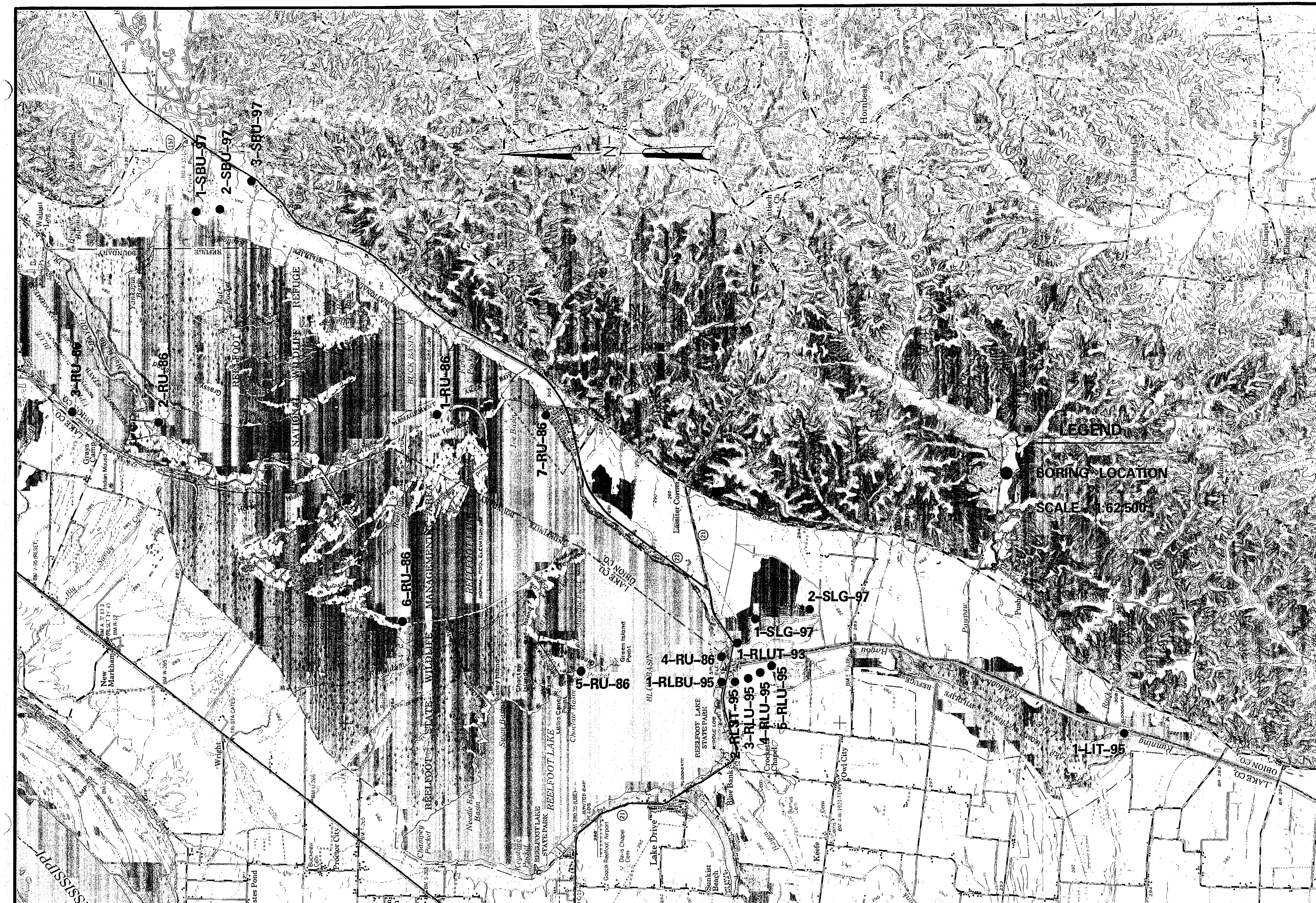


U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
MEMPHIS, TENNESSEE

ENVIRONMENTAL RESTORATION
MISSISSIPPI RIVER AND TRIBUTARIES

REELFOOT LAKE, TENNESSEE & KENTUCKY
GENERAL VICINITY & GEOLOGICAL MAP

DRAWING III-1



US Army Corps of Engineers Memphis District

U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
MEMPHIS, TENNESSEE

ENVIRONMENTAL RESTORATION NATURAL RESOURCES PROJECT

MISSISSIPPI RIVER AND TRIBUTARIES CONSTRUCTION

REELFOOT LAKE, TENNESSEE & KENTUCKY

BORINGS LOCATION MAP

DRAWING
III–2

UNIFIED SOIL CLASSIFICATION

MAJOR DIVISION			TYPE	LETTER SYMBOL	SYMBOL	TYPICAL NAMES
COARSE - GRAINED SOILS More than half of material is larger than no. 200 sieve size.	GRAVEL More than half of coarse fraction larger than no. 4 sieve size.		CLEAN GRAVEL (little or no fines)	GW		GRAVEL, Well graded, gravel-sand mixture, little or no fines
				GP		GRAVEL, Poorly graded, gravel-sand mixture, little or no fines
			GRAVEL w/FINES (Appr. amt. of fines)	GM		SILTY GRAVEL, gravel-sand-silt mixtures
				GC		CLAYEY GRAVEL, gravel-sand-clay mixtures
	SAND More than half of coarse fraction smaller than no. 4 sieve size.		CLEAN SAND (little or no fines)	SW		SAND, Well graded, gravelly sands
				SP		SAND, Poorly graded, gravelly sands
			SAND w/FINES (Appr. amt. of fines)	SM		SILTY SAND, sand - silt mixtures
				SC		CLAYEY SAND, sand - clay mixtures
FINE-GRAINED SOILS	More than half the material is smaller than no. 200 sieve size.		SILTS & CLAYS (Liquid Limit <50)	ML		SILT & very fine sand, silty or clayey fine sand or clayey silt with slight plasticity
				CL		LEAN CLAY; Sandy Clay; Silty Clay; of low to medium plasticity
				OL		ORGANIC SILTS, and organic silty clays of low plasticity
			SILTS & CLAYS (Liquid Limit >50)	MH		SILT, fine sandy or silty soil with high plasticity
				CH		FAT CLAY, inorganic clay of high plasticity
				OH		ORGANIC CLAYS, of medium to high plasticity, organic silts
HIGH ORGANIC SOIL				Pt		PEAT, and other highy organic soil
WOOD				WD		WOOD
MIXED SAMPLE				VM		Variable mixed silts, clays and sands
NO SAMPLE						

NOTE: Soils possessing characteristics of two groups are designated by combinations of group symbols.

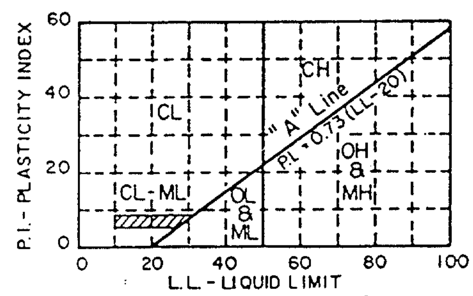
DESCRIPTIVE SYMBOL

COLOR

COLOR	SYMBOL
Tan	T
Yellow	Y
Red	R
Black	BK
Gray	Gr
Light Gray	lGr
Dark Gray	dGr
Brown	Br
Light Brown	lBr
Dark Brown	dBr
Brownish-Gray	brGr
Grayish-Brown	grBr
Greenish-Gray	gnGr
Grayish-Green	grGn
Green	Gn
Blue	Bl
Blue-Green	BlGn
White	Wh
Mottled	Mot

CONSISTENCY FOR COHESIVE SOILS

CONSISTENCY	COHESION IN LBS/SQ.FT. FROM UNCONFINED COMPRESSION TEST	SYMBOL
Very Soft	< 250	vSo
Soft	250-500	So
Medium	500-1000	M
Stiff	1000-2000	St
Very Stiff	2000-4000	vSt
Hard	> 4000	H



PLASTICITY CHART

For classification of fine grained soils

MODIFICATIONS

MODIFICATION	SYMBOL
Traces	Tr-
Fine	F
Medium	M
Coarse	C
Concretions	cc
Rootlets	rt
Lignite fragments	lg
Shale fragments	sh
Sandstone fragments	sds
Shell fragments	slf
Organic matter	O
Clay strata-lenses	CS
Silt strata-lenses	SIS
Sand strata-lenses	SS
Sandy	S
Gravelly	G
Boulders	B
Slickensides	SL
Wood	Wd
Oxidized	Ox
Saturated	sat
Lumps of clay	Clp
Laminated	lm

SAMPLER & SIDE SYMBOLS

JAR SAMPLE

7/6 9/6 STANDARD PENETRATION TEST. (140 lb. hammer dropped 30"). Values to right are resulting blows/inches.

UNDISTURBED THIN WALL SHELBY TUBE

CHANNEL GRADE

WATER LOCATION AT TIME INDICATED

TEST RESULTS

WATER % - Natural water content expressed as percentage of dry weight.

LIQUID LIMIT & PLASTIC LIMIT - Water content expressed as percentage of dry weight for described states.

COH & TORVANE - Cohesive values in lbs./sq. ft. as determined by unconfined compression and torvane tests, respectively.

DRY WT. - Dry weight in lbs./cubic ft.

SAND % - Sand content expressed as percentage of dry weight.

D_{10} SIZE - Grain diameter in millimeter for which 10 % of the soil is finer and 90 % coarser.

WES TEST - Denotes type & location of tests. Test results are available for inspection in U.S. Army Engineer District. Types are as follows:

- C - Location of consolidation test
- S - Location of consolidated - drained direct shear test
- R - Location of consolidated - undrained triaxial compression test
- Q - Location of unconsolidated - undrained triaxial compression test
- T - Location of sample subject to combination of above tests

GENERAL NOTES:

While the borings are representative of subsurface conditions at their respective locations and for their respective vertical reaches, local variations characteristic of subsurface materials of the region are anticipated and, if encountered, such variations will not be considered as differing materially within purview of the contract clause entitled, "Differing Site Conditions".

Ground - water elevations shown on boring logs represent ground-water surfaces encountered in such borings on the dates shown. Absence of water surface data on certain borings indicates that no ground-water data are available from the borings but does not necessarily mean that ground-water will not be encountered at the locations or within the vertical reaches of such borings.

Consistency of cohesive soils shown on the boring logs is based on driller's log and visual examination and is approximate, except within those vertical reaches of the borings where shear strengths from unconfined compression tests are shown.

STANDARD

BORING LEGEND

DEPARTMENT OF THE ARMY

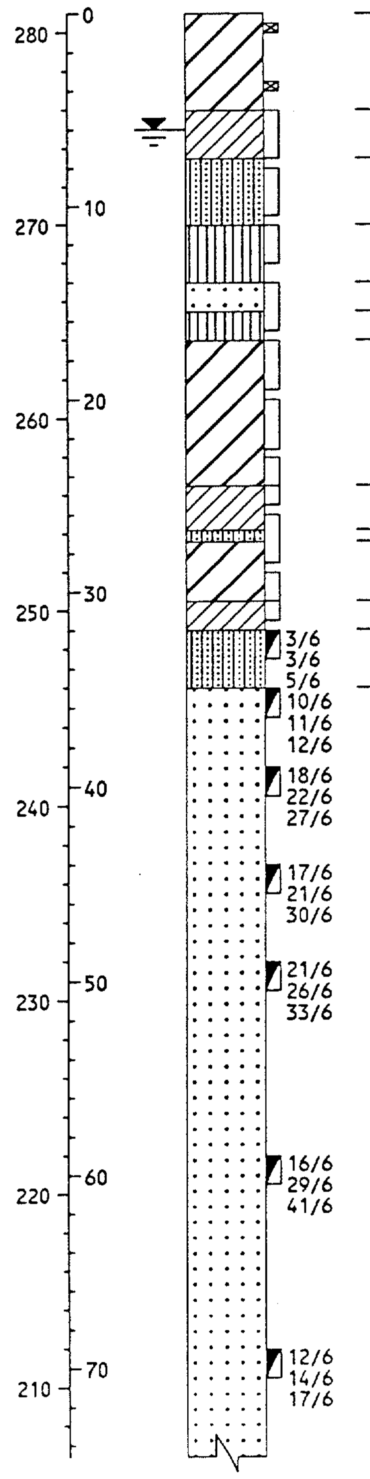
MEMPHIS DISTRICT CORPS OF ENGINEERS

BORING LOG

BORING 2-RLST-95

PROJECT: REELFOOT LAKE - ALTERNATIVE SPILLWAY
LOCATION:
LATITUDE: 36 21'04" N LONGITUDE: 89 24'31" W
DRILLED (BY) MEMPHIS DISTRICT (METHOD) AUG, 5" TUBE, SS (DATE) 7 - 9 AUGUST 1995
DEPTH TO WATER WAS 6.0 FEET ON 7 AUGUST 1995 GROUND ELEVATION: 280.96

ELEVATION / DEPTH	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA



FIELD TEST DATA	USCS	DESCRIPTION	WATER %	Liquid Limit	Plastic Limit	COH (psf)	Torvane (psf)	Dry Wt. (pcf)	Sand %	D 10 SIZE	WES TEST
	CH	Br, St									
		Br, M	29	65	20						
	CL	Br, Ox, SIS	27	40	20	524	840	90.9			
	SM	Gr, sat									
	ML	Gr, S, sat									
	SP	Gr, F									
	ML	(Pushover Sample)									
	CH	Gr, S, CS, sat	45	55	20	452	520	75.9			
		Gr, SIS, Wd									
		Gr, Wd	47	84	23	759	840	73.7			
		Gr	36	58	21	800		85.7			CQS
	CL	Gr	36	47	21	1079	800	85.6			
		Gr	45	49	21	392	500	76.2			
	SM	Gr	54	66	22	760	680	68.2			
	CH	Gr, Wd	38	45	22	680		83.9			CQS
		Gr	38	45	22	669	540	82.3			
	CL	Gr, SIS									
3/6 3/6 5/6	SM	Gr, sat (5" Tube Refused)									
10/6 11/6 12/6	SP	Gr, F, sat									
18/6 22/6 27/6		Gr, M, sat								0.220	
17/6 21/6 30/6		Gr, M, sat									
21/6 26/6 33/6		Gr, M, sat								0.218	
16/6 29/6 41/6		Gr, M, sat									
12/6 14/6 17/6		Gr, M, sat									

BORING LOG

BORING 2-RLST-95

PROJECT: REELFOOT LAKE - ALTERNATIVE SPILLWAY
LOCATION:
LATITUDE: 36 21'04" N LONGITUDE: 89 24'31" W
DRILLED (BY) MEMPHIS DISTRICT (METHOD) AUG, 5" TUBE, SS (DATE) 7 - 9 AUGUST 1995
DEPTH TO WATER WAS 6.0 FEET ON 7 AUGUST 1995 GROUND ELEVATION: 280.96

ELEVATION	DEPTH	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	DESCRIPTION	WATER %	Liquid Limit	Plastic Limit	COH (psf)	Torvane (psf)	Dry Wt. (pcf)	Sand %	D 10 SIZE	WES TEST
200	80	19/6 14/6 15/6		Gr, M, sat								0.190	
190	90	29/6 39/6 46/6		Gr, M, sat									
180	100	37/6 58/6		Gr, F, sat, Lignite								0.133	
170	110	20/6 29/6 34/6		Gr, M - F, sat								0.190	
160	120	50/6 36/6 41/6		Gr, M, sat								0.228	
150	130	74/6		Gr, F, sat								0.164	
140	140		GM	Gr, S, sat									

BORING LOG

BORING 2-RLST-95

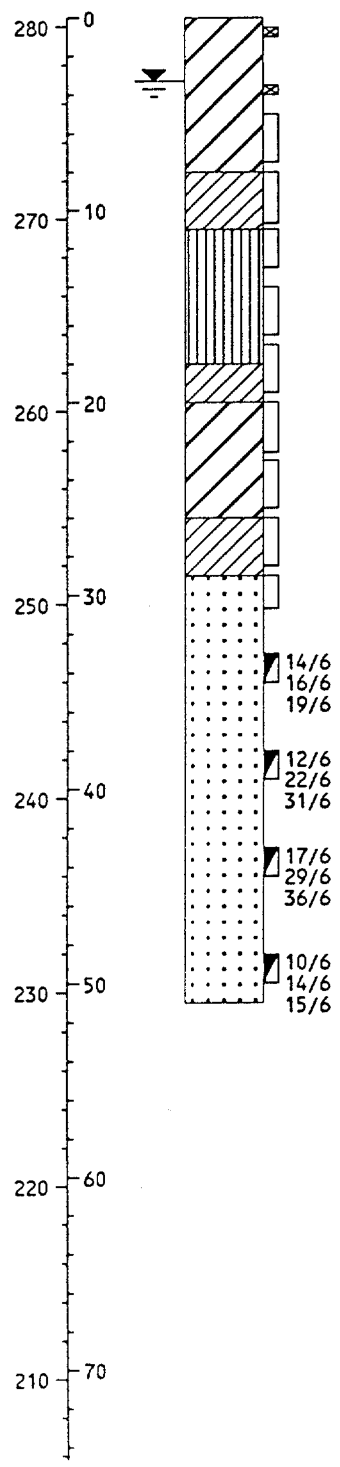
PROJECT: REELFOOT LAKE - ALTERNATIVE SPILLWAY
LOCATION:
LATITUDE: 36 21'04" N   LONGITUDE: 89 24'31" W
DRILLED (BY) MEMPHIS DISTRICT (METHOD) AUG, 5" TUBE, SS   (DATE) 7 - 9 AUGUST 1995
DEPTH TO WATER WAS 6.0 FEET ON 7 AUGUST 1995   GROUND ELEVATION: 280.96

ELEVATION / DEPTH	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	DESCRIPTION	WATER %	Liquid Limit	Plastic Limit	COH (psf)	Torvane (psf)	Dry Wt. (pcf)	Sand %	D 10 SIZE	WES TEST
130 / 150	72/6	SP	Gr, M, sat								0.165	
120 / 160	72/6		Gr, M, sat								0.187	
110 / 170			Sand & Gravel									
100 / 180												
90 / 190												
80 / 200												
70 / 210												
60 / 220												

BORING LOG

BORING 3-RLU-95

PROJECT: REELFOOT LAKE - ALTERNATIVE SPILLWAY
LOCATION:
LATITUDE: 36 20'57.5" N LONGITUDE: 89 24'30" W
DRILLED (BY) MEMPHIS DISTRICT (METHOD) AUG, 5" TUBE, SS (DATE) 23 AUGUST 1995
DEPTH TO WATER WAS 3.3 FEET ON 23 AUGUST 1995 GROUND ELEVATION: 280.46

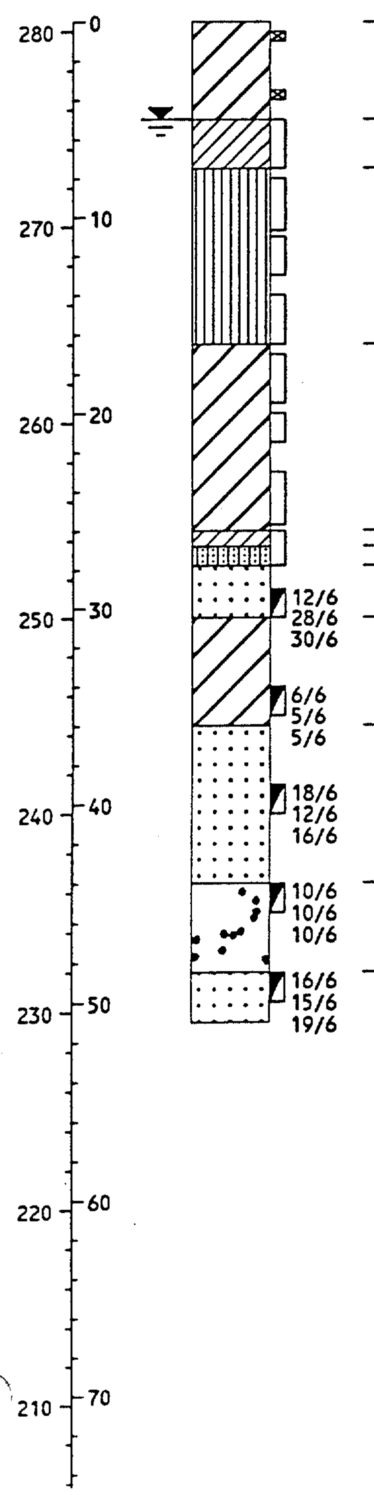


SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	DESCRIPTION	WATER %	Liquid Limit	Plastic Limit	COH (psf)	Torvane (psf)	Dry Wt. (pcf)	Sand %	D 10 SIZE	WES TEST
	CH	Br, S, M									
		Gr, M	28	41	18						
		Gr (Pushover)	38	68	21	451	1060	81.5			
	CL	Gr	27	39	20	1039	1540	96.3			
	ML	Gr, CS, SS	31								
		Gr, CS, SS	43								
		Gr, CS, SS	19								
	CL	Gr, SIS	37	40	23	599	400	86.4			
	CH	Gr, SIS, SS, rt	54								
		Gr, SIS, SS	36	59	24	1274	560	86.9			
	CL	Gr, SIS	40	45	20	719	480	83.1			
	SP	Gr, F, SIS								0.081	
14/6 16/6 19/6		Gr, M, sat								0.286	
12/6 22/6 31/6		Gr, M, sat									
17/6 29/6 36/6		Gr, M, sat									
10/6 14/6 15/6		Gr, G, sat								0.171	

BORING LOG

BORING 4-RLU-95

PROJECT: REELFOOT LAKE - ALTERNATIVE SPILLWAY
LOCATION:
LATITUDE: 36 20'49.1" N LONGITUDE: 89 24'23.7" W
DRILLED (BY) MEMPHIS DISTRICT (METHOD) AUG, 5" TUBE, SS (DATE) 22 AUGUST 1995
DEPTH TO WATER WAS 5.0 FEET ON 22 AUGUST 1995 GROUND ELEVATION: 280.48

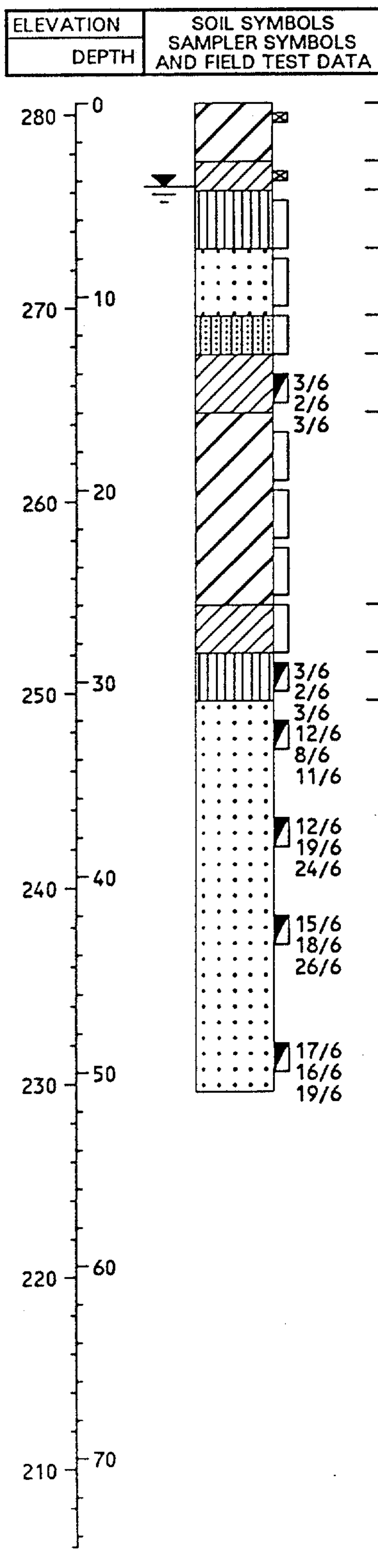


SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	DESCRIPTION	WATER %	Liquid Limit	Plastic Limit	COH (psf)	Torvane (psf)	Dry Wt. (pcf)	Sand %	D 10 SIZE	WES TEST
	CH	Br, M									
		Br, M	27	52	16						
	CL	Br, Ox	26	30	22	586	760	94.6			
	ML	Gr	31								
		Gr, SS	33								
		Gr	30								
	CH	Gr, SIS	48	72	24	619	780	75.0			
		Gr, SL	46	85	25	600		75.3			Q S
		Gr	47	71	21	624	660	76.4			
	CL	Gr, SIS	39	43	28	1006	460	83.6			
	SM	Gr, CS									
12/6 28/6 30/6	SP	Gr, F, sat									
	CH										
6/6 5/6 5/6		Gr, SS, M	59								
	SP										
18/6 12/6 16/6		Gr, F, CS, sat									
10/6 10/6 10/6	GP	Gr, S, sat								0.221	
16/6 15/6 19/6	SP	Gr, G, sat								0.248	

BORING LOG

BORING 5-RLU-95

PROJECT: REELFOOT LAKE - ALTERNATIVE SPILLWAY
LOCATION:
LATITUDE: 36 20'40.3" N LONGITUDE: 89 24'17.9" W
DRILLED (BY) MEMPHIS DISTRICT (METHOD) AUG, 5" TUBE, SS (DATE) 15, 17, 21 AUG 95
DEPTH TO WATER WAS 4.3 FEET ON 17 AUGUST 1995 GROUND ELEVATION: 280.59

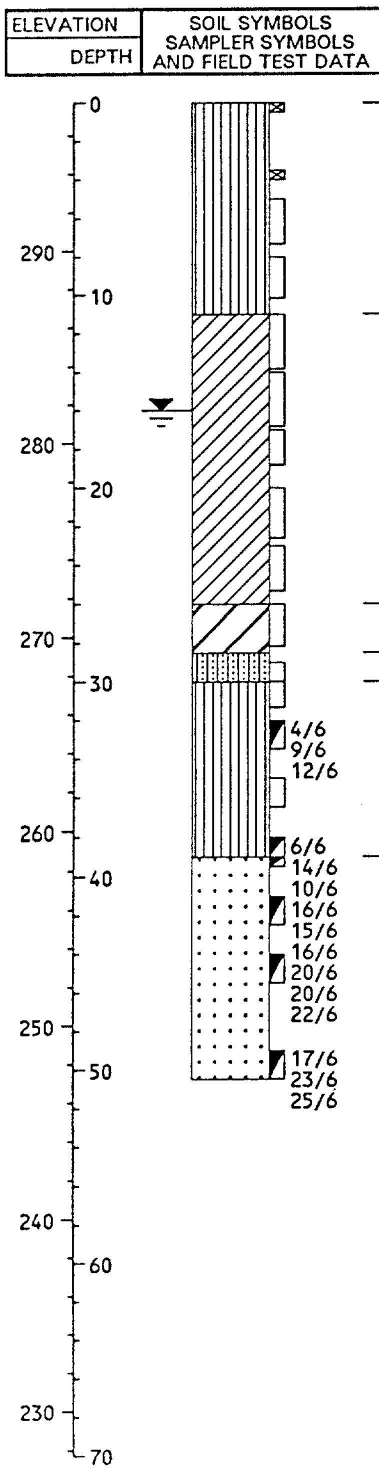


USCS	DESCRIPTION	WATER %	Liquid Limit	Plastic Limit	COH (psf)	Torvane (psf)	Dry Wt. (pcf)	Sand %	D 10 SIZE	WES TEST
CH	Br & Gr, M									
CL	Br & Gr, SS, M	28								
ML	Br, CS	30								
SP	Br, F, SIS									
SM	Gr									
CL	Gr, SS, M									
CH	Br	50	77	25	492	800	71.5			
	Gr	45	91	27	726	1100	76.8			
	Gr, SIS, SS	34	50	25	1337	1100	88.0			
CL	Gr, SIS	38	43	25	666	500	85.0			
ML	Gr, S, sat (Pushover)									
SP	Gr, F, sat								0.073	
	Br, F, sat								0.144	
	Br, F, sat									
	Br, F, sat									

BORING LOG

BORING 1-SBU-97

PROJECT: REELFOOT LAKE - SEDIMENT RETENTION BASIN
LOCATION:
LATITUDE: 36 26'31.4" N LONGITUDE: 89 18'49.2" W STATION:
DRILLED (BY) MEMPHIS DISTRICT (METHOD) AUG, 5" TUBE, SS (DATE) 3-4 SEPT 1997
DEPTH TO WATER WAS 16.0 FEET ON 3 SEPTEMBER 1997 GROUND ELEVATION: 297.7



USCS	DESCRIPTION	WATER %	Liquid Limit	Plastic Limit	COH (psf)	Torvane (psf)	Dry Wt. (pcf)	Sand %	D 10 SIZE	WES TEST
ML	Br									
	Br	29								
	Gr,CS	30								
	Gr,CS,O	39								
CL	Gr,O	43	40	22	500	660	76.6			
	Gr	34	37	19	248	380	89.1			
	Gr	32	33	19						
	Gr	31	39	19	531	740	92.2			
	Br&Gr	34	42	21	654	760	88.5			
CH	Br&Gr,SIS	37	67	22	887	820	84.6			
SM	Br									
ML	Br&Gr,Ox	26								
	Gr	28								
	Gr,G	24								
	Gr	24								
SP	Gr,M,G,CS									
	Gr,M,G,CS									
	Gr,M								0.160	
	Gr,F								0.998	

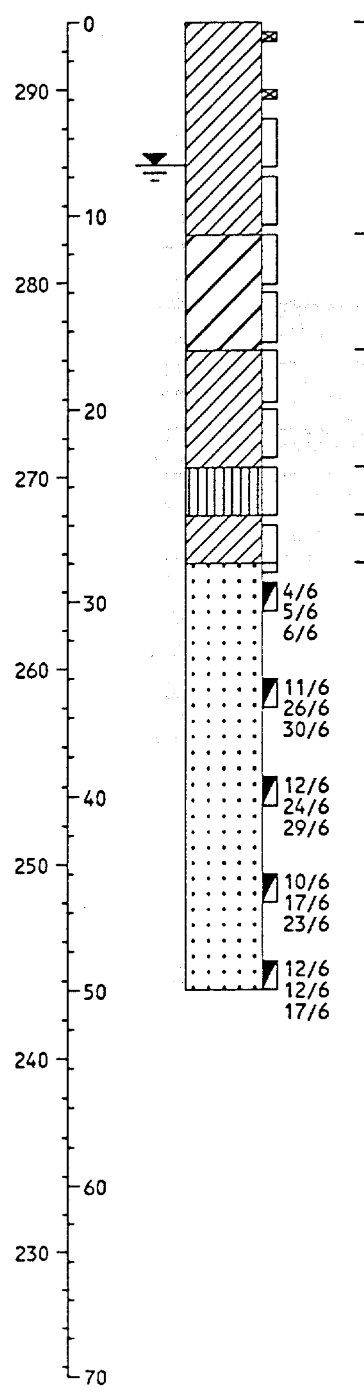
PLATE III-11

BORING LOG

BORING 2-SBU-97

PROJECT: REELFOOT LAKE - SEDIMENT RETENTION BASIN
LOCATION:
LATITUDE: 36 26'16.1" N LONGITUDE: 89 18'44.1" W STATION:
DRILLED (BY) MEMPHIS DISTRICT (METHOD) AUG, 5" TUBE, SS (DATE) 10-11 SEPT 1997
DEPTH TO WATER WAS 7.4 FEET ON 11 SEPTEMBER 1997 GROUND ELEVATION: 293.49

ELEVATION / DEPTH — SOIL SYMBOLS, SAMPLER SYMBOLS AND FIELD TEST DATA

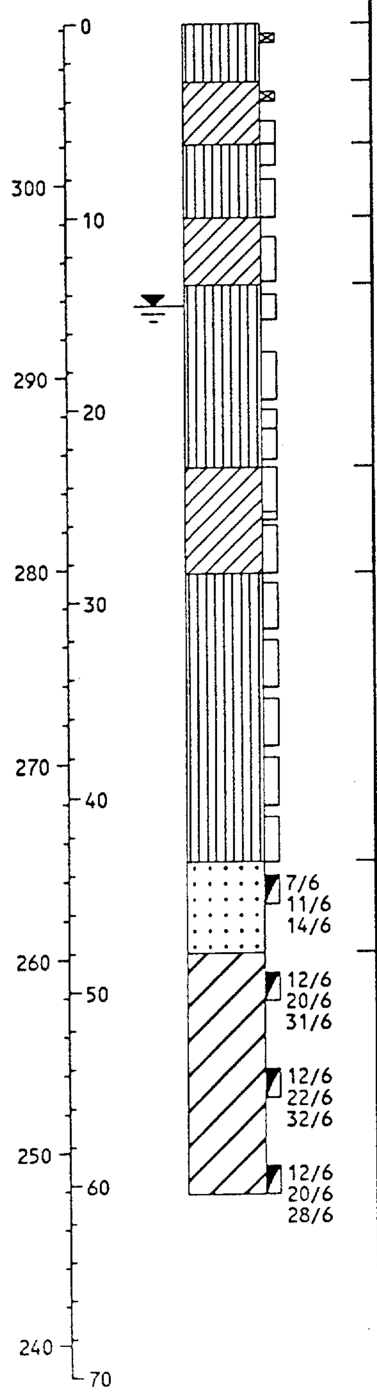


USCS	DESCRIPTION	WATER %	Liquid Limit	Plastic Limit	COH (psf)	Torvane (psf)	Dry Wt. (pcf)	Sand %	D 10 SIZE	WES TEST
CL	Br,M									
	Br,M	19	43	18						
	Br,Ox	22	49	18	2162	MAX.	102.4			
	Br,Ox	25	29	18	676	900	99.1			
CH	Gr,OX	33	74	19	1870	MAX.	89.1			
	Gr	31	69	20	966	1640	93.3			
CL	Br	28	37	21	911	800	97.1			
	Br&Gr,Ox	28	40	19	1178	1100	96.6			
ML	Br	30								
CL	Br,Ox	27	33	18	1063	1020	104.7			
SP	Br,F Br,F to M								0.171	
	Br,M								0.179	
	Br,M									
	Br,M									
	Gr,S,G,CS									

BORING LOG

BORING 3-SBU-97

PROJECT: REELFOOT LAKE - SEDIMENT RETENTION BASIN
LOCATION:
LATITUDE: 36 25'58.8" N LONGITUDE: 89 18'18.3" W STATION:
DRILLED (BY) MEMPHIS DISTRICT (METHOD) AUG, 5" TUBE, SS (DATE) 8-10 SEPT 1997
DEPTH TO WATER WAS 14.6 FEET ON 9 SEPTEMBER 1997 GROUND ELEVATION: 308.3



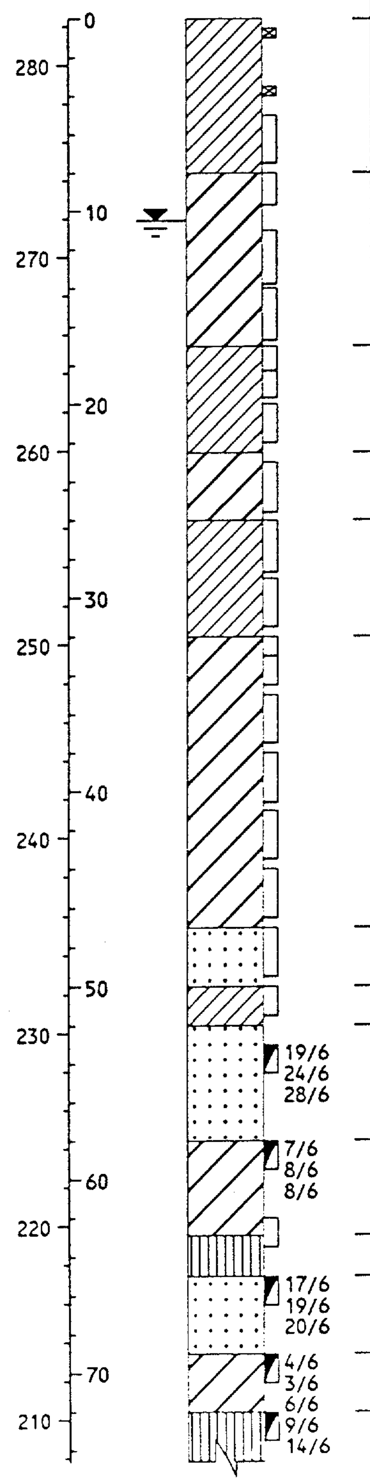
Field Test Data	USCS	DESCRIPTION	WATER %	Liquid Limit	Plastic Limit	COH (psf)	Torvane (psf)	Dry Wt. (pcf)	Sand %	D 10 SIZE	WES TEST
	ML	Br									
	CL	Br,M	24	33	21						
		Br,Ox	27	40	20	1096	1220	98.0			
	ML	Br	28								
		Br									
	CL	Br	29	41	19	1606	700	95.6			
	ML	Br	28								
		Br,CS	29								
		Br,CS	30								
		Br	30								
	CL	Br	30	34	23	974	860	94.5			
		Gr	30	48	22	1233	860	94.2			
		Gr	28	46	19	846	660	96.9			
	ML	Br&Gr	31								
		Gr	28								
		Gr	30								
		Gr	36								
		Gr	36								
7/6 11/6 14/6	SP	Gr,F								0.091	
12/6 20/6 31/6	CH	Gr,H,SIS	24								
12/6 22/6 32/6		Gr,SIS	32								
12/6 20/6 28/6		Gr,SIS	25								

PLATE III-13

BORING LOG

BORING 1-LIT-95

PROJECT: REELFOOT LAKE - LAKE ISOM
LOCATION: 12 FEET SOUTH OF CL LEVEE & 64 FEET WEST OF BM LI-GPS-1-95
LATITUDE: 36 17'07.7" N LONGITUDE: 89 25'09.5" W
DRILLED (BY) MEMPHIS DISTRICT (METHOD) AUG, 5" TUBE, SS (DATE) 24 & 28 AUG 1995
DEPTH TO WATER WAS 10.5 FEET ON 24 AUGUST 1995 GROUND ELEVATION: 282.4

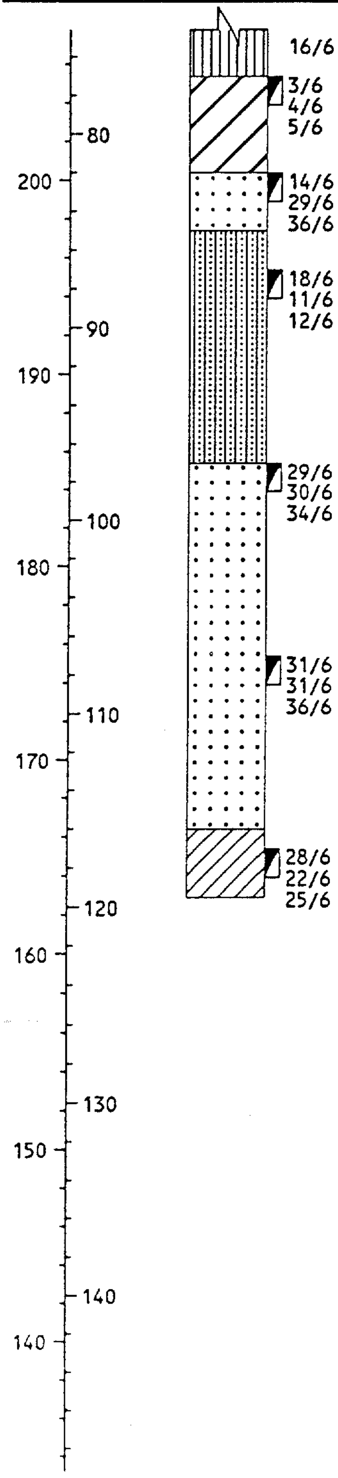


SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	DESCRIPTION	WATER %	Liquid Limit	Plastic Limit	COH (psf)	Torvane (psf)	Dry Wt. (pcf)	Sand %	D 10 SIZE	WES TEST
	CL	Br, M									
		Br, M	19	43	16						
		Br, SS, cc	25	49	18	1481	1480	100.4			
	CH	Br	31	60	21	1537	1680	93.0			
		Br, Ox	39	80	25	1072	1500	83.4			
		Br & Gr, Ox	41	74	26	774	1320	82.0			
	CL	Gr, Ox	33	37	19	500		90.7			CQS
		Br & Gr, Ox	34	36	24	497	600	90.2			
		Gr, SIS	33	38	18	425	820	90.6			
	CH	Gr, SIS	43	63	17	773	960	80.4			
	CL	Gr, SIS	35	36	25	725	520	90.9			
		Gr, SIS	25	39	26	779	660	88.6			
	CH	Gr, Ox	51	68	22	540		71.8			CQS
		Gr, SIS	45	61	27	878	600	78.1			
		Gr, SIS	51	61	19	753	560	70.9			
		Gr, SIS	40	56	26	900	760	81.4			
		Gr, SIS	50	64	20	696	600	72.1			
		Gr, SIS	47	69	18	725	580	73.8			
	SP	Gr, F, SIS, CS									
	CL	Gr, SIS	34	35	22	633		86.9			
19/6 24/6 28/6	SP	Gr, M, sat								0.209	
7/6 8/6 8/6	CH	Gr, SIS, So	35								
		Gr (Tube Refused)	38	62	18						
	ML	Gr	34								
17/6 19/6 20/6	SP	Gr, F, sat									
4/6 3/6 6/6	CH	Gr, SIS, So	38								
9/6 14/6	ML	Gr, CS	38								

BORING LOG

BORING 1-LIT-95

PROJECT: REELFOOT LAKE - LAKE ISOM
LOCATION: 12 FEET SOUTH OF CL LEVEE & 64 FEET WEST OF BM LI-GPS-1-95
LATITUDE: 36 17'07.7" N LONGITUDE: 89 25'09.5" W
DRILLED (BY) MEMPHIS DISTRICT (METHOD) AUG, 5" TUBE, SS (DATE) 24 & 28 AUG 1995
DEPTH TO WATER WAS 10.5 FEET ON 24 AUGUST 1995 GROUND ELEVATION: 282.4

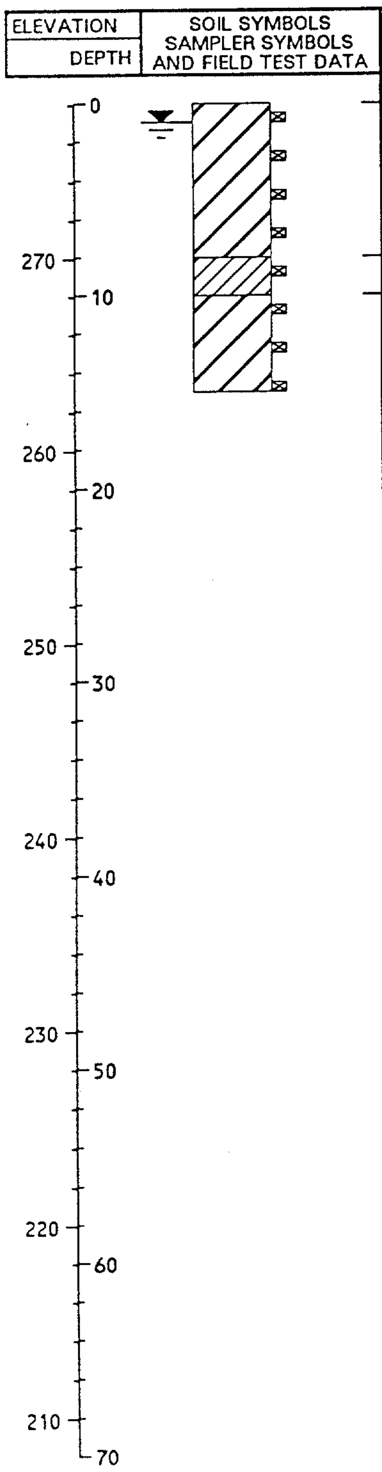


SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	DESCRIPTION	WATER %	Liquid Limit	Plastic Limit	COH (psf)	Torvane (psf)	Dry Wt. (pcf)	Sand %	D 10 SIZE	WES TEST
16/6											
3/6 4/6 5/6	CH	Gr, SIS, So	38								
14/6 29/6 36/6	SP	Gr, F, CS									
18/6 11/6 12/6	SM	Gr, sat							73		
29/6 30/6 34/6	SP	Gr, F, sat								0.130	
31/6 31/6 36/6		Gr, F, sat									
28/6 22/6 25/6	CL	Gr, S	38								

BORING LOG

BORING 1-SLG-97

PROJECT: REELFOOT LAKE - SHELBY LAKE
LOCATION:
LATITUDE: 36 20'36.6" N   LONGITUDE: 89 23'37.0" W   STATION:
DRILLED (BY) MEMPHIS DISTRICT (METHOD) HAND AUGER   (DATE) 16 SEPTEMBER 1997
DEPTH TO WATER WAS 1.0 FEET ON 16 SEPTEMBER 1997   GROUND ELEVATION: 278.08



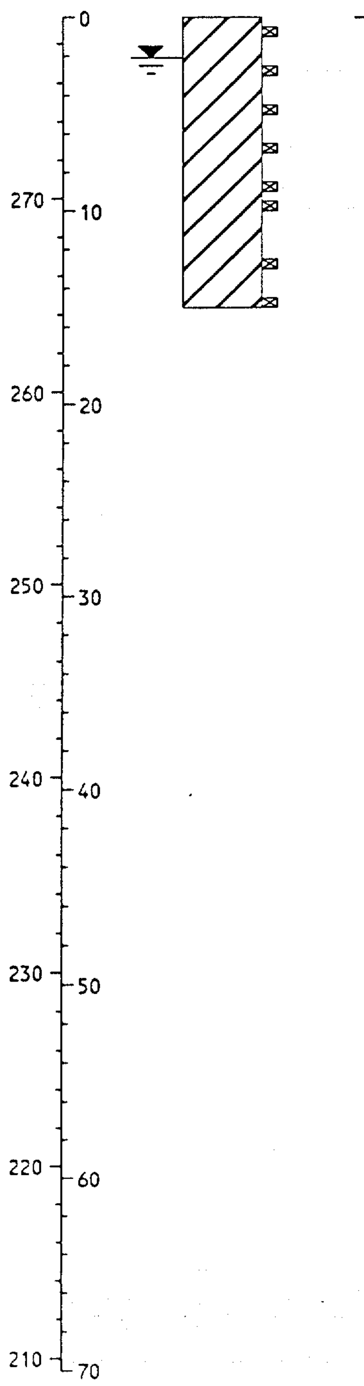
USCS	DESCRIPTION	WATER %	Liquid Limit	Plastic Limit	COH (psf)	Torvane (psf)	Dry Wt. (pcf)	Sand %	D 10 SIZE	WES TEST
CH	Br,M									
	Gr,M	52	92	26						
	Gr,M	39	81	25						
	Gr,M	38								
CL	Gr,So	44	49	23						
CH	Gr,M	36	74	25						
	Gr,M	42								
	Gr,M	44								

PLATE III-16

BORING LOG

BORING 2-SLG-97

PROJECT: REELFOOT LAKE - SHELBY LAKE
LOCATION:
LATITUDE: LONGITUDE: STATION:
DRILLED (BY) MEMPHIS DISTRICT (METHOD) HAND AUGER (DATE) 16 SEPTEMBER 1997
DEPTH TO WATER WAS 2.1 FEET ON 17 SEPTEMBER 1997 GROUND ELEVATION: 279.36



ELEVATION / DEPTH	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	DESCRIPTION	WATER %	Liquid Limit	Plastic Limit	COH (psf)	Torvane (psf)	Dry Wt. (pcf)	Sand %	D 10 SIZE	WES TEST
		CH	Br									
			Gr,M	51								
			Gr,M	32	67	20						
			Gr,M	55								
			Gr,M	49								
			Gr,M	54	88	24						
			Gr,M	43								
			Gr,M	40								

BORING LOG

BORING 1-RLUT-93

PROJECT: REELFOOT LAKE
LOCATION: SEE LOCATION MAP FOR BORING LOCATION
LATITUDE: LONGITUDE: STATION:
DRILLED (BY) MEMPHIS DISTRICT (METHOD) AUG, 5" TUBE, SS (DATE) 27-28 APRIL 1993
DEPTH TO WATER WAS 5.3 FEET ON 27 APRIL 1993 GROUND ELEVATION: 287.0

SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	DESCRIPTION	WATER %	Liquid Limit	Plastic Limit	COH (psf)	Torvane (psf)	Dry Wt. (pcf)	Sand %	D 10 SIZE	WES TEST
	CL	Br, M									
		Br, M	28	32	24						
	CH	Gr	50	89	34	377	1040	71.0			
	CL	Gr, SIS	24	34	19						
	CH	Gr	28	54	22	510	MAX.	93.7			
	ML	Gr	33								
3/6 3/6 4/6		Gr							80		
5/6 5/6 9/6	SM	Gr							70		
5/6 9/6 8/6	ML	Gr, S							40		
5/6 3/6 2/6	CH	Gr, M, SIS	41								
		Gr	54	77	28	694	1840	70.4			
	SP	Gr, F	40	71	25	1652	760	80.7			
	CH	Gr, SIS								0.139	
4/6 9/6 17/6		Br, F									
10/6 16/6 10/6		Br, F, CS									
17/6 21/6 23/6		Br, F								0.274	
13/6 22/6 35/6		Br, F									
17/6		Gr, F								0.119	

LAKE ELEVATION WAS 282.7 ON 27 APRIL 1993.

PLATE III-18

BORING LOG

BORING 1-RLUT-93

PROJECT: REELFOOT LAKE
LOCATION: SEE LOCATION MAP FOR BORING LOCATION
LATITUDE: LONGITUDE: STATION:
DRILLED (BY) MEMPHIS DISTRICT (METHOD) AUG, 5" TUBE, SS (DATE) 27-28 APRIL 1993
DEPTH TO WATER WAS 5.3 FEET ON 27 APRIL 1993 GROUND ELEVATION: 287.0

ELEVATION	DEPTH	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	DESCRIPTION	WATER %	Liquid Limit	Plastic Limit	COH (psf)	Torvane (psf)	Dry Wt. (pcf)	Sand %	D 10 SIZE	WES TEST
	70	21/6 35/6											
210													
	80	16/6 30/6 20/6	SP	Gr, F									
200													
	90	20/6 20/6 21/6		Gr, F								0.149	
190													
	100	38/6 58/6		Gr, F									
180													
	110	21/6 42/6 50/6	CL	Br	35	43	21						
170													
	120												
160													
	130												
150													
	140												

LAKE ELEVATION WAS 282.7 ON 27 APRIL 1993.

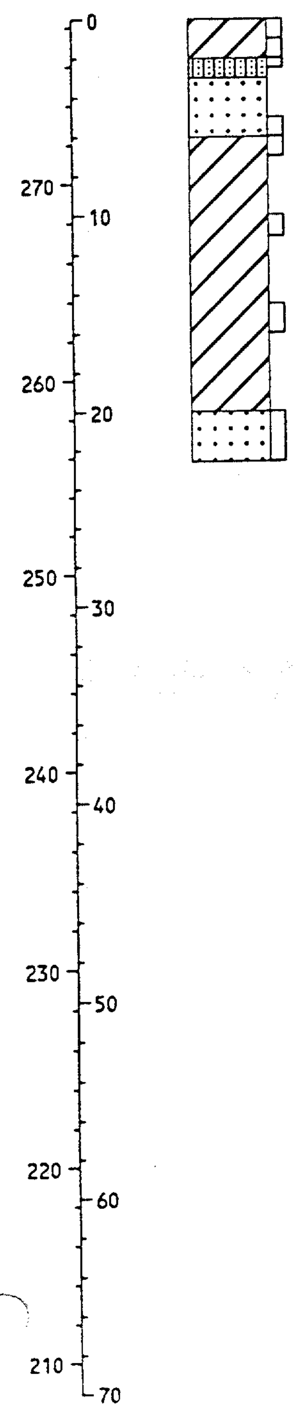
PLATE III-19

BORING LOG

BORING 1-RU-86

PROJECT: REELFOOT LAKE
LOCATION: SEE LOCATION MAP FOR BORING LOCATION
LATITUDE: LONGITUDE: STATION:
DRILLED (BY) MEMPHIS DISTRICT (METHOD) 3" TUBE (DATE) 24 JUNE 1986
DEPTH TO WATER WAS FEET ON GROUND ELEVATION: 278.4

ELEVATION / DEPTH — SOIL SYMBOLS, SAMPLER SYMBOLS AND FIELD TEST DATA



USCS	DESCRIPTION	WATER %	Liquid Limit	Plastic Limit	COH (psf)	Torvane (psf)	Dry Wt. (pcf)	Sand %	D 10 SIZE	WES TEST
CH	Gr, O	203	155	44						
	Gr, O	216	153	41						
SM	Gr, O									
SP	Gr, F								0.18	
	Gr	73	83	23						C
CH	Gr	65	95	28	371		61.2			
	Gr	100	104	26						C
SP	Gr, F, CS								0.11	

LAKE ELEVATION WAS 282.3 FEET ON 24 JUNE 1986.

BORING LOG

BORING 2-RU-86

PROJECT: REELFOOT LAKE
LOCATION: SEE BORING LOCATION MAP FOR BORING LOCATION
LATITUDE: LONGITUDE: STATION:
DRILLED (BY) MEMPHIS DISTRICT (METHOD) 3" TUBE (DATE) 25 JUNE 1986
DEPTH TO WATER WAS FEET ON GROUND ELEVATION: 279.3

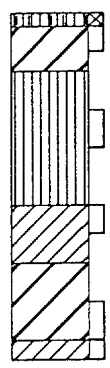
ELEVATION / DEPTH	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	DESCRIPTION	WATER %	Liquid Limit	Plastic Limit	COH (psf)	Torvane (psf)	Dry Wt. (pcf)	Sand %	D 10 SIZE	WES TEST
0		CL	Gr, O	175								
		CH	Gr	43	82	25						C
			Gr	39	61	20						
270 / 10			Gr, rt, Wd	45	61	21	127		77.4			
			Gr, Decayed wood	38	68	20						C
260 / 20												
250 / 30												
240 / 40												
230 / 50												
220 / 60												
210 / 70												

LAKE ELEVATION WAS 282.3 FEET ON 25 JUNE 1986.

BORING LOG

BORING 3-RU-86

PROJECT: REELFOOT LAKE
LOCATION: SEE BORING LOCATION MAP FOR BORING LOCATION
LATITUDE: LONGITUDE: STATION:
DRILLED (BY) MEMPHIS DISTRICT (METHOD) 3" TUBE (DATE) 25-26 JUNE 1986
DEPTH TO WATER WAS FEET ON GROUND ELEVATION: 279.8



ELEVATION / DEPTH	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	DESCRIPTION	WATER %	Liquid Limit	Plastic Limit	COH (psf)	Torvane (psf)	Dry Wt. (pcf)	Sand %	D 10 SIZE	WES TEST
0		OL	Gr	39	80	24	767		81.9			
		CH	Gr, O									
		ML	Gr, CS	32								
270 / 10		CL	Gr, SIS	33	43	20	461		89.3			
		CH	Gr	51	58	21						C
		CL	Gr	34	37	23	471		90.4			

Elevation/depth scale: 270–10, 260–20, 250–30, 240–40, 230–50, 220–60, 210–70

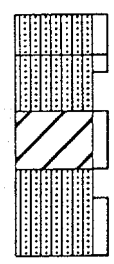
LAKE ELEVATION WAS 282.3 FEET ON 25 JUNE 1986.

PLATE III-22

BORING LOG

BORING 4-RU-86

PROJECT: REELFOOT LAKE
LOCATION: SEE BORING LOCATION MAP FOR BORING LOCATION
LATITUDE: LONGITUDE: STATION:
DRILLED (BY) MEMPHIS DISTRICT (METHOD) 3" TUBE (DATE) 30 JUNE 1986
DEPTH TO WATER WAS FEET ON GROUND ELEVATION: 280.2



ELEVATION / DEPTH	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	DESCRIPTION	WATER %	Liquid Limit	Plastic Limit	COH (psf)	Torvane (psf)	Dry Wt. (pcf)	Sand %	D 10 SIZE	WES TEST
280 / 0		SM	Gr, CS									
		SM	Gr							79		
		CH	Gr, SS	37	83	24	1022		84.6			
270 / 10		SM	Gr	28								
260 / 20												
250 / 30												
240 / 40												
230 / 50												
220 / 60												
70												

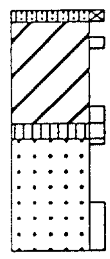
LAKE ELEVATION WAS 282.2 FEET ON 30 JUNE 1986.

PLATE III-23

BORING LOG

BORING 5-RU-86

PROJECT: REELFOOT LAKE
LOCATION: SEE BORING LOCATION MAP FOR BORING LOCATION
LATITUDE: LONGITUDE: STATION:
DRILLED (BY) MEMPHIS DISTRICT (METHOD) 3" TUBE (DATE) 1 JULY 1986
DEPTH TO WATER WAS FEET ON GROUND ELEVATION: 280.1



ELEVATION / DEPTH	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	DESCRIPTION	WATER %	Liquid Limit	Plastic Limit	COH (psf)	Torvane (psf)	Dry Wt. (pcf)	Sand %	D 10 SIZE	WES TEST
280 — 0		SM	Gr, O	31	58	22	1167		90.4			
		CH	Gr									
			Gr	29	51	18	1188		93.9			
		ML	Gr	36	34	24						C
		SP	Gr, F									
270 — 10			Br & Gr, F								0.16	
260 — 20												
250 — 30												
240 — 40												
230 — 50												
220 — 60												
70												

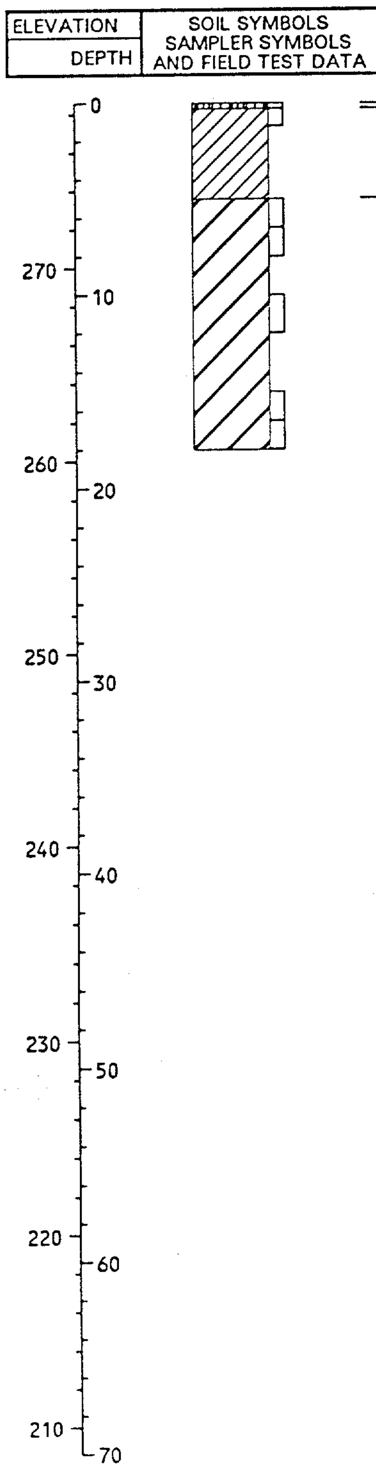
LAKE ELEVATION WAS 282.1 FEET ON 1 JULY 1986.

PLATE III-24

BORING LOG

BORING 6-RU-86

PROJECT: REELFOOT LAKE
LOCATION: SEE BORING LOCATION MAP FOR BORING LOCATION
LATITUDE: LONGITUDE: STATION:
DRILLED (BY) MEMPHIS DISTRICT (METHOD) 3" TUBE (DATE) 1 JULY 1986
DEPTH TO WATER WAS FEET ON GROUND ELEVATION: 278.6



USCS	DESCRIPTION	WATER %	Liquid Limit	Plastic Limit	COH (psf)	Torvane (psf)	Dry Wt. (pcf)	Sand %	D 10 SIZE	WES TEST
OL	BK	37	44	20	256		84.8			
CL	Gr									
	Gr	38	59	24	435		81.4			
	Gr	37	51	22						C
CH	Gr	44	92	27	1169		77.3			
	Gr	52	88	27	819		71.4			
	Gr	47	70	24						C

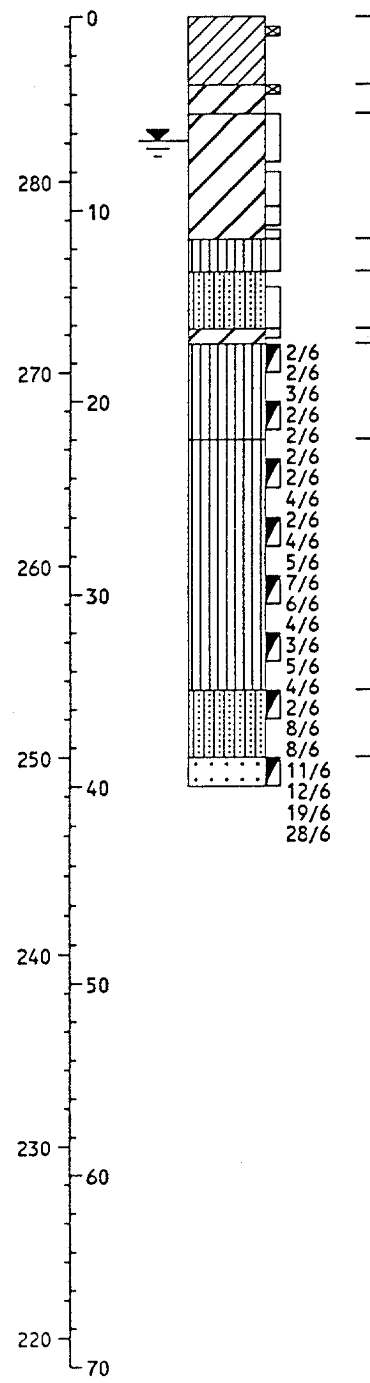
LAKE ELEVATION WAS 282.1 FEET ON 1 JULY 1986.

BORING LOG

BORING 7-RU-86

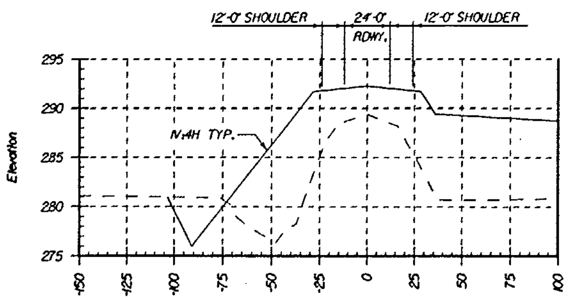
PROJECT: REELFOOT LAKE
LOCATION: SEE BORING LOCATION MAP FOR BORING LOCATION
LATITUDE: LONGITUDE: STATION:
DRILLED (BY) MEMPHIS DISTRICT (METHOD) AUG, 5" TUBE, SS (DATE) 8 JULY 1986
DEPTH TO WATER WAS 6.4 FEET ON 8 JULY 1986 GROUND ELEVATION: 288.5

ELEVATION / DEPTH — SOIL SYMBOLS, SAMPLER SYMBOLS AND FIELD TEST DATA

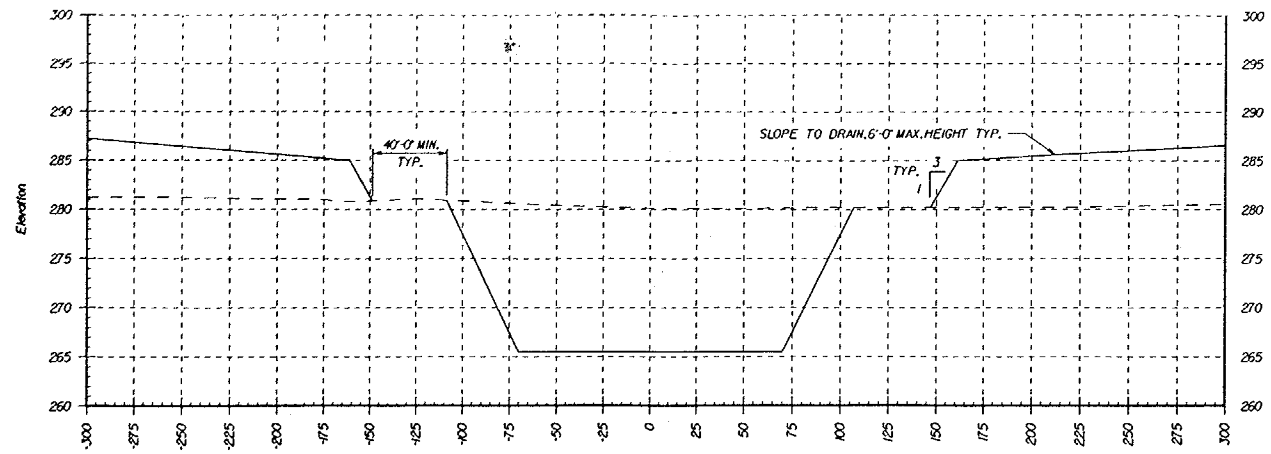


USCS	DESCRIPTION	WATER %	Liquid Limit	Plastic Limit	COH (psf)	Torvane (psf)	Dry Wt. (pcf)	Sand %	D 10 SIZE	WES TEST
CL	Br, S, G, M	23								
CH	Br & Gr, M	26	53	18						
	Br, Ox	25	64	18	2791		98.5			
CH	Br & Gr, Ox, SL	30	76	20	1202		90.9			
	Gr	31	73	20						C
	Br & Gr, Ox	31	61	20	1815		94.2	24		
ML	Br, S, CS									
SM	Br							75		
CH	Br, SIS, SS									
	Br, CS									
ML	Gr, CS									
	Gr, S							20		
	Gr, S							24		
ML	Gr, S									
	Gr, S									
SM	Gr							77	0.004	
SP	Br & Gr, M								0.14	

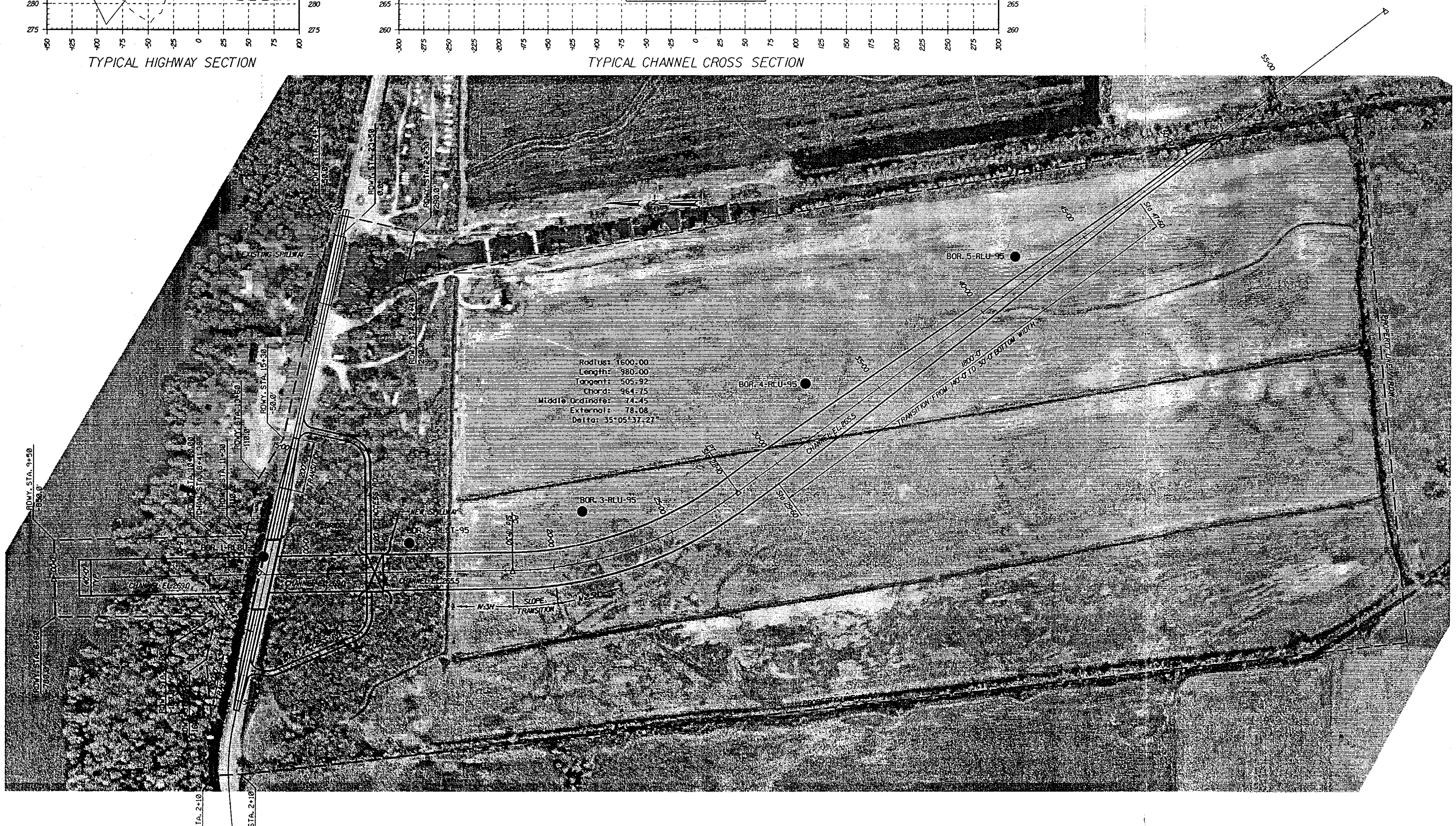
LAKE ELEVATION WAS 282.0 ON 8 JULY 1986.



TYPICAL HIGHWAY SECTION



TYPICAL CHANNEL CROSS SECTION



R.O.W. PLAN
SCALE 1" : 400'

US Army Corps of Engineers Memphis District

U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
MEMPHIS, TENNESSEE

ENVIRONMENTAL RESTORATION NATURAL RESOURCES PROJECT — MISSISSIPPI RIVER AND TRIBUTARIES CONSTRUCTION

REELFOOT LAKE, TENNESSEE & KENTUCKY
ALTERNATIVE SPILLWAY
RIGHT-OF WAY & TYPICAL SECTIONS

DRAWING
III-27

FINAL SITE PLAN

SCALE 1" = 200'

NOTES:
1. HIGHWAY SLOPES ARE 1V:4H. ALL OTHER SLOPES ARE 1V:3H UNLESS NOTE OTHERWISE.

US Army Corps of Engineers
Memphis District

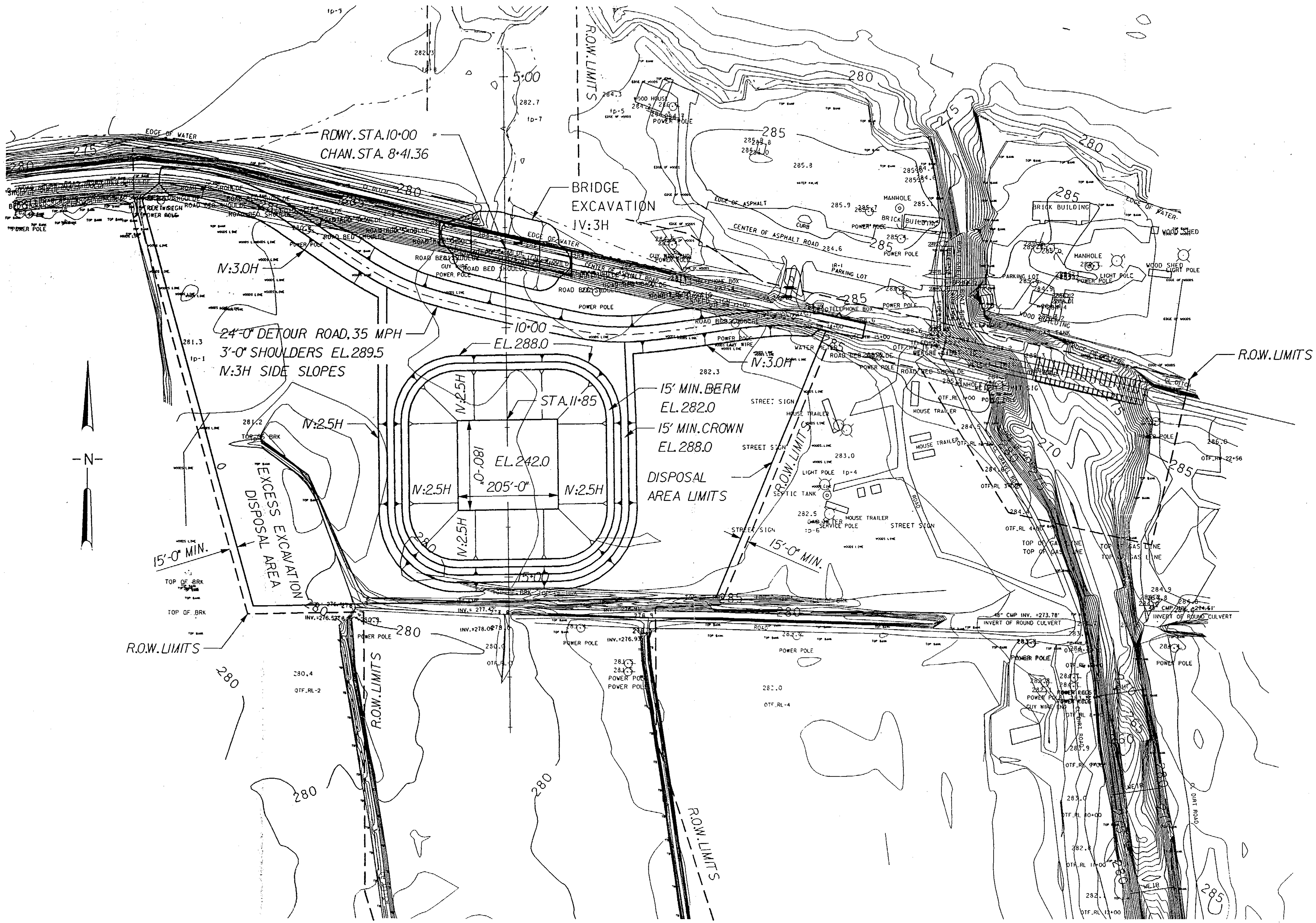
U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
MEMPHIS, TENNESSEE

ENVIRONMENTAL RESTORATION
NATURAL RESOURCES PROJECT

MISSISSIPPI RIVER AND TRIBUTARIES
CONSTRUCTION

REELFOOT LAKE, TENNESSEE & KENTUCKY
ALTERNATIVE SPILLWAY
FINAL SITE PLAN

DRAWING
III-28



CONSTRUCTION EXCAVATION PLAN
SCALE 1" = 200'

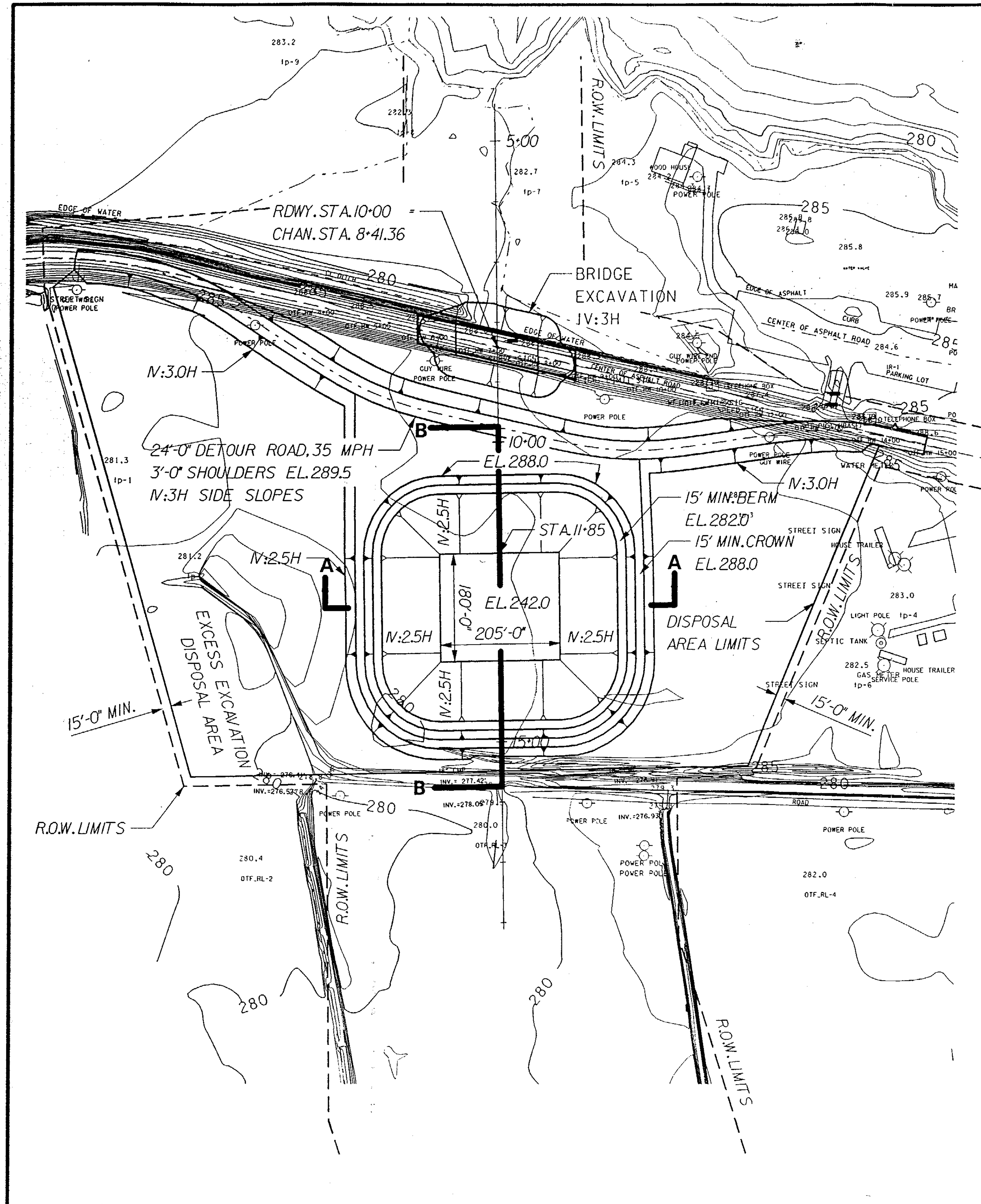
US Army Corps of Engineers Memphis District

U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
MEMPHIS, TENNESSEE

ENVIRONMENTAL RESTORATION
NATURAL RESOURCES PROJECT
MISSISSIPPI RIVER AND TRIBUTARIES
CONSTRUCTION

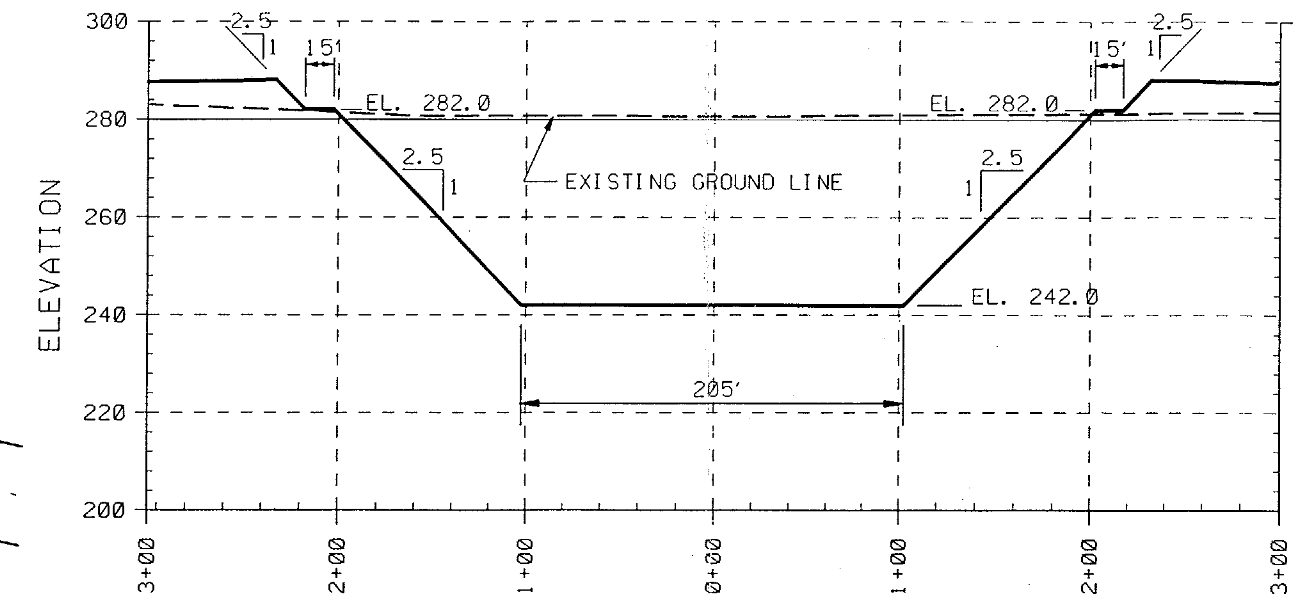
REELFOOT LAKE, TENNESSEE & KENTUCKY
ALTERNATIVE SPILLWAY
CONSTRUCTION EXCAVATION PLAN

DRAWING
III-29



CONSTRUCTION EXCAVATION PLAN

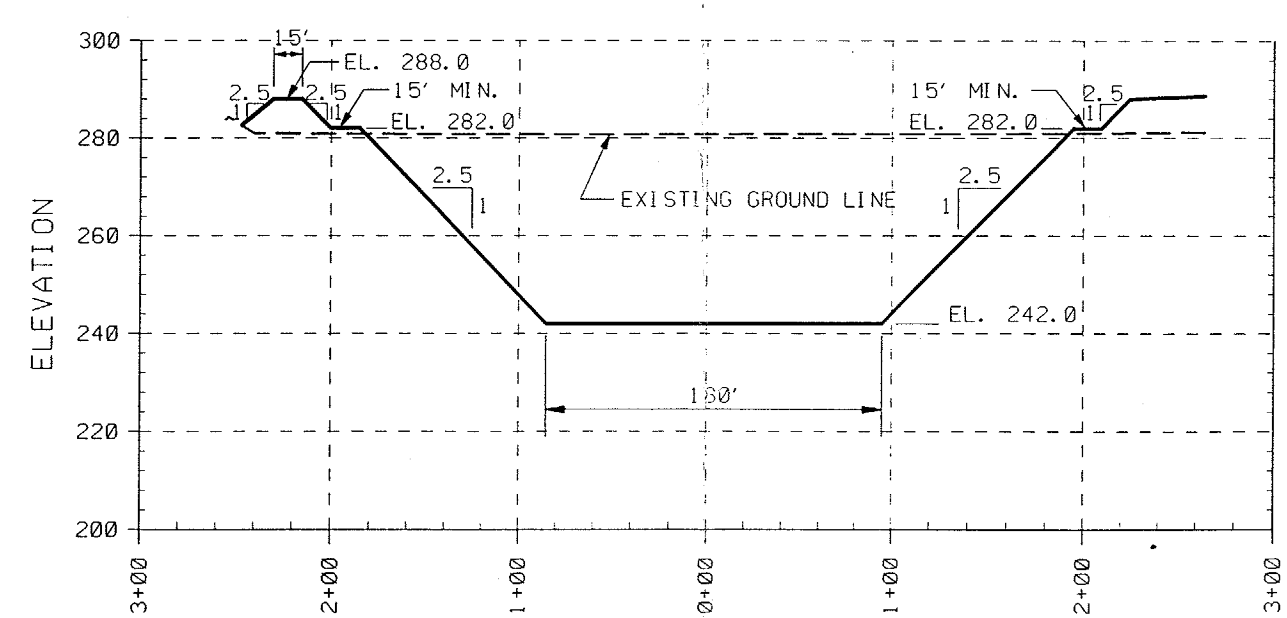
SCALE 1" = 200



SECTION A-A

SCALE 1" = 40' VERTICAL

SCALE 1" = 100' HORIZONTAL



SECTION B-B

SCALE 1" = 40' VERTICAL

SCALE 1" = 100' HORIZONTAL

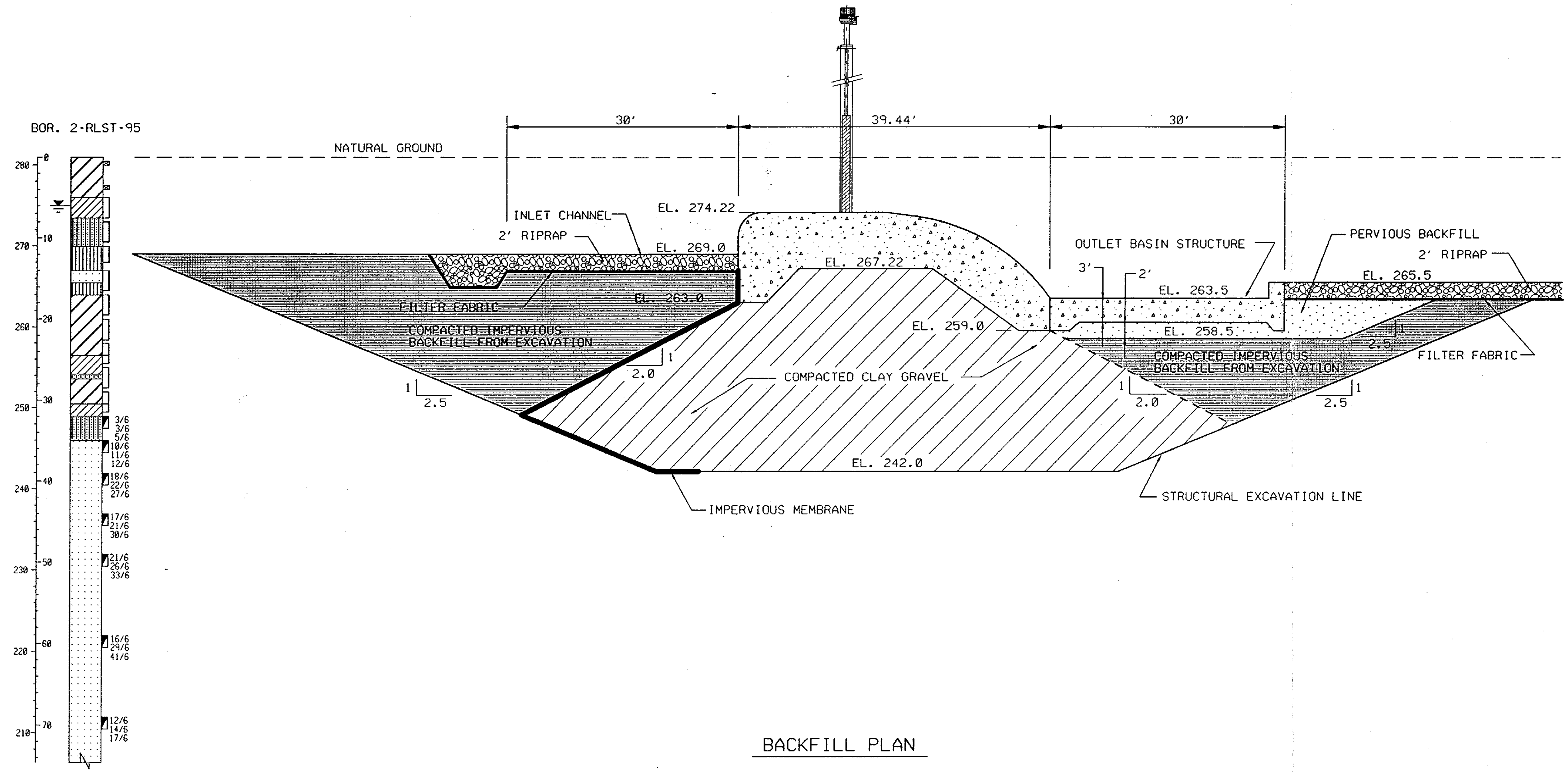
US Army Corps of Engineers Memphis District

U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
MEMPHIS, TENNESSEE

MISSISSIPPI RIVER AND TRIBUTARIES
ENVIRONMENTAL RESTORATION
NATURAL RESOURCES PROJECT
CONSTRUCTION

REELFOOT LAKE, TENNESSEE & KENTUCKY
ALTERNATIVE SPILLWAY
COFFERDAM AND EXCAVATION PLAN

DRAWING
III-30



BACKFILL PLAN

US Army Corps of Engineers Memphis District

U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
MEMPHIS, TENNESSEE

ENVIRONMENTAL RESTORATION
NATURAL RESOURCES PROJECT

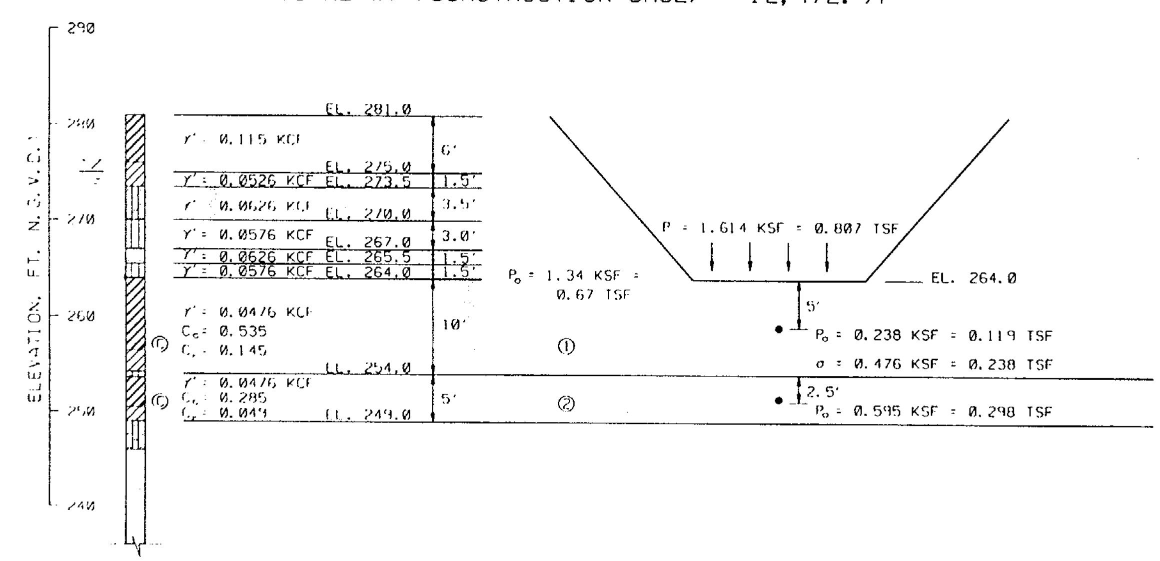
MISSISSIPPI RIVER AND TRIBUTARIES
CONSTRUCTION

REELFOOT LAKE, TENNESSEE & KENTUCKY
ALTERNATIVE SPILLWAY
BACKFILL PLAN

DRAWING
III-31

SPILLWAY STRUCTURE IS 39.44 FT WIDE X 196 FT LONG

TOTAL WT (CONSTRUCTION CASE) = 12,472.94^K



ASSUME EXCAVATION TO EL. 264.0 AND ASSUME SOIL REBOUNDS TOTALLY AND THEN STRUCTURAL LOAD IS APPLIED. USED CONSOLIDATION TEST RESULTS FROM SAMPLES 9 AND 11 FROM BORING 2-RLST-95

DETERMINE SETTLEMENT DUE TO STRUCTURAL LOADING OF 0.807 TSF

SAMPLE NO. TEST	LAYER	MIDPOINT P_o (TSF)	e_o	ΔP (TSF)	$P_o + \Delta P$ (TSF)	e_T	Δe $(e_o - e_T)$	H (FT)	$\Delta H = \frac{H\Delta e}{1+e_o}$
9	1	0.119	1.485	0.801	0.920	1.408	0.077	10	0.310′
11	2	0.298	1.090	0.752	1.050	1.025	0.065	5	0.156′
									0.466′ = 5.59"

SETTLEMENT APPEARS EXCESSIVE

VALUES OF ΔP WERE DETERMINED FROM BOUSSINESQ STRESS CURVES FROM NAVFAC DM 7.1, P. 7.1-168

ENVIRONMENTAL RESTORATION NATURAL RESOURCES PROJECT
MISSISSIPPI RIVER AND TRIBUTARIES CONSTRUCTION

REELFOOT LAKE, TENNESSEE & KENTUCKY
SPILLWAY STRUCTURE
SETTLEMENT ANALYSIS

U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
MEMPHIS, TENNESSEE

US Army Corps of Engineers
Memphis District

DRAWING
III-22

RECURRENCE EQUATIONS - CENTRAL UNITED STATES [1]

REGION	SEISMIC AREA "A" (km^2)	a	M_bMAX
NEW MADRID "A"	22,506	3.90	7.5
NEW MADRID "B"	27,506	2.99	6.5

[1] BOUNDARIES OF SEISMIC SOURCE ZONES AND COEFFICIENTS FOR MAGNITUDE-RECURRENCE EQUATIONS OBTAINED FROM WES MP S-73-1, "STATE-OF-ART FOR ASSESSING EARTHQUAKE HAZARDS IN THE UNITED STATES, REPORT 12, CREDIBLE EARTHQUAKES FOR THE CENTRAL UNITED STATES", BY NUTTLI & HERRMANN, 1978.

ASSUMPTIONS:

(1) 100-YEAR DESIGN LIFE FOR STRUCTURE.

(2) THE STRUCTURE IS LOCATED WITHIN THE NEW MADRID SEISMIC REGION "A" (FIG 3, MP S-73-1) (REF 1).

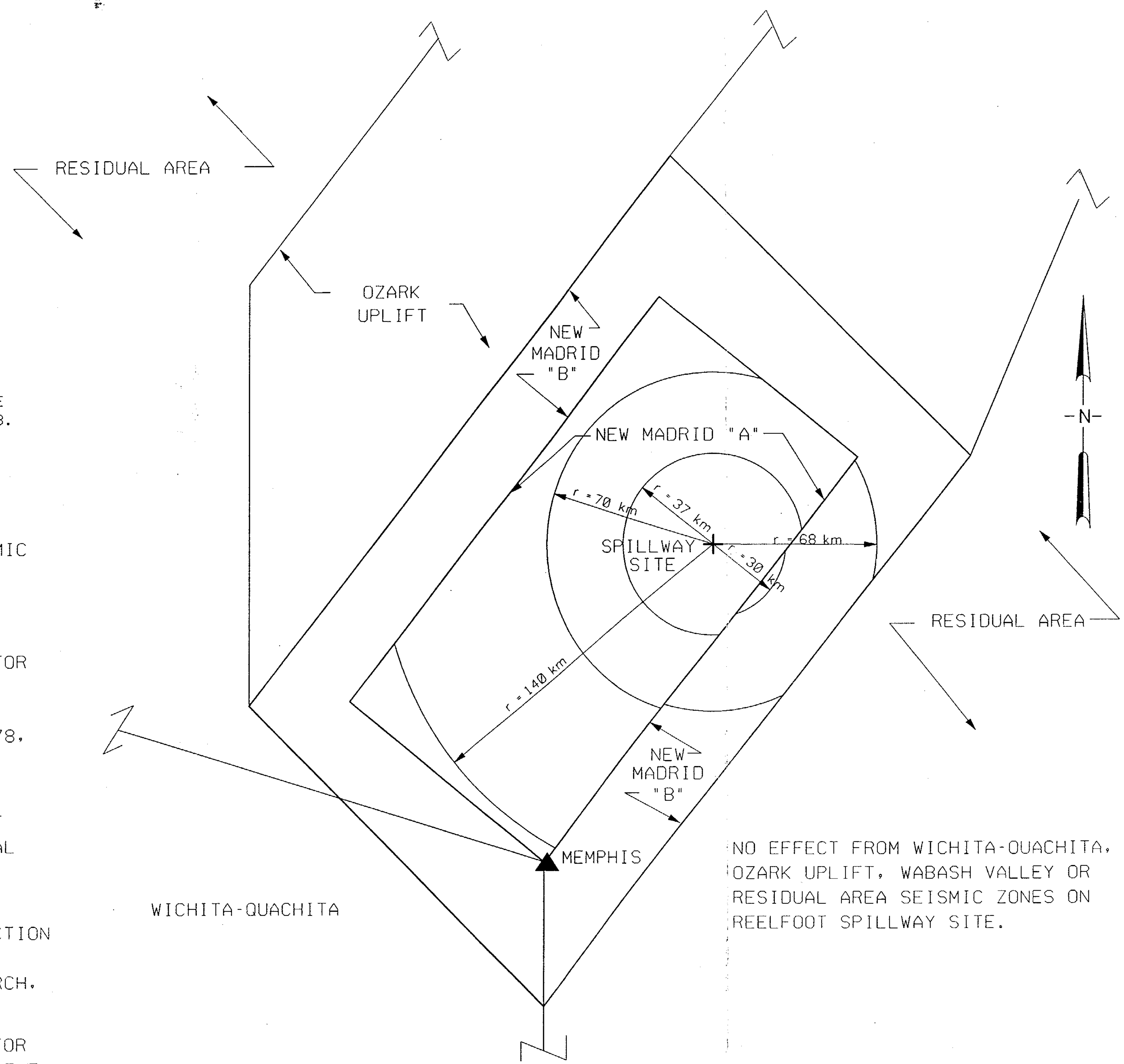
REFERENCES:

(1) NUTTLI, O.W. AND HERRMANN, R.B., "STATE-OF-THE-ART FOR ASSESSING EARTHQUAKE HAZARDS IN THE UNITED STATES, CREDIBLE EARTHQUAKE FOR THE CENTRAL UNITED STATES", MISCELLANEOUS PAPERS S-73-1, REPORT 12, DECEMBER 1978, U.S. ARMY ENGINEER WATERWAYS EXPERIMENT STATION, CE, VICKSBURG, MS.

(2) NUTTLI, O. W. AND HERRMANN, R. B., "GROUND MOTION OF MISSISSIPPI VALLEY EARTHQUAKES", JOURNAL OF TECHNICAL TOPICS IN CIVIL ENGINEERING, ASCE, VOL. 110, NO. 1, MAY, 1984, PP. 54-68.

(3) SEED, H.B. AND IDRISS, I.M., "EVALUATION OF LIQUEFACTION POTENTIAL USING FIELD PERFORMANCE DATA", JOURNAL OF GEOTECHNICAL ENGINEERING, ASCE, VOL. 109, NO. 3, MARCH, 1983, PP. 459-482.

(4) SEED, H.B. AND IDRISS, I.M., "SIMPLIFIED PROCEDURE FOR EVALUATING SOIL LIQUEFACTION POTENTIAL", JOURNAL OF THE SOIL MECHANICS AND FOUNDATIONS DIVISION, ASCE, VOL. 97, NO. SM9, SEPT., 1971, PAGES 1255 AND 1265.



NO EFFECT FROM WICHITA-QUACHITA, OZARK UPLIFT, WABASH VALLEY OR RESIDUAL AREA SEISMIC ZONES ON REELFOOT SPILLWAY SITE.

SEISMIC SOURCE ZONES - CENTRAL UNITED STATES [1]

SCALE: 1" = 50 KM

US Army Corps of Engineers Memphis District

U.S. ARMY ENGINEER DISTRICT CORPS OF ENGINEERS MEMPHIS, TENNESSEE

ENVIRONMENTAL RESTORATION NATURAL RESOURCES PROJECT

MISSISSIPPI RIVER AND TRIBUTARIES CONSTRUCTION

REELFOOT LAKE, TENNESSEE & KENTUCKY

SEISMIC RISK ANALYSIS

DRAWING III-33

I. EVALUATE LIQUEFACTION POTENTIAL AT SITE OF SPILLWAY. ASSUME WATER TABLE AT 6' DEPTH (EL. 275.0).

γ_{clay} = 0.115 KCF; γ'_{clay} = 0.0526 KCF; γ_{silt} = 0.120 KCF; γ'_{silt} = 0.0576 KCF;
γ_{sand} = 0.125 KCF; γ'_{sand} = 0.0626 KCF.

SEMI-EMPIRICAL PROCEDURE TO EVALUATE LIQUEFACTION POTENTIAL

BORING 2-RLST-95

DEPTH	ELEV	SOIL TYPE	N_f	σ_o (ksf)	σ'_o (ksf)	N_1	C_N	N_2	τ/σ'_o	r_d	a/g *
(FIRST 20' ± OF SOIL WILL BE REPLACED BY STRUCTURE. FIRST 32' CONSISTS OF CH & CL)											
32	249	SM	8	3.73	2.10	8	0.98	8	0.11	0.92	0.10
35	246	SP	23	4.10	2.29	19	0.95	18	0.20	0.89	0.19 **
39	242	SP	49	4.60	2.54	-	0.91	45	0.50	0.87	0.49
44	237	SP	51	5.23	2.85	-	0.86	44	0.50	0.82	0.51
49	232	SP	59	5.85	3.17	-	0.83	49	0.50	0.76	0.55
59	222	SP	70	7.10	3.79	-	0.78	55	0.50	0.70	0.59
69	212	SP	31	8.35	4.42	-	0.72	22	0.23	0.60	0.31
79	202	SP	29	9.60	5.04	-	0.67	19	0.21	0.55	0.31
89	192	SP	85	10.85	5.67	-	0.63	54	0.50	0.52	0.77
99	182	SP	100+	12.10	6.30	-	0.60	60	0.50	0.51	0.79
109	172	SP	63	13.35	6.92	-	0.57	36	0.45	0.51	0.70
119	162	SP	77	14.60	7.55	-	0.54	42	0.50	0.51	0.78
129	152	SP	100+	(ASSUMED TO BE NON-LIQUEFIABLE)							
139	142	GP	100+	(ASSUMED TO BE NON-LIQUEFIABLE)							
149	132	SP	100+								
159	122	SP	100+								

** CONTROLLING STRATUM

$* \; a/g = \left(\frac{\tau}{\sigma'_o}\right)\left(\frac{\sigma'_o}{\sigma_o}\right)\left(\frac{1}{0.65\, r_d}\right)$ = ACCELERATION AT WHICH LIQUEFACTION WILL OCCUR.

FROM THE ANALYSIS ABOVE. IT IS OBVIOUS FROM THE ACCELERATION VALUES AT WHICH LIQUEFACTION WILL OCCUR THAT THE SM AND SP LAYERS LOCATED AT A DEPTH BETWEEN 32 AND 35 FEET (EL. 249.0-246.0) ARE THE MOST SUSCEPTIBLE TO LIQUEFACTION. BASED ON OTHER CONCERNS REGARDING THE STRUCTURAL STABILITY OF THE SPILLWAY FOUNDATION. IT HAS BEEN PROPOSED TO OVER-EXCAVATE THE IN-SITU CLAYS DOWN TO SAND AND BACKFILL WITH A COMPACTED CLAY GRAVEL MATERIAL. TO REMEDIATE THE RISK OF LIQUEFACTION IN THESE UPPER SAND LAYERS BETWEEN EL. 249.0 AND 246.0. IT IS RECOMMENDED TO EXTEND THE OVER-EXCAVATION DOWN TO EL. 242.0 AND REPLACE THESE SANDS WITH A COMPACTED CLAY GRAVEL BACKFILL ALSO. THIS BEING THE CASE. THE NEXT LAYERS OF SAND THAT ARE THE MOST SUSCEPTILBLE TO LIQUEFACTION OCCUR AT DEPTHS BETWEEN 69 AND 79 FEET. A RISK ANALYSIS AS PRESENTED BELOW WAS PERFORMED TO DETERMINE THE COST EFFECTIVENESS OF ABATING THE LIQUEFACTION POTENTIAL IN THESE LAYERS.

II. UTILIZING THE ATTENUATION RELATIONSHIP CURVES FOR THE MISSISSIPPI VALLEY (SEE FIG. 6, REF. 2) AND THE CHART EVALUATING LIQUEFACTION POTENTIAL FOR DIFFERENT MAGNITUDE (M_b) EARTHQUAKES (SEE FIG. 13, REF. 3) THEN THE MAXIMUM DISTANCE (r) FROM THE EPICENTER TO THE SITE THAT WOULD PRODUCE LIQUEFACTION CAN BE DETERMINED FOR DIFFERENT MAGNITUDES OF EARTHQUAKES. FURTHERMORE, THE TOTAL ANNUAL RISK FACTOR (R_T) FOR AN EARTHQUAKE TO CAUSE LIQUEFACTION OF THE CONTROLLING SAND STRATUM CAN BE DETERMINED.

CALCULATION OF ANNUAL RISK FACTOR OF LIQUEFACTION

BORING 2-RLST-95, STRATUM AT 79' DEPTH, N_2 = 19, σ_o = 9.60, σ'_o = 5.04

EXISTING CONDITIONS

NEW MADRID 'A' SEISMIC RISK ZONE
AREA = 22,506 KM^2; LOG N_c = a - 0.92 M_b; a = 3.90; M_b (MAX) = 7.5; $N_c = 10^{(a - 0.92 M_b)}$

M_b	N_c	I	τ/σ'_o	a/g	a_h	r(km)	P_A	P_{AVG}	RISK
5.25	0.11749		0.300	0.4407	432	1.3	0.000236		
		0.09350						0.008563	0.000801
6.0	0.02399		0.265	0.3893	382	11.0	0.01689		
		0.00829						0.03637	0.000302
6.2	0.01570		0.230	0.3379	331	20.0	0.05584		
		0.00897						0.13792	0.001237
6.6	0.00673		0.203	0.2982	293	43.0	0.2200		
		0.00440						0.4372	0.001924
7.1	0.00233		0.181	0.2659	261	89.0	0.6543		
		0.00133						0.8238	0.001096
7.5	0.00100		0.160	0.2350	231	160.0	0.9933		
								RISK TOTAL	0.005360

CALCULATION OF ANNUAL RISK FACTOR OF LIQUEFACTION

BORING 2-RLST-95, STRATUM AT 79' DEPTH, N_2 = 19, σ_o = 9.60, σ'_o = 5.04

EXISTING CONDITIONS

NEW MADRID 'B' SEISMIC RISK ZONE
AREA = 27,506 KM^2; LOG N_c = a - 0.92 M_b; a = 2.99; M_b (MAX) = 6.5; $N_c = 10^{(a - 0.92 M_b)}$

M_b	N_c	I	τ/σ'_o	a/g	a_h	r(km)	P_A	P_{AVG}	RISK
5.25	0.01445		0.300	0.4407	432	1.3	(NO RISK TO SPILLWAY)		
		0.011499							
6.0	0.002951		0.265	0.3893	382	11.0	(NO RISK TO SPILLWAY)		
		0.001019							
6.2	0.001932		0.230	0.3379	331	20.0	(NO RISK TO SPILLWAY)		
		0.000909						0.01135	0.000010
6.5	0.001023		0.210	0.3085	303	40.0	0.0227		
								RISK TOTAL	0.000010

III. IF THE CONTROLLING STRATUM AT A DEPTH OF 79 FT WAS DENSIFIED THE RISK OF LIQUEFACTION COULD BE REDUCED. ASSUMING DENSIFICATION OF THE EXISTING MATERIALS TO A RELATIVE DENSITY OF 80% WOULD RESULT IN A FIELD N VALUE OF N_f = 45 (SEE FIG. 13, REF. 4) WHICH TRANSLATES TO N_2 = 30. THE RISK OF LIQUEFACTION IS COMPUTED BELOW. AS SHOWN IN PARAGRAPH II OVER 99% OF THE TOTAL RISK IS DERIVED FROM THE NEW MADRID 'A' SEISMIC ZONE. THEREFORE, IN THE FOLLOWING ANALYSES IT IS ASSUMED THAT THE TOTAL RISK APPROXIMATES THE RISK FOR THE NEW MADRID 'A' SEISMIC ZONE.

CALCULATION OF ANNUAL RISK FACTOR OF LIQUEFACTION

BORING 2-RLST-95, STRATUM AT 79' DEPTH, σ_o = 9.60, σ'_o = 5.04

IMPROVED CONDITIONS

NEW MADRID 'A' SEISMIC RISK ZONE
AREA = 22,506 KM^2; LOG N_c = a - 0.92 M_b; a = 3.90; M_b (MAX) = 7.5; $N_c = 10^{(a - 0.92 M_b)}$

M_b	N_c	I	τ/σ'_o	a/g	a_h	r(km)	P_A	P_{AVG}	RISK
5.25	0.11749		0.500	0.7345	721	1.0	0.00014		
		0.09350						0.00014	0.000013
6.0	0.02399		0.500	0.7345	721	1.0	0.00014		
		0.00829						0.00014	0.000001
6.2	0.01570		0.400	0.5876	576	1.0	0.00014		
		0.00897						0.02527	0.000227
6.6	0.00673		0.335	0.4921	483	19.0	0.05039		
		0.00440						0.16625	0.000732
7.1	0.00233		0.295	0.4333	425	49.0	0.2821		
		0.00133						0.50205	0.000668
7.5	0.00100		0.260	0.3819	375	100.0	0.7220		
								RISK TOTAL	0.001641

IV. DETERMINE COST EFFECTIVENESS OF GROUND MODIFICATION TO REDUCE THE RISK OF LIQUEFACTION. BASED ON FINDINGS FROM OTHER SIMILAR PROJECTS. GROUND DENSIFICATION AT THESE DEPTHS WOULD BE MOST COST EFFECTIVE BY USING VIBROFLOTATION TECHNIQUES. ASSUME DAMAGE DUE TO LIQUEFACTION EQUALS APPROXIMATELY $3,500,000.00 (INITIAL COST OF STRUCTURE MINUS SALVAGE VALUE OF GATES, HOISTS, ETC. COST EFFECTIVENESS IS MEASURED AS SAVINGS IN ANNUAL DAMAGES. ANNUAL DAMAGES ARE CONVERTED TO PRESENT WORTH BASED ON 100-YEAR LIFE OF STRUCTURE AND AN INTEREST RATE (i) EQUAL TO 7.375%.

Σ RISKS FOR SPILLWAY - EXISTING CONDITIONS

ZONE 'A' = 0.005360
ZONE 'B' = 0.000010
TOTAL = 0.005370

Σ RISKS FOR SPILLWAY - IMPROVED CONDITIONS
(ASSUMING DENSIFICATION OF SANDS TO DEPTH OF APPROXIMATE 85' TO A RELATIVE DENSITY = 80%)

ZONE 'A' = 0.001641
ZONE 'B' = NO RISK
TOTAL = 0.001641

BENEFITS:
SAVINGS IN ANNUAL DAMAGES = $3,500,000 (0.005370 - 0.001641) = $13,052.00
PRESENT WORTH (n = 100 YEARS; i = 7.375; A = 13,052)

$$\text{PRESENT WORTH} = A\left(\frac{(1+i)^n - 1}{i(1+i)^n}\right) = 13,052\left(\frac{(1+0.07375)^{100}-1)}{0.07375(1+0.07375)^{100}}\right)$$

= $176,775.00

COSTS:
ASSUME VIBROFLOTATION WILL BE USED TO DENSIFY SANDS FROM EL. 248.0 TO EL. 196.0.

VOLUME OF SAND TO BE DENSIFIED
250' X 150' X (248-196) = 1,950,000 FT^3 = 72,250 YD^3

72,250 YD^3 AT $5.50/$YD^3$ = $397,375

BENEFITS/COSTS:

$$B/C = \frac{176,775}{397,375} = 0.44$$

V. IN SUMMARY IT IS NOT COST EFFECTIVE TO PERFORM GROUND DENSIFICATION OF DEEPER SANDS FOR THE PURPOSE OF REDUCING OR ELIMINATING THE RISKS OF EARTHQUAKE INDUCED LIQUEFACTION.

DEFINITIONS:

$1/N_c$ = RECURRENCE PERIOD IN YEARS.
LOG N_c = a - 0.92 M_b (PARA. 50, REF. 1)
a = COEFFICIENT INDICATIVE OF THE NUMBER OF EVENTS PER SEISMIC AREA 'A' UP TO 100,000 km^2
M_b = BODY WAVE MAGNITUDE.
N_f = FIELD RESULTS OF STANDARD PENETRATION TEST (SPT) BLOWS/FT.
N_1 = RESULTS OF SPT CORRECTED FOR PRESENCE OF FINES.

$$N_1 = 15 + \frac{(N - 15)}{2}$$

C_n = CORRECTION FACTOR FOR EFFECTIVE OVERBURDEN PRESSURE (FIG. 4, REF. 3)
N_2 = N_1 CORRECTED FOR EFFECTIVE OVERBURDEN PRESSURE.

$$N_2 = C_c N_1$$

σ_o = TOTAL OVERBURDEN PRESSURE (ksf)
σ'_o = EFFECTIVE OVERBURDEN PRESSURE (ksf)
τ/σ'_o = CYCLIC STRESS RATIO CAUSING LIQUEFACTION AT σ'_o = 1 tsf (FIG. 9, REF. 3)
r_d = STRESS REDUCTION COEFFICIENT (FIG. 4, REF. 4)
a/g = ACCELERATION/GRAVITY AT WHICH LIQUEFACTION WILL OCCUR

$a/g = \left(\frac{\tau}{\sigma'_o}\right)\left(\frac{\sigma'_o}{\sigma_o}\right)\left(\frac{1}{0.65\, r_d}\right)$ (EQ. 1, REF. 3)

r = MAXIMUM RADIUS FROM EPICENTER AT WHICH LIQUEFACTION WILL OCCUR
I = INCREMENTAL RECURRENCE ($N_{C(n)} - N_{C(n+1)}$)

P_A = AREA OF SEISMIC SOURCE ZONE ENCOMPASSED BY RADIUS "r" DIVIDED BY SEISMIC AREA 'A' FOR 'A' ≤ 100,000 km^2

$$P_{AVG} = \frac{(P_{A(n)} + P_{A(n+1)})}{2}$$

RISK = I X P_{AVG} = INCREMENTAL ANNUAL RISK FACTOR FOR AN EARTHQUAKE OF M_b = X TO CAUSE LIQUEFACTION OF THE CONTROLLING SAND STRATUM.

US Army Corps of Engineers Memphis District

U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
MEMPHIS, TENNESSEE

ENVIRONMENTAL RESTORATION / NATURAL RESOURCES PROJECT
MISSISSIPPI RIVER AND TRIBUTARIES CONSTRUCTION

REELFOOT LAKE, TENNESSEE & KENTUCKY

SEISMIC RISK ANALYSIS

DRAWING III-34

ARBITRARY COMPUTER COORDINATES

BORING 2-RLST-95
COMPUTER RESULTS
*EDGE METHOD (GEOSLOPE)

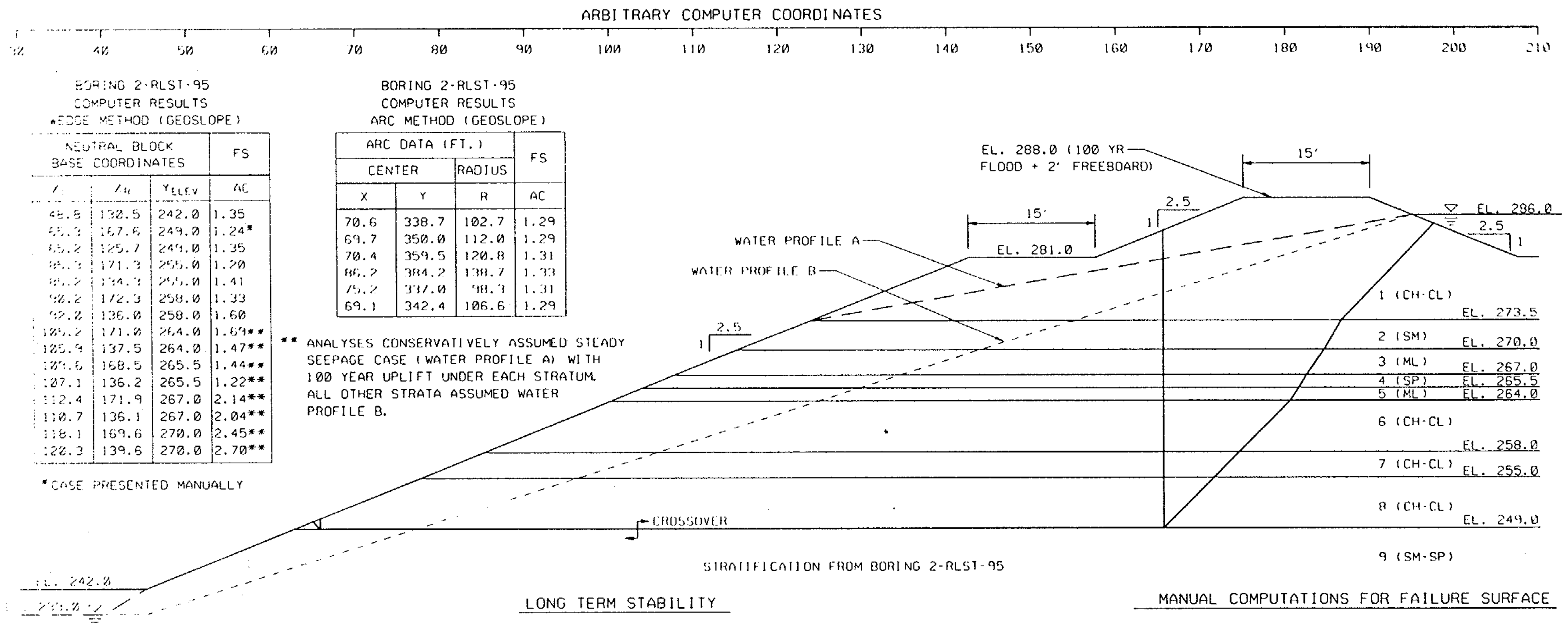
NEUTRAL BLOCK BASE COORDINATES			FS
X_L	X_R	Y_{ELEV}	AC
48.8	130.5	242.0	1.35
65.3	167.6	249.0	1.24*
65.2	125.7	249.0	1.35
85.3	171.3	255.0	1.20
85.2	134.3	255.0	1.41
90.2	172.3	258.0	1.33
92.2	136.0	258.0	1.60
105.2	171.0	264.0	1.69**
105.9	137.5	264.0	1.47**
101.6	168.5	265.5	1.44**
107.1	136.2	265.5	1.22**
112.4	171.9	267.0	2.14**
110.7	136.1	267.0	2.04**
118.1	169.6	270.0	2.45**
122.3	139.6	270.0	2.70**

*CASE PRESENTED MANUALLY

BORING 2-RLST-95
COMPUTER RESULTS
ARC METHOD (GEOSLOPE)

ARC DATA (FT.)			FS
CENTER		RADIUS	
X	Y	R	AC
70.6	338.7	102.7	1.29
69.7	350.0	112.0	1.29
70.4	359.5	120.8	1.31
86.2	384.2	138.7	1.33
75.2	337.0	98.3	1.31
69.1	342.4	106.6	1.29

** ANALYSES CONSERVATIVELY ASSUMED STEADY SEEPAGE CASE (WATER PROFILE A) WITH 100 YEAR UPLIFT UNDER EACH STRATUM. ALL OTHER STRATA ASSUMED WATER PROFILE B.



LONG TERM STABILITY

$$FS = \frac{\tan\phi}{\tan i} = \frac{2.5[7.5\tan 23^\circ + 12\tan 30^\circ + 4.5\tan 28^\circ + 15\tan 21^\circ]}{39.0}$$

FS = $\underline{1.17}$ O.K. FOR CONSTRUCTION DURATION

SOIL SHEAR STRENGTHS - BORING 2-RLST-95

SOIL NO.	SOIL TYPE	ELEVATION (N.G.V.D.)	UNIT WT. (PCF)	Q φ (°)	Q C (PSF)	R φ (°)	R C (PSF)	S φ (°)	S C (PSF)
1	CH-CL	281.0-273.5	115	0	675	12	335	23	0
2	SM	273.5-270.0	125	30	0	30	0	30	0
3	ML	270.0-267.0	120	20	300	20	300	28	0
4	SP	267.0-265.5	125	30	0	30	0	30	0
5	ML	265.5-264.0	120	20	300	20	300	28	0
6	CH-CL	264.0-258.0	110	0	675	10	335	21	0
7	CH-CL	258.0-255.0	110	0	400	10	200	21	0
8	CH-CL	255.0-249.0	110	0	675	10	335	21	0
9	SM-SP	BELOW 249.0	125	30	0	30	0	30	0

NOTE: ANALYSES PERFORMED ASSUMING DEWATERING SYSTEM IN OPERATION AND WATER TABLE AT EL. 239.0 AT THE BOTTOM OF EXCAVATION AND AT EL. 286.0 ON THE COFFERDAM. ANALYSES ALSO ASSUMED A MINIMUM BERM WIDTH OF 15 FEET. ACTUAL BERM WIDTH WILL LIKELY BE LARGER TO ACCOMMODATE CONSTRUCTION OPERATIONS AND EQUIPMENT.

MANUAL COMPUTATIONS FOR FAILURE SURFACE

$D_A = W \tan(45^\circ + \phi/2)$

$D_A = 8.85 + 3.77\tan 60^\circ + 4.8\tan 55^\circ + 2.05\tan 60^\circ + 3.74\tan 55^\circ + 18.98 + 10.9 + 21.13$

$D_A = \underline{82.14}^K$

$D_P = W \tan(45^\circ - \phi/2)$

$D_P = 0.16\tan 30^\circ + 0.10$

$D_P = \underline{0.10}^K$

$R_A = 2[W\tan\phi + CH\tan(45^\circ - \phi/2)]$

$R_A = 2[2.17\tan 30^\circ + 2.71\tan 20^\circ + 1.16\tan 30^\circ + 1.71\tan 20^\circ + 0.675(11) + 0.3(3)\tan 35^\circ + 0.3(1.5)\tan 35^\circ + 0.675(6) + 0.4(3) + 0.675(6)]$

$R_A = \underline{42.40}^K$

$R_B = W\tan\phi + CL$

$R_B = 30.19\tan 30^\circ + 0.675(62.13)$

$R_B = \underline{59.37}^K$

$R_P = 2[W\tan\phi + CH\tan(45^\circ + \phi/2)]$

$R_P = 2[0.08\tan 30^\circ + 0.675(0.3)]$

$R_P = \underline{0.5}^K$

$$FS = \frac{R_A + R_B + R_P}{D_A - D_P} = \frac{42.40 + 59.37 + 0.5}{82.14 - 0.10}$$

FS = $\underline{1.25}$

DRAWING
III-35

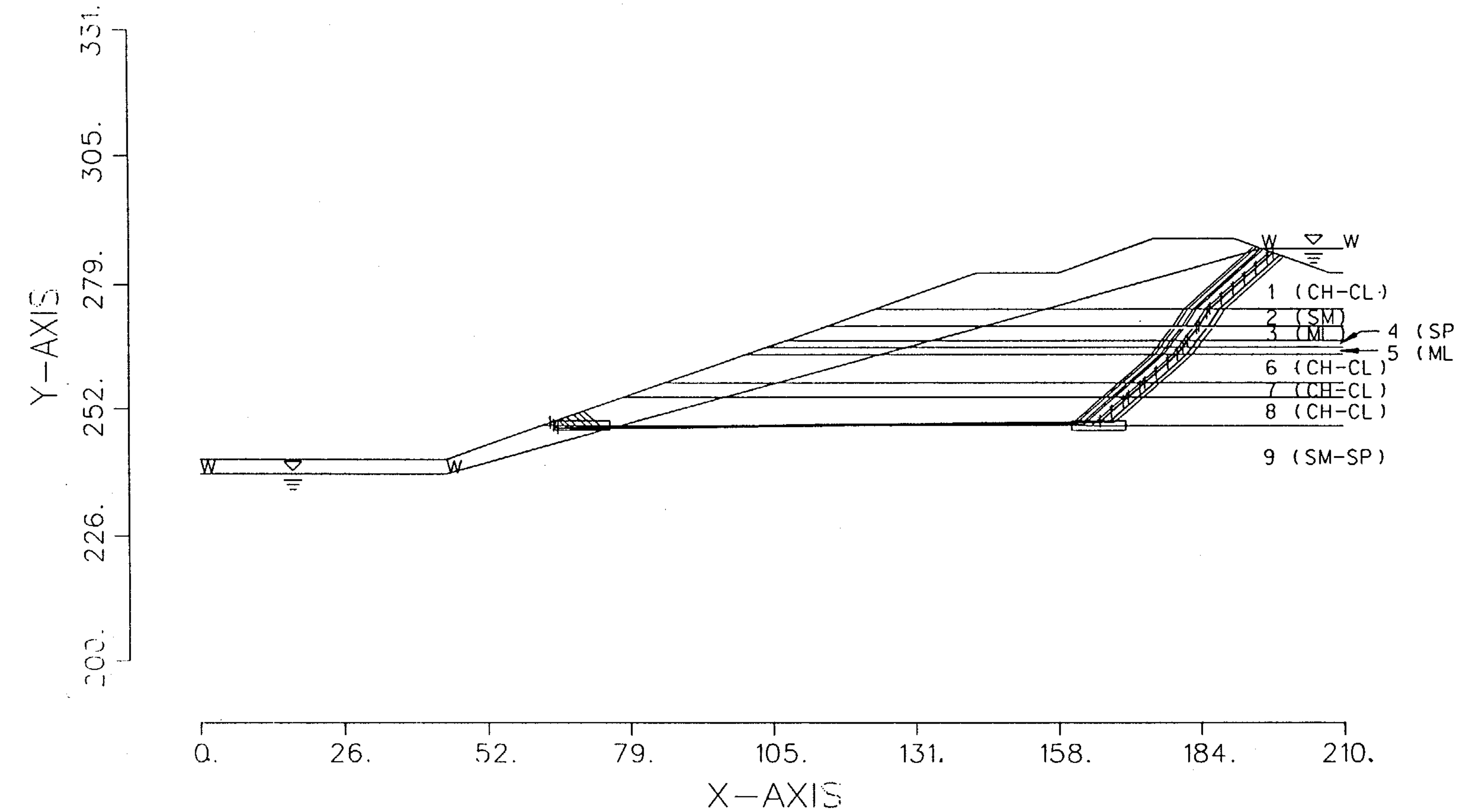
ENVIRONMENTAL RESTORATION NATURAL RESOURCES PROJECT — MISSISSIPPI RIVER AND TRIBUTARIES CONSTRUCTION

REELFOOT LAKE, TENNESSEE & KENTUCKY
STRUCTURAL EXCAVATION & COFFERDAM
STABILITY ANALYSIS

US ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
MEMPHIS, TENNESSEE

US Army Corps of Engineers Memphis District

REELFOOT LAKE SPILLWAY EXCAVATION SLOPES
1V ON 2.5H BOR. 2-RLST-95
99 SURFACES HAVE BEEN GENERATED
10 MOST CRITICAL OF SURFACES GENERATED
MINIMUM FACTOR OF SAFETY = 1.241



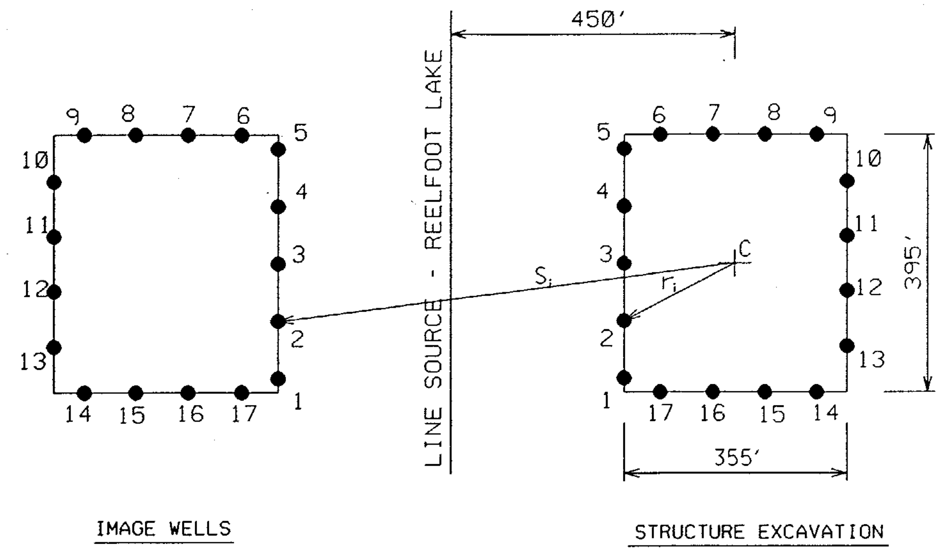
ENVIRONMENTAL RESTORATION NATURAL RESOURCES PROJECT — MISSISSIPPI RIVER AND TRIBUTARIES CONSTRUCTION

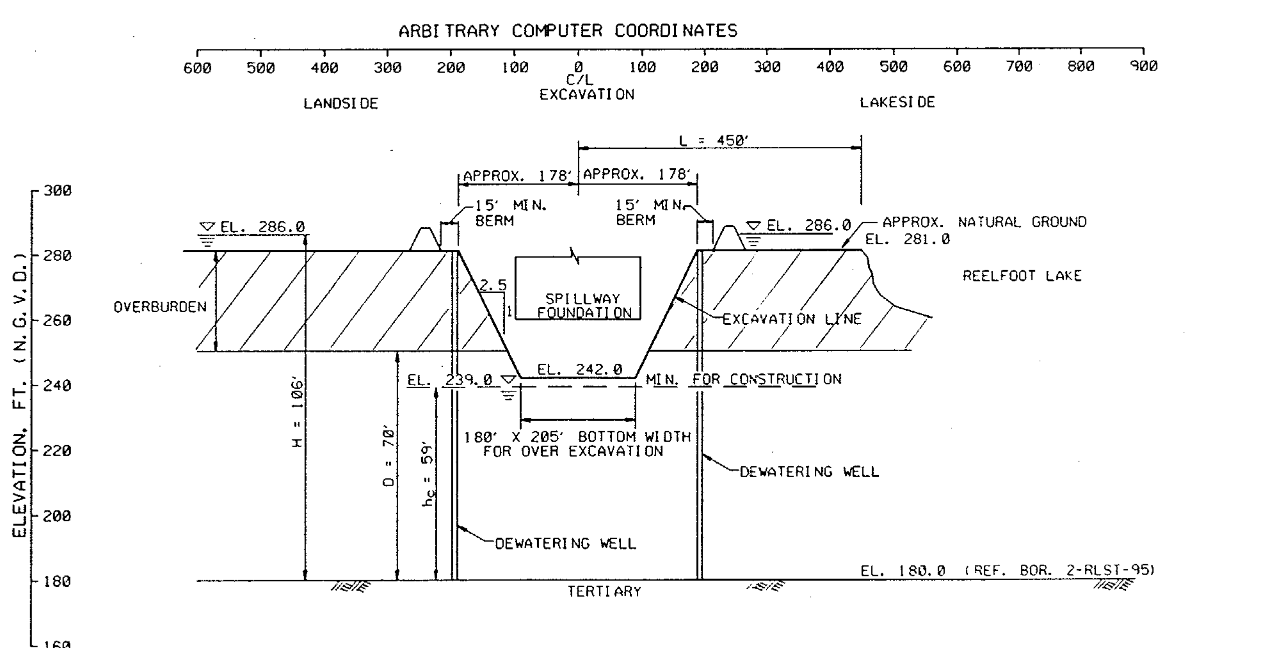
REELFOOT LAKE, TENNESSEE & KENTUCKY
STRUCTURAL EXCAVATION AND COFFERDAM STABILITY ANALYSIS

U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
MEMPHIS, TENNESSEE

U.S. Army Corps of Engineers
Memphis District

DRAWING
III-36





SECTION SHOWN IS PARALLEL WITH INLET CHANNEL THROUGH THE CENTER OF THE EXCAVATION AND SPILLWAY

ASSUMPTIONS

1. TERTIARY DEPOSITS ARE LOCATED AT APPROXIMATE EL. 180.0. BORING 2-RLST-95 DID NOT INDICATE ANY DISTINGUISHING CLAY LAYERS AT THE DEPTH WHERE TERTIARY WAS PRESUMED; HOWEVER, THERE WAS INDICATION OF SOME DENSE LIGNITE AND GRAVEL AT THIS DEPTH. ALSO, BORING 1-RLUT-93 TAKEN ON THE EAST SIDE OF THE EXISTING SPILLWAY INDICATES TERTIARY AT EL. 178. GEOLOGICAL MAPS INDICATE TERTIARY SURFACE AT EL. 200±
2. THE DEWATERING SYSTEM WILL BE REQUIRED TO LOWER THE WATER TABLE TO EL. 239.0 OR THREE FEET BELOW THE EXCAVATION.
3. HEAD WATER ELEVATION OF 286.0 IS BASED ON THE 100 YEAR FLOOD FREQUENCY OF REELFOOT LAKE.
4. REELFOOT LAKE IS LOCATED APPROXIMATELY 450 FEET AWAY FROM THE CENTERLINE OF THE EXCAVATION.
5. PERMEABILITY OF AQUIFER, K_h = 1054 X 10^{-4} CM/SEC = 0.207 FT/MIN.
6. DEPTH OF AQUIFER = EL. 249 - 180 = 69 FT, SAY 70 FT.
7. ASSUME DEEP WELLS WILL BE REQUIRED TO LOWER WATER TABLE 3 FEET BELOW THE BOTTOM OF EXCAVATION IN THE FOUNDATION SANDS. A VACUUM WELL POINT SYSTEM OR BENTONITE SLURRY CUTOFF TRENCH WILL BE REQUIRED IN THE SILTS AND SILTY SANDS FROM APPROXIMATE EL. 274 - 264 AS INDICATED BY BORING 2-RLST-95.

REFERENCE: TM 5-818-5, P. 4-15, EQ. 6

FOR A DEEP WELL SYSTEM
DETERMINE EQUIVALENT RADIUS A_E OF WELL SYSTEM

$$A_E = (4/\pi)\sqrt{b_1 \times b_2}$$

$$A_E = (4/\pi)\sqrt{(355/2)(395/2)} = 238.4 \text{ FT} = r_w$$

ESTIMATE TOTAL FLOW USING RADIUS A_E OF AN EQUIVALENT LARGE DIAMETER WELL FOR ARTESIAN FLOW CONDITIONS

$$Q_T = \frac{2\pi KD(H-h_c)}{\ln(2L/r_w)}$$

$$Q_T = \frac{2\pi(0.207)(70)(106-59)}{\ln(900/238.4)} = 3221 \text{ CFM} = 24,093 \text{ GPM}$$

ASSUME 12" DIAMETER FULLY PENTRATING DEEP WELLS WILL BE USED. MAXIMUM FLOW FOR 12" WELLS IS APPROXIMATELY 1400 GPM.

NO. OF WELLS = 24,093 GPM/1400 GPM = 17.2, SAY 17 WELLS

FLOW PER WELL 24,093/17 = 1417 GPM = 189.4 CFM = Q_w

DETERMINE HEAD AT CENTER OF EXCAVATION ASSUMING METHOD OF IMAGE WELLS AND LAYOUT SHOWN.

HEAD AT POINT C

WELL	S_i	r_i	$\ln S_i/r_i$
1	744	250	1.09
2	728	198	1.30
3	723	178	1.40
4	728	198	1.30
5	744	250	1.09
6	805	231	1.25
7	887	201	1.48
8	968	203	1.56
9	1049	236	1.49
10	1085	218	1.60
11	1078	182	1.78
12	1078	182	1.78
13	1085	218	1.60
14	1049	236	1.49
15	968	203	1.56
16	887	201	1.48
17	805	231	1.25
			24.50

F'_c = 24.5 (189.4) = 4640.3

$H-h_c = F'_c/2\pi KD = 4640.3/[2\pi(0.207)(70)] = 50.97'$

HEAD AT CENTER OF EXCAVATION
h_c = 106 - 50.97 = 55.03' < 59' O.K.

REFERENCES: 1. TM 5-818-5, 'DEWATERING AND GROUNDWATER FOR DEEP EXCAVATIONS'
2. 'FOUNDATION ENGINEERING', LEONARDS, G.A. MCGRAW-HILL, 1962, P. 241-347

NOTE: DEWATERING ANALYSIS AND LAYOUT ARE FOR ESTIMATING PURPOSES ONLY.

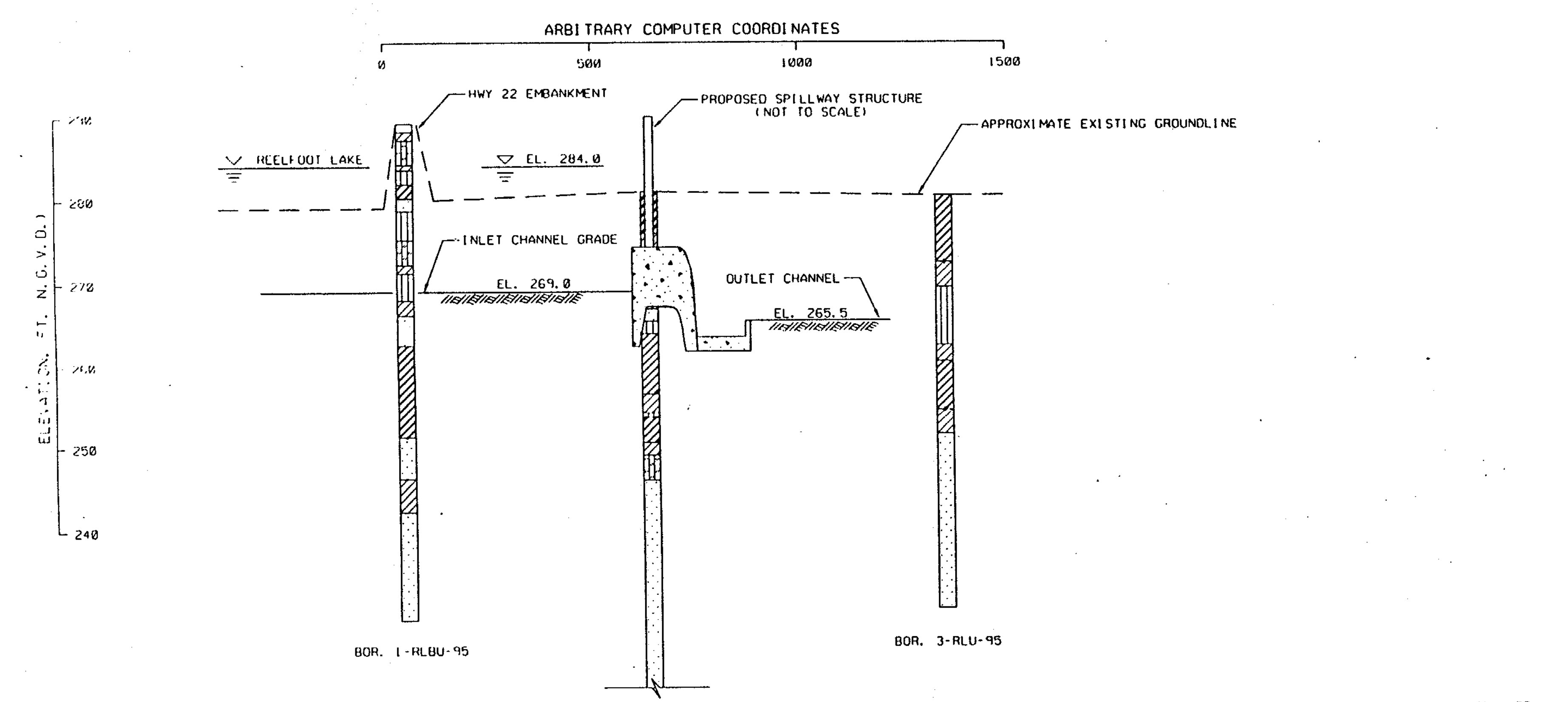
US Army Corps of Engineers Memphis District

U.S. ARMY ENGINEER DISTRICT CORPS OF ENGINEERS MEMPHIS, TENNESSEE

ENVIRONMENTAL RESTORATION NATURAL RESOURCES PROJECT
MISSISSIPPI RIVER AND TRIBUTARIES CONSTRUCTION

REELFOOT LAKE, TENNESSEE & KENTUCKY
ALTERNATIVE SPILLWAY
DEWATERING ANALYSIS

DRAWING III-37



...AGE ANALYSIS BASED ON GUIDANCE FROM TM NO. 3-424. BORINGS 1-RLBU-95 AND 2-RLST-95 AND ...TED PERMEABILITIES RESULTED IN A PREDICTED LANDSIDE HEAD AT THE TOE OF THE SPILLWAY OF ...ET WITH A LAKE ELEVATION OF 284.0 AND THE TAILWATER AT CHANNEL BOTTOM (ELEVATION 265.5) ...ING IN A NET HEAD OF ELEVATION OF 273.4.

...ER, GROUND WATER IN ALL FIVE BORINGS TAKEN FOR THE SPILLWAY, INLET AND OUTLET CHANNELS ...TED GROUND WATER LEVELS BETWEEN 277.0 AND 281.0 IN THE GENERAL AREA. THEREFORE, UPLIFT IN ... OF ELEVATION 281.0 WOULD BE ANTICIPATED. PROJECT FLOOD ON THE MISSISSIPPI RIVER LOCATED ...LFOOT LAKE (<5 MILES) IS APPROXIMATE ELEVATION 295.0 AND REELFOOT LAKE HAS A MAXIMUM ...ELEVATION AROUND 286.0.

...THICKNESS ABOVE THE LOWER SAND AQUIFER BASED ON A CHANNEL GRADE OF ELEVATION 265.5 ...FROM BORING 3-RLU-95 IS APPROXIMATELY 14 FEET.

...DETERMINE MAXIMUM HEAD FOR UPLIFT FACTOR OF SAFETY EQUAL TO 1.3.

ASSUME CLAY AND OUTLET CHANNEL PROTECTION @ 115 LBS/FT^3

FS_{UPLIFT} = (115-62.4)14 / ΔH(62.4) = 1.3   ΔH = 9.1 FT.

HEAD ELEVATION = 9.1 + 265.5 = 274.6

274.6 < 281.0   ∴ UPLIFT RELIEF REQUIRED - ASSUME RELIEF WELLS

TWO ADDITIONAL BORINGS WILL BE TAKEN AT THE SITE. FOUR PIEZOMETERS WILL ALSO BE INSTALLED AT LEAST 6 MONTHS BEFORE CONSTRUCTION TO ASSESS GROUND WATER LEVELS. BASED ON EXISTING AND NEW INFORMATION, A RELIEF WELL SYSTEM WILL BE DESIGNED DOWNSTREAM OF THE SPILLWAY STRUCTURE TO A MINIMUM OF 100 FEET PAST THE RIPRAP PROTECTION TO RELIEVE EXCESSIVE UPLIFT PRESSURES.

ESTIMATED LENGTH - 300 FEET.

WELL SPACING - ASSUME 60 FEET.

REQUIRED WELLS - 2[300/60 + 1] = 12 WELLS

TYPICAL WELL DESIGN

LOCATION - 10' FROM TOP BANK

SIZE - 8' DIAMETER WELLS

MATERIAL - STAINLESS STEEL

RISER - 35', SCREEN - 50'

TOTAL LENGTH - 85'

HORIZONTAL DESCHARGE INTO RIPRAP SLOPE AT ELEVATION 272.0 IN THE OUTLET CHANNEL.

ENVIRONMENTAL RESTORATION NATURAL RESOURCES PROJECT — MISSISSIPPI RIVER AND TRIBUTARIES CONSTRUCTION

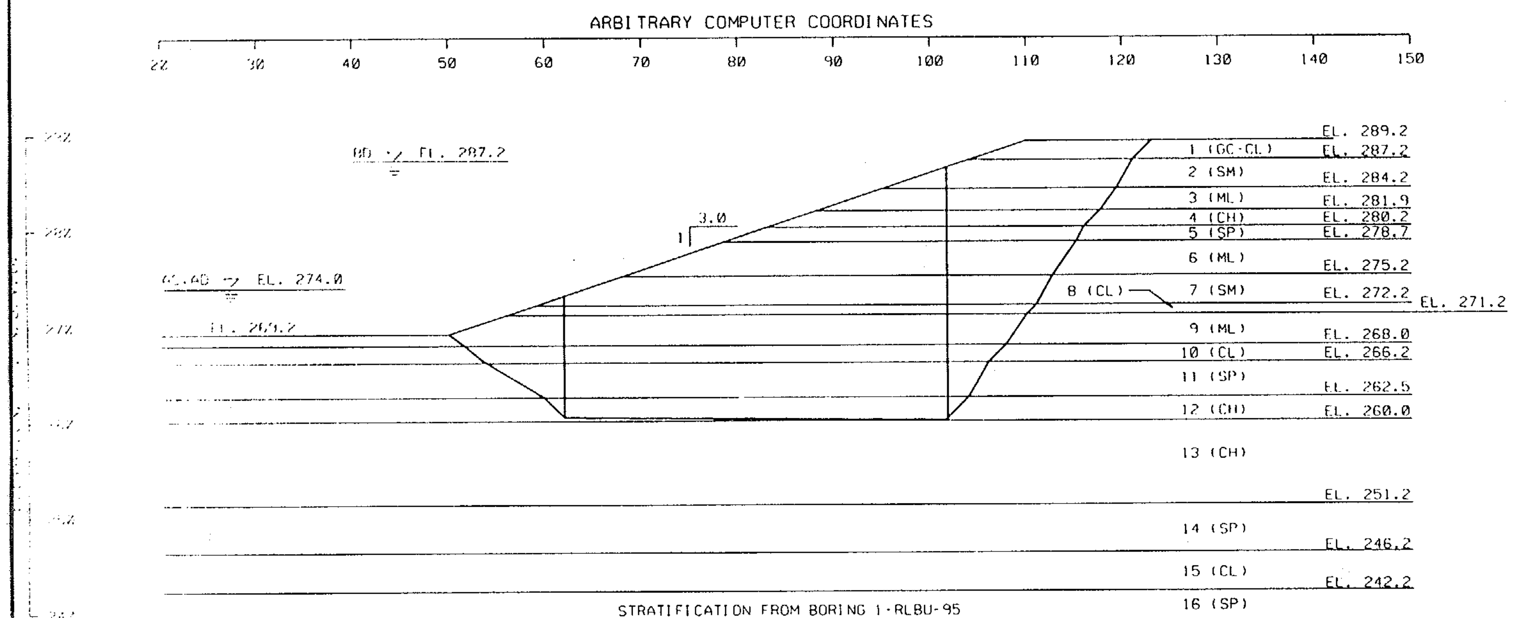
REELFOOT LAKE, TENNESSEE & KENTUCKY

SEEPAGE AND PRESSURE RELIEF ANALYSIS

US ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
MEMPHIS, TENNESSEE

US Army Corps of Engineers
Memphis District

DRAWING



BORING 1-RLBU-95
COMPUTER RESULTS - ARC METHOD (GEOSLOPE)

ARC DATA (FT.)			FACTORS OF SAFETY	
CENTER		RADIUS		
X	Y	R	AC	SD
76.4	314.6	54.0	1.69	1.37
78.5	312.9	52.2	1.70	1.38
72.3	321.8	61.1	1.73	1.40
73.9	309.5	48.4	1.73	1.41
69.8	315.5	54.6	1.76	1.44
74.3	317.1	55.1	1.76	1.45
71.8	319.1	57.1	1.76	1.46

BORING 1-RLBU-95
COMPUTER RESULTS - WEDGE METHOD (GEOSLOPE)

NEUTRAL BLOCK BASE COORDINATES			FACTORS OF SAFETY	
X_L	X_R	ELEV.	AC	SD
91.8	107.0	281.9	4.85	4.28
86.8	106.1	280.2	3.19	2.29
82.9	106.7	278.7	3.03	2.05
71.6	102.0	275.5	2.75	1.69
63.7	104.2	272.2	1.96	1.52
62.6	104.8	271.2	2.06	1.77
57.3	92.6	268.0	1.97	1.68
51.6	101.1	266.2	1.62	1.36
62.0	99.9	262.5	1.50	1.25
61.1	101.5	260.0	1.42	1.20 *
67.8	97.2	251.2	1.49	1.33
66.7	99.0	246.2	1.63	1.45
68.0	97.4	242.2	1.60	1.43

*CASE PRESENTED MANUALLY

LONG TERM STABILITY

$$\text{FS} = \frac{2.5[10.8 \tan 28° + 7.5 \tan 30° + 1.7 \tan 25°]}{20}$$

FS = [illegible]

SOIL SHEAR STRENGTHS, BORING 1-RLBU-95

SOIL NO.	SOIL TYPE	ELEVATION (N.G.V.D.)	UNIT WT. (PCF)	Q		R		S	
				ϕ (°)	C (PSF)	ϕ (°)	C (PSF)	ϕ (°)	C (PSF)
1	GC-CL	289.2-287.2	120	0	750	14	375	28	0
2	SM	287.2-284.2	125	30	0	30	0	30	0
3	ML	284.2-281.9	120	20	300	20	300	28	0
4	CH	281.9-280.2	115	0	550	12	275	25	0
5	SP	280.2-278.7	125	30	0	30	0	30	0
6	ML	278.7-275.2	120	20	300	20	300	28	0
7	SM	275.2-272.2	125	30	0	30	0	30	0
8	CL	272.2-271.2	120	0	400	14	200	28	0
9	ML	271.2-268.0	120	20	300	20	300	28	0
10	CL	268.0-266.2	120	0	400	14	200	28	0
11	SP	266.2-262.5	125	30	0	30	0	30	0
12	CH	262.5-260.0	110	0	400	10	200	21	0
13	CH	260.0-251.2	110	0	625	10	315	21	0
14	SP	251.2-246.2	125	30	0	30	0	30	0
15	CL	246.2-242.2	120	0	715	13	360	26	0
16	SP	BELOW 242.2	125	35	0	35	0	35	0

MANUAL COMPUTATIONS FOR FAILURE SURFACE

$D_A = W \tan(45° + \phi/2)$

$D_A = 0.24 + 0.73 \tan 60° + 1.28 \tan 55° + 1.60 + 0.94 \tan 60° + 3.71 \tan 55° + 3.15 \tan 60° + 1.99 + 4.55 \tan 55° + 3.83 + 4.66 \tan 60° + 5.52$

$D_A = \underline{43.30}^K$

$D_P = W \tan(45° - \phi/2)$

$D_P = 1.36 + 2.33 \tan 30° + 0.3 + 0.08 \tan 35°$

$D_P = \underline{3.06}^K$

$R_A = 2[W \tan\phi + CH \tan(45° - \phi/2)]$

$R_A = 2[0.57 \tan 30° + 0.84 \tan 20° + 0.56 \tan 30° + 2.11 \tan 20° + 1.77 \tan 30° + 2.74 \tan 20° + 2.95 \tan 30° + 0.75(2) + 0.3(2.3) \tan 35° + 0.55(1.7) + 0.3(3.5) \tan 35° + 0.4(1.0) + 0.3(3.2) \tan 35° + 0.4(1.8) + 0.4(2.4)]$

$R_A = \underline{23.71}^K$

$R_B = W \tan\phi + CL$

$R_B = 0.4(39.6)$

$R_B = \underline{15.8}^K$

$R_P = 2[W \tan\phi + CH \tan(45° + \phi/2)]$

$R_P = 2[2.33 \tan 30° + 0.08 \tan 20° + 0.4(2.1) + 0.4(1.8) + 0.3(1.2) \tan 55°]$

$R_P = \underline{6.90}^K$

$$FS = \frac{R_A + R_B + R_P}{D_A - D_P} = \frac{23.71 + 15.8 + 6.90}{43.30 - 3.06}$$

$FS = \underline{1.15}$

DRAWING III-39

ENVIRONMENTAL RESTORATION NATURAL RESOURCES PROJECT

MISSISSIPPI RIVER AND TRIBUTARIES CONSTRUCTION

REELFOOT LAKE, TENNESSEE & KENTUCKY
INLET CHANNEL
STABILITY ANALYSIS

US ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
MEMPHIS, TENNESSEE

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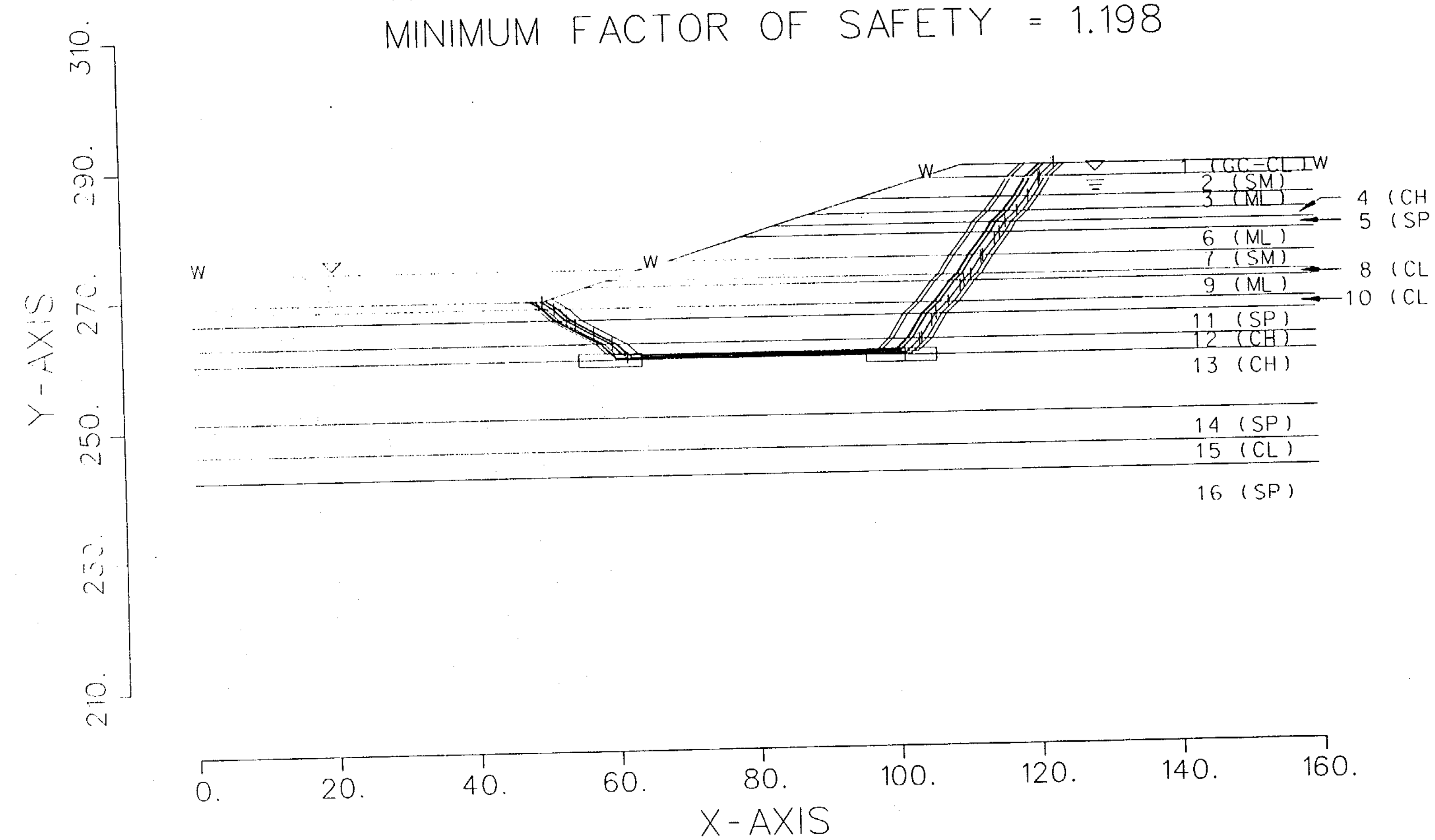
Checked by: NEW | Checked by: NEW | Feature table: NONE | Color table: | Plot date: 01-JUN-1998
Submitted by: RONALD D. SMITH, P.E. Chief, Geotechnical Design Section

US Army Corps of Engineers Memphis District

REELFOOT LAKE BORING 1-RLBU-95
1V ON 3.0H CHANNEL SLOPES
99 SURFACES HAVE BEEN GENERATED
10 MOST CRITICAL OF SURFACES GENERATED
MINIMUM FACTOR OF SAFETY = 1.198



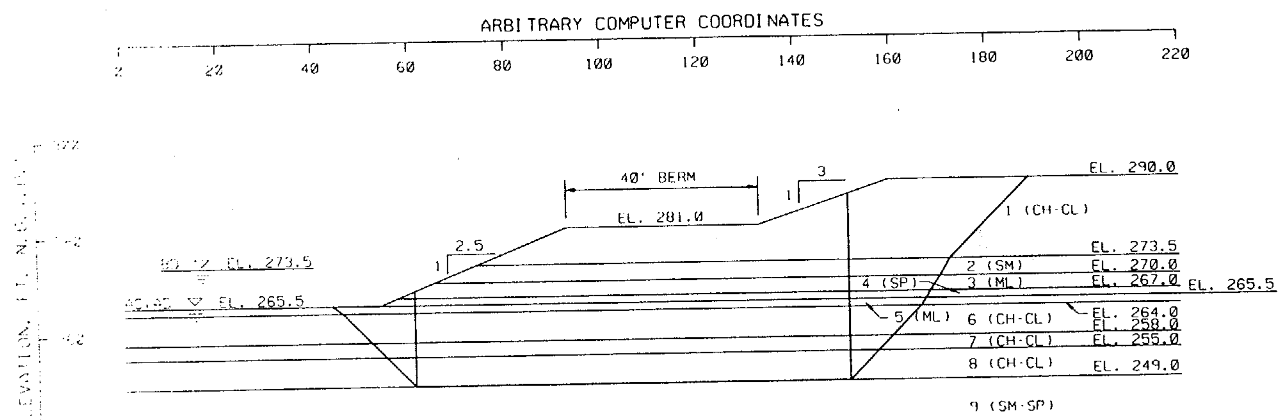
ENVIRONMENTAL RESTORATION NATURAL RESOURCES PROJECT
MISSISSIPPI RIVER AND TRIBUTARIES CONSTRUCTION

REELFOOT LAKE, TENNESSEE & KENTUCKY
INLET CHANNEL
STABILITY ANALYSIS

U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
MEMPHIS, TENNESSEE

US Army Corps of Engineers
Memphis District

DRAWING
III-40



STRATIFICATION FROM BORING 2-RLST-95

LONG TERM STABILITY

$$FS = \frac{\tan \phi}{\tan i} = \frac{2.5(7.5 \tan 23° + 3.5 \tan 28° + 4.5 \tan 30°)}{15.5}$$

FS = 1.23 GOVERNS

BORING 2-RLST-95
COMPUTER RESULTS - ARC METHOD (GEOSLOPE)

ARC DATA (FT.)			FACTORS OF SAFETY	
CENTER		RADIUS		
X	Y	R	AC	SD
72.5	288.9	33.8	1.95	1.89
95.0	347.6	98.5	2.06	2.02
96.7	344.7	95.0	2.10	2.05
85.2	375.8	120.5	2.22	2.12
100.1	341.3	89.8	2.19	2.15

BORING 2-RLST-95
COMPUTER RESULTS - WEDGE METHOD (GEOSLOPE)

NEUTRAL BLOCK BASE COORDINATES			FACTORS OF SAFETY	
X_L	X_R	ELEV.	AC	SD
77.3	156.3	273.5	4.13	4.07
59.7	153.7	255.0	1.60	1.56
60.9	151.2	249.0	1.77	1.75
68.3	90.4	270.0	3.35	2.93
60.2	87.3	267.0	2.81	2.17
57.4	85.9	265.5	2.73	2.07
56.8	85.7	264.0	2.69	2.39
59.3	85.3	255.0	1.74	1.68
58.9	86.9	249.0	1.84	1.78

SOIL SHEAR STRENGTHS - BORING 2-RLST-95

SOIL NO.	SOIL TYPE	ELEVATION (N.G.V.D.)	UNIT WT. (PCF)	Q φ (°)	Q C (PSF)	R φ (°)	R C (PSF)	S φ (°)	S C (PSF)
1	CH-CL	290.0-273.5	115	0	675	12	335	23	0
2	SM	273.5-270.0	125	30	0	30	0	30	0
3	ML	270.0-267.0	120	20	300	20	300	28	0
4	SP	267.0-265.5	125	30	0	30	0	30	0
5	ML	265.5-264.0	120	20	300	20	300	28	0
6	CH-CL	264.0-258.0	110	0	675	10	335	21	0
7	CH-CL	258.0-255.0	110	0	400	10	200	21	0
8	CH-CL	255.0-249.0	110	0	400	10	200	21	0
9	SM-SP	BELOW 249.0	125	30	0	30	0	30	0

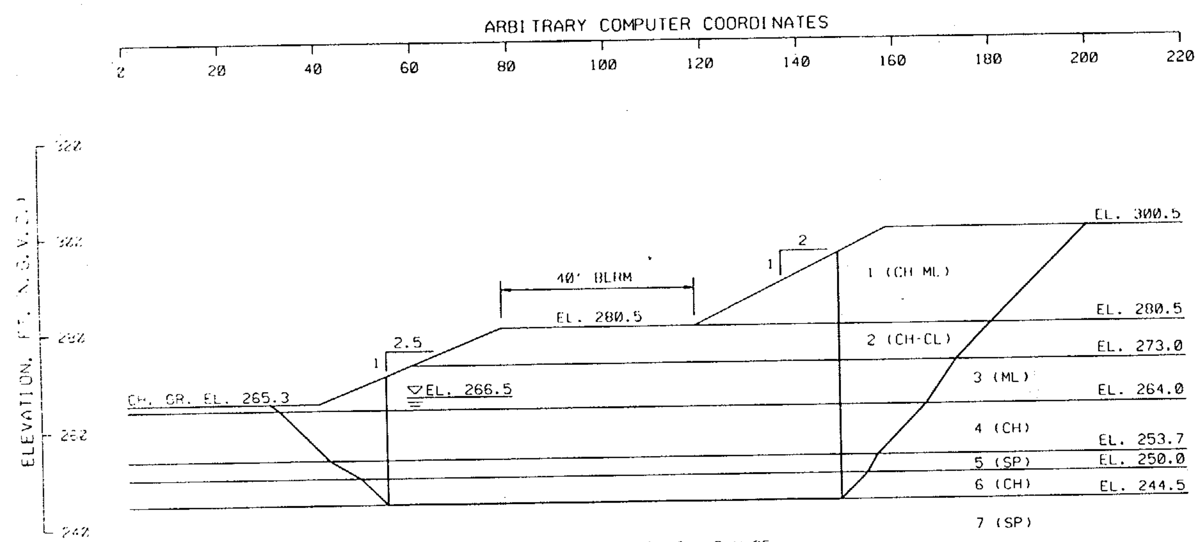
DRAWING III-41

ENVIRONMENTAL RESTORATION NATURAL RESOURCES PROJECT
MISSISSIPPI RIVER AND TRIBUTARIES CONSTRUCTION

REELFOOT LAKE, TENNESSEE & KENTUCKY
OUTLET CHANNEL
STABILITY ANALYSIS

U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
MEMPHIS, TENNESSEE

US Army Corps of Engineers
Memphis District



STRATIFICATION FROM BORING 4-RLU-95

BORING 4-RLU-95
COMPUTER RESULTS
WEDGE METHOD (GEOSLOPE)

NEUTRAL BLOCK BASE COORDINATES			FS
X_L	X_R	Y_{ELEV}	AC
59.3	152.5	273.0	2.16
41.9	151.1	264.0	1.68
52.2	148.8	253.7	1.27
56.4	149.0	250.0	1.33
54.0	148.0	244.5	1.24*
52.2	151.4	253.7	2.13
53.2	152.4	250.0	2.09
55.0	150.8	244.5	2.00

*FAILURE SURFACE SHOWN

LONG TERM STABILITY

$$FS = \frac{TAN\ \phi}{TAN\ i} = \frac{2.5[7.5\ TAN\ 21° + 7.7\ TAN\ 28°]}{15.2}$$

$FS = \underline{1.15}$

DESIGN STRENGTHS - BORING 4-RLU-95

SOIL NO.	SOIL TYPE	ELEVATION (N.G.V.D.)	UNIT WT. (PCF)	Q ϕ (°)	Q C (PSF)	S ϕ (°)	S C (PSF)
1	CH ML	300.5-280.5	110	0	500	21	0
2	CH-CL	280.5-273.0	120	0	670	21	0
3	ML	273.0-264.0	120	20	300	28	0
4	CH	264.0-253.7	115	0	650	24	0
5	SP	253.7-250.0	125	30	0	30	0
6	CH	250.0-244.5	115	0	750	21	0
7	SP	BELOW 244.5	125	32	0	32	0

MANUAL COMPUTATIONS FOR FAILURE SURFACE

$D_A = W\ TAN\ (45° + \phi/2)$
$D_A = 22.00 + 19.88 + 22.80\ TAN\ 55° + 44.08 + 9.61\ TAN\ 60° + 24.99$
$D_A = \underline{160.16}^K$

$D_P = W\ TAN\ (45° - \phi/2)$
$D_P = 8.02 + 5.97\ TAN\ 30° + 3.61 + 0.07\ TAN\ 35°$
$D_P = \underline{15.13}^K$

$R_A = 2[W\ TAN\ \phi + CH\ TAN\ (45° - \phi/2)]$
$R_A = 2[0.5(20) + 0.67(7.5) + 22.80\ TAN\ 20° + 0.3(9)\ TAN\ 35° + 0.65(10.3) + 9.61\ TAN\ 30° + 0.75(5.5)]$
$R_A = \underline{83.16}^K$

$R_B = W\ TAN\ \phi + CL$
$R_B = 0.75(94)$
$R_B = \underline{70.5}^K$

$R_P = 2[W\ TAN\ \phi + CH\ TAN\ (45° + \phi/2)]$
$R_P = 2[0.75(5.5) + 5.97\ TAN\ 30° + 0.65(10.3) + 0.07\ TAN\ 20° + 0.3(1.3)\ TAN\ 55°]$
$R_P = \underline{30.73}^K$

$$FS = \frac{R_A + R_B + R_P}{D_A - D_P} = \frac{83.16 + 70.5 + 30.73}{160.16 - 15.13}$$

$FS = \underline{1.27}$

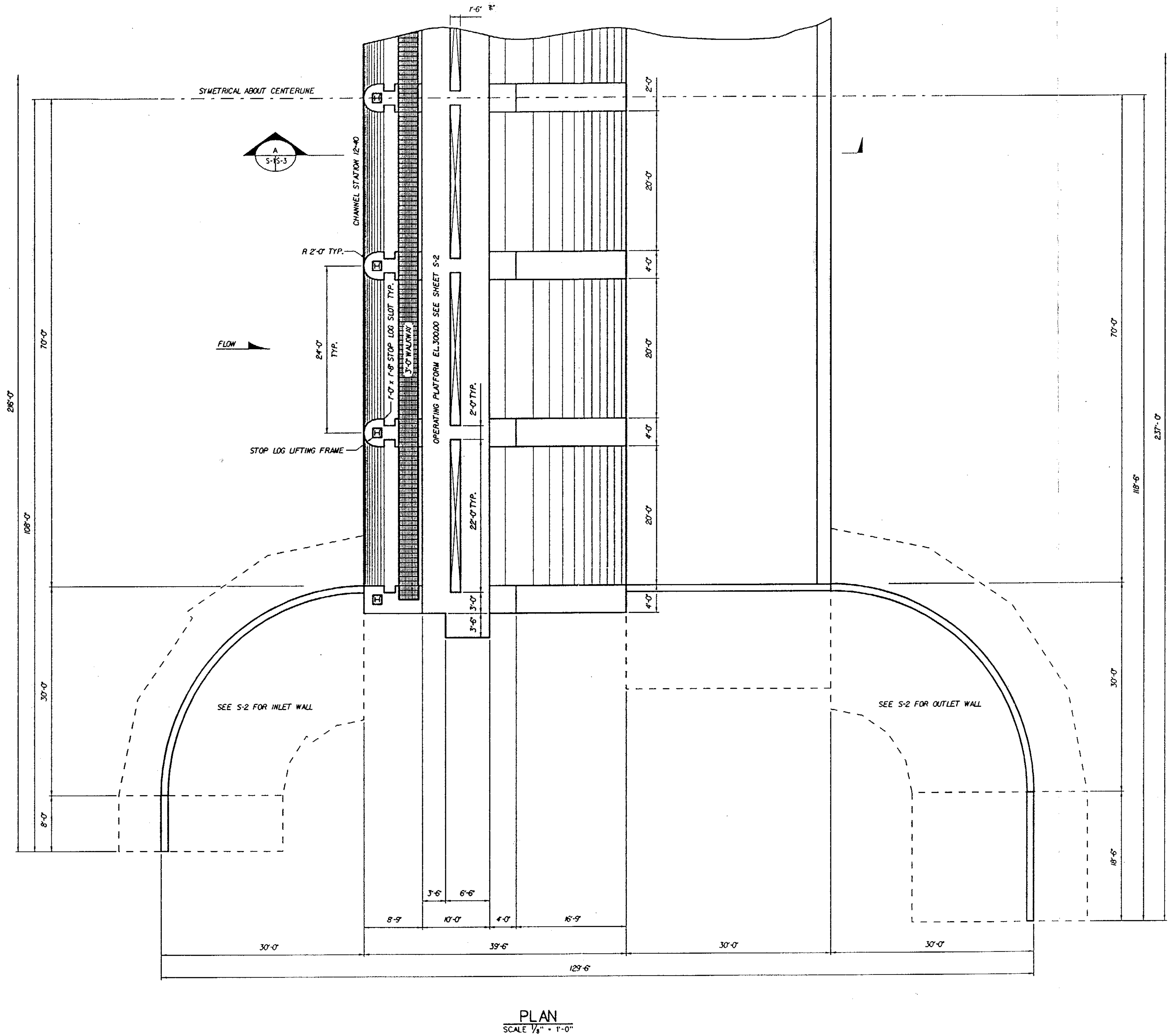
ENVIRONMENTAL RESTORATION NATURAL RESOURCES PROJECT
MISSISSIPPI RIVER AND TRIBUTARIES CONSTRUCTION

REELFOOT LAKE, TENNESSEE & KENTUCKY
OUTLET CHANNEL
STABILITY ANALYSIS

US ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
MEMPHIS, TENNESSEE

US Army Corps of Engineers Memphis District

DRAWING
III-42



PLAN
SCALE 1/8" = 1'-0"

US Army Corps of Engineers Memphis District

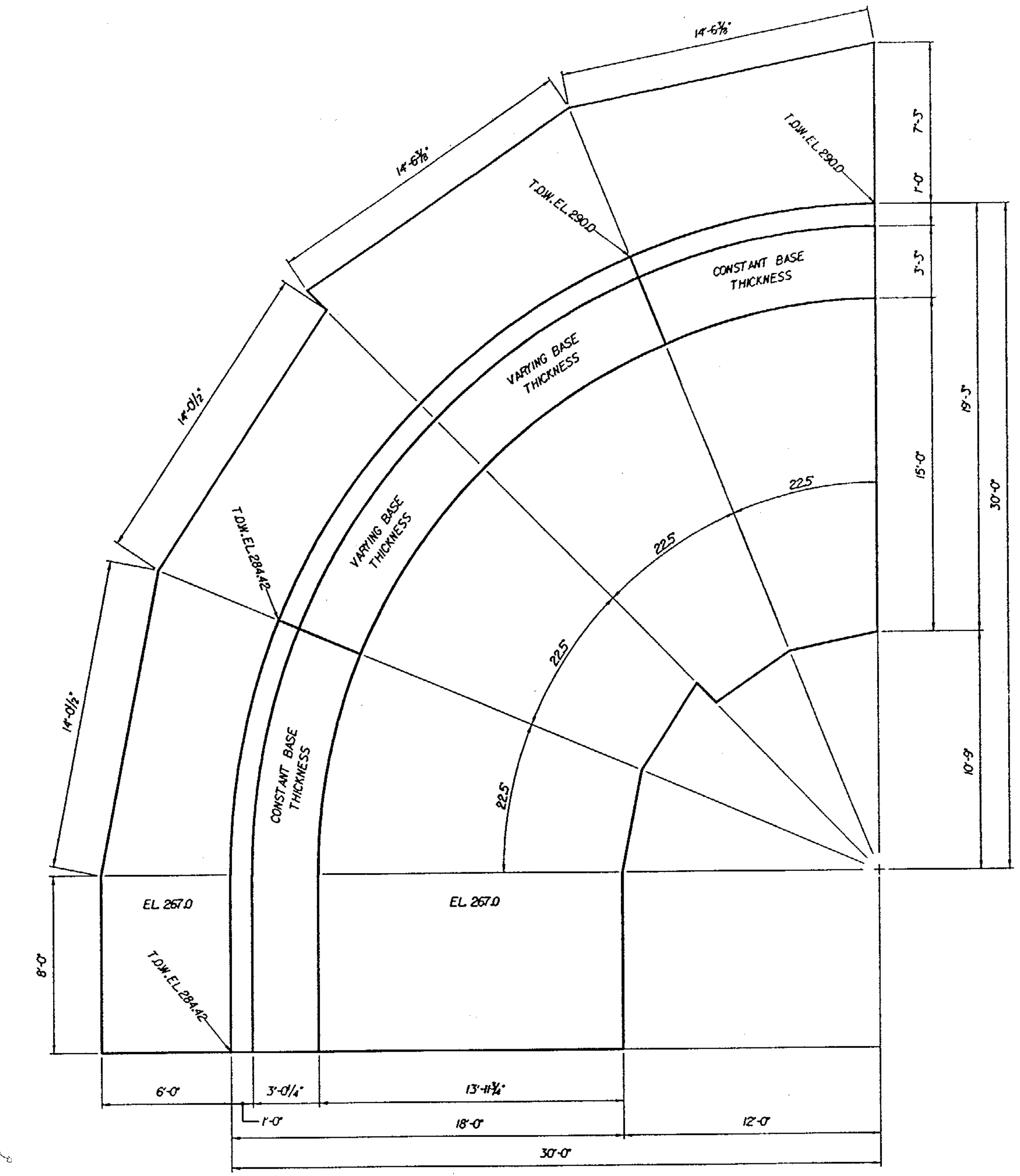
U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
MEMPHIS, TENNESSEE

ENVIRONMENTAL RESTORATION
NATURAL RESOURCES PROJECT

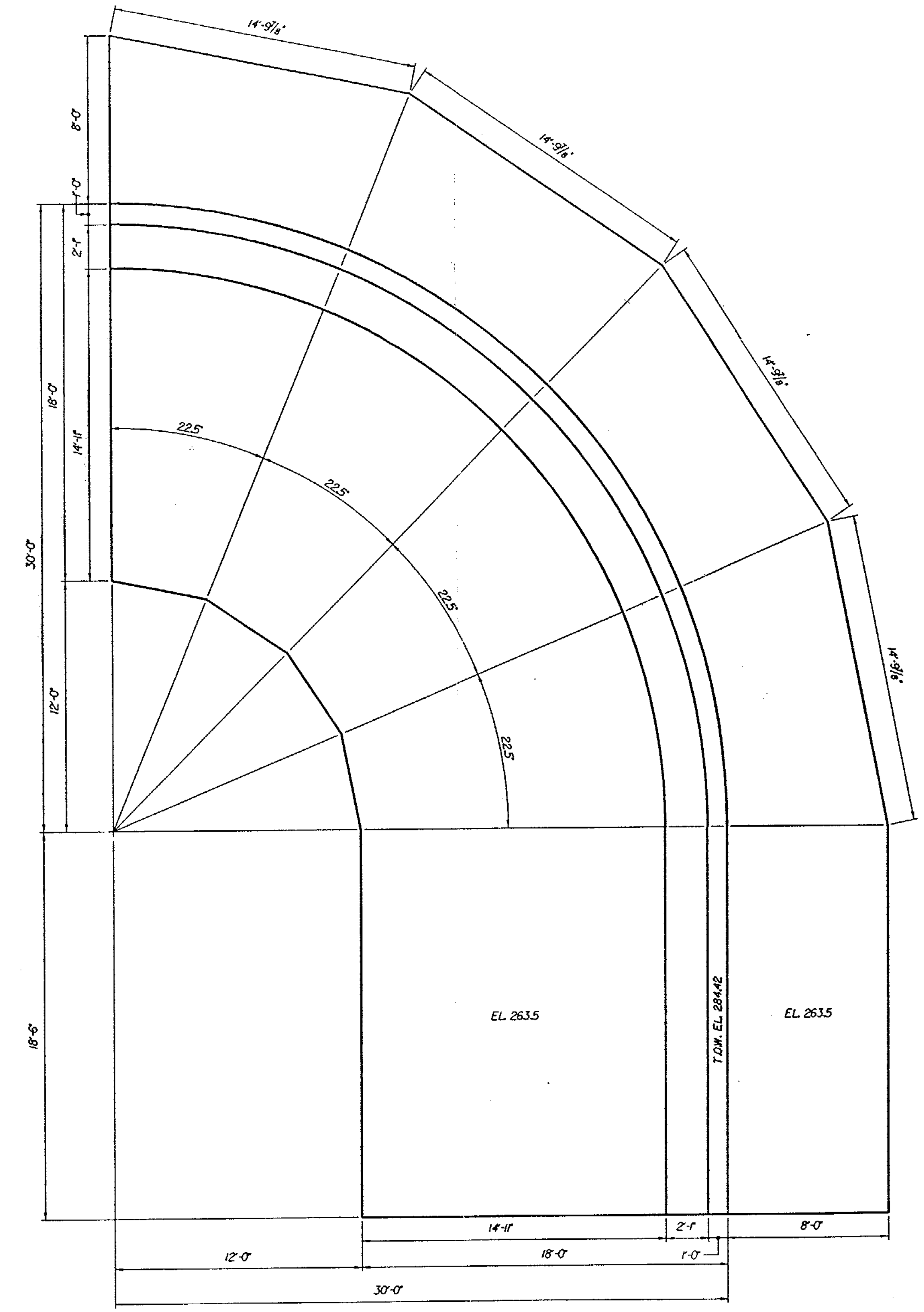
MISSISSIPPI RIVER AND TRIBUTARIES
CONSTRUCTION

REELFOOT LAKE, TENNESSEE & KENTUCKY
ALTERNATIVE SPILLWAY
SPILLWAY PLAN

DRAWING
III-43



INLET WING WALL
SCALE ¼" = 1'-0"



OUTLET WING WALL
SCALE ¼" = 1'-0"

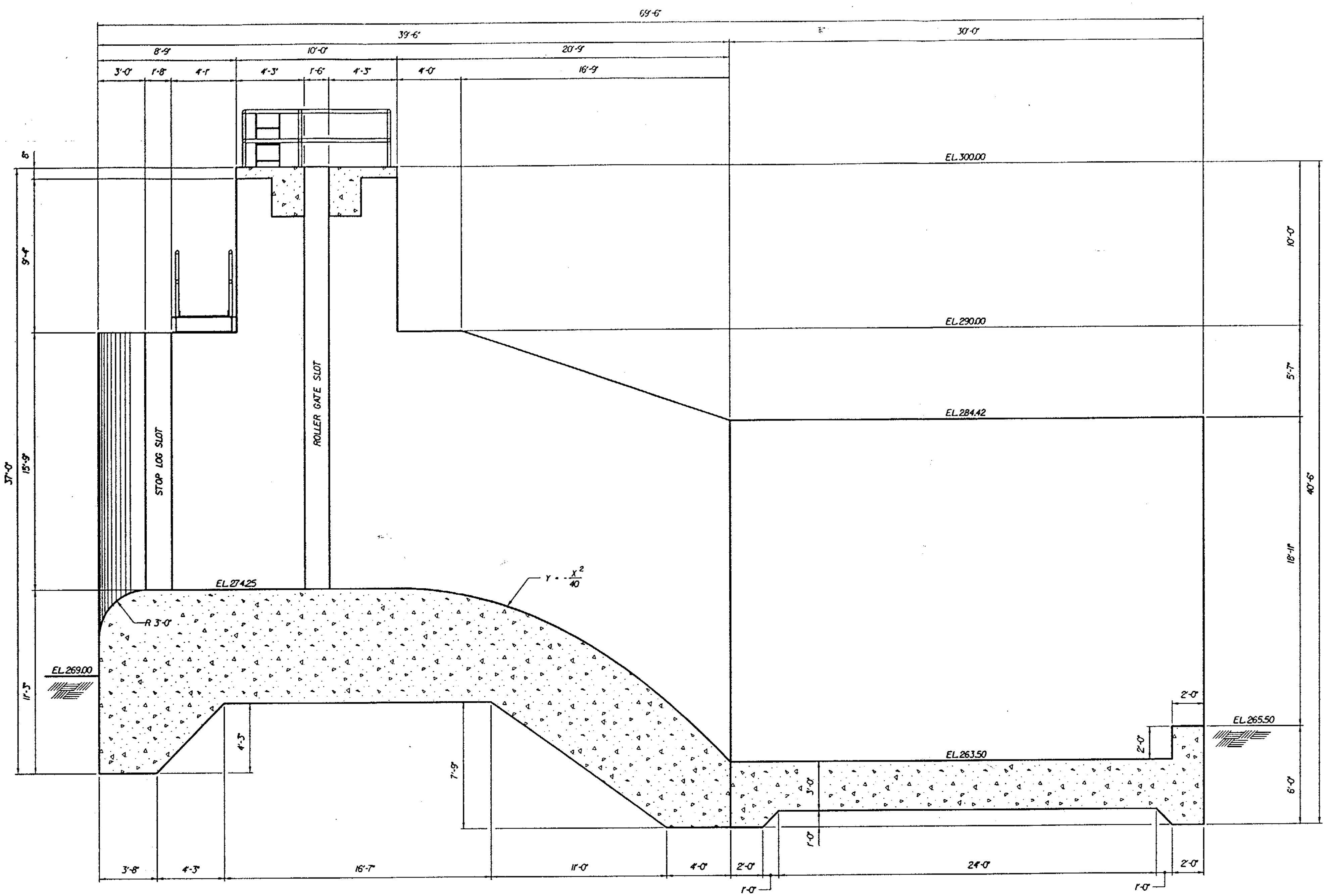
US Army Corps of Engineers Memphis District

U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
MEMPHIS, TENNESSEE

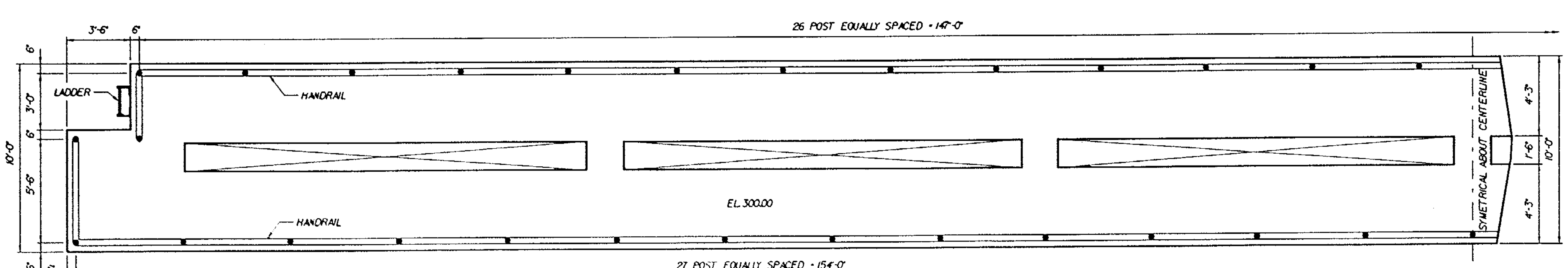
ENVIRONMENTAL RESTORATION NATURAL RESOURCES PROJECT
MISSISSIPPI RIVER AND TRIBUTARIES CONSTRUCTION

REELFOOT LAKE, TENNESSEE & KENTUCKY
ALTERNATIVE SPILLWAY
SPILLWAY SECTION

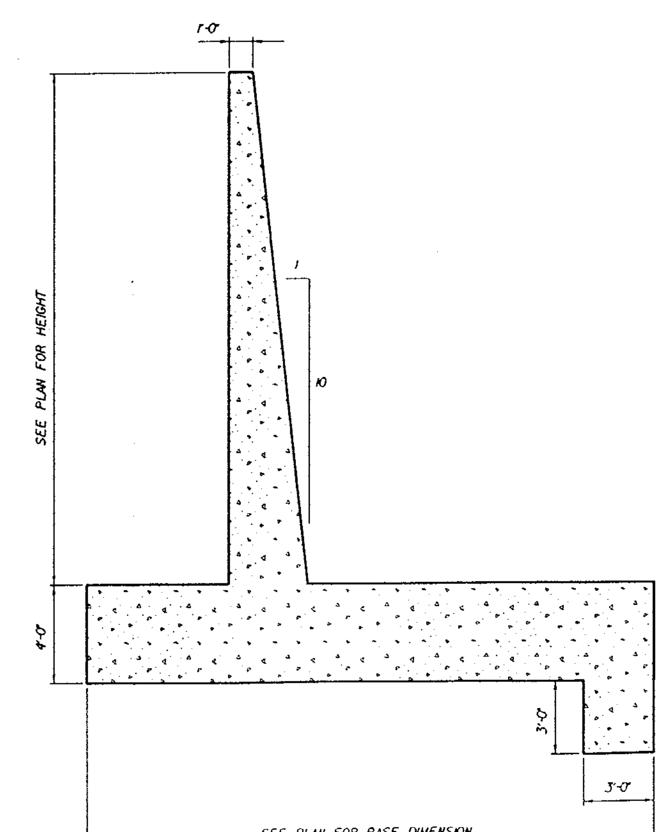
DRAWING
III-44



SECTION A / S-TS-2
SCALE 1/4" = 1'-0"



OPERATING PLATFORM
SCALE 1/4" = 1'-0"



TYPICAL WING WALL SECTION
SCALE 1/4" = 1'-0"

US Army Corps of Engineers Memphis District

Symbol	Description	Date	Appr.

U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
MEMPHIS, TENNESSEE

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Checked by: NEN	Checked by: NEN	Feature table: NONE	Color table: bw.ctb	Plot date: 31-MAY-1996
Submitted by: RONALD O. SMITH, P.E. Chief, Geotechnical Design Section				
Design file name				
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ENVIRONMENTAL RESTORATION NATURAL RESOURCES PROJECT
MISSISSIPPI RIVER AND TRIBUTARIES CONSTRUCTION

REELFOOT LAKE, TENNESSEE & KENTUCKY
ALTERNATIVE SPILLWAY
SPILLWAY SECTIONS

DRAWING
III-45

SUMMARY OF MAIN SPILLWAY STRUCTURE STABILITY ANALYSIS

LOADING CASE	DESCRIPTION	RESULTANTS VERTICAL (K)	RESULTANTS HORIZONTAL (K)	RESULTANTS MOMENT (FT•K)	$X_{c.g.}$ (FT)	RESULTANT RATIO	ECCENTRICITY (FT)	$q_{max.}$ (KSF)	$q_{min.}$ (KSF)	SLIDING FACTOR OF SAFETY	BEARING FACTOR OF SAFETY
1	CONSTRUCTION WITH EQ.: OPERATING EQUIPMENT SUCH AS GATES NOT INSTALLED. MOMENTS ABOUT HEEL & TOE										
	MOMENTS ABOUT HEEL	9452	1418	86107	9.11	0.27	10.61	4.23	-0.99		
	MOMENTS ABOUT TOE	9452	1418	172258	18.22	0.46	1.50	1.99	1.25	2.7	>8.8
1A	CONSTRUCTION WITH EQ.: ALL GATE EQUIPMENT INSTALLED										
	MOMENTS ABOUT HEEL	9705	1456	169214	17.44	0.44	2.28	2.24	1.09		
	MOMENTS ABOUT TOE	9705	1456	178287	18.37	0.47	1.35	2.00	1.32	2.7	8.8
2	HIGH NORMAL POOL W/O EQ.										
2U	HORIZONTAL FORCES NOT EQUAL ZERO	5159	2815	76661	14.86	0.38	4.86	1.54	0.23	2.0	7.7
2B	HORIZONTAL FORCES EQUAL ZERO	5159	0	82290	15.95	0.40	3.77	1.39	0.38		
3	HIGH NORMAL POOL WITH EQ.										
3U	HORIZONTAL FORCES NOT EQUAL ZERO	5159	4504	53741	10.42	0.26	9.30	2.13	-0.37	1.03	1.7
3B	HORIZONTAL FORCES EQUAL ZERO	5159	0	62749	12.16	0.31	7.56	1.90			
4	UNUSUAL LOADING - 100 YR.										
4U	HORIZONTAL FORCES NOT EQUAL ZERO	4992	3310	67301	13.48	0.34	6.24	1.67	0.04	1.7	5.5
4B	HORIZONTAL FORCES EQUAL ZERO	4992	0	73921	14.81	0.38	4.91	1.49	0.22		
5	LOW POOL WITH EQ.										
5U	HORIZONTAL FORCES NOT EQUAL ZERO	5935	2609	87496	14.74	0.37	4.98	1.79	0.25	1.8	5.6
5B	HORIZONTAL FORCES EQUAL ZERO	5935	0	92714	15.62	0.40	4.10	1.65	0.38		
6	LOW POOL W/O EQ.										
6U	HORIZONTAL FORCES NOT EQUAL ZERO	5935	1041	108473	18.28	0.46	4.98	1.24	0.79	4.5	17.1
6B	HORIZONTAL FORCES EQUAL ZERO	5935	0	110554	18.63	0.47	4.10	1.19	0.85		

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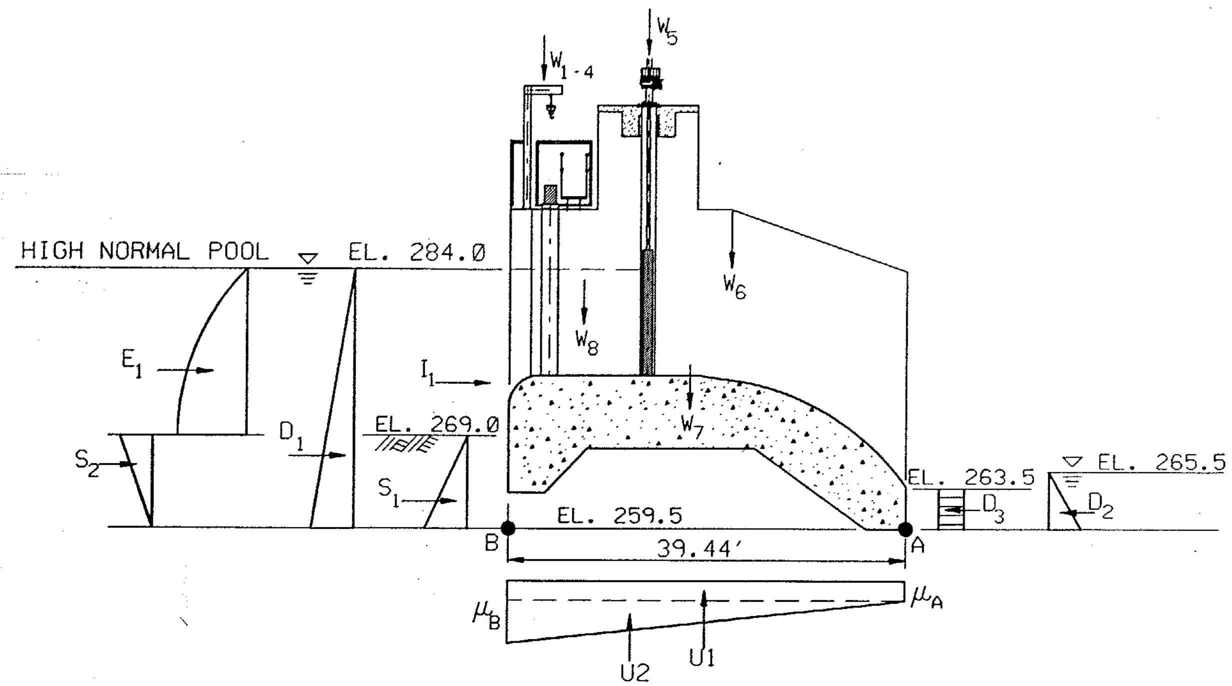
ENVIRONMENTAL RESTORATION NATURAL RESOURCES PROJECT — MISSISSIPPI RIVER AND TRIBUTARIES CONSTRUCTION

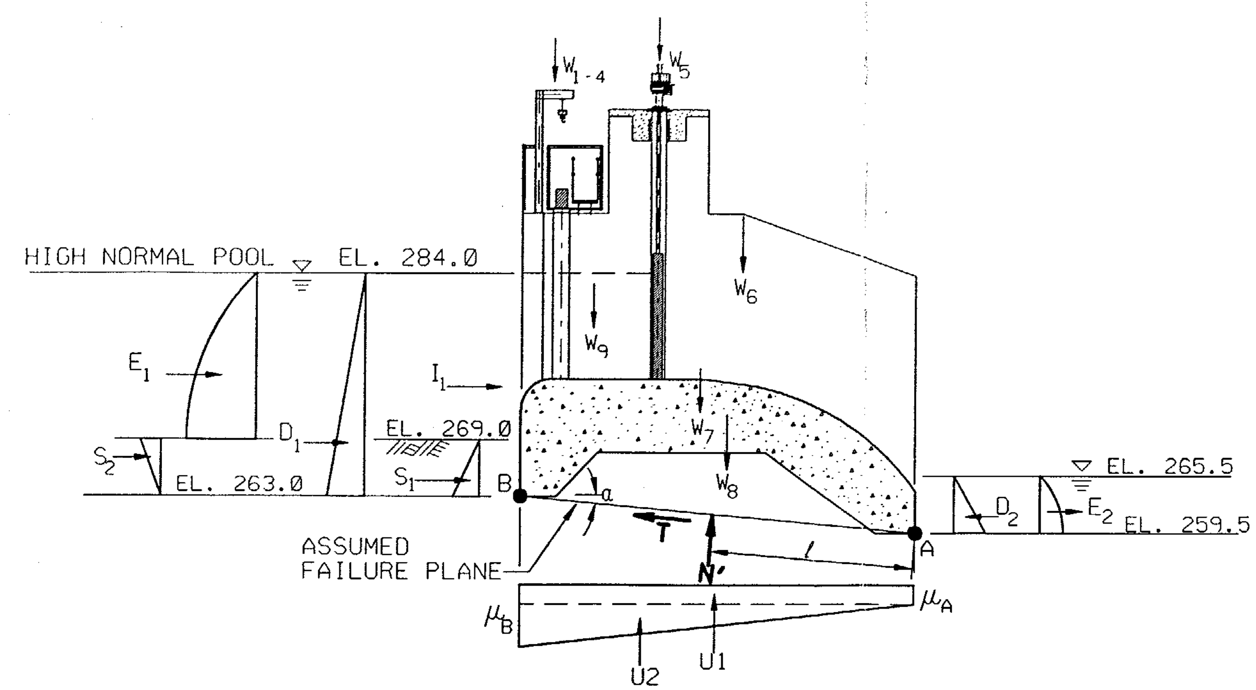
REELFOOT LAKE, TENNESSEE & KENTUCKY
ALTERNATIVE SPILLWAY
STABILITY ANALYSIS

U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
MEMPHIS, TENNESSEE

US Army Corps of Engineers
Memphis District



CASE 3 - HIGH NORMAL POOL WITH E.Q.
(OVERTURNING ANALYSIS)
NTS



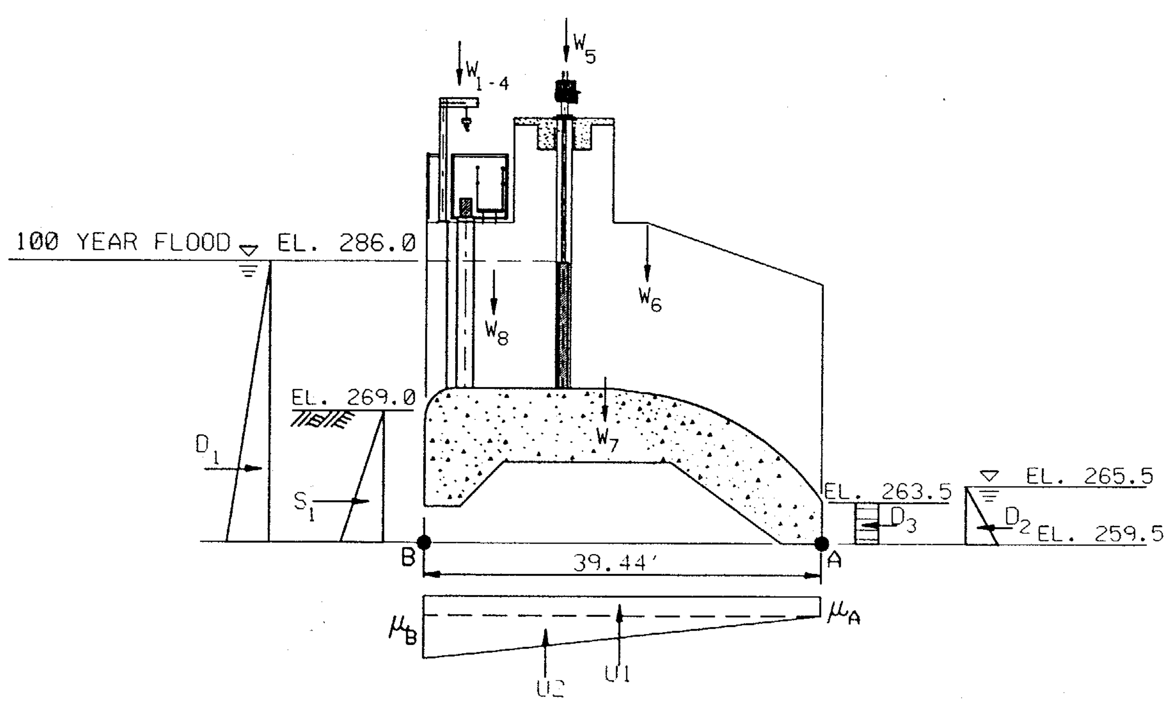
CASE 3 - HIGH NORMAL POOL WITH E.Q.
(SLIDING AND BEARING ANALYSIS)
NTS

CASES PRESENTED

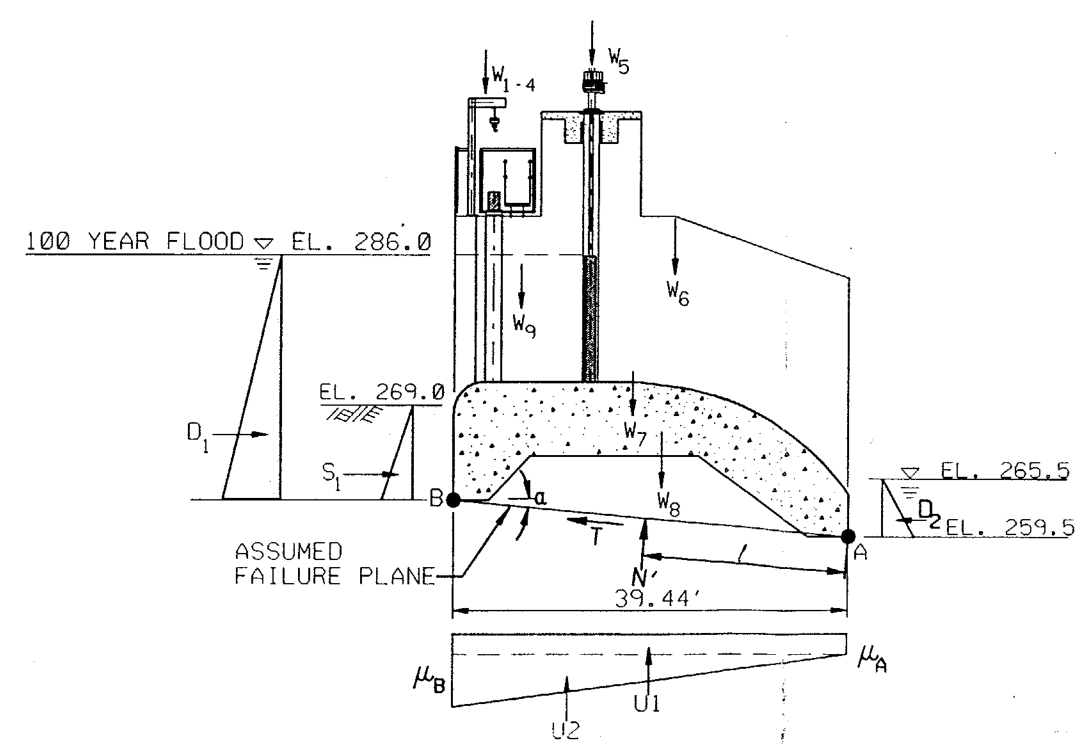
LOADING CASE	OVERTURNING RATIO	FACTOR OF SAFETY	
		SLIDING	BEARING
3 HIGH NORMAL POOL WITH EARTHQUAKE; a=0.15g	0.31 (RESULTANT WITHIN BASE)	1.03 (1.1)	1.7 (>1.0)
4 UNUSUAL LOADING-100 YR. FLOOD W/O EARTHQUAKE	0.38 (>0.25)	1.7 (1.33)	5.5 (2.0)

NOTES:

1. ALL ANALYSES PERFORMED IN ACCORDANCE WITH EM1110-2-2502, DATED 29 SEP 89.
2. VALUES WITHIN PARENTHESIS IN SUMMARY TABLE ARE MINIMUMS PRESENTED IN TABLE 4-1, "RETAINING WALL STABILITY CRITERIA", IN REFERENCED EM1110-2-2502.



CASE 4 - UNUSUAL LOADING
(OVERTURNING ANALYSIS)
NTS



CASE 4 - UNUSUAL LOADING
(SLIDING AND BEARING ANALYSIS)
NTS

US Army Corps of Engineers Memphis District

U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
MEMPHIS, TENNESSEE

ENVIRONMENTAL RESTORATION NATURAL RESOURCES PROJECT
MISSISSIPPI RIVER AND TRIBUTARIES CONSTRUCTION

REELFOOT LAKE, TENNESSEE & KENTUCKY
ALTERNATIVE SPILLWAY
STRUCTURE STABILITY

DRAWING
III-47

OVERTURNING ANALYSIS - ALTERNATIVE SPILLWAY LOADING CASE 3

ASSUMPTIONS:

FULL HYDROSTATIC PRESSURES DEVELOP AT 'B'
UPLIFT OCCURS ALONG LINE A-B
HORIZONTAL FORCES ARE BALANCED BY CONTACT WITH DOWNSTREAM STILLING BASIN: $\Sigma F_h = 0$

COMPUTE DRIVING FORCES OF WATER AND SOIL

$D_1 = 0.5(\gamma)(H)^2(L)$
$D_1 = 0.5(0.0624)(24.5)^2(148) = 2771.7^K$

$S_1 = 0.5(\gamma)(H)^2(L)(k_o)$
S_1 $0.5(0.0626)(9.5)^2(148)(0.5) = 209.04^K$

COMPUTE FORCES DUE TO SEISMIC LOADING

DETERMINE SPILLWAY STRUCTURE DEAD LOAD CENTER OF GRAVITY FOR APPLICATION OF SEISMIC LOADING

ITEM NO.	DESCRIPTION OF ITEM	FORCES	MOMENT ARMS		SUM OF MOMENTS	
			X_{ARM}	Y_{ARM}	Y-AXIS	X-AXIS
W_1	GRATES, HAND RAILS, ETC.	4.3	33.94	30.50	146.6	131.8
W_2	GRATES, HAND RAILS, ETC.	4.3	31.36	30.50	135.5	131.8
W_3	STOP LOGS	31.0	35.44	22.70	1098.6	703.7
W_4	STOP LOG CRANE BEAM, FRAME & HOIST	10.7	37.44	30.50	399.1	325.1
W_5	TOP SLAB & BEAM	253.8	25.69	39.26	6520.1	9964.2
W_5	GATES	186.0	25.69	22.70	4778.3	4222.2
W_5	HOIST	15.9	25.69	40.50	408.5	644.0
W_6	WALLS	2533.1	17.81	22.80	45114.9	57755.1
W_7	FOUNDATION	6665.6	20.98	9.09	139843.2	60589.8
	SUMMATION	9705.0			198445.0	134468.0

$X_{CG} = 198445/9705 = 20.45$
$Y_{CG} = 134468/9705 = 13.86$

INERTIA FORCE ON STRUCTURE, $I_1 = (0.15)(\text{DEAD LOAD}) = (0.15)(9705^K) = 1455.8^K$

HYDRODYNAMIC FORCE DUE TO WATER ABOVE GROUND:

$P_E = (2/3)(C_E)(K_h)(h)^2 = E_{(1\text{- FOOT})} = (2/3)(0.051)(0.15)(15)^2$
$= 1.1475$ K/FT. THEREFORE $E_1 = 1.1475$ K/FT (148 FT) $= 169.83^K$

INCREASE IN SOIL PRESSURE (ACTIVE SIDE ONLY) DUE TO EARTHQUAKE:

FROM EQ. 3-71, $S_2 = k_h[(\gamma)(H)^2/2(\tan\alpha - \tan\beta)] + k_h[(\gamma_s - \gamma)(H_s)^2/2(\tan\alpha)]$

ASSUMING $\gamma_s = \gamma$; $S_2 = K_h[(\gamma)(H)^2/2(\tan\alpha - \tan\beta)]$

BACKFILL SLOPE ANGLE, $\beta = 0$

FROM EQ. 3-56, $\alpha = \tan^{-1}[(C_1 + \text{SQRT}(C_1^2 + 4C_2))/2]$

FROM EQ. 3-57, $C_1 = 2(\tan\phi - k_h)/(1 + k_h \tan\phi) = 2(\tan 30^\circ - 0.15)/(1 + 0.15(\tan 30^\circ)) = 0.7866$

FROM EQ. 3-58, $C_2 = [\tan\phi(1 - \tan\phi\tan\beta) - (\tan\beta + k_h)] / [\tan\phi(1 + k_h \tan\phi)]$, AND SINCE $\beta = 0$, $C_2 = 0.6812$

AND $\alpha = \tan^{-1}[0.7866 + \text{SQRT}(0.7866^2 + 4(0.6812))/2] = 52.59^\circ$

AND $S_2 = 0.15[(0.1258)(9.5)^2 / 2(\tan 52.59)] = 0.649$ K/FT; FOR L = 148', $S_2 = 96.1^K$ @ $y = 2/3(9.5) = 6.33'$

COMPUTE UPLIFT FORCE 'U'

$\mu(A) = 6.0(0.0624) = 0.3744$ KSF

$\mu(B) = 24.5(0.0624) = 1.5288$ KSF

$U_1 = (0.3744)(39.44)(148) = 2185.4^K$

$U_2 = 0.5(1.5288 - 0.3744)(39.44)(148) = 3369.2^K$

COMPUTE RESISTING FORCES OF WATER AND STILLING BASIN

$D_2 = 0.5(0.0624)(6.0)^2(148) = 166.2^K$

$D_3 = 4535.8^K$. AS REQUIRED TO BALANCE HORIZONTAL FORCES

STRUCTURE WEIGHTS AND MOMENTS ABOUT POINT 'A'

ITEM NO.	DESCRIPTION OF ITEM	FORCES (KIPS)				MOMENT ARMS (FT)		SUM OF MOMENTS (FT)	
		VERTICAL		HORIZONTAL					
		DOWN	UP	LEFT	RIGHT	X_{ARM}	Y_{ARM}	OVERTURN	RESIST
W_1	GRATES, HAND RAILS, ETC.	4.3				33.94			147
W_2	GRATES, HAND RAILS, ETC.	4.3				31.36			135
W_3	STOP LOGS	31.0				35.44			1099
W_4	STOP LOG CRANE BEAM, FRAME & HOIST	10.7				37.44			399
W_5	TOP SLAB, BEAM, GATES & HOIST	455.7				25.69			11707
W_6	WALLS	2533.1				17.81			45115
W_7	FOUNDATION	6665.6				20.98			139843
W_8	WATER ON INLET SLAB	1009.0				32.57			32863
D_1	WATER - ACTIVE SIDE				2771.7		8.17	22645	
D_2	WATER - PASSIVE SIDE			166.2			2.00		332
U_1	UPLIFT		2185.4			19.72		43096	
U_2	UPLIFT		3369.2			26.29		88576	
S_1	SOIL - ACTIVE SIDE				209.0		3.17	663	
D_3	STILLING BASIN REACTION TO DRIVING FORCES (SUM OF HORIZONTAL FORCES = 0)			4535.8			2.00		9072
EARTHQUAKE INDUCED FORCES									
I_1	INERTIA FORCE OF STRUCTURE DEAD WEIGHT				1455.8		13.86	20177	
E_1	WATER ABOVE GROUND LEVEL				169.8		15.50	2632	
S_2	ADDITIONAL SOIL REACTION				96.1		6.33	608	
	SUMMATION	10714.0	5555.0	4702.0	4702.0			178398	240712

SUMMARY OF RESULTS FOR OVERTURNING ANALYSIS

ΣFORCES(HORIZONTAL) = 0.0 ΣFORCES(VERTICAL) = 5159^K ΣMOMENTS = 62314 FT-K

X_R (RESULTANT) = Σ MOMENTS/ Σ FORCES(VERTICAL) = 62314 FT-K / 5159^K = 12.1' FROM 'A'

BASE WIDTH (X-DIRECTION) = 39.44'

BASE LENGTH (Z-DIRECTION) = 148.0'

RESULTANT RATIO = X_R / BASE WIDTH = 12.1/39.44 = 0.31 (SLIGHTLY LESS THAN 100%)

EM 1110-2-2505, TABLE 4-1 REQUIRES THE RESULTANT TO BE WITHIN BASE - OK. CRITERIA SATISFIED

ECCENTRICITY, $e = (0.5)(\text{WIDTH}) - X_R = (0.5)(39.44) - 12.1 = 7.62$

ESTIMATE SOIL PRESSURES (ASSUMES BASE TOTALLY IN COMPRESSION)

$q_{(maximum)} = [V/(B)(L)][1 + 6(e)/B] = 1.91$ KSF

$q_{(minimum)} = [V/(B)(B)][1 - 6(e)/L] = -0.14$ KSF

SLIDING ANALYSIS - ALTERNATIVE SPILLWAY LOADING CASE 3 CONTD

ASSUMPTIONS:
SAME AS OVERTURNING EXCEPT THAT HORIZONTAL FORCES ARE NOT BALANCED: $\Sigma F_h \neq 0$
AND SOIL ABOVE LINE A-B IS INCLUDED IN STRUCTURE WEIGHT

COMPUTE DRIVING FORCES OF WATER AND SOIL

$D_1 = 0.5(\gamma)(H^2)(L)$
$D_1 = 0.5(0.0624)(21.0)^2(148) = 2036.36^K$

$S_1 = 0.5(\gamma)(H^2)(L)(k_a)$
$S_1 = 0.5(0.0626)(6.0)^2(148)(0.5) = 83.38^K$

COMPUTE FORCES DUE TO SEISMIC LOADING

DETERMINE SPILLWAY STRUCTURE DEAD LOAD CENTER OF GRAVITY INCLUDING THE WEIGHT OF THE SOIL BENEATH THE STRUCTURE.

ITEM NO.	DESCRIPTION OF ITEM	FORCES	MOMENT ARMS		SUM OF MOMENTS	
			X_{ARM}	Y_{ARM}	Y-AXIS	X-AXIS
W_1	GRATES, HAND RAILS, ETC.	4.3	33.94	30.50	146.6	131.8
W_2	GRATES, HAND RAILS, ETC.	4.3	31.36	30.50	135.5	131.8
W_3	STOP LOGS	31.0	35.44	22.70	1098.6	703.7
W_4	STOP LOG CRANE BEAM, FRAME & HOIST	10.7	37.44	30.50	399.1	325.1
W_5	TOP SLAB & BEAM	253.8	25.69	39.26	6520.1	9964.2
W_5	GATES	186.0	25.69	22.70	4778.3	4222.2
W_5	HOIST	15.9	25.69	40.50	408.5	644.0
W_6	WALLS	2533.1	17.81	22.80	45114.9	57755.1
W_7	FOUNDATION	6665.6	20.98	9.09	139843.2	60589.8
W_8	SOIL BENEATH THE STRUCTURE	2557.1	21.00	4.36	53699.1	11149.0
W_9	WATER ON INLET SLAB	1009.0	32.57	19.61	32863.1	19786.5
	SUMMATION	13271.0			285007.0	165403.0

$X_{CG} = 285007/13271 = 21.48$
$Y_{CG} = 165403/13271 = 12.46$

INERTIA FORCE ON STRUCTURE, $I_1 = (0.15)(\text{DEAD LOAD}) = (0.15)(13271.26^K) = 1990.69^K$

HYDRODYNAMIC FORCE DUE TO WATER ABOVE GROUND ON ACTIVE SIDE:

$P_E = (2/3)(C_E)(K_h)(h)^2 = E_{(1-\text{FOOT})} = (2/3)(0.051)(0.15)(15)^2$
$= 1.1475$ K/FT, THEREFORE $E_1 = 1.1475$ K/FT (148FT $= 169.83^K$

HYDRODYNAMIC FORCE DUE TO WATER ABOVE GROUND ON PASSIVE SIDE:

$P_E = (2/3)(C_E)(K_h)(h)^2 = E_{(1-\text{FOOT})} = (2/3)(0.051)(0.15)(6)^2$
$= 0.1836$ K/FT, THEREFORE $E_2 = 0.1836$ K/FT (148 FT) $= 27.17^K$

INCREASE IN SOIL PRESSURE (ACTIVE SIDE ONLY) DUE TO EARTHQUAKE:

$S_2 = [(0.15)(83.38)] + (0.15)[(15)(0.0624)(6)(148) + (0.5)(0.0624)(6)^2(148)]$
$= 162.12^K$

COMPUTE UPLIFT FORCE 'U'

$\mu(A) = 6.0(0.0624) = 0.3744$ KSF

$\mu(B) = 21.0(0.0624) = 1.3104$ KSF

$U_1 = (0.3744)(39.44)(148) = 2185.4^K$

$U_2 = 0.5(1.3104 - 0.3744)(39.44)(148) = 2731.77^K$

COMPUTE RESISTING FORCES OF WATER AND STILLING BASIN

$D_2 = 0.5(0.0624)(6.0)^2(148) = 166.23^K$

STRUCTURE WEIGHTS AND MOMENTS ABOUT POINT 'A'

ITEM NO.	DESCRIPTION OF ITEM	FORCES (KIPS)				MOMENT ARMS (FT)		SUM OF MOMENTS (FT)	
		VERTICAL		HORIZONTAL					
		DOWN	UP	LEFT	RIGHT	X_{ARM}	Y_{ARM}	OVERTURN	RESIST
W_1	GRATES, HAND RAILS, ETC.	4.3				33.94			147
W_2	GRATES, HAND RAILS, ETC.	4.3				31.36			135
W_3	STOP LOGS	31.0				35.44			1099
W_4	STOP LOG CRANE BEAM, FRAME & HOIST	10.7				37.44			399
W_5	TOP SLAB, BEAM, GATES & HOIST	455.7				25.69			11707
W_6	WALLS	2533.1				17.81			45115
W_7	FOUNDATION	6665.6				20.98			139843
W_8	WATER ON INLET SLAB	1009.0				32.57			32863
W_9	SOIL BENEATH THE STRUCTURE	2557.1				21.00			53699
D_1	WATER - ACTIVE SIDE				2036.4		10.50	21382	
D_2	WATER - PASSIVE SIDE			166.2			2.00		332
U_1	UPLIFT		2185.4			19.72		43096	
U_2	UPLIFT		2731.8			26.29		71818	
S_1	SOIL - ACTIVE SIDE				83.4		5.50	459	
D_3	STILLING BASIN REACTION TO DRIVING FORCES (SUM OF HORIZONTAL FORCES NOT EQUAL TO ZERO)								
	EARTHQUAKE INDUCED FORCES								
I_1	INERTIA FORCE OF STRUCTURE DEAD WEIGHT				1990.7		12.46	24804	
E_1	WATER ABOVE GROUND LEVEL (US)				169.8		15.50	2632	
S_2	ADDITIONAL SOIL REACTION				162.1		7.50	1216	
E_2	WATER ABOVE GROUND LEVEL (DS)				27.2		2.40	65	
	SUMMATION	13271.0	4917.0	166.2	4470.0			165472	285340

SUMMARY OF RESULTS FOR SLIDING ANALYSIS

ΣFORCES(HORIZONTAL) $= 4304^K \rightarrow$ ΣFORCES(VERTICAL) $= 8354^K \downarrow$ ΣMOMENTS = 119868 FT-K

COMPUTE FACTOR OF SAFETY (FS) AGAINST SLIDING

$\alpha = \tan^{-1}(3.5/39.44) = -5.07^\circ$

$N' = (\Sigma \text{ VERTICAL FORCES}) \cos\alpha + (\Sigma \text{ HORIZONTAL FORCES}) \sin\alpha$
$N' = 8354 \cos -5.07^\circ + 4304 \sin -5.07^\circ = 7941^K$

$T = (\Sigma \text{ HORIZONTAL FORCES}) \cos\alpha - (\Sigma \text{ VERTICAL FORCES}) \sin\alpha$
$T = 4304 \cos -5.07^\circ - 8354 \sin -5.07^\circ = 5052^K$

$FS = (N' \tan\phi + cL)/T$
$FS = 7941 \tan 33^\circ/5025 = 1.03$ (SLIGHTLY LESS THAN 1.1, BUT O.K. DUE TO CONSERVATIVE METHOD OF SEISMIC LOADING)

BEARING CAPACITY ANALYSIS

$Q = B'[(\xi_{cd}\xi_{ci}\xi_{ct}\xi_{cg}cN_c) + (\xi_{qd}\xi_{qi}\xi_{qt}\xi_{qg}q_oN_q) + (0.5\bullet\xi_{\gamma d}\xi_{\gamma i}\xi_{\gamma t}\xi_{\gamma g}B'\gamma N_\gamma)]$

$\ell = \Sigma$ MOMENTS / $N' = 119868$ FT-K/7941 = 15.09'

$e = (B/2) - \ell = 4.63$

B' (EFFECTIVE BASE WIDTH) $= B - (2)(e) = 30.18'$

COMPUTE BEARING CAPACITY FACTORS

$D = 4.0$ $B = 39.44$ $\phi = 33^\circ$ $C = 0.0$ $\alpha = -5.07^\circ$ $\beta = 0.0$ RADS

$T = 5025^K$ $N' = 7941^K$ $\delta = \tan^{-1}(T/N') = 32.32^\circ$

NOTE: N_q, NC, AND N_γ AVAILABLE FROM TABLE 5-1, PGS 5-5 & 5-6 OF EM 1110-2-2502

$N_q = 26.09$

$N_c = 38.64$ FOR $\phi > 0$; $N_c = 5.14$ FOR $\phi = 0$

$N_\gamma = 26.17$

$\xi_{cd} = 1 + 0.2(D/B')\tan(45^\circ + \phi/2) = 1 + 0.2(4.0/30.18)\tan(45^\circ + 33/2) = 1.05$

$\xi_{ci} = \xi_{qi} = (1 - \delta^\circ/90^\circ)^2 = (1 - 32.32/90^\circ)^2 = 0.41$

$\xi_{qd} = \xi_{\gamma d} = 1 + 0.1(D/B')\tan(45^\circ + \phi/2) = 1 + 0.1(4.0/30.18)\tan(45^\circ + 33/2)$
$= 1.02$ (FOR $\phi > 10^\circ$)

$\xi_{qt} = \xi_{\gamma t} = (1 - \alpha\tan\phi)^2$ *(WHERE α IS IN RADIANS)* $= (1 - -0.0885 \tan 33^\circ)^2 = 1.12$

$\xi_{qg} = \xi_{\gamma g} = (1 - \tan\beta)^2 = (1 - \tan 0)^2 = 1.0$

$\xi_{cg} = \xi_{qg} - [(1 - \xi_{qg})/N_c \bullet \tan\phi)] = 1.0 - [(1.0 - 1.0)/(38.64 \bullet \tan 33^\circ)]$
$= 1.0$ (FOR $\phi > 0^\circ$)

q_o = (MINIMUM EFFECTIVE OVERBURDEN PRESSURE)

$q_o = 4.0(0.0626) = 0.2504$

$\xi_{\gamma i} = (1 - \delta/\phi)^2 = (1 - 32.32/33)^2 = 0.00042$

$\gamma = 0.0626$

COMPUTE BEARING CAPACITY

$Q = B'[(\xi_{cd}\xi_{ci}\xi_{ct}\xi_{cg}cN_c) + (\xi_{qd}\xi_{qi}\xi_{qt}\xi_{qg}q_oN_q) + (0.5\bullet\xi_{\gamma d}\xi_{\gamma i}\xi_{\gamma t}\xi_{\gamma g}B'\gamma N_\gamma)]$

$Q = (30.18)[0 + (1.02)(0.41)(1.12)(1.0)(0.2504)(26.09)$
$+ 0.5(1.02)(0.00042)(1.12)(1.0)(30.18)(0.0626)(26.17)] = 89.69$ K-FT

TOTAL $Q = 148'(89.69) = 13274.3^K$

COMPUTE FACTOR OF SAFETY

FS = ALLOWABLE/ACTUAL = Q/N' = 13274/7941 = 1.7 > 1.0 (O.K.)

US Army Corps of Engineers Memphis District

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ENVIRONMENTAL RESTORATION NATURAL RESOURCES PROJECT
MISSISSIPPI RIVER AND TRIBUTARIES CONSTRUCTION

REELFOOT LAKE, TENNESSEE & KENTUCKY
ALTERNATIVE SPILLWAY
STABILITY ANALYSIS - LOADING CASE 3

DRAWING
III-49

OVERTURNING ANALYSIS - ALTERNATIVE SPILLWAY LOADING CASE 4

ASSUMPTIONS:
FULL HYDROSTATIC PRESSURES DEVELOP AT 'B'

UPLIFT OCCURS ALONG LINE A-B

HORIZONTAL FORCES ARE BALANCED BY CONTACT WITH DOWNSTREAM STILLING BASIN: $\Sigma F_h = 0$

COMPUTE DRIVING FORCES OF WATER AND SOIL

$D_1 = 0.5(\gamma)(H^2)(L)$
$D_1 = 0.5(0.0624)(26.6)^2(148) = 3267.2^K$

$S_1 = 0.5(\gamma)(H^2)(L)(k_o)$
S_1 $0.5(0.0626)(9.5)^2(148)(0.5) = 209.04^K$

COMPUTE UPLIFT FORCE 'U'

$\mu(A) = 6.0(0.0624) = 0.3744$ KSF (ASSUMES CREEP PATH IS HORIZONAL AT EL. 259.5)

$\mu(B) = 26.6(0.0624) = 1.6598$ KSF

$U_1 = (0.3744)(39.44)(148) = 2185.4^K$

$U_2 = 0.5(1.6598 - 0.3744)(39.44)(148) = 3752.1^K$

COMPUTE RESISTING FORCES OF WATER AND STILLING BASIN

$D_2 = 0.5(0.0624)(6.0)^2(148) = 166.2^K$

$D_3 = 3310.1^K$, AS REQUIRED TO BALANCE HORIZONTAL FORCES

STRUCTURE WEIGHTS AND MOMENTS ABOUT POINT 'A'

ITEM NO.	DESCRIPTION OF ITEM	FORCES (KIPS)				MOMENT ARMS (FT)		SUM OF MOMENTS (FT)	
		VERTICAL		HORIZONTAL					
		DOWN	UP	LEFT	RIGHT	X_{ARM}	Y_{ARM}	OVERTURN	RESIST
W_1	GRATES, HAND RAILS, ETC.	4.3				33.94			147
W_2	GRATES, HAND RAILS, ETC.	4.3				31.36			135
W_3	STOP LOGS	31.0				35.44			1099
W_4	STOP LOG CRANE BEAM, FRAME & HOIST	10.7				37.44			399
W_5	TOP SLAB, BEAM, GATES & HOIST	455.7				25.69			11707
W_6	WALLS	2533.1				17.81			45115
W_7	FOUNDATION	6665.6				20.98			139843
W_8	WATER ON INLET SLAB	1225.2				32.57			39905
D_1	WATER - ACTIVE SIDE				3267.2		8.87	28980	
D_2	WATER - PASSIVE SIDE			166.2			2.00		332
U_1	UPLIFT		2185.4			19.72		43096	
U_2	UPLIFT		3752.1			26.29		98643	
S_1	SOIL - ACTIVE SIDE				209.0		3.17	663	
D_3	STILLING BASIN REACTION TO DRIVING FORCES (SUM OF HORIZONTAL FORCES = 0)			3310.1			2.00		6620
	SUMMATION	10930.0	5938.0	3476.0	3476.0			171381	245302

SUMMARY OF RESULTS FOR OVERTURNING ANALYSIS

ΣFORCES(HORIZONTAL) = 0.0 ΣFORCES(VERTICAL) = 4992^K ↓ ΣMOMENTS = 73921 FT-K

X_R (RESULTANT) = ΣMOMENTS/ΣFORCES(VERTICAL) = 73921 FT-K / 4992^K = 14.81' FROM 'A'

BASE WIDTH (X-DIRECTION) = 39.44'

BASE LENGTH (Z-DIRECTION) = 148.0'

RESULTANT RATIO = X_R / BASE WIDTH = 14.81/39.44 = 0.38

EM 1110-2-2502, TABLE 4-1 REQUIRES THE MINIMUM BASE AREA IN COMPRESSION TO BE 75% (RESULTANT RATIO $\geq$ 0.25) - OK, CRITERIA SATISFIED

ECCENTRICITY, e = (0.5)(WIDTH) - X_R = (0.5)(39.44) - 14.81 = 4.91

ESTIMATE SOIL PRESSURES (ASSUMES BASE TOTALLY IN COMPRESSION)

$q_{(maximum)} = [V/(B)(L)][1 + 6(e)/B] = 1.49$ KSF

$q_{(minimum)} = [V/(B)(L)][1 - 6(e)/B] = 0.22$ KSF

SLIDING ANALYSIS

ASSUMPTIONS:
HORIZONTAL FORCES ARE NOT BALANCED: $\Sigma F_h \neq 0$

DETERMINE SPILLWAY STRUCTURE DEAD LOAD CENTER OF GRAVITY INCLUDING THE WEIGHT OF THE SOIL BENEATH THE STRUCTURE.

COMPUTER DRIVING FORCES OF WATER AND SOIL

$D_1 = 0.5(\gamma)(H)^2(L)$
$D_1 = 0.5(0.0624)(21.0)^2(148) = 2036.36^K$

$S_1 = 0.5(\gamma)(H)^2(L)(k_o)$
$S_1 = 0.5(0.0626)(6.0)^2(148)(0.5) = 83.38^K$

COMPUTE UPLIFT FORCE 'U'

$\mu(A) = 6.0(0.0624) = 0.3744$ KSF

$\mu(B) = 21.0(0.0624) = 1.3104$ KSF

$U_1 = (0.3744)(39.44)(148) = 2185.4^K$

$U_2 = 0.5(1.3104 - 0.3744)(39.44)(148) = 2731.77^K$

COMPUTE RESISTING FORCES OF WATER AND STILLING BASIN

$D_2 = 0.5(0.0624)(6.0)^2(148) = 166.23^K$

STRUCTURE WEIGHTS AND MOMENTS ABOUT POINT 'A'

ITEM NO.	DESCRIPTION OF ITEM	FORCES (KIPS)				MOMENT ARMS (FT)		SUM OF MOMENTS (FT)	
		VERTICAL		HORIZONTAL					
		DOWN	UP	LEFT	RIGHT	X_{ARM}	Y_{ARM}	OVERTURN	RESIST
W_1	GRATES, HAND RAILS, ETC.	4.3				33.94			147
W_2	GRATES, HAND RAILS, ETC.	4.3				31.36			135
W_3	STOP LOGS	31.0				35.44			1099
W_4	STOP LOG CRANE BEAM, FRAME & HOIST	10.7				37.44			399
W_5	TOP SLAB, BEAM, GATES & HOIST	455.7				25.69			11707
W_6	WALLS	2533.1				17.81			45115
W_7	FOUNDATION	6665.6				20.98			139843
W_8	WATER ON INLET SLAB	1225.2				32.57			39905
W_9	SOIL BENEATH THE STRUCTURE	2557.1				21.00			53699
D_1	WATER - ACTIVE SIDE				2464.0		11.20	27597	
D_2	WATER - PASSIVE SIDE			166.2			2.00		332
U_1	UPLIFT		2185.4			19.72		43096	
U_2	UPLIFT		3114.2			26.29		81872	
S_1	SOIL - ACTIVE SIDE				83.4		5.50	459	
	NO STILLING BASIN REACTION TO DRIVING FORCES (SUM OF HORIZONTAL FORCES NOT EQUAL TO ZERO)								
	SUMMATION	13487.0	5300.0	166.2	2547.0			153024	292381

SUMMARY OF RESULTS FOR SLIDING ANALYSIS

ΣFORCES(HORIZONTAL) = 2381^K ⟶ ΣFORCES(VERTICAL) = 8187^K ↓ ΣMOMENTS = 139357 FT-K

COMPUTE FACTOR OF SAFETY (FS) AGAINST SLIDING

$a = \text{TAN}^{-1}(3.5/39.44) = -5.07°$

$N' = (\Sigma \text{ VERTICAL FORCES}) \text{ COS } a + (\Sigma \text{ HORIZONTAL FORCES}) \text{ SIN } a$
$N' = 8187 \text{ COS } -5.07° + 2381 \text{ SIN } -5.07° = 7945^K$

$T = (\Sigma \text{ HORIZONTAL FORCES}) \text{ COS } a - (\Sigma \text{ VERTICAL FORCES}) \text{ SIN } a$
$T = 2381 \text{ COS } -5.07° - 8187 \text{ SIN } -5.07° = 3095^K$

$FS = (N' \text{ TAN } \phi + cL)/T$
$FS = 7945 \text{ TAN } 33°/3095 = 1.67 > 1.33$ THEREFORE O.K.

BEARING CAPACITY ANALYSIS

$$Q = B'[(\xi_{cd}\xi_{ci}\xi_{ct}\xi_{cg}cN_c) + (\xi_{qd}\xi_{qi}\xi_{qt}\xi_{qg}q_oN_q) + (0.5*\xi_{\gamma d}\xi_{\gamma i}\xi_{\gamma t}\xi_{\gamma g}B'\gamma N_\gamma)]$$

$l = \Sigma$MOMENTS / N' = 139357 FT-K/7945^K = 17.54'

$e = (B/2) - l = 2.18$

B' (EFFECTIVE BASE WIDTH) = B - (2)(e) = 35.07'

COMPUTE BEARING CAPACITY FACTORS

D = 4.0 B = 39.44 $\phi = 33°$ C = 0.0 $a = -5.07°$ $\beta = 0.0$ RADS

T = 3095^K N' = 7945^K $\delta = \text{TAN}^{-1}(T/N') = 21.3°$

NOTE: N_q, NC, AND N_γ AVAILABLE FROM TABLE 5-1, PGS 5-5 & 5-6 OF EM 1110-2-2502

$N_q = 26.09$

$N_c = 38.64$ FOR $\phi>0$; $N_c = 5.14$ FOR $\phi = 0$

$N_\gamma = 26.17$

$\xi_{cd} = 1 + 0.2(D/B')\text{TAN}(45° + \phi/2) = 1 + 0.2(4.0/35.07)\text{TAN}(45° + 33/2) = 1.04$

$\xi_{ci} = \xi_{qi} = (1 - \delta°/90°)^2 = (1 - 21.3/90°)^2 = 0.58$

$\xi_{qd} = \xi_{\gamma d} = 1 + 0.1(D/B')\text{TAN}(45° + \phi/2) = 1 + 0.1(4.0/35.07)\text{TAN}(45° + 33/2)$
$= 1.02$ (FOR $\phi > 10°$)

$\xi_{qt} = \xi_{\gamma t} = (1 - a \text{ TAN } \phi)^2$ *(WHERE a IS IN RADIANS)* $= (1 - -0.0885 \text{ TAN } 33°)^2 = 1.12$

$\xi_{qg} = \xi_{\gamma g} = (1 - \text{TAN } \beta)^2 = (1 - \text{TAN } 0)^2 = 1.0$

$\xi_{cg} = \xi_{qg} - [(1 - \xi_{qg})/N_c * \text{TAN } \phi)] = 1.0 - [(1.0 - 1.0)/(38.64 * \text{TAN } 33°)]$
$= 1.0$ (FOR $\phi > 0°$)

q_o = (MINIMUM EFFECTIVE OVERBURDEN PRESSURE)

$q_o = 4.0(0.0626) = 0.2504$

$\xi_{\gamma i} = (1 - \delta/\phi)^2 = (1 - 21.3/33)^2 = 0.1257$

$\gamma = 0.0626$

COMPUTE BEARING CAPACITY

$$Q = B'[(\xi_{cd}\xi_{ci}\xi_{ct}\xi_{cg}cN_c) + (\xi_{qd}\xi_{qi}\xi_{qt}\xi_{qg}q_oN_q) + (0.5*\xi_{\gamma d}\xi_{\gamma i}\xi_{\gamma t}\xi_{\gamma g}B'\gamma N_\gamma)]$$

$Q = (35.07)[0 + (1.02)(0.58)(1.12)(1.0)(0.2504)(26.09)$
$+ 0.5(1.02)(0.1257)(1.12)(1.0)(35.07)(0.0626)(26.17)] = 296.47$ K-FT

TOTAL Q = 148'(296.47) = 43878^K

COMPUTE FACTOR OF SAFETY

FS = ALLOWABLE/ACTUAL = Q/N' = 43878/7945 = 5.5 > 2.0 (O.K.)

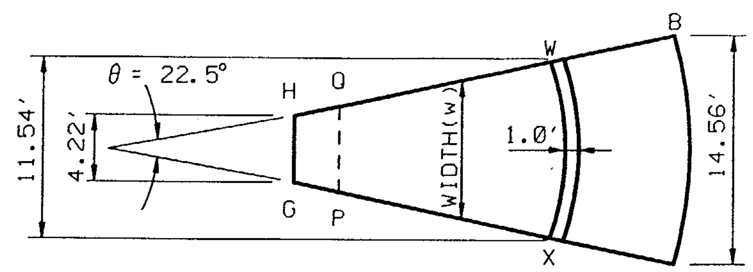
US Army Corps of Engineers Memphis District

U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
MEMPHIS, TENNESSEE

ENVIRONMENTAL RESTORATION NATURAL RESOURCES PROJECT
MISSISSIPPI RIVER AND TRIBUTARIES CONSTRUCTION

REELFOOT LAKE, TENNESSEE & KENTUCKY
ALTERNATIVE SPILLWAY
STABILITY ANALYSIS - LOADING CASE 4

DRAWING
III-50



PLAN
NTS

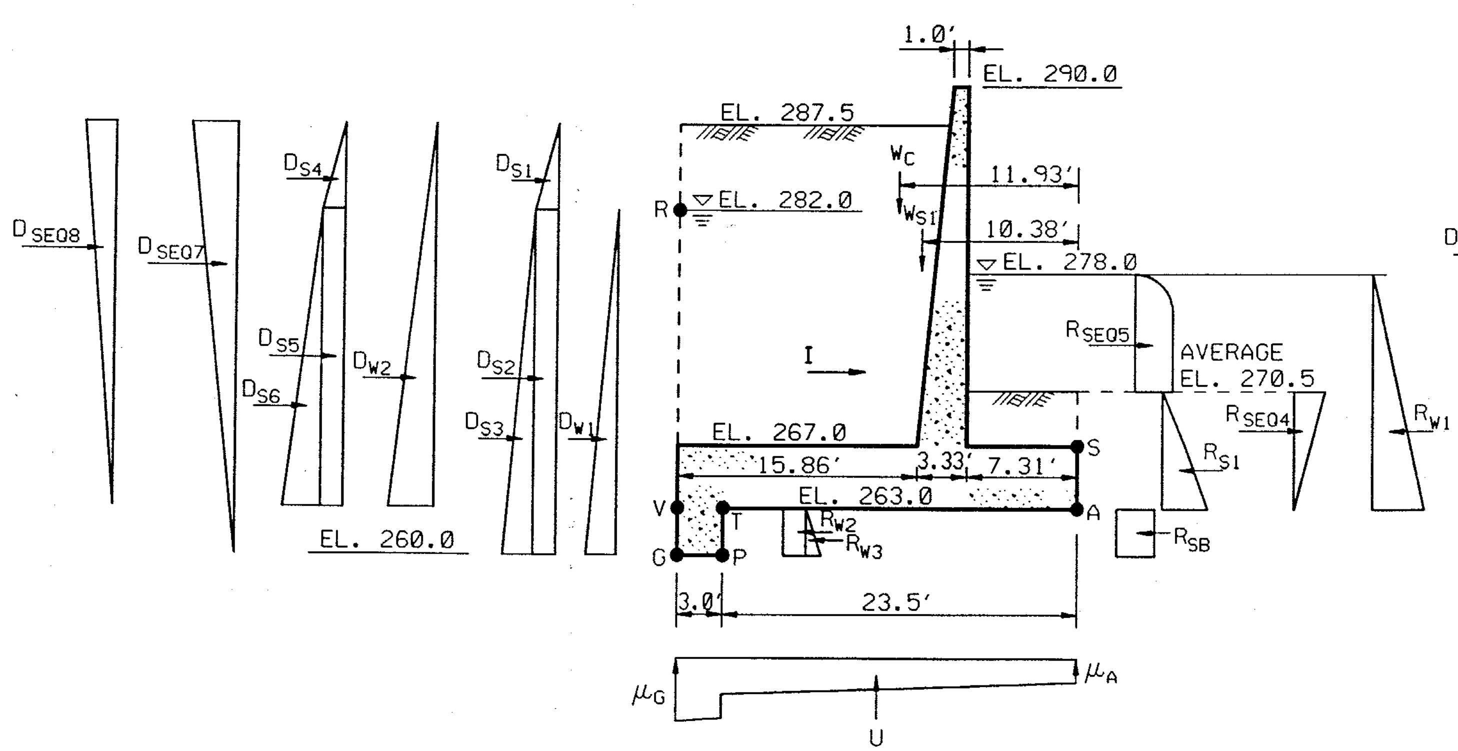
SUMMARY TABLE

LOADING CASE	OVERTURNING RATIO	FACTOR OF SAFETY	
		SLIDING	BEARING
A *NORMAL LAKE LEVEL IN BACKFILL (EL. 282.0) WITH 4' DRAWDOWN	0.34 (RESULTANT WITHIN MID THIRD)	1.9 (1.33)	4.6 (3.0)
B NORMAL LAKE LEVEL IN BACKFILL & TAILWATER (EL. 282.0) WITH EARTHQUAKE, a=0.15g	0.07 (RESULTANT WITHIN BASE)	1.16 (1.1)	1.24 (>1.0)
C HIGH NORMAL WATER TABLE IN BACKFILL (EL.276.0) WITH MINIMUM TAILWATER (EL. 270.5)	0.36 (RESULTANT WITHIN MID THIRD)	2.1 (1.5)	5.3 (3.0)
D SAME AS CASE A WITH EARTHQUAKE; a=0.15g	0.21 (RESULTANT WITHIN BASE)	1.2 (1.1)	1.7 (>1.0)
E LANDSIDE CRACK - FULL HYDROSTATIC PRESSURES	0.52 (RESULTANT WITHIN MID THIRD)	13.0 (1.5)	17.6 (3.0)
F CONSTRUCTION CASE - NO WATER	0.46 (>0.25)	5.3 (1.33)	15.2 (2.0)

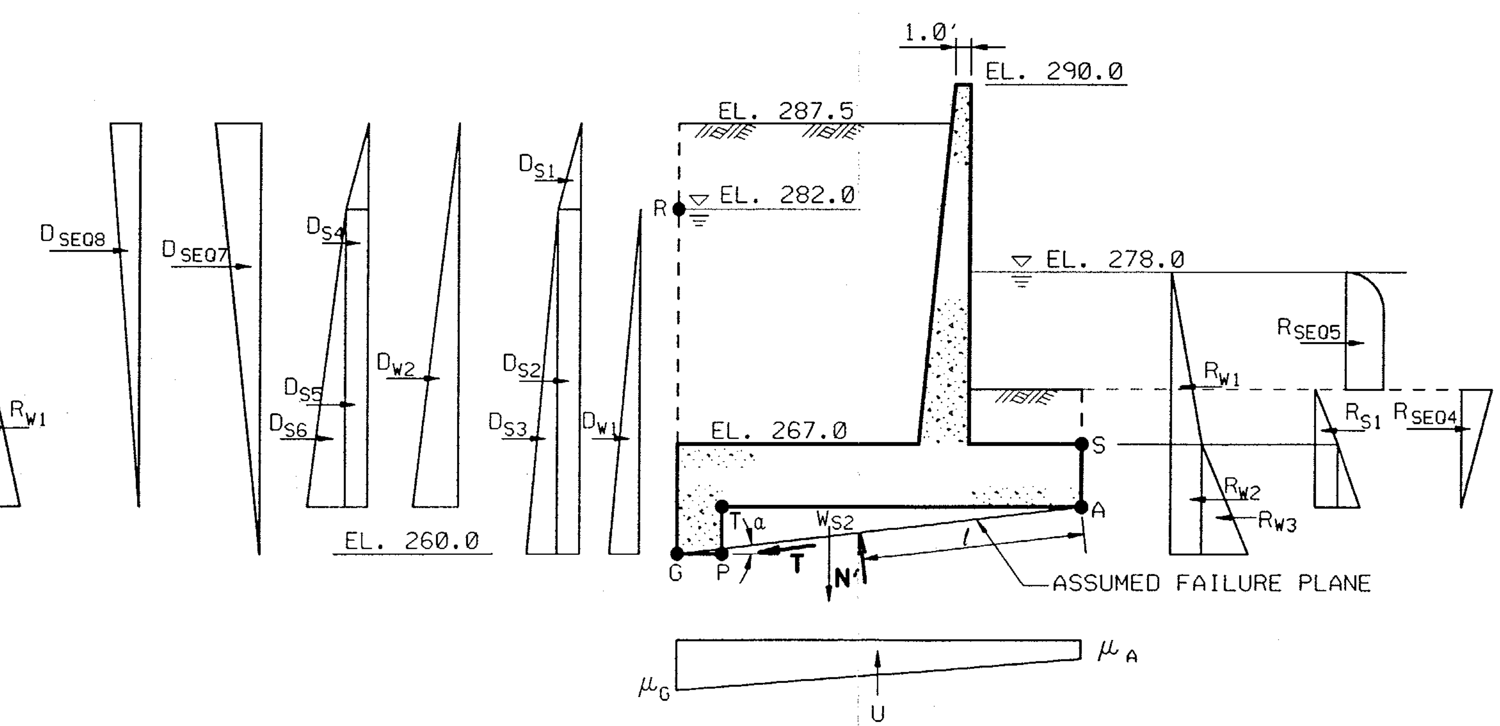
*CASE PRESENTED

NOTES:

1. ALL ANALYSES PERFORMED IN ACCORDANCE WITH EM1110-2-2502, DATED 29 SEP 89.
2. VALUES WITHIN PARENTHESIS IN SUMMARY TABLE ARE MINIMUMS PRESENTED IN TABLE 4-1 ,"RETAINNING WALL STABILITY CRITERIA", IN REFERENCED EM1110-2-2502.



ELEVATION
(OVERTURNING ANALYSIS)
NTS
CASE A WATER LOADING



ELEVATION
(SLIDING AND BEARING ANALYSIS)
NTS
CASE A WATER LOADING

NOTE: D_{SE08}, D_{SE07}, I, R_{SE04} & R_{SE05} ARE FOR CASE B & D LOADING OR EARTHQUAKE LOADING.

US Army Corps of Engineers Memphis District

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ENVIRONMENTAL RESTORATION NATURAL RESOURCES PROJECT
MISSISSIPPI RIVER AND TRIBUTARIES CONSTRUCTION

REELFOOT LAKE, TENNESSEE & KENTUCKY
ALTERNATIVE SPILLWAY
INLET WINGWALL STABILITY ANALYSIS - LOADING CASE A

DRAWING
III-51

OVERTURNING ANALYSIS - INLET WINGWALL CASE A

COMPUTE HYDROSTATIC PRESSURES

SEEPAGE PATH = [RGAS] = 56.17'

POINT	SEEPAGE HEAD (FT Y)	+ POSITION HEAD (FT Y)	= TOTAL HEAD (FT Y)	(KSF)
R	(56.17/56.17)(4.0)	-4.0	0.00	0.000
V	(37.17/56.17)(4.0)	15.0	17.65	1.101
G	(34.17/56.17)(4.0)	18.0	20.43	1.275
A	(7.5/56.17)(4.0)	15.0	15.53	0.969
S	(3.5/56.17)(4.0)	11.0	11.25	0.702

PRORATE HEAD LOSS FROM 'G' TO 'A' FOR POINTS 'P' & 'T'

SEEPAGE LOSS ALONG 'GA' = (26.67/56.17)(4.0)γ = 1.899γ

SEEPAGE PATH 'GPTA' = 29.5'

μ(P) = 1.275 - (3/29.5)(1.899 γ) = 1.263 KSF

μ(T) = 1.275 - (6/29.5)(1.899 γ) - 3.0γ = 1.063 KSF

COMPUTE UPLIFT FORCE 'U' FOR OVERTURNING ANALYSES

U = (AREA)(PRESSURE)

WIDTH (W) AS F(X), X MEASURED FROM 'G' : W = 4.22 + 0.3902X

μ AS F(X), X MEASURED FROM 'G': X = 0 TO 3 μ = 1.275 - 0.0040X
X = 3 TO 26.5 μ = 1.075 - 0.004X

APPROXIMATE INTEGRATION OF (PRESSURE)*(AREA) BY ASSUMING 1' WIDE STRIPS.

$X_{MIDPOINT}$	AREA (A)	PRESSURE (P)	FORCE	MOMENT ARM	MOMENT
0.50	4.42	1.27	5.62	26.20	147.26
1.50	4.81	1.27	6.10	25.20	153.67
2.50	5.20	1.27	6.57	24.20	159.05
3.50	5.59	1.06	5.93	23.20	137.49
4.50	5.98	1.06	6.32	22.20	140.23
5.50	6.37	1.05	6.70	21.20	142.11
6.50	6.76	1.05	7.09	20.20	143.16
7.50	7.15	1.05	7.47	19.20	143.39
8.50	7.54	1.04	7.85	18.20	142.79
9.50	7.93	1.04	8.22	17.20	141.39
10.50	8.32	1.03	8.59	16.20	139.18
11.50	8.71	1.03	8.96	15.20	136.19
12.50	9.10	1.03	9.32	14.20	132.41
13.50	9.49	1.02	9.69	13.20	127.87
14.50	9.88	1.02	10.05	12.20	122.56
15.50	10.27	1.01	10.40	11.20	116.50
16.50	10.66	1.01	10.75	10.20	109.69
17.50	11.05	1.01	11.10	9.20	102.15
18.50	11.44	1.00	11.45	8.20	93.89
19.50	11.83	1.00	11.79	7.20	84.91
20.50	12.22	0.99	12.13	6.20	75.23
21.50	12.61	0.99	12.47	5.20	64.85
22.50	13.00	0.99	12.80	4.20	53.78
23.50	13.39	0.98	13.14	3.20	42.03
24.50	13.78	0.98	13.46	2.20	29.62
25.50	14.17	0.97	13.79	1.20	16.55
26.50	14.56	0.97	14.11	0.20	2.82

SUMMATION U = 261.87 MOMENTS = 2,900.77

$X_{CENTROID}$ = 11.08

COMPUTE DRIVING FORCES ACTING ON [GH]

D_{W1} = (0.5)(1.275)(22.0)(4.22) = 59.19^K @ Y = 4.33'

K_o = 1 - SIN Φ = 1 - SIN 30° = 0.50

D_{S1} = 0.5(0.125)(5.5)^2(4.22)(0.5) = 3.99^K @ Y = 20.83'

D_{S2} = 5.50(0.125)(22.0)(0.50)(4.22) = 31.91^K @ Y = 8.0'

D_{S3} = 0.5(γB)H^2[GH]K_o WHERE γB = ((22.0)(0.125) - 1.275)/22.0 = 0.06705

D_{S3} = 0.50(0.06705)(22.0)^2(4.22)(0.50) = 34.23^K @ Y = 4.33'

COMPUTE DRIVING FORCES ACTING ON [WX-GH]

D_{W2} = (0.5)(1.10)(19.0)(7.32) = 76.56^K @ Y = 6.33'

D_{S4} = 0.5(0.125)(5.5)^2(7.32)(0.5) = 6.92^K @ Y = 20.83'

D_{S5} = 5.5(0.125)(19.0)(0.50)(7.32) = 47.81^K @ Y = 9.50'

D_{S6} = 0.5(γB)H^2[WX-GH]K_o WHERE γB = ((19.0)(0.125) - 1.101)/19.0 = 0.0670

D_{S6} = 0.50(0.0670)(19.0)^2(7.32)(0.50) = 44.30^K @ Y = 6.33'

COMPUTE RESISTING FORCES

R_{W1} = 0.5(0.624)(15.0)^2(14.56) = 102.21^K @ Y = 5.0'

R_{W2} = 1.063(3.0)(5.41) = 17.25^K @ Y = -1.5'

R_{W3} = 0.5(1.263 - 1.063)(3.0)(5.414) = 1.62^K @ Y = -2.0'

R_{S1} = 0.5(0.0626)(7.5)^2(0.50)(14.56) = 12.82^K @ Y = 2.5'

SUM MOMENTS ABOUT [AB] - OVERTURNING

ITEM NO.	DESCRIPTION OF ITEM	FORCES (KIPS) VERTICAL DOWN	FORCES (KIPS) VERTICAL UP	FORCES (KIPS) HORIZONTAL LEFT	FORCES (KIPS) HORIZONTAL RIGHT	MOMENT ARM (FT) X_{ARM}	MOMENT ARM (FT) Y_{ARM}	MOMENTS (KIP-FT) OVERTURN	MOMENTS (KIP-FT) RESIST
W_c	WEIGHT OF CONCRETE	241.0				10.38			2501.58
W_{S1}	WEIGHT OF SOIL ABOVE HEEL	374.9				11.93			4472.56
W_{S2}	(WEIGHT SOIL BENEATH BASE SLAB-N/A)								
U	UPLIFT		261.87			11.08		2901.52	
	DRIVING FORCES ALONG FACE (GH)								
D_{W1}					59.19		4.33	256.29	
D_{S1}					3.99		20.83	83.11	
D_{S2}					31.91		8.00	255.28	
D_{S3}					34.23		4.33	148.22	
	DRIVING FORCES ALONG FACE (WX-GH)								
D_{W2}					76.56		6.33	484.62	
D_{S4}					6.92		20.83	144.14	
D_{S5}					47.81		9.50	454.20	
D_{S6}					44.30		6.33	280.42	
R_{W1}	RESISTING WATER ON (AB)			102.21			5.00		511.05
R_{W2}	RESISTING WATER ON (PT)			17.25			-1.50		-25.88
R_{W3}	RESISTING WATER ON (PT)			1.62			-2.00		-3.24
R_{S1}	RESISTING SOIL ABOVE 'A'			12.82			2.50		32.05
R_{S8}	REQUIRED SOIL FORCE FOR BALANCE OF FORCES			171.00			-1.50		-256.50
	SUMMATION	615.9	261.87	304.90	304.90			5007.80	7231.60

SUMMARY OF RESULTS FOR OVERTURNING ANALYSIS

ΣFORCES (HORIZONTAL) = 0.0 ΣFORCES (VERTICAL) = 354.03^K↓ Σ MOMENTS = 2223.8 FT-K

X_R (RESULTANT) = ΣMOMENTS / ΣFORCES (VERTICAL) = 2.223.8 FT-K/ 354.03^K = 6.28' FROM [AB]

COMPUTER BASE WIDTH @ X_R : W = 4.22 + 0.3902(26.7 - 6.28) = 12.19'

BASE AREA FROM X_R TO [AB] ≈ 0.5(12.19 + 14.56)(6.28) = 83.99 SQ. FT.

TOTAL BASE AREA = 251.0 SQ. FT.

RESULTANT RATIO (BASED ON AREA) = 83.99/251.0 = 0.34 > 0.33, THEREFORE O.K.

SLIDING ANALYSIS

COMPUTE W_{S2} AND ITS CENTROID (WEIGHT OF SOIL BENEATH SLAB)

$X_{MIDPOINT}$	AREA (A)	SOIL THICKNESS (T)	SOIL WEIGHT (W)	MOMENT ARM	MOMENT
0.50	4.42	0.00	0.00	26.20	0.00
1.50	4.81	0.00	0.00	25.20	0.00
2.50	5.20	0.00	0.00	24.20	0.00
3.50	5.59	2.61	1.82	23.20	42.22
4.50	5.98	2.49	1.86	22.20	41.36
5.50	6.37	2.38	1.90	21.20	40.18
6.50	6.76	2.27	1.92	20.20	38.72
7.50	7.15	2.16	1.93	19.20	37.00
8.50	7.54	2.04	1.93	18.20	35.06
9.50	7.93	1.93	1.91	17.20	32.93
10.50	8.32	1.82	1.89	16.20	30.65
11.50	8.71	1.71	1.86	15.20	28.25
12.50	9.10	1.60	1.81	14.20	25.76
13.50	9.49	1.48	1.76	13.20	23.21
14.50	9.88	1.37	1.69	12.20	20.64
15.50	10.27	1.26	1.61	11.20	18.08
16.50	10.66	1.15	1.53	10.20	15.57
17.50	11.05	1.03	1.43	9.20	13.13
18.50	11.44	0.92	1.32	8.20	10.79
19.50	11.83	0.81	1.20	7.20	8.60
20.50	12.22	0.70	1.06	6.20	6.59
21.50	12.61	0.58	0.92	5.20	4.78
22.50	13.00	0.47	0.77	4.20	3.21
23.50	13.39	0.36	0.60	3.20	1.92
24.50	13.78	0.25	0.42	2.20	0.93
25.50	14.17	0.13	0.24	1.20	0.28
26.50	14.56	0.02	0.04	0.20	0.01

SUMMATION W_{S2} = 33.40 MOMENTS = 479.87

$X_{CENTROID}$ = 14.37

COMPUTE DRIVING FORCES

(SAME AS FOR OVERTURNING ANALYSIS)

US Army Corps of Engineers Memphis District

U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
MEMPHIS, TENNESSEE

ENVIRONMENTAL RESTORATION NATURAL RESOURCES PROJECT
MISSISSIPPI RIVER AND TRIBUTARIES CONSTRUCTION

REELFOOT LAKE, TENNESSEE & KENTUCKY
ALTERNATIVE SPILLWAY
INLET WINGWALL STABILITY ANALYSIS - LOADING CASE A

DRAWING
III-52

SLIDING ANALYSIS - INLET WINGWALL CASE A CONT

COMPUTE UPLIFT FORCE 'U' FOR SLIDING ANALYSIS

U = (AREA)(PRESSURE)

WIDTH (W) AS F(X), X MEASURED FROM 'G' : $W = 4.22 + 0.3902X$

μ AS F(X), X MEASURED FROM [GH]: $\mu = 1.275 - 0.01146X$

APPROXIMATE INTEGRATION OF (PRESSURE)(AREA) BY ASSUMING 1' WIDE STRIPS.

$X_{MIDPOINT}$	AREA (A)	PRESSURE (P)	FORCE	MOMENT ARM	MOMENT
0.50	4.42	1.27	5.60	26.20	146.82
1.50	4.81	1.26	6.04	25.20	152.31
2.50	5.20	1.25	6.48	24.20	156.70
3.50	5.59	1.23	6.90	23.20	160.03
4.50	5.98	1.22	7.31	22.20	162.31
5.50	6.37	1.21	7.72	21.20	163.57
6.50	6.76	1.20	8.11	20.20	163.84
7.50	7.15	1.19	8.50	19.20	163.15
8.50	7.54	1.18	8.88	18.20	161.53
9.50	7.93	1.17	9.24	17.20	158.99
10.50	8.32	1.15	9.60	16.20	155.58
11.50	8.71	1.14	9.95	15.20	151.30
12.50	9.10	1.13	10.30	14.20	146.20
13.50	9.49	1.12	10.63	13.20	140.30
14.50	9.88	1.11	10.95	12.20	133.63
15.50	10.27	1.10	11.27	11.20	126.20
16.50	10.66	1.09	11.57	10.20	118.05
17.50	11.05	1.07	11.87	9.20	109.21
18.50	11.44	1.06	12.16	8.20	99.71
19.50	11.83	1.05	12.44	7.20	89.56
20.50	12.22	1.04	12.71	6.20	78.79
21.50	12.61	1.03	12.97	5.20	67.44
22.50	13.00	1.02	13.22	4.20	55.53
23.50	13.39	1.01	13.47	3.20	43.09
24.50	13.78	0.99	13.70	2.20	30.14
25.50	14.17	0.98	13.93	1.20	16.71
26.50	14.56	0.97	14.14	0.20	2.83

SUMMATION U = 279.66 MOMENTS = 3,153.55

$X_{CENTROID}$ = 11.28

COMPUTE RESISTING FORCES

$R_{W1} = 0.5(0.0624)(15)^2(14.56) = 102.21^K$ @ Y = 5.33'

$R_{W2} = 0.969(3.0)(5.41) = 15.73^K$ @ Y = - 1.5'

$R_{W3} = 0.5(1.275 - 0.969)(3.0)(5.41) = 2.48^K$ @ Y = -2.0

SUM MOMENTS ABOUT [AB] - SLIDING

ITEM NO.	DESCRIPTION OF ITEM	FORCES (KIPS) VERTICAL DOWN	VERTICAL UP	HORIZONTAL LEFT	HORIZONTAL RIGHT	MOMENT ARM (FT) X_{ARM}	Y_{ARM}	MOMENTS (KIP-FT) OVERTURN	RESIST
W_c	WEIGHT OF CONCRETE	241.0				10.38			2501.58
W_{S1}	WEIGHT OF SOIL ABOVE HEEL	374.9				11.93			4472.56
W_{S2}	WEIGHT SOIL BENEATH BASE SLAB	33.4				14.37			479.96
U	UPLIFT		279.66			11.28		3154.56	
DRIVING FORCES ALONG FACE (GH)									
D_{W1}					59.19		4.33	256.29	
D_{S1}					3.99		20.83	83.11	
D_{S2}					31.91		8.00	255.28	
D_{S3}					34.23		4.33	148.22	
DRIVING FORCES ALONG FACE (WX-GH)									
D_{W2}					76.56		6.33	484.62	
D_{S4}					6.92		20.83	144.14	
D_{S5}					47.81		9.50	454.20	
D_{S6}					44.30		6.33	280.42	
R_{W1}	RESISTING WATER ON (AB)			102.21			5.33		544.78
R_{W2}	RESISTING WATER ON (PT)			15.73			-1.50		-23.60
R_{W3}	RESISTING WATER ON (PT)			2.48			-2.00		-4.96
R_{S1}	RESISTING SOIL ABOVE 'A'			12.82			2.50		32.05
	SUMMATION	649.3	279.7	133.2	304.9			5260.8	8002.4

SUMMARY OF RESULTS FOR SLIDING ANALYSIS

ΣFORCES (HORIZONTAL) = 171.7^K → ΣFORCES (VERTICAL) = 369.6^K ↓ ΣMOMENTS = 2741.6 FT-K

COMPUTE FACTOR OF SAFETY (FS) AGAINST SLIDING

$\alpha = TAN^{-1}(3.0/26.7) = 6.41^\circ$

$N' = (\Sigma \text{ VERTICAL FORCES}) \cos\alpha + (\Sigma \text{ HORIZONTAL FORCES}) \sin\alpha$

$N' = 369.6 \cos 6.41^\circ + 171.7 \sin 6.41^\circ = 386.5^K$

$T = (\Sigma \text{ HORIZONTAL FORCES}) \cos\alpha - (\Sigma \text{ VERTICAL FORCES}) \sin\alpha$

$T = 171.7 \cos 6.41^\circ - 369.6 \sin 6.41^\circ = 129.4^K$

$FS = (N' \tan\phi + CL)/T$

FS = 386.5 TAN 33°/129.4 = 1.94 > 1.33, THEREFORE O.K.

BEARING CAPACITY ANALYSIS

COMPUTE EFFECTIVE BASE WIDTH

l = 2741.6/386.5 = 7.09' FROM [AB]

BASE AREA FROM l = 7.09 FEET TO [AB] = 94.0 SQ.FT. = A1

ASSUME SYMMETRICAL AREA (A2) FROM l = 7.09' TOWARD [GH].

THEREFORE, BASE AREA NOT SYMMETRICAL ABOUT LOAD = A3 = TOTAL AREA - 2(A1)

A3 = 251.0 - 2(94.0) = 63.0 SQ. FT.

SOLVE QUADRATIC EQUATION FOR DISTANCE FROM [GH] SUCH THAT A3 = 63.0 SQ. FT.

$A3 = [4.22 + (4.22 + 0.3902X)/2]X$

X = 10.16'

B' (EFFECTIVE BASE WIDTH) = B (TOTAL BASE WIDTH) - X (WIDTH NOT SYMMETRICAL TO LOAD)

B' = 26.7 - 10.16 = 16.5'

COMPUTE BEARING CAPACITY FACTORS

$Q = B'[(\xi_{cd}\xi_{ci}\xi_{ct}\xi_{cg} c N_c) + (\xi_{qd}\xi_{qi}\xi_{qt}\xi_{qg} q_o N_q) + (0.5 \cdot \xi_{\gamma d}\xi_{\gamma i}\xi_{\gamma t}\xi_{\gamma g} B' \gamma N_\gamma)]$

D = 7.5 B = 27.3 $\phi = 33^\circ$ C = 0.0 $\alpha = 6.41^\circ$ β = 0.0 RADS

T = 129.4 N' = 386.5 $\delta = TAN^{-1}(T/N') = 18.51^\circ$ B' = 16.5'

$\xi_{cd} = 1 + 0.2(D/B') \tan(45^\circ + \phi/2) = 1 + 0.2(7.5/16.5) \tan(45^\circ + 33/2) = 1.17$

$\xi_{ci} = \xi_{qi} = (1 - \delta^\circ/90^\circ)^2 = (1 - 18.51/90^\circ)^2 = 0.63$

NOTE: N_q, NC, AND N_γ AVAILABLE FROM TABLE 5-1, PGS 5-5 & 5-6 OF EM 1110-2-2502

$N_q = 26.09$

$N_c = 38.64$

$N_\gamma = 26.17$

$\xi_{qd} = \xi_{\gamma d} = 1 + 0.1(D/B') \tan(45^\circ + \phi/2) = 1 + 0.1(7.5/16.5) \tan(45^\circ + 33/2) = 1.08$

$\xi_{qt} = \xi_{\gamma t} = (1 - \alpha \tan\phi)^2$ *(WHERE α IS IN RADIANS)* $= (1 - 0.1119 \tan 33^\circ)^2 = 0.86$

$\xi_{qg} = \xi_{\gamma g} = 1 - \tan\beta)^2 = (1 - \tan 0)^2 = 1.0$

$\xi_{cg} = \xi_{qg} - [(1 - \xi_{qg})/N_c \cdot \tan\phi)] = 1.0 - [(1.0 - 1.0)/(38.64 \cdot \tan 33^\circ)] = 1.0$ (FOR $\phi > 0$)

q_o = (MINIMUM EFFECTIVE OVERBURDEN PRESSURE)

$q_o = 7.5(0.125 - 0.969/15) = 0.453$

$\xi_{\gamma i} = (1 - \delta/\phi)^2 = (1 - 18.51/33)^2 = 0.1928$

$\gamma = 0.125 - (0.969/15) = 0.0604$

COMPUTE BEARING CAPACITY

$Q = (AREA)[(\xi_{cd}\xi_{ci}\xi_{ct}\xi_{cg} c N_c) + (\xi_{qd}\xi_{qi}\xi_{qt}\xi_{qg} q_o N_q) + (0.5 \cdot \xi_{\gamma d}\xi_{\gamma i}\xi_{\gamma t}\xi_{\gamma g} B' \gamma N_\gamma)]$

$Q = 2(94.0)[(1.08)(0.63)(0.86)(1.0)(0.453)(26.09)$
$+ (0.5)(1.08)(0.1928)(0.86)(1.0)(16.8)(0.0604)(26.17)]$

$Q = 1747^K$

COMPUTE FACTOR OF SAFETY

FS = ALLOWABLE/ACTUAL = Q/N' = 1747/386.5 = 4.5 > 2.0 (O.K.)

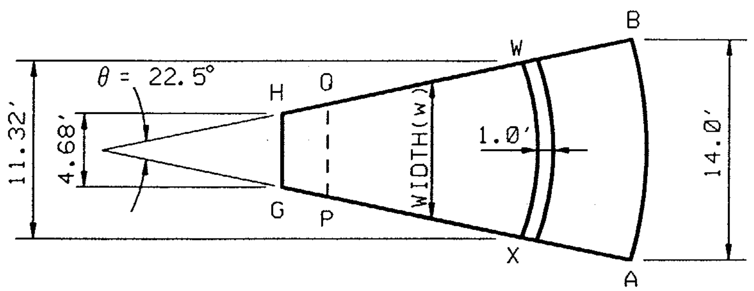
US Army Corps of Engineers Memphis District

U.S. ARMY ENGINEER DISTRICT CORPS OF ENGINEERS MEMPHIS, TENNESSEE

ENVIRONMENTAL RESTORATION NATURAL RESOURCES PROJECT — MISSISSIPPI RIVER AND TRIBUTARIES CONSTRUCTION

REELFOOT LAKE, TENNESSEE & KENTUCKY
ALTERNATIVE SPILLWAY
INLET WINGWALL STABILITY ANALYSIS - LOADING CASE A

DRAWING III-53



PLAN

NTS

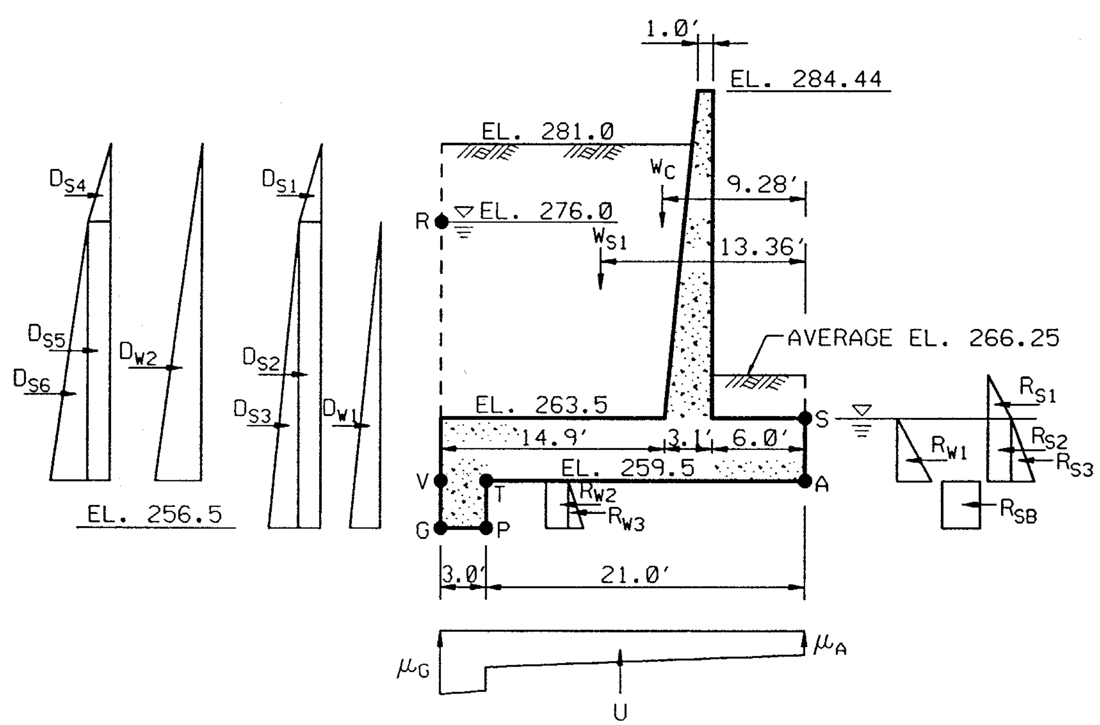
SUMMARY TABLE

LOADING CASE	OVERTURNING RATIO	FACTOR OF SAFETY	
		SLIDING	BEARING
A *HIGH NORMAL WATER TABLE IN BACKFILL W/ MINIMUM TAILWATER	0.39 (RESULTANT WITHIN MID THIRD)	1.6 (1.5)	4.9 (3.0)
B SAME AS CASE A WITH EARTHQUAKE: a=0.15g	0.21 (RESULTANT WITHIN BASE)	1.1 (1.1)	2.0 (>1.0)
C LANDSIDE CRACK - FULL HYDROSTATIC PRESSURES	0.50 (RESULTANT WITHIN MID THIRD)	6.0 (1.5)	14.3 (3.0)
D CONSTRUCTION CASE - NO WATER	0.46 (>0.25)	3.0 (1.33)	12.4 (2.0)

*CASE PRESENTED

NOTES:

1. ALL ANALYSES PERFORMED IN ACCORDANCE WITH EM1110-2-2502, DATED 29 SEP 89.
2. VALUES WITHIN PARENTHESIS IN SUMMARY TABLE ARE MINIMUMS PRESENTED IN TABLE 4-1, 'RETAINING WALL STABILITY CRITERIA', IN REFERENCED EM1110-2-2502.

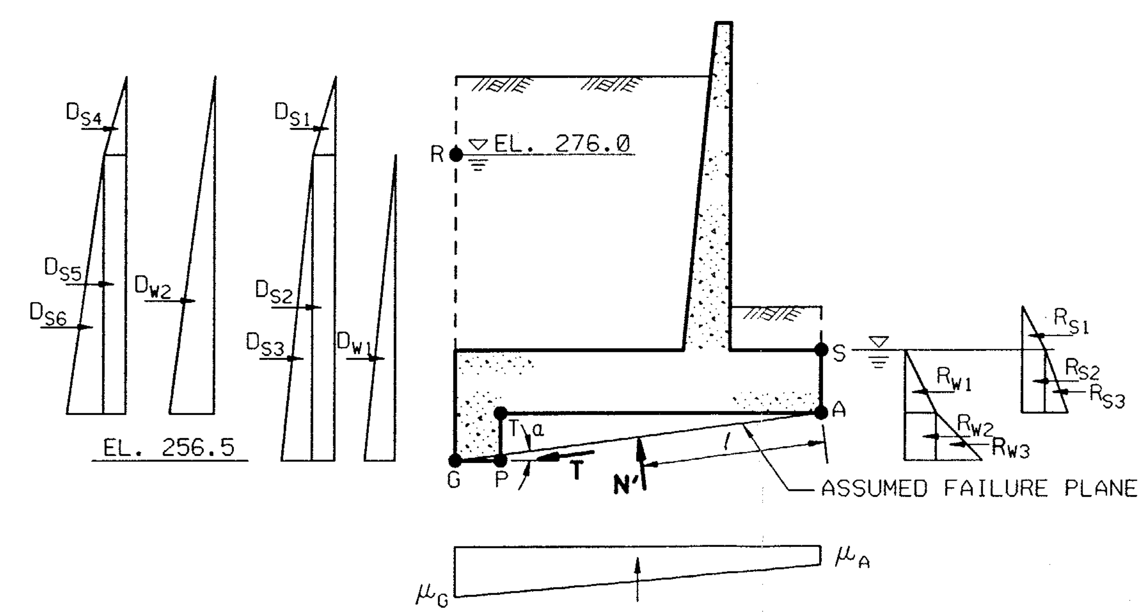


ELEVATION

(OVERTURNING ANALYSIS)

NTS

CASE A



ELEVATION

(SLIDING AND BEARING ANALYSIS)

NTS

CASE A

US Army Corps of Engineers Memphis District

U.S. ARMY ENGINEER DISTRICT CORPS OF ENGINEERS MEMPHIS, TENNESSEE

ENVIRONMENTAL RESTORATION NATURAL RESOURCES PROJECT

MISSISSIPPI RIVER AND TRIBUTARIES CONSTRUCTION

REELFOOT LAKE, TENNESSEE & KENTUCKY

ALTERNATIVE SPILLWAY

OUTLET WINGWALL STABILITY ANALYSIS - LOADING CASE A

DRAWING

III-54

OVERTURNING ANALYSIS - OUTLET WINGWALL CASE A

COMPUTE HYDROSTATIC PRESSURES

SEEPAGE PATH = [RGAS] = 47.69'

POINT	SEEPAGE HEAD (FT γ)	+ POSITION HEAD (FT γ)	= TOTAL HEAD (FT γ)	(KSF)
R	(47.69/47.69)(12.5)	-12.5	0.00	0.00
V	(31.19/47.69)(12.5)	4.0	12.18	0.760
G	(28.19/47.69)(12.5)	7.0	14.39	0.898
A	(4.0/47.69)(12.5)	4.0	5.05	0.315
S	(0/47.69)(12.5)	0.0	0.00	0.00

PRORATE HEAD LOSS FROM 'G' TO 'A' FOR POINTS 'P' & 'T'

SEEPAGE LOSS ALONG 'GA' = (24.19/47.69)(12.5)γ = 6.34'γ

SEEPAGE PATH 'GPTA' = 27'

μ(P) = 0.898 - (3/27)(6.34'γ) = 0.854 KSF

μ(T) = 0.898 - (6/27)(6.34'γ) - 3.0γ = 0.623 KSF

COMPUTE UPLIFT FORCE 'U' FOR OVERTURNING ANALYSES

U = (AREA)(PRESSURE)

WIDTH (W) AS F(X), X MEASURED FROM 'G' : W = 4.68 + 0.38833X

μ AS F(X), X MEASURED FROM 'G': X = 0 TO 3 μ = 0.898 - 0.014667X
X = 3 TO 24 μ = 0.667 - 0.014667X

APPROXIMATE INTEGRATION OF (PRESSURE) * (AREA) BY ASSUMING 1' WIDE STRIPS.

$X_{MIDPOINT}$	AREA (A)	PRESSURE (P)	FORCE	MOMENT ARM	MOMENT
0.50	4.87	0.89	4.34	23.50	102.02
1.50	5.26	0.88	4.61	22.50	103.72
2.50	5.65	0.86	4.87	21.50	104.65
3.50	6.04	0.62	3.72	20.50	76.22
4.50	6.43	0.60	3.86	19.50	75.33
5.50	6.82	0.59	4.00	18.50	73.93
6.50	7.20	0.57	4.12	17.50	72.07
7.50	7.59	0.56	4.23	16.50	69.78
8.50	7.98	0.54	4.33	15.50	67.09
9.50	8.37	0.53	4.42	14.50	64.03
10.50	8.76	0.51	4.49	13.50	60.65
11.50	9.15	0.50	4.56	12.50	56.97
12.50	9.53	0.48	4.61	11.50	53.03
13.50	9.92	0.47	4.65	10.50	48.86
14.50	10.31	0.45	4.68	9.50	44.50
15.50	10.70	0.44	4.70	8.50	39.98
16.50	11.09	0.42	4.71	7.50	35.34
17.50	11.48	0.41	4.71	6.50	30.61
18.50	11.86	0.40	4.69	5.50	25.82
19.50	12.25	0.38	4.67	4.50	21.01
20.50	12.64	0.37	4.63	3.50	16.21
21.50	13.03	0.35	4.58	2.50	11.45
22.50	13.42	0.34	4.52	1.50	6.78
23.50	13.81	0.32	4.45	0.50	2.22

SUMMATION FORCES = 107.16 MOMENTS = 1,262.28

$X_{CENTROID}$ = 11.78 FROM 'A'

COMPUTE DRIVING FORCES ACTING ON [GH]

D_{W1} = (0.5)(0.898)(19.5)(4.68) = 40.98^K @ Y = 3.5'

K_o = 1 - SIN ϕ = 1 - SIN 30° = 0.50

D_{S1} = 0.5(0.125)(5.0)^2(4.68)(0.5) = 3.66^K @ Y = 18.17'

D_{S2} = 5.0(0.125)(19.5)(0.50)(4.68) = 28.52^K @ Y = 6.75'

D_{S3} = 0.5(γB)H^2[GH]K_o WHERE γB = ((19.5)(0.125) - 0.898)/19.5 = 0.0789

D_{S3} = 0.50(0.0789)(19.5)^2(4.68)(0.50) = 35.10^K @ Y = 3.50'

COMPUTE DRIVING FORCES ACTING ON [WX-GH]

D_{W2} = (0.5)(0.760)(16.5)(6.64) = 41.63^K @ Y = 5.5'

D_{S4} = 0.5(0.125)(5.0)^2(6.64)(0.5) = 5.19^K @ Y = 18.17'

D_{S5} = 5.0(0.125)(16.5)(0.50)(6.64) = 34.24^K @ Y = 5.50'

D_{S6} = 0.5(γB)H^2[WX-GH]K_o WHERE γB = ((16.5)(0.125) - 0.760)/16.5 = 0.0789

D_{S6} = 0.50(0.0789)(16.5)^2(6.64)(0.50) = 35.68^K @ Y = 5.50'

COMPUTE RESISTING FORCES

R_{W1} = 0.5(0.315)(4.0)(14.0) = 8.82^K @ Y = 1.33'

R_{W2} = 0.623(3.0)(5.84) = 10.91^K @ Y = -1.5'

R_{W2} = 0.5(0.854 - 0.623)(3.0)(5.84) = 2.02^K @ Y = -2.0'

R_{S1} = 0.5(0.125)(2.75)^2(0.50)(11.67) = 2.76^K @ Y = 4.92'

R_{S2} = 2.75(0.125)(4.0)(0.50)(14.0) = 9.63^K @ Y = 2.0'

R_{S3} = 0.5(γB)H^2[AB]K_o WHERE γB = ((4.0)(0.125) - 0.315)/4.0 = 0.04625

R_{S3} = 0.50(0.04625)(4.0)^2(14.0) (0.50) = 2.59^K @ Y = 1.33'

R_{SB} = AS REQUIRED TO BALANCE HORIZONTAL FORCES

SUM MOMENTS ABOUT [AB] - OVERTURNING

ITEM NO.	DESCRIPTION OF ITEM	FORCES (KIPS) VERTICAL DOWN	VERTICAL UP	HORIZONTAL LEFT	HORIZONTAL RIGHT	MOMENT ARM (FT) X_{ARM}	Y_{ARM}	MOMENTS (KIP-FT) OVERTURN	RESIST
W_c	WEIGHT OF CONCRETE	206.1				9.28			1912.70
W_{S1}	WEIGHT OF SOIL ABOVE HEEL	235.7				13.36			3149.49
W_{S2}	(WEIGHT SOIL BENEATH BASE SLAB-N/A)								
U	UPLIFT		107.2			11.78		1262.34	
DRIVING FORCES ALONG FACE (GH)									
D_{W1}					41.0		3.50	143.43	
D_{S1}					3.7		18.17	66.50	
D_{S2}					28.5		6.75	192.51	
D_{S3}					35.1		3.50	122.85	
DRIVING FORCES ALONG FACE (WX-GH)									
D_{W2}					41.6		5.50	228.97	
D_{S4}					5.2		18.17	94.30	
D_{S5}					34.2		5.50	188.32	
D_{S6}					35.7		5.50	196.24	
R_{W1}	RESISTING WATER ON (AB)			8.8			1.33		11.73
R_{W2}	RESISTING WATER ON (PT)			10.9			-1.50		-16.37
R_{W3}	RESISTING WATER ON (PT)			2.0			-2.00		-4.04
R_{S1}	RESISTING SOIL ABOVE 'A'			2.8			4.92		13.58
R_{S2}	RESISTING SOIL ABOVE 'A'			9.6			2.00		19.26
R_{S3}	RESISTING SOIL ABOVE 'A'			2.6			1.33		3.44
R_{SB}	REQUIRED SOIL FORCE FOR BALANCE OF FORCES			188.3			-1.50		-282.45
SUMMATION		442.0	107.2	225.0	225.0			2495.00	4807.00

SUMMARY OF RESULTS FOR OVERTURNING ANALYSIS

ΣFORCES (HORIZONTAL) = 0.0 ΣFORCES (VERTICAL) = 335K ↓ ΣMOMENTS = 2312 FT-K

X_R (RESULTANT) = ΣMOMENTS / ΣFORCES (VERTICAL) = 2312 FT-K/ 335^K = 6.91' FROM [AB]

COMPUTER BASE WIDTH @ X_R : W = 4.68 + 0.38833(24 - 6.91) = 11.31'

BASE AREA FROM X_R TO [AB] = 0.5(11.31+14.0)(6.91) = 87.5 SQ. FT.

TOTAL BASE AREA = 0.5(4.68 + 14.0)(24) = 224.2 SQ. FT.

RESULTANT RATIO (BASED ON AREA) = 87.5/224.2 = 0.39 > 0.33, THEREFORE O.K.

SLIDING ANALYSIS

COMPUTE W_{S2} AND ITS CENTROID (WEIGHT OF SOIL BENEATH SLAB)

$X_{MIDPOINT}$	AREA (A)	SOIL THICKNESS (T)	SOIL WEIGHT (W)	MOMENT ARM	MOMENT
0.50	4.87	0.00	0.00	23.50	0.00
1.50	5.26	0.00	0.00	22.50	0.00
2.50	5.65	0.00	0.00	21.50	0.00
3.50	6.04	2.56	1.93	20.50	39.66
4.50	6.43	2.44	1.96	19.50	38.19
5.50	6.82	2.31	1.97	18.50	36.45
6.50	7.20	2.19	1.97	17.50	34.47
7.50	7.59	2.06	1.96	16.50	32.30
8.50	7.98	1.94	1.93	15.50	29.96
9.50	8.37	1.81	1.90	14.50	27.49
10.50	8.76	1.69	1.85	13.50	24.94
11.50	9.15	1.56	1.79	12.50	22.33
12.50	9.53	1.44	1.71	11.50	19.70
13.50	9.92	1.31	1.63	10.50	17.09
14.50	10.31	1.19	1.53	9.50	14.54
15.50	10.70	1.06	1.42	8.50	12.08
16.50	11.09	0.94	1.30	7.50	9.74
17.50	11.48	0.81	1.17	6.50	7.58
18.50	11.86	0.69	1.02	5.50	5.61
19.50	12.25	0.56	0.86	4.50	3.88
20.50	12.64	0.44	0.69	3.50	2.42
21.50	13.03	0.31	0.51	2.50	1.27
22.50	13.42	0.19	0.31	1.50	0.47
23.50	13.81	0.06	0.11	0.50	0.05

SUMMATION W_{S2} = 29.51 MOMENTS = 380.22

$X_{CENTROID}$ = 12.88

COMPUTE DRIVING FORCES

(SAME AS FOR OVERTURNING ANALYSIS)

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ALTERNATIVE SPILLWAY
OUTLET WINGWALL STABILITY ANALYSIS - LOADING CASE A

U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
MEMPHIS, TENNESSEE

DRAWING III-55

SLIDING ANALYSIS - OUTLET WINGWALL CASE A CONTD

COMPUTE UPLIFT

μ AS F(X), X MEASURED FROM 'G'

μ = 0.898 - 0.024292X

$X_{MIDPOINT}$	AREA (A)	PRESSURE (P)	FORCE	MOMENT ARM	MOMENT
0.50	4.87	0.89	4.32	23.50	101.47
1.50	5.26	0.86	4.53	22.50	102.01
2.50	5.65	0.84	4.73	21.50	101.72
3.50	6.04	0.81	4.91	20.50	100.65
4.50	6.43	0.79	5.07	19.50	98.85
5.50	6.82	0.76	5.21	18.50	96.38
6.50	7.20	0.74	5.33	17.50	93.31
7.50	7.59	0.72	5.43	16.50	89.67
8.50	7.98	0.69	5.52	15.50	85.54
9.50	8.37	0.67	5.58	14.50	80.97
10.50	8.76	0.64	5.63	13.50	76.01
11.50	9.15	0.62	5.66	12.50	70.72
12.50	9.53	0.59	5.67	11.50	65.17
13.50	9.92	0.57	5.66	10.50	59.39
14.50	10.31	0.55	5.63	9.50	53.46
15.50	10.70	0.52	5.58	8.50	47.42
16.50	11.09	0.50	5.51	7.50	41.34
17.50	11.48	0.47	5.43	6.50	35.27
18.50	11.86	0.45	5.32	5.50	29.27
19.50	12.25	0.42	5.20	4.50	23.39
20.50	12.64	0.40	5.06	3.50	17.70
21.50	13.03	0.38	4.90	2.50	12.24
22.50	13.42	0.35	4.72	1.50	7.07
23.50	13.81	0.33	4.52	0.50	2.26

SUMMATION U = 125.10 MOMENTS = 1,491.31

$X_{CENTROID}$ = 11.92

COMPUTE RESISTING FORCES

R_{W1} = 0.5(0.315)(4)(14.0) = 8.82^K @ Y = 1.33'

R_{W2} = 0.315(3.0)(5.84) = 5.52^K @ Y = - 1.5'

R_{W2} = 0.5((0.898-0.315)(3.0)(5.84) = 5.11^K @ Y = -2.0

R_{S1}, R_{S2}, R_{S3} = SAME AS FOR OVERTURNING ANALYSIS

R_{SB} = NOT APPLICABLE

SUM MOMENTS ABOUT [AB] - SLIDING

ITEM NO.	DESCRIPTION OF ITEM	FORCES (KIPS) VERTICAL DOWN	UP	HORIZONTAL LEFT	RIGHT	MOMENT ARM (FT) X_{ARM}	Y_{ARM}	MOMENTS (KIP-FT) OVERTURN	RESIST
W_c	WEIGHT OF CONCRETE	206.1				9.28			1912.70
W_{S1}	WEIGHT OF SOIL ABOVE HEEL	235.7				13.36			3149.49
W_{S2}	WEIGHT SOIL BENEATH BASE SLAB	29.5				12.88			380.09
U	UPLIFT		125.1			11.92		1491.19	
DRIVING FORCES ALONG FACE (GH)									
D_{W1}					41.0		3.50	143.43	
D_{S1}					3.7		18.17	66.50	
D_{S2}					28.5		6.75	192.51	
D_{S3}					35.1		3.50	122.85	
DRIVING FORCES ALONG FACE (WX-GH)									
D_{W2}					41.6		5.50	228.97	
D_{S4}					5.2		18.17	94.30	
D_{S5}					34.2		5.50	188.32	
D_{S6}					35.7		5.50	196.24	
R_{W1}	RESISTING WATER ON (AB)			8.8			1.33		11.73
R_{W2}	RESISTING WATER ON (PT)			5.5			-1.50		-8.28
R_{W3}	RESISTING WATER ON (PT)			5.1			-2.00		-10.21
R_{S1}	RESISTING SOIL ABOVE 'A'			2.8			4.92		13.58
R_{S2}	RESISTING SOIL ABOVE 'A'			9.6			2.00		19.26
R_{S3}	RESISTING SOIL ABOVE 'A'			2.6			1.33		3.44
R_{SB}									
	SUMMATION	471.0	125.0	34.0	225.0			2724.00	5472.00

SUMMARY OF RESULTS FOR SLIDING ANALYSIS

ΣFORCES (HORIZONTAL) = 191^K → ΣFORCES (VERTICAL) = 346^K ↓ ΣMOMENTS = 2747 FT-K ↶

COMPUTE FACTOR OF SAFETY (FS) AGAINST SLIDING

α = TAN^{-1}(3.0/24.0) = 7.125°

N' = (Σ VERTICAL FORCES) COS α + (Σ HORIZONTAL FORCES) SIN α
N' = 346 COS 7.125° + 191 SIN 7.125° = 367.0^K

T = (Σ HORIZONTAL FORCES) COS α - (Σ VERTICAL FORCES) SIN α
T = 191 COS 7.125° - 346 SIN 7.125° = 146.6^K

FS = (N' TAN ϕ + CL)/T
FS = 367 TAN 33°/146.6 = 1.63 > 1.50 , THEREFORE O.K.

BEARING CAPACITY ANALYSIS

COMPUTE EFFECTIVE BASE WIDTH

l = 2747/367 = 7.49' FROM [AB]

BASE AREA FROM = 7.49 FEET TO [AB] = 94.0 SQ.FT. = A1

ASSUME SYMMETRICAL AREA (A2) FROM l = 7.49' TOWARD [GH] .

THEREFORE, BASE AREA NOT SYMMETRICAL ABOUT LOAD = A3 = TOTAL AREA - 2(A1)

A3 = 224.2 - 2(94.0) = 36.2 SQ. FT.

SOLVE QUADRATIC EQUATION FOR DISTANCE FROM [GH] SUCH THAT A3 = 36.2 SQ. FT.

A3 = [4.68 + (4.68+0.38833X)/2]X

X = 6.16'

B' (EFFECTIVE BASE WIDTH) = B (TOTAL BASE WIDTH) - X (WIDTH NOT SYMMETRICAL TO LOAD)

B' = 24.0 - 6.16 = 17.84'

COMPUTE BEARING CAPACITY FACTORS

$Q = B'[(\xi_{cd}\xi_{ci}\xi_{ct}\xi_{cg}cN_c) + (\xi_{qd}\xi_{qi}\xi_{qt}\xi_{qg}q_o N_q) + (0.5 \cdot \xi_{\gamma d}\xi_{\gamma i}\xi_{\gamma t}\xi_{\gamma g}B'\gamma N_\gamma)]$

D = 6.75 B = 24.0 ϕ = 33° C = 0.0 α = 7.125° β = 0.0 RADS

T = 146.6^K N' = 367.0^K δ = TAN^{-1}(T/N') = 21.77° B' = 17.84'

ξ_{cd} = 1 + 0.2(D/B') TAN(45° + ϕ/2) = 1 + 0.2(6.75/17.84) TAN(45° + 33/2) = 1.14

$\xi_{ci} = \xi_{qi}$ = (1 - δ°/ 90°)^2 = (1 - 21.77/90°)^2 = 0.57

NOTE: N_q, NC, AND N_γ AVAILABLE FROM TABLE 5-1, PGS 5-5 & 5-6 OF EM 1110-2-2502

N_q = 26.09

N_c = 38.64

N_γ = 26.17

$\xi_{qd} = \xi_{\gamma d}$ = 1 + 0.1(D/B') TAN(45° + ϕ/2) = 1 + 0.1(6.75/17.84) TAN(45° + 33/2) = 1.07

$\xi_{qt} = \xi_{\gamma t}$ = (1 - α TAN ϕ)^2 *(WHERE α IS IN RADIANS)* = (1 - 0.1244 TAN 33°)^2 = 0.84

$\xi_{qg} = \xi_{\gamma g}$ = 1 - TAN β)^2 = (1 - TAN 0)^2 = 1.0

$\xi_{cg} = \xi_{qg}$ - [(1 - ξ_{qg})/N_c $\cdot$ TAN ϕ)] = 1.0 - [(1.0 - 1.0)/(38.64 $\cdot$ TAN 33°)] = 1.0 (FOR ϕ > 0°)

q_o = (MINIMUM EFFECTIVE OVERBURDEN PRESSURE)

q_o = 2.75(0.125) + 4(0.0626) = 0.59

$\xi_{\gamma i}$ = (1 - δ/ϕ)^2 = (1 - 21.77/33)^2 = 0.12

γ = 0.0626

COMPUTE BEARING CAPACITY

$Q = (AREA)[(\xi_{cd}\xi_{ci}\xi_{ct}\xi_{cg}cN_c) + (\xi_{qd}\xi_{qi}\xi_{qt}\xi_{qg}q_o N_q) + (0.5 \cdot \xi_{\gamma d}\xi_{\gamma i}\xi_{\gamma t}\xi_{\gamma g}B'\gamma N_\gamma)]$

Q = 2(94.0)[0 + (1.07)(0.57)(0.84)(1.0)(0.59)(26.09)
+ 0.5(1.07)(0.12)(0.84)(1.0)(17.84)(.0626)(26.17)]

Q = 188(0 + 7.89 + 1.58) = 1780^K

COMPUTE FACTOR OF SAFETY

FS = ALLOWABLE/ACTUAL = Q/N' = 1780/367 = 4.85 > 3.0 (O.K.)

US Army Corps of Engineers Memphis District

U.S. ARMY ENGINEER DISTRICT
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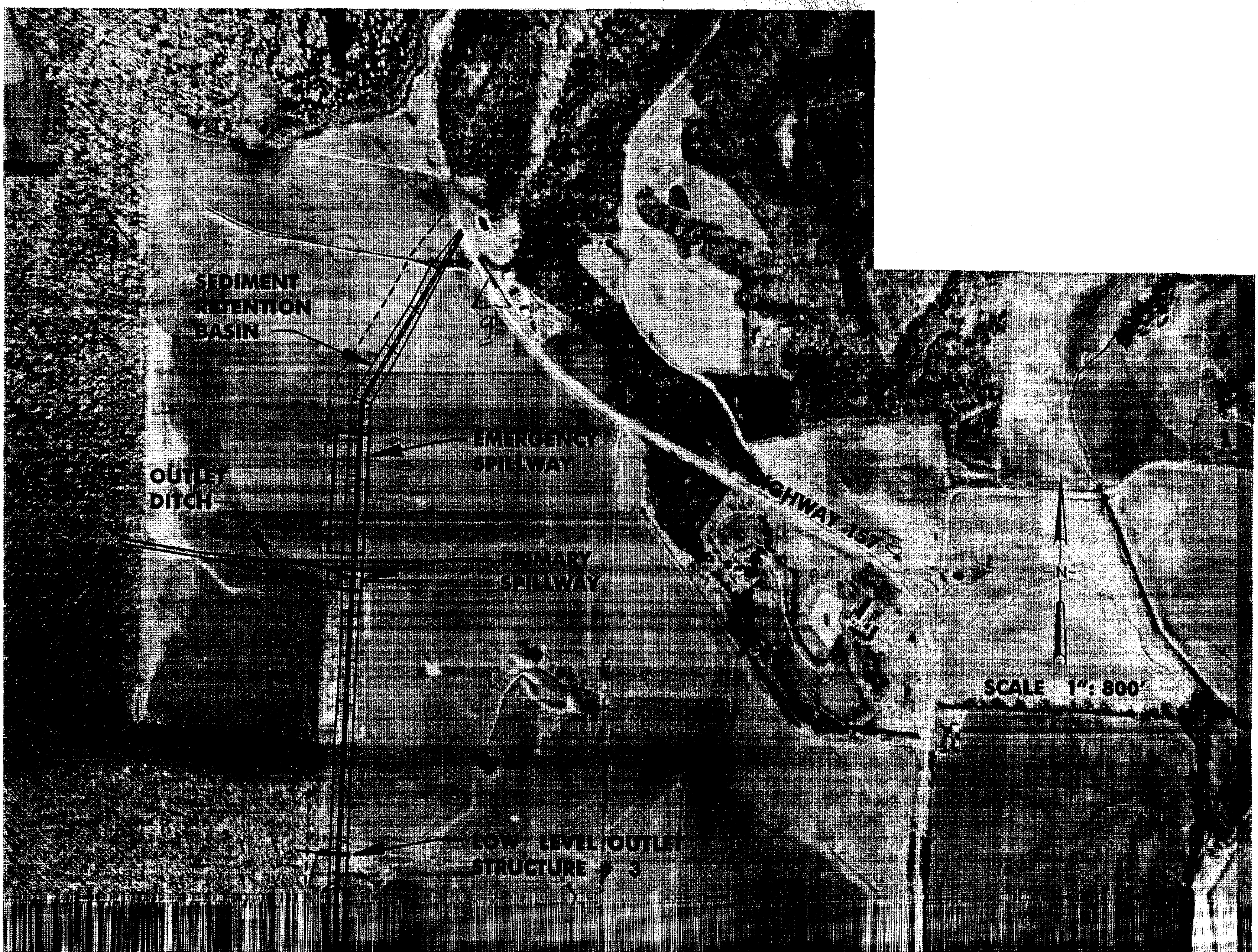
MEMPHIS, TENNESSEE

ENVIRONMENTAL RESTORATION NATURAL RESOURCES PROJECT
MISSISSIPPI RIVER AND TRIBUTARIES CONSTRUCTION

REELFOOT LAKE, TENNESSEE & KENTUCKY
ALTERNATIVE SPILLWAY
OUTLET WINGWALL STABILITY ANALYSIS - LOADING CASE A

DRAWING III-56



SEDIMENT RETENTION BASIN
EMERGENCY SPILLWAY
OUTLET DITCH
PRIMARY SPILLWAY
GHWAY 157
N
SCALE 1": 800'
LOW LEVEL OUTLET STRUCTURE # 3

DRAWING III-57

ENVIRONMENTAL RESTORATION NATURAL RESOURCES PROJECT

MISSISSIPPI RIVER AND TRIBUTARIES CONSTRUCTION

REELFOOT LAKE, TENNESSEE & KENTUCKY

SEDIMENT RETENTION BASIN

U.S ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
MEMPHIS, TENNESSEE

Designed by: DSC | Drawn by: DSC | Checked by: NEN | Checked by: NEN | Plotted by: chastad | Plot scale: 800.000000:1.000000 | Pen table: d:\users\mphscadd\...\pen

Submitted by: RONALD D. SMITH, P.E. Chief, Geotechnical/Design Section | Feature table: NONE | Color table: color.ctb | Plot date: 02-JUN-1999

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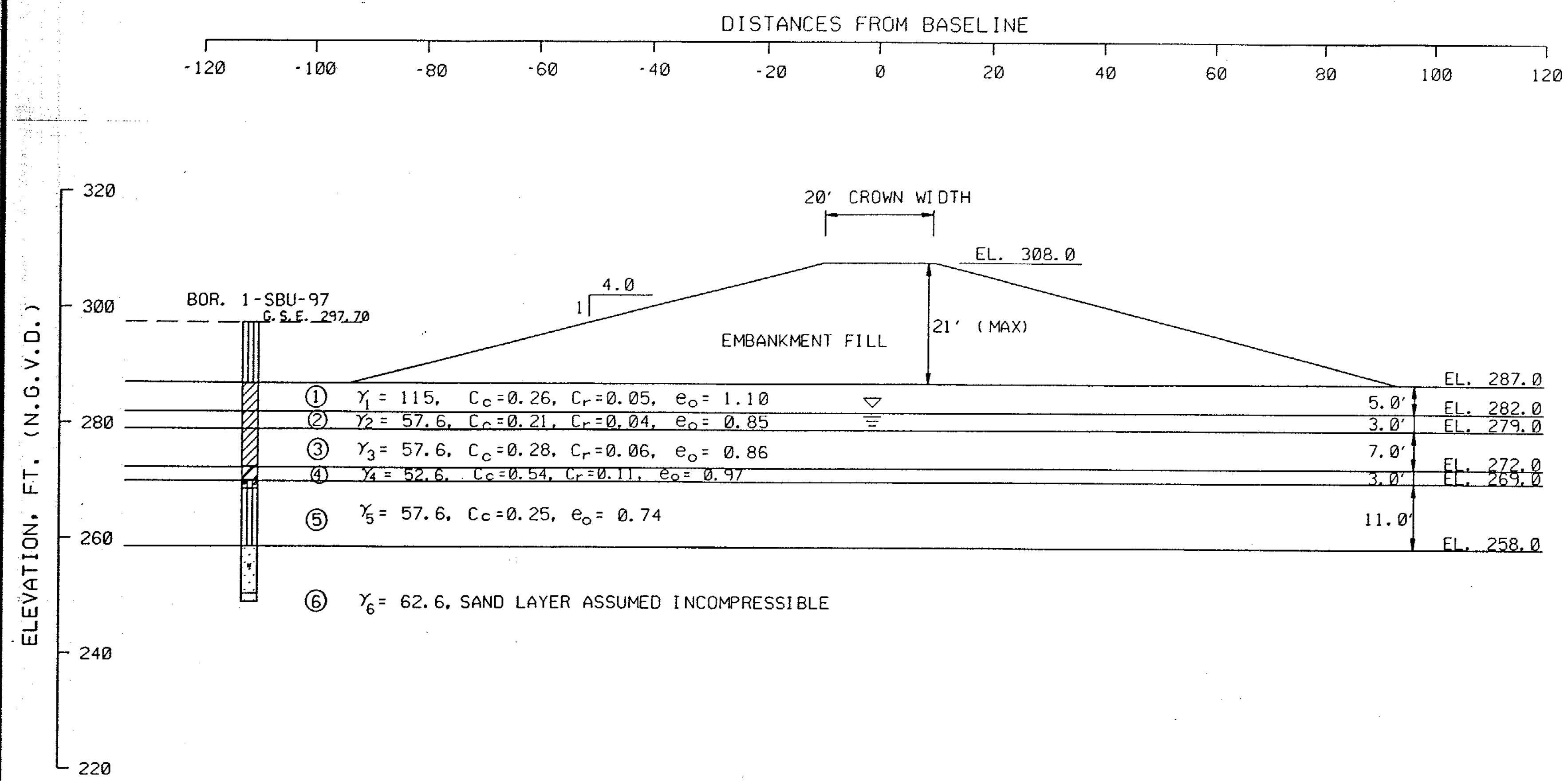
US Army Corps of Engineers Memphis District



LAYER	ELEVATION	DEPTH TO MIDPOINT	H (FT)	C_c	C_r	e_o	$S = \frac{C_r H}{1 + e_o} \text{LOG} \frac{P_o + \Delta P}{P_o}$
1	287.0-282.0	2.50	5.0	0.26	0.05	1.10	0.47 *
2	282.0-279.0	6.50	3.0	0.21	0.04	0.85	0.04
3	279.0-272.0	11.50	7.0	0.28	0.06	0.86	0.13
4	272.0-269.0	16.50	3.0	0.54	0.11	0.97	0.06
5	269.0-258.0	23.50	11.0	0.25	--	0.74	0.57
							1.27 FT = 15.2 INCHES

* OBTAINED USING C_r VALUES FROM INITIAL LOADING UNTIL $P_o + \Delta P = \sigma_p'$ AND THEN C_c VALUES FOR REMAINDER OF LOADING

CONCLUSION

THE ANALYSIS PRESENTED IS FOR THE MAXIMUM EXPECTED FILL FOR THE EMBANKMENT WHICH OCCURS NEAR THE PROPOSED SITE OF THE PRIMARY SPILLWAY. BORING 1-SBU-97 IS THE CLOSEST BORING TO THE NORTHERN PORTION OF THE RETENTION BASIN IN WHICH RIGHT OF ENTRY WAS DENIED. ADDITIONAL BORINGS AND LAB TEST DATA ON THE NORTHERN PORTION OF THE RETENTION BASIN NEAR THE PRIMARY AND EMERGENCY SPILLWAYS WILL BE REQUIRED PRIOR TO CONSTRUCTION.

ASSUMPTIONS

1. USE STRATIFICATION AND SOIL TEST DATA FROM BORING 1-SBU-97

2. MAXIMUM FILL IS APPROXIMATELY 21 FEET ABOVE NATURAL GROUND NEAR THE LOCATION OF THE PROPOSED PRIMARY SPILLWAY. THE SURCHARGE LOAD FROM THE FILL WAS COMPUTED AS :

$$P = 21 \text{ FT } (0.120 \text{ KCF}) = 2.52 \text{ KSF}$$

3. THE FILL REQUIRED FOR THE EMBANKMENT SHALL BE PLACED TO ELEVATION 308.0 USING SEMI COMPACTED EFFORT. HOWEVER, THE FILL REQUIRED FOR THE EMBANKMENT AT THE PRIMARY AND EMERGENCY SPILLWAYS SHALL BE COMPACTED TO A MINIMUM OF 95% STANDARD PROCTOR DENSITY TO STRUCTURE GRADE.

4. THE VALUES FOR THE COMPRESSION INDEX, C_c, WERE BASED ON THE EMPIRICAL RELATIONSHIP C_c = 0.01(LL - 13) AS SHOWN IN EM 1110-1-1904, TABLE 3-7. THESE VALUES WERE ALSO COMPARED TO THE EMPIRICAL CORRELATION BETWEEN COMPRESSION INDEX AND IN SITU WATER CONTENT FOR CLAY AND SILT DEPOSITS AS SHOWN IN FIGURE 16.3, 'SOIL MECHANICS IN ENGINEERING PRACTICE', 3RD ED., 1996, BY TERZAGHI, PECK, AND MESRI

5. THE INCREASE IN VERTICAL STRESS, ΔP, FROM THE SURCHARGE OF THE FILL WAS DETERMINED FROM THE 'TABLES OF BOUSSINESQ COEFFICIENTS FOR VERTICAL STRESS INDUCTION', NEW ORLEANS DISTRICT, MARCH 1969.

6. BASED ON A REVIEW OF WATER CONTENTS, ATTERBERG LIMITS, AND THE UNDRAINED SHEAR STRENGTHS OF THE CLAY SAMPLES FROM BORING 1-SBU-97, IT APPEARS THAT THE MAJORITY OF THE CLAY IS SLIGHTLY PRECONSOLIDATED. IN LAYER 1, THE PRECONSOLIDATION STRESS IS BETWEEN THE PRESENT STRESS AND ANTICIPATED STRESS AFTER FILL PLACEMENT ($P_o + \Delta P$). IN LAYERS 2-4, THE PRECONSOLIDATION STRESS IS APPROXIMATELY EQUAL TO THE ANTICIPATED STRESS AFTER FILL PLACEMENT. ($P_o + \Delta P$)

$$\sigma_p' = \frac{C_u}{0.11 + 0.0037 \text{ PI}} \qquad \text{(EQ. 1-2, EM 1110-1-1904)}$$

WHERE σ_p' = PRECONSOLIDATION STRESS, KSF
C_u = UNDRAINED SHEAR STRENGTH, KSF
PI = PLASTICITY INDEX, PERCENT

LAYER NO.	ELEVATION	PI	C_u (KSF)	σ_p' (KSF)	P_o (KSF)	ΔP (KSF)	$P_o + \Delta P$ (KSF)
1	287.0-282.0	18	0.350	1.98	0.288	2.52	2.81
2	282.0-279.0	14	0.500	3.09	0.661	2.50	3.16
3	279.0-272.0	20	0.600	3.26	0.949	2.46	3.41
4	272.0-269.0	45	0.800	2.89	1.230	2.40	2.95
5	269.0-258.0	--	--	--	1.626	2.30	3.74

7. IT IS ANTICIPATED THAT THE MAJORITY OF LOADING FROM THE 21 FEET OF FILL WILL FALL ALONG THE RECOMPRESSION PORTION OF THE CONSOLIDATION CURVES RATHER THAN THE VIRGIN COMPRESSION PORTION. THEREFORE, C_r VALUES WERE USED TO DETERMINE THE ESTIMATED SETTLEMENT (RECOMPRESSION INDEX).

THESE C_r VALUES WERE ESTIMATED AS 15 TO 20 PERCENT OF C_c VALUES BASED ON EXPERIENCE AND NUMEROUS CONSOLIDATION TESTS ON THIS PROJECT AND OTHER PROJECTS WITHIN THE MEMPHIS DISTRICT.

ADDITIONAL TESTING IN ALL STRATA WILL BE REQUIRED TO DETERMINE IF THESE ASSUMPTIONS ARE VALID.

8. THE ESTIMATED TOTAL FOUNDATION SETTLEMENT AT THE PRIMARY SPILLWAY WAS ESTIMATED TO BE APPROXIMATELY 15 INCHES. TIME-RATE OF CONSOLIDATION WAS DETERMINED USING THE CORPS OF ENGINEERS PROGRAM 'CSETT', WHICH USES TERZAGHI'S CLASSICAL THEORY OF ONE DIMENSIONAL CONSOLIDATION. THE RESULTS INDICATE THAT PRELOADING THE PRIMARY SPILLWAY SITE TO AN ELEVATION OF 308.0 FOR A PERIOD OF 6 MONTHS WILL RESULT IN OVER 95% OF THE ESTIMATED SETTLEMENT OCCURING DURING THE PRELOAD PERIOD.

9. A SETTLEMENT ANALYSIS USING BORING NO. 1-SBU-97 WAS ALSO PERFORMED AT THE LOCATION OF THE LOW LEVEL OUTLET STRUCTURE NO. 2. THE ESTIMATED SETTLEMENT FOR THE EMBANKMENT FOUNDATION RANGED FROM APPROXIMATELY 8 INCHES NEAR THE CENTERLINE TO LESS THAN 1 INCH AT THE TOE.

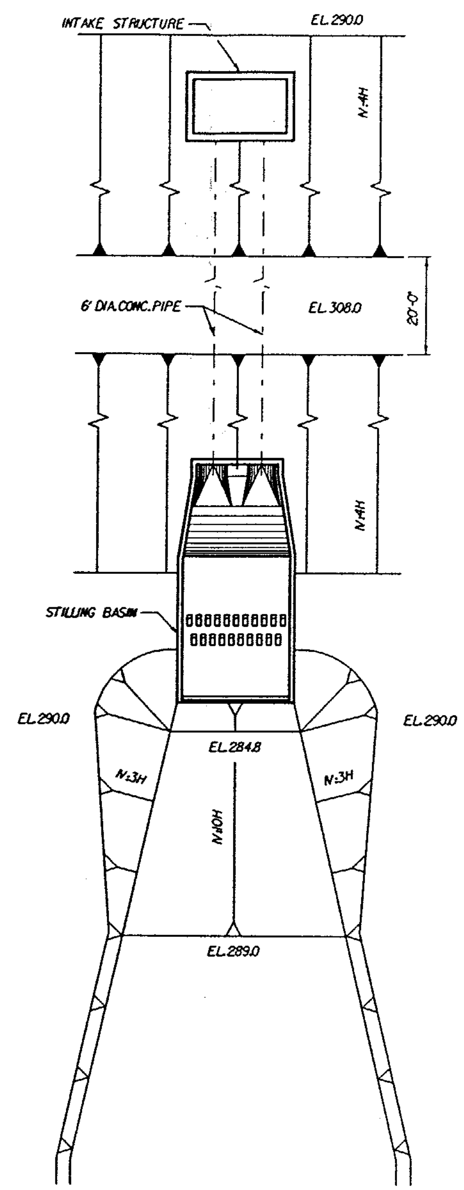
US Army Corps of Engineers Memphis District

U.S. ARMY ENGINEER DISTRICT CORPS OF ENGINEERS MEMPHIS, TENNESSEE

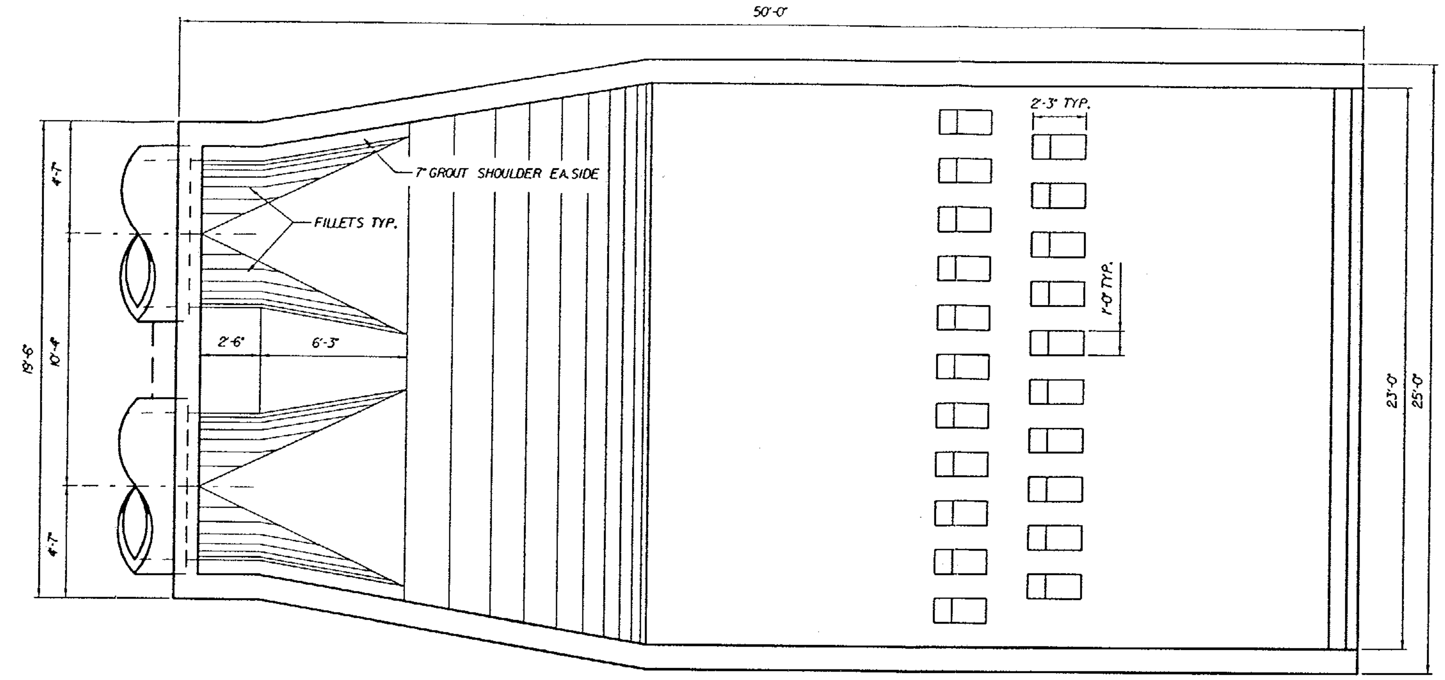
ENVIRONMENTAL RESTORATION NATURAL RESOURCES PROJECT
MISSISSIPPI RIVER AND TRIBUTARIES CONSTRUCTION

REELFOOT LAKE, TENNESSEE & KENTUCKY
SEDIMENT RETENTION BASIN
SETTLEMENT ANALYSIS

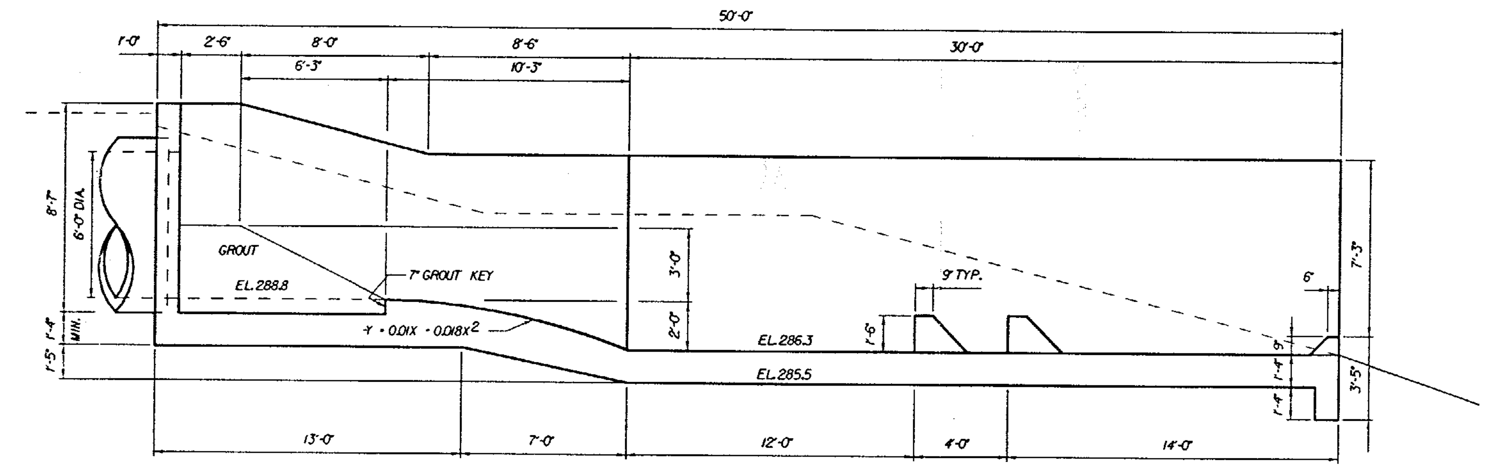
DRAWING
III-58



SITE PLAN
SCALE 1" = 20'-0"

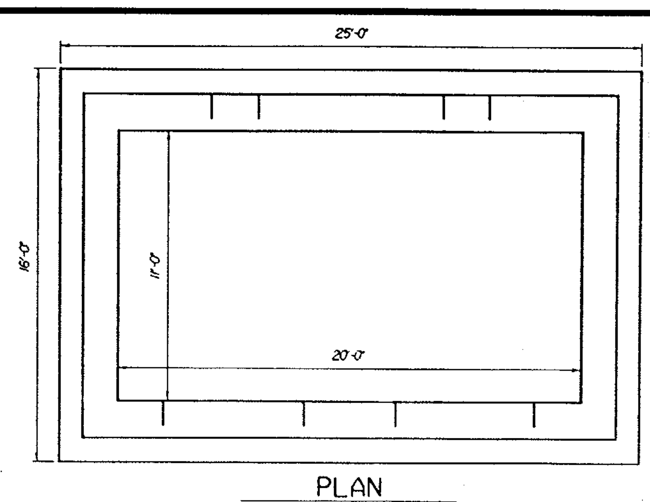


PLAN

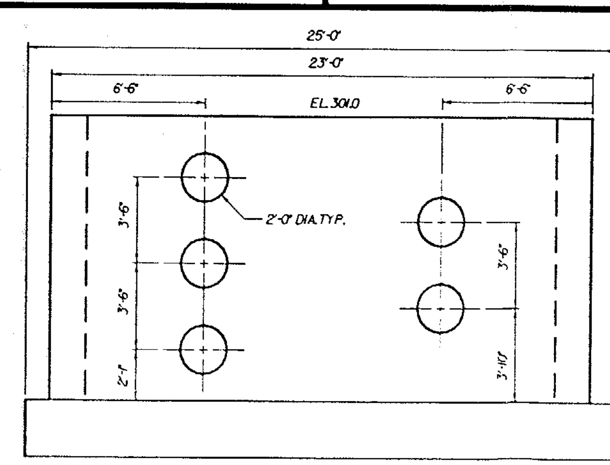


SECTION

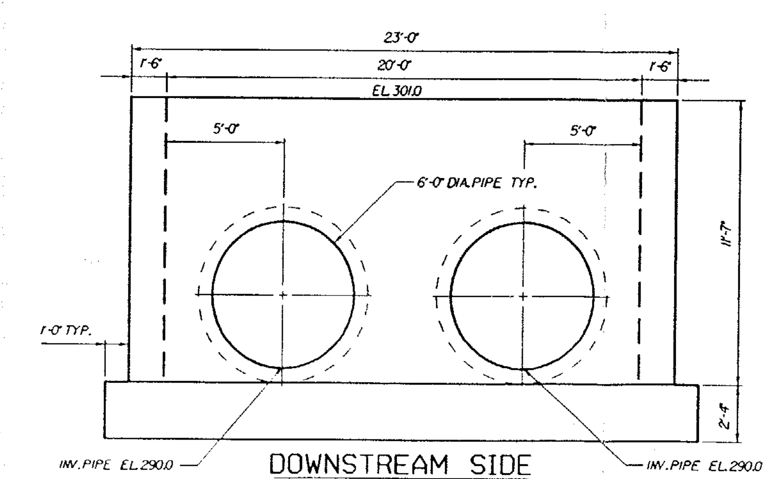
STILL BASIN
SCALE 1/4" = 1'-0"



PLAN



UPSTREAM SIDE



DOWNSTREAM SIDE

INTAKE STRUCTURE
SCALE 1/4" = 1'-0"

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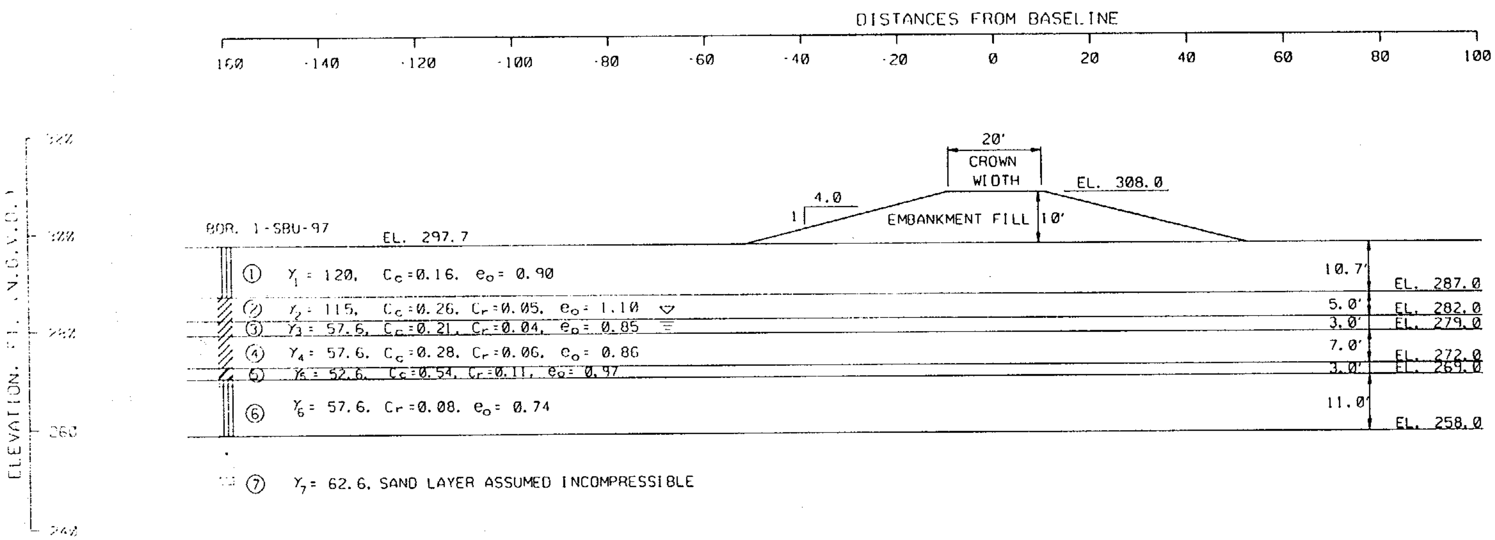
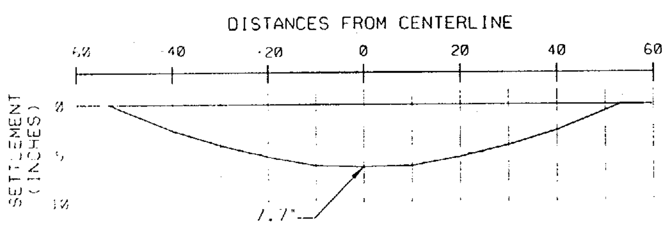
SEDIMENT RETENTION BASIN
LOW LEVEL SPILLWAY

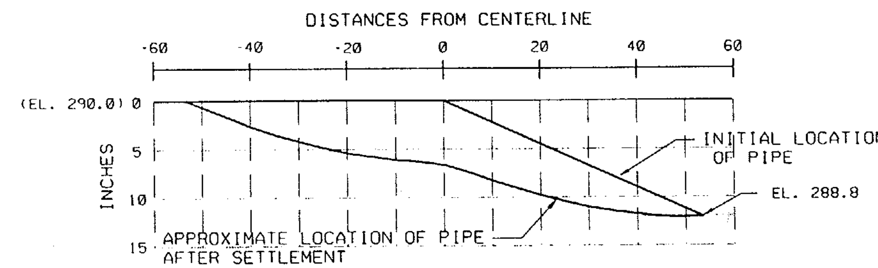
DRAWING
III-59



NOTE: MAXIMUM SETTLEMENT AT THE CENTER OF THE EMBANKMENT IS ESTIMATED AT 7.7 INCHES AS SHOWN BELOW.



SETTLEMENT AT LOW LEVEL STRUCTURE •2



VERTICAL DISPLACEMENT OF OUTLET PIPE

DRAWING III-60

ENVIRONMENTAL RESTORATION NATURAL RESOURCES PROJECT

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LOW LEVEL - OUTLET STRUCTURE
SETTLEMENT ANALYSIS

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