HISTORY OF CONSTRUCTION

CFR 257.73(c)(1)

Fly Ash Pond

Big Sandy Plant
Louisa, Kentucky

October, 2016

Prepared for : Kentucky Power – Big Sandy Plant

Louisa, Kentucky

Prepared by: American Electric Power Service Corporation

1 Riverside Plaza

Columbus, OH 43215

GERS-16-063

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Attachments

Attachment A – Location Map
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1.0 OBJECTIVE

This report was prepared by AEP- Geotechnical Engineering Services (GES) section to fulfill requirements of CFR 257.73(c)(1) with an evaluation of the facility.

2.0 DESCRIPTION OF CCR THE IMPOUNDMENT

The Big Sandy Power Plant is located north of the City of Louisa, Lawrence County, Kentucky. It is owned and operated by Kentucky Power. The facility operates two surface impoundments for storing CCRs called the Fly Ash Pond and the Bottom Ash Pond. This report deals with the History of Construction for the Fly Ash Pond.

The Fly Ash Pond is a valley impoundment with a main dam and a saddle dam. The Big Sandy Fly Ash Pond received sluiced fly ash and waste water from the plant via the bottom ash pond. Bottom Ash excavated from the Big Sandy Bottom Ash Pond is also placed within the Fly Ash Pond.

The Big Sandy Power Plant has ceased burning coal and has been refueled for natural gas. The Fly Ash Pond currently receives waste water from the plant for discharge through the permitted outfall.

3.0 SUMMARY OF OWNERSHIP 257.73(c)(1)(i)

[The name and address of the person(s) owning or operating the CCR unit: the name associated with the CCR unit: and the identification number of the CCR unit if one has been assigned by the state.]

The Big Sandy Power Plant is located at 23000 Highway 23, Lousia, KY 41230 near the City of Louisa, Lawrence County, Kentucky. It is owned and operated by Kentucky Power. The facility's Kentucky Dam ID number is 0367.

4.0 LOCATION OF THE CCR UNIT 257.73 (c)(1)(ii)

[The location of the CCR unit identified on the most recent U.S. Geological Survey (USGS) 7 ½ minute or 15 minute topographic quadrangle map, or a topographic map of equivalent scale if a USGS map is not available.]

A location map is included in Attachment A.

5.0 STATEMENT OF PURPOSE 257.73 (c)(1)(iii)

[A statement of the purpose for which the CCR unit is being used.]

The Big Sandy Fly Ash Pond received sluiced fly ash and waste water from the plant via the bottom ash pond. Bottom Ash excavated from the Big Sandy Bottom Ash Pond is also placed within the Fly Ash Pond.

The Big Sandy Power Plant has ceased burning coal and has been refueled for natural gas. The Fly Ash Pond currently receives waste water from the plant for discharge through the permitted outfall.

6.0 NAME AND SIZE OF WATERSHED THE CCR UNIT IS LOCATED 257.73 (c)(1)(IV)

[The name and size in acres of the watershed within which the CCR unit is located.]

The Big Sandy Fly Ash Pond is located in the Big Sandy Water Shed (HUC 05070204) which is 258,956.8 acres (404.62 square miles). Locally the impoundment is across the Horseford Creek and has a drainage area of approx. 675 acres.

7.0 DESCRIPTION OF THE FOUNDATION AND ABUTMENT MATERIALS 257.73(c)(1)(v)

[A description of the physical and engineering properties of the foundation and abutment materials on which the CCR unit is located.]

The foundation soils at the main dam are primarily residual and colluvial in nature, with minor amounts of alluvim in the valley bottom. The foundation soils in the area of the main dam consists of stratified soils ranging from clay to sand and containing particles of weathered rock. Based on soil borings the original thickness of the foundation soils (overburden) ranged in thickness from about 5-ft at the east side of the valley to 36-ft on the west side. The overburden thickness was approximately 28-ft thick in the center of the valley. On the east abutment the overburden was 5-10 feet thick and on the west abutment it was thicker upto approx.20-feet with bedrock outcrops on both abutments.

With the exception of a relatively thin “Clay and Weathered Shale” layer under the original (Stage I) embankment, the foundation soils have been modeled as similar “overburden” units because the foundation soils are stratified with properties as identified below.

Material	Unit Weight (pcf)	Cohesion (psf)	Friction Angle (°)
Clay and Weathered Shale	135	1000	23
Main Dam Overburden “A”	135	0	25
Main Dam Overburden “B”	135	1000	23
Saddle Dam Overburden	135	0	34

The major bedrock at the site is sandstone with various degrees of gradation and cementation. The sandstone strata vary in thickness from a few inches to over 100-ft within the upper portions of the abutment slopes. The sandstone alternates with layers of siltstone, shale, and claystone ranging in thickness from a fraction of an inch to approx. 20-ft. Additionally there is an approx. 2-ft coal seam at approx. elev. 600 on the abutments.

<u>8.0</u> DESCRIPTION OF EACH CONSTRUCTED ZONE OR STAGE OF THE CCR UNIT 257.73 (C)(1)(VI)

[A statement of the type, size, range, and physical and engineering properties of the materials used in constructing each zone or stage of the CCR unit; and the approximate dates of construction of each successive stage of construction of the CCR unit.]

Note: The Big Sandy Fly Ash Pond has also historically been referenced as the Horseford Creek Fly Ash Dam in various design reports and drawings. As such the design reports and drawings in Attachments B and C reference the Horseford Creek Fly Ash Dam.

The Big Sandy Fly Ash Pond was designed and constructed in three phases.

Stage I was constructed to a crest elevation of approx. 625 between 1968 and 1970. Construction was halted or periodically slowed due to excessive embankment deformations. The Stage I dam was designed as a homogeneous compacted impervious fill with a 20-ft wide key in the foundation soil to bedrock. The Stage I dam raised approximately 85 feet above the valley floor at the maximum section and had a crest length of 650-ft and was 25-ft wide. The slopes of the dam were 2.5H:1V from the foundation soils to elev. 580. From elev. 580 to 625 the slopes were 2H:1V. During Stage I construction as deformations were observed rock fill was added to both the upstream and downstream toes. The rock fill was placed approximately 125-ft from the upstream toe to elevation 573 (Approx. 33-ft thick). At the downstream toe the rock fill was placed to elev. 566 (Approx. 30-ft thick) approximately 100-ft from the toe. Drawings for the Stage I construction are included in Attachment C. There is no design report available associated with the Stage I construction.

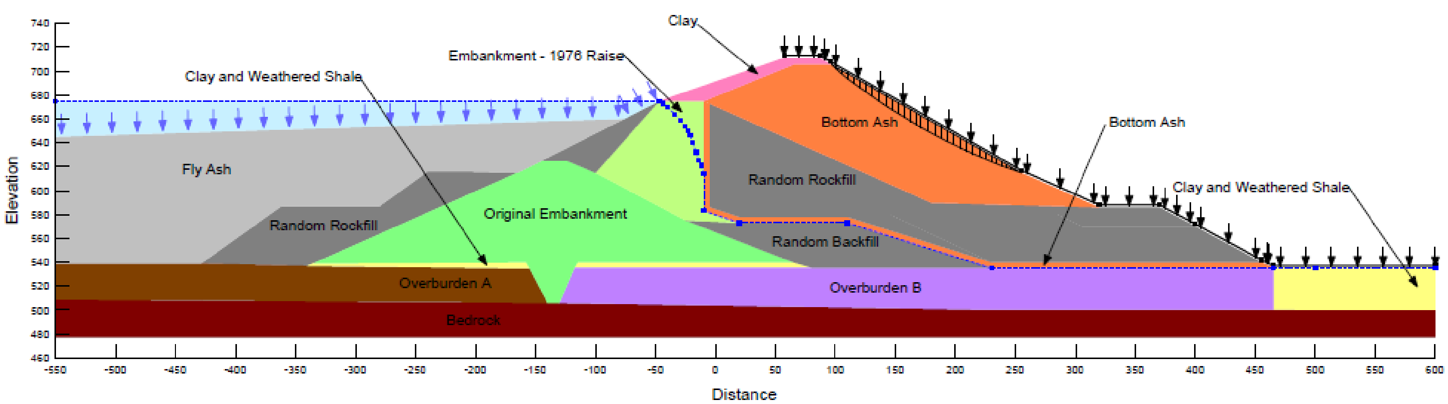
Stage II was constructed to a crest elevation of 675 in 1977 and 1978. The Stage II construction is a zoned embankment with a compacted upstream random rock fill, a central impervious clay core, a vertical chimney drain and a downstream rock fill zone. The Stage II crest is approximately 800-ft long and 55-ft wide. The Stage II impervious core was placed over the downstream slope of the Stage I dam. The upstream rock fill extends from the clay core over the Stage I crest and is placed over the Stage I Upstream slope and rock fill. The upstream slope varies form 2.6H:1V and 2H:1V with a rock fill bench at 625. Downstream of the clay core is a 10-ft wide chimney drain that connects to a 6-8-ft blanket drain. The downstream rock fill is located adjacent to the chimney drain and over the blanket drain. The downstream slope varies from 2H:1V to 1.4H:1V with and approximately 250-ft wide toe berm below elevation 600. The Design report for the Stage II raising are in Attachment B and Design Drawings in Attachment C.

The Stage II construction included the construction of the Saddle Dam. The Saddle Dam is a zone filled dam with an impervious clay core tied into bedrock with a 30-ft wide key trench. A random rock fill was placed on the upstream and downstream shells with a chimney drain located along the downstream face of the clay core. The design report and drawings for the Stage II Saddle Dam are in Attachments B and C.

Stage III raising was designed in 1993 and constructed in incremental stages from 1995 to 2010 with a final crest elevation of 711. An impervious clayey soil layer was constructed as the upstream slope from the Stage II crest tying into the Stage II impervious core. The downstream shell was constructed of bottom ash with a riprap face to minimize riprap. The Stage III crest is approximately 1,000-ft Long and 30-ft wide. The Upstream slope was constructed with a 2-3/4H:1V slope. The downstream slope varies with a 1-3/4H:1 slope from crest to Elev 615 and a 2-1/4H:1 slope to the Stage II rock fill (approx. Elev. 590). The design report and drawings for the Stage III (1993) dam raising are included in Attachments B and C.

The Stage III raising included the reconstruction of the Saddle Dam to raise it to elevation 711. Reconstruction of the saddle dam consists of excavating and reconstructing the clay key trench to bedrock with an upstream clay soil shell. The reconstructed saddle dam has a downstream shell of bottom ash with a riprap layer for erosion protection. The crest of the Saddle dam is approx. 30-ft wide by Approx. 500-ft long. The upstream slope of the saddle dam was constructed at a 2-3/4H:1V slope and the downstream slope was reconstructed at 1-3/4H:1V The reconstructed saddle dam included placing Roller Compacted Concrete (RCC) in the Emergency Spillway constructed as part of the Stage II raising (refer to Section 9 regarding information on the apprutenances). The design report and drawings for the Stage III (1993) dam raising are included in Attachments B and C.

Material	Unit Weight (pcf)	Cohesion (psf)	Friction Angle (°)
Stage I Original Embankment	130	0	25
Stage II (1976) Embankment	135	0	25
Random Rock Fill	110	320	26
Bottom Ash	70	0	41
Stage III Clay	135	340	20



<u>9.0</u> ENGINEERING STRUCTURES AND APPURTENANCES, 257.73 (c)(1)(vii)

[At a scale that details engineering structures and appurtenances relevant to the design, construction, operation, and maintenance of the CCR unit, detailed dimensional drawings of the CCR unit, including a plan view and cross sections of the length and width of the CCR unit, showing all zones, foundation improvements, drainage provisions, spillways, diversion ditches, outlets, instrument locations, and slope protection…]

Detailed Engineering drawings are included in Attachment C.

A map with instrumentation locations is provided in Attachment D.

<u>10.0</u> SUMMARY OF POOL SURFACE ELEVATIONS, AND MAXIMUM DEPTH OF CCR, 257.73 (c)(1)(vii)

[…in addition to the normal operating pool surface elevation and the maximum pool elevation following peak discharge from the inflow design flood, the expected maximum depth of CCR within the CCR surface impoundment.]

The Fly Ash Pond is constructed and designed with a maximum capacity of approximately 12,000 acre-ft of storage to an elevation of 705-ft. With recent plant operations the water elevation was generally maintained at a relatively consistent water elevation of 670.5-ft at the outfall structure at the dam. However, because of the disposition of ash in the pond the water elevation across the pond varies up to an approximate high of 697-ft in the back of the pond. Depth of water varies in various pools with the deepest being approximately 67-ft near the outfall structure to less than one (1) foot across many areas of the pond. It is estimated that approximately 400 acre-ft of free water was impounded prior to closure activities commenced.

Current water management activities related to closure of the pond have lowered the water elevation across the site by 5 to 10 feet and is highly dependent on rainfall events. All free water will eventually be removed from the impoundment as part of the closure activities.

Based on an original pond bottom of approximately elevation 540 the thickness of CCR deposited in the pond varies from approx. 83-ft (Elev. 623) near the outfall structure to approx. 157-ft (Elev. 697) at the far end where ash was most recently deposited. It is estimated that the average thickness across the pond is 140-ft corresponding to an average elevation of 680. Based on the stage storage curve for the pond and using the average elevation of 680 it is estimated that 8,200 acre-ft of CCR is currently impounded which would represent the maximum amount ever stored at the facility.

<u>11.0</u> FEATURES THAT COULD ADVERSELY AFFECT OPERATION DUE TO MALFUNCTION OR MIS-OPERATION (257.73 (c)(1)(vii))

[…and any identificable natural or manmade features that could adversely affect operations of the CCR unit due to malfunction or mis-operation]

In the event of malfunction or mis-operation of any of the pond's appurtenances the pond's operations could be adversely affected. These structures include the outlet structure and piping to the downstream outlet. See the design drawings in Attachment C for location and details of all appurtenances

<u>12.0</u> DESCRIPTION OF THE TYPE, PURPOSE AND LOCATION OF EXISTING INSTRUMENTATION 257.73 (c)(1)(viii)

[A description of the type, purpose, and location of existing instrumentation.]

The Main dam currently has 16 survey monuments and 3 slope inclinometers to monitor movement of the dam structure. Additionally there are 17 piezometers at the Main Dam to monitor pore water conditions within the dam. No instrumentation is located at the Saddle Dam. A location map is provided in Attachment D.

<u>13.0</u> AREA – CAPACITY CURVES FOR THE CCR UNIT 257.73 (c)(1)(ix)

[Area-capacity curves for the CCR unit.]

Area – Capacity curves have been developed for each stage of construction. For the Stage I construction it is reported as a table on drawing 12-3038 included in Attachment C. For Stage II and III it is included graphically as part the hydrology and hydraulics evaluation included in Attachment B. It should be noted that the Stage II and Stage III Area-Capacity curves are based on Elev. 600 and Elev. 650 having zero storage respectively.

<u>14.0</u> DESCRIPTION OF EACH SPILLWAY AND DIVERSION 257.73 (c)(1)(x)

[A description of each spillway and diversion design features and capacities and calculations used in their determination.]

<u>Primary Spillway</u>

There have been two Primary spillways located at the Main Dam. As part of the Stage I construction the A sloping shaft spillway with stop logs was constructed along the hillside near the right abutment. The sloping concrete riser connected to a 42-in diameter concrete pipe that ran through the Stage I dam and discharging into an open channel at the downstream toe. This structure was abandoned filled with concrete as part of the stage II construction. The outlet end of this Stage I outlet pipe is still visible.

As part of Stage II a new Primary (or Service) Spillway structure was constructed near the left abutment. The structure consists of a vertical, reinforced concrete, drop inlet. The Primary Spillway incorporates a twin shaft weir design with slide gates at the bottom of each shaft. Concrete stoplogs are added to to the weirs to control the water elevations in the pond. As part of the Stage II construction the spillway structure was built to approx. elev. 700. A floating skimmer system surrounds the structure to control the discharge of fly ash and debris. Water discharging over the spillway flows through a 30-in. dia. concrete pipe which was installed on a concrete cradle

<u>Emergency Spillway</u>

An Emergency Spillway was constructed with each construction stage. As part of Stage I the emergency spillway was constructed into rock on the hillside of the left abutment. The Stage I emergency spillway had an overflow elevation of approx. 618 The excavation of the Stage I Emergency spillway was incorporated in the construction of the Primary Spillway installed during Stage II.

For Stage II a new emergency spillway was constructed in bedrock on the left abutment of the Saddle Dam. The Stage II emergency spillway was 40-50-ft wide with an elevation of 671. The Stage II emergency spillway was filled with roller compacted concrete as part of the reconstruction of the saddle dam as part of the Stage III construction.

With the reconstruction of the Saddle Dam a new emergency spillway was excavated as part of Stage III construction. The Stage III emergency spillway was also excavated into bedrock on the left abutment of the saddle dam and had an overflow elevation of 706.25

15.0 SUMMARY CONSTRUCTION SPECIFICATIONS AND PROVISIONS FOR SURVEILLANCE, MAINTENANCE AND REPAIR 257.73 (c)(1)(xi)

[The construction specifications and provisions for surveillance, maintenance, and repair of the CCR unit.]

The construction of the dams for the Big Sandy Fly Ash Pond were designed and constructed over multiple years as part of three distinct designs and construction. The 1976 and 1993 design reports are included in Attachment B. A design report for the original 1968 construction is not available information is limited to the available drawings included in Attachment C.

As required by the CCR rules the Fly Ash Pond is inspected at least every 7 days by a qualified person. Also as a requirement of the CCR rules the impoundment is also inspected annual by a professional engineer. Additionally, as a requirement by the State of Kentucky the impoundment is inspected once a month.

If repairs are found to be necessary during any inspection they will be completed as needed.

16.0 RECORD OR KNOWLEDGE OF STRUCTURAL INSTABILITY 257.73 (c)(1)(xii)

[Any record or knowledge of the structural instability of the CCR unit.]

Structural instability is reported during the Stage I and Stage II construction processes as discussed below. Remedial measures were taken at that time and Stage III construction was managed to control the creation of similar issues:

- As part of the Stage 1 construction there was excessive deformation observed that resulted in the construction of the rockfill buttresses on the upstream and downstream toes.
- During Stage II construction, sloughing occurred near the left abutment of the Stage I dam. A stabilizing berm was constructed to control movement and was incorporated into the Stage II clay core.

ATTACHMENT A

LOCATION MAP

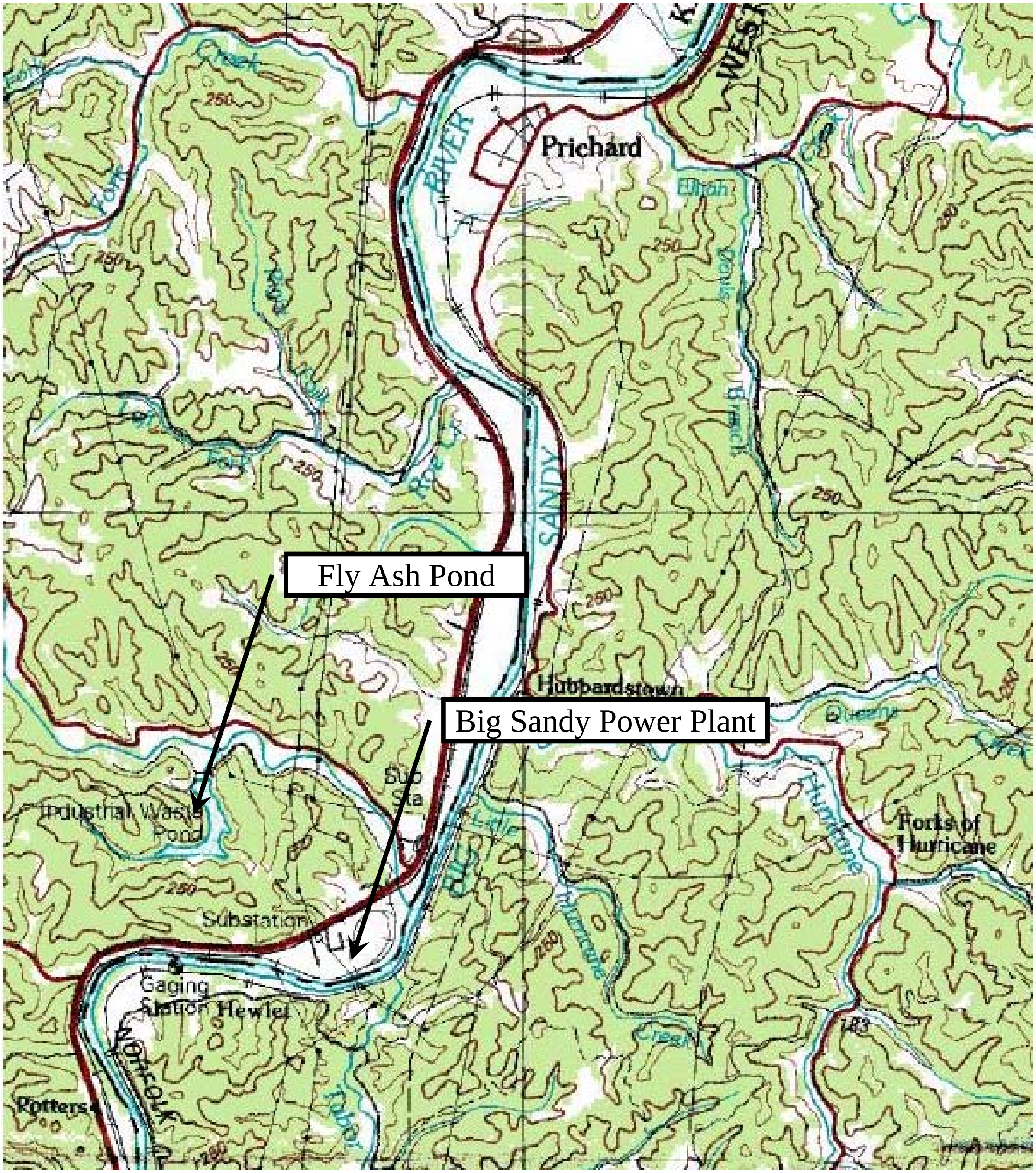


IMAGE DATE: 07/01/1975

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	CHA	**Figure 1** **Project Location Map**
Scale: 1" = 1 mile	**Project No.: 20085.7000.1510**	**American Electric Power Corp.** **Big Sandy Power Plant** **Louisa, KY**

ATTACHMENT B

DESIGN REPORTS

ATTACHMENT B-1

STAGE II Design Report

KENTUCKY POWER COMPANY
BIG SANDY PLANT
FLY ASH RETENTION DAM
STAGE 2 RAISING
DESIGN REPORT

PREPARED BY
AMERICAN ELECTRIC POWER SERVICE CORP.
CIVIL ENGINEERING DIVISION
SOILS, FOUNDATION AND HYRDO SECTION

KENTUCKY POWER COMPANY
BIG SANDY PLANT
FLY ASH RETENTION DAM
STAGE 2 RAISING
DESIGN REPORT

PREPARED BY
AMERICAN ELECTRIC POWER SERVICE CORP.
CIVIL ENGINEERING DIVISION
SOILS, FOUNDATION AND HYRDO SECTION

CERTIFICATE

I certify that the work reported herein was subject to my review.

John Longo

Commonwealth of Kentucky, P.E. License No. 10238

Signature John J. Longo

Date May 17, 1977

Seal

BIG SANDY PLANT

FLY ASH RETENTION DAM

STAGE 2 RAISING

DESIGN REPORT

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KENTUCKY POWER COMPANY
BIG SANDY PLANT
FLY ASH RETENTION DAM

INTRODUCTION

Fly Ash slurry from the Big Sandy Thermal Generating Plant is currently being retained behind an existing dam, referred to as the Big Sandy Fly Ash Retention Dam. This structure, consisting of compacted earth and rockfill, is classified as a CLASS-B structure in accordance with the design criteria for dams established by the Department of Natural Resources and Environmental Protection of the Commonwealth of Kentucky.

The Big Sandy Fly Ash Retention Dam is located on Horseford Creek approximately 1000 feet upstream from its confluence with Blaine Creek in Lawrence County, Kentucky, Latitude & Longitude N 38°11'40", W 82°38'. A location map for the subject dam is shown on Dwg. No. 984-C-100.

Construction of the existing dam was completed in 1970 under construction permit 186 issued Oct. 8, 1968 by the Department of Natural Resources, Division of Water. The dam rises approximately 85 feet above the valley floor at the maximum section, to a crest elevation of 625 feet M.S.L., with a crest length of 650 feet. Currently fly ash is being sluiced into the reservoir at the rate of 411,000 cubic yards per year and it has now become necessary to raise this dam in order to provide additional ash storage capacity for future operation of the plant.

It is proposed to raise the existing dam to a crest elevation of 675 feet, M.S.L. This raising will provide additional storage life of 18 years or a total storage capacity of approximately 7.5 million cubic yards of fly ash.

The documents and drawings listed herein are submitted as part of this final design report for this proposed raising.

1. <u>Geotechnical Investigation</u>

The geotechnical investigation for the proposed dam raising was conducted by American Electric Power Service Corp. in consultation with Casagrande Consultants. All soil testing for the dam foundation and embankment materials was undertaken by Casagrande Consultants. The results of all geotechnical site exploration work and laboratory tests are presented in "Geotechnical Investigations For Proposed Raising Of The Big Sandy Fly Ash Retention Dam to Elevation 675" Volumes I, II and III, prepared by Casagrande Consultants.

2. <u>Geotechnical Calculations</u>

The geotechnical calculations for the proposed raising were made by American Electric Power Service Corporation and are summarized in Section 1 of this design report.

3. <u>Hydrology/Hydraulic Calculations</u>

All hydrologic/hydraulic calculations and graphs were made by Harza Engineering Company and are summarized in Section 2 of this design report.

4. Construction Drawings

Construction Drawings were prepared by Harza Engineering Company. A complete list of these drawings is given below.

984-C-100	PROJECT AND BORROW AREA LOCATION MAP
984-C-101	DAM PLAN SHEET 1 OF 3
984-C-102	DAM SECTIONS SHEET 2 OF 3
984-C-102A	DAM SECTIONS SHEET 3 OF 3
984-C-103	SADDLE DIKE AND EMERGENCY SPILLWAY
984-C-104	SERVICE SPILLWAY EARTHWORK DETAILS
984-C-105	RISER PLAN AND SECTIONS
984-C-106	RISER MISCELLANEOUS DETAILS
984-C-107	RISER REINFORCING DETAILS SHEET 1 OF 2
984-C-108	RISER REINFORCING DETAILS SHEET 2 OF 2
984-C-109	SPILLWAY CONDUIT SECTIONS & DETAILS
984-C-110	EXISTING SERVICE SPILLWAY ABANDONMENT DETAILS

5. Construction Specifications

Construction specification for all work items required for the dam raising, were prepared by American Electric Power Service Corporation.

GEOTECHNICAL DESIGN OF DAM
SECTION 1

Embankment Materials

The materials to be used for construction of the enlarged dam embankment will be obtained from the following sources:

(a) Material for the rock fill zones will be obtained from Borrow Area I.

(b) Overburden from Borrow Areas I, II and III will be used for the impervious zone in the dam.

(c) Suitable overburden excavation from the dam foundation and emergency spillway area will be used for the impervious and random fill areas.

(d) Bottom Ash produced at the Big Sandy Plant will be used for the transition zone.

(e) Imported sand and gravel or crushed stone or a combination of crushed stone and gravel will be used for the drainage zone.

Dam Foundation

The borings indicate that the overburden in the bottom of the valley, within the foundation area of the existing dam and the proposed additional embankment area consist of stratisfied soils ranging from clay to sand, and containing particles of weathered rock. The clayey soils are generally of medium plasticity. "Q"-tests made on samples of overburden from within the foundation area gave a shear strength value of 0.6 TON/sq. ft., this value was used in the Total Stress

Analyses that governed the design of the dam. "S"-tests made on the same material type gave a shear strength angle of 31.1 degrees.

The rock formations at this site are all of sedimentary origin. The major bedrock type is sandstone of various gradations and degrees of cementation. The sandstone alternates with layers of siltstone, cemented shale and clay shale ranging in thickness from a fraction of an inch to approximately 20 feet.

Slope Stability Calculations

The "Janbu Method" of analysis was used for evaluating stability. Total stress, effective stress and earthquake conditions were analyzed for the proposed raising.

A. Total Stress Analysis

After studying the "Q"-test results, it was realized that the strength of the foundation material, immediately after construction, would govern the design of the dam. Therefore, it was decided to adopt a conservative approach for studying this condition. The lowest value obtained for the foundation material from the "Q"-tests was used in this analysis, C=1,200PSF. Since the strength value used in the analysis is, conservative, it was decided to design in the total stress analysis for a factor of safety of 1.2.

In order to obtain a factor of safety greater than 1.2, for the critical shear plane given in the downstream slope of section A-A, a berm 215 feet in length was required. The berm height, at elevation 590 feet M.S.L., was governed by the critical shear plane given in section B-B. The critical shear plane analyzed in section

C-C, located on the left abutment, required no berm and a factor of safety of 1.24 was obtained. This can be explained by the fact that the bed rock elevation is much higher which in effect reduces the height of the embankment in this area.

B. <u>Effective Stress Analysis</u>

"S"-strength tests made on samples of the embankment fill and the overburden material directly downstream of the dam yielded a friction angle of 31.1 degrees for both materials. This value was used in the effective stress analysis, and it was elected to design for a factor of safety of 1.7.

The dam section established in the total stress analysis was re-evaluated using the effective stress method. As a result a factor of safety of 2.03 was calculated for the downstream slope and 2.17 for the upstream slope of section A-A.

C. <u>Earthquake Condition</u>

The dam is located in a zone 1 seismic risk area, it is, however, close to Zone 2, in which moderate damage to structures can occur during an earthquake. Therefore, the Zone 2 risk was assumed in the calculations. The Corps of Engineers' Engineering Manual EM 1110-2-1902 recommends that a seismic coïfficient of 0.1g be applied for the dynamic stability analysis of structures located in this zone.

A factor of safety of 1.32 was calculated for the downstream slope of section A-A.

Conclusion

From the factors of safety obtained using the total stress analysis, construction condition, and the effective stress analysis, long term condition, the calculations indicated stability of the dam will increase considerably with time. A summary of the results of the two analyses of the stability calculations yielding the respective factors of safety given below.

TYPE OF ANALYSIS	LOADING CONDITION	SECTION	FACTOR OF SAFETY
TOTAL STRESS	CONSTRUCTION (D.S.)	A-A	1.22
TOTAL STRESS	CONSTRUCTION (U.S.)	A-A	1.22
TOTAL STRESS	CONSTRUCTION (D.S.)	B-B	1.26
TOTAL STRESS	CONSTRUCTION (D.S.)	C-C	1.24
EFFECTIVE STRESS	LONG TERM (D.S.)	A-A	2.03
EFFECTIVE STRESS	LONG TERM (U.S.)	A-A	2.17
EFFECTIVE STRESS	EARTHQUAKE (D.S.)	A-A	1.32

Seepage Calculations

A flow net was developed for the steady state seepage condition, with the maximum reservoir level at elevation 670 feet. To simplify plotting of the flow net, which is attached, it is assumed that all clay zones in the dam and the clay overburden are homogeneous and have the same coefficient of permeability 1.98 x 10-7 ft./min. The elevation of bedrock was assumed to be 515 feet; therefore, resulting a total head loss of 155 feet through the dam.

As shown on the flow net 8 full flow channels (nf), and approximately 6.5 equi-potential drops (nd), have been drawn. The seepage through the dam can be estimated by the equation given below.

Q=(K)(H)(L)(nf/nd)

K=1.98 x 10-7 ft./min. (Permeablilty Coeff. of Clay Material)

H=155 ft. (Total Head Loss)

L=800 ft. (Crest Length)

nf=8 (Number of Flow Channels)

nd=6.5 (Number of Equi-Potential Drops)

Q=.23 gpm (Total Seepage Through Dam)

The calculated seepage through the dam (Q), is found to be .23 gpm., which was considered very low. The downstream sandstone zone will be adequate to pass this quantity of water, however, a drainage zone will be installed in the dam foundation as a precaution should the sandstone rock fill zone not function as designed for any reason to dissipate the seepage.

HYDROLOGY/HYDRAULICS REPORT

SECTION 2

Introduction

The Big Sandy Project consists of raising an existing dam 50 feet to accommodate additional fly ash storage. As a component of the dam construction, a new service spillway and emergency spillway will be constructed. The existing service spillway will be used for diversion during construction, but will be abandoned following construction completion.

Hydrology

Appendix A, attached, summarizes the development of the design storm hydrographs used to perform the flood routings. The design storms consisted of the 25 year, freeboard and emergency hydrographs. In addition, a 100 year storm hydrograph was computed for flood routing through the existing service spillway to test its adequacy during construction diversion.

Hydraulics

The spillway design consists both of a service spillway and an emergency spillway. The service spillway will discharge normal operational flow plus storms of a magnitude less than the 25 year flood. The emergency spillway will operate at storms of greater intensity than the 25 year storm.

The design of the service spillway consists of a reinforced concrete, drop-inlet type spillway located on the left abutment of the dam. The intake is an 85 foot high, vertical, decanting riser structure with concrete stoplogs provided to raise the overflow weir crest elevation concurrently with the rising fly ash elevation. (The maximum riser elevation was set at El. 700 anticipating a third stage dam raising.) The spillway consists of a double weir design with slide gates at the location shown on Fig. 11. The service spillway is intended to operate using both weirs under all normal and flood conditions. In the unlikely event that a stoplog should break, a slide gate can be operated to terminate flow through either bay of the riser. This will prevent fly ash from being discharged. The other cell would continue to function. The single cell operation is considered a remote possibility.

The water discharged over the weir will flow through a sloping 30-inch diameter prestressed concrete cylinder pipe conduit, terminating in a riprap lined plunge pool stilling basin downstream. Both the service spillway riser and conduit will be founded on rock.

The emergency spillway consists of a 15 foot wide channel located in a saddle near the south-eastern portion of the reservoir (See Dwg. 984C 100). The invert of the emergency spillway channel will be founded in rock and is at El. 671.0.

Flood Routings

Table 1 and Figures 3-10 show the results of the flood routing studies. Table 1 summarizes the results of both the hydrologic and hydraulic studies. Figures 3-10 show the inflow/outflow hydrographs, water surface elevation versus time and spillway rating curves for both double and single weir operation. Figure 12 shows the reservoir area-volume curves.

The results of the flood routing through the existing service spillway are given on Figures 1 and 2. The flood routing reveals that the existing service spillway is adequate to pass the 100 year storm without overtopping the existing dam crest El. 625.0.

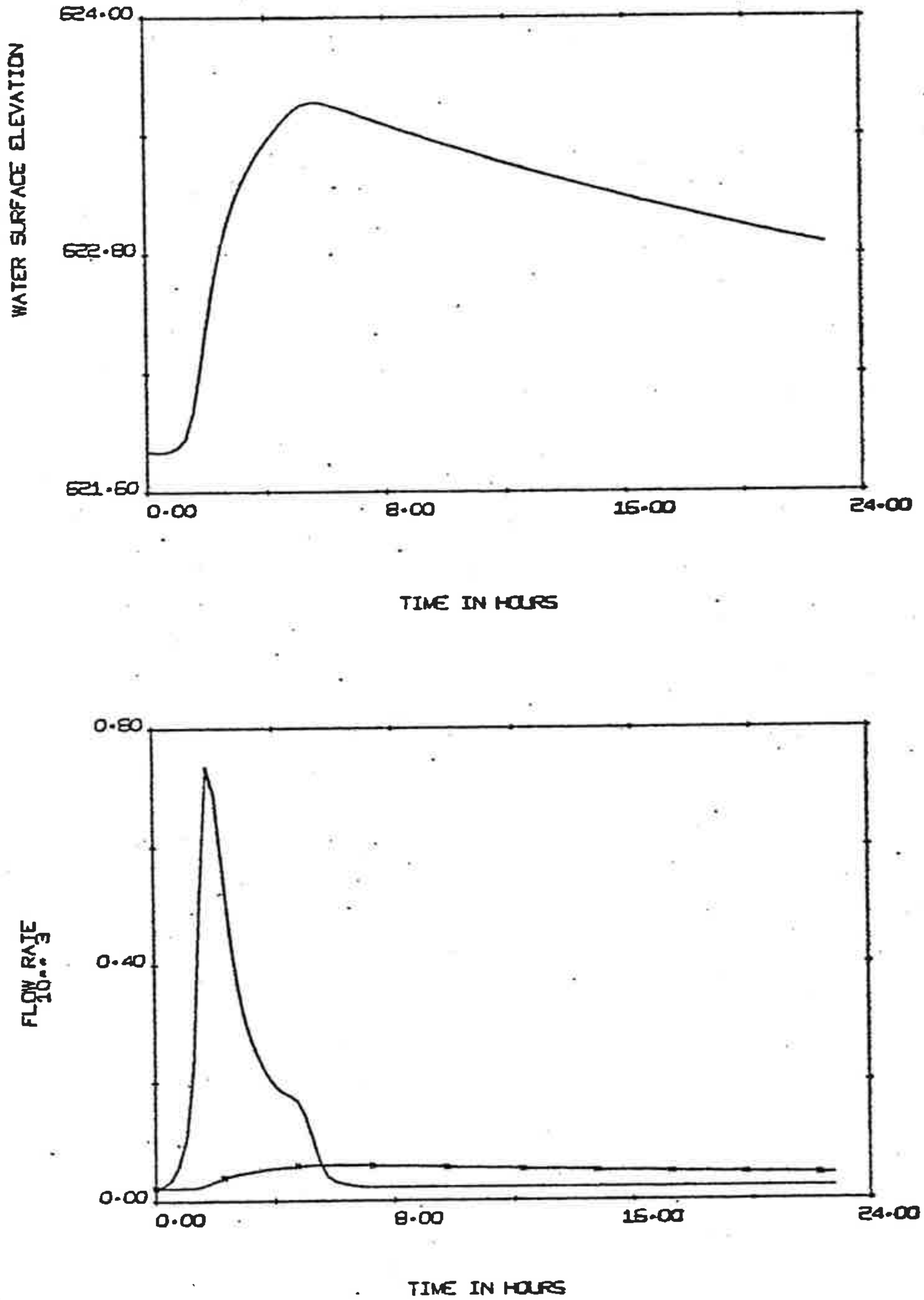
TABLE 1

SUMMARY OF HYDROLOGIC AND HYDRAULIC STUDIES

	DOUBLE RISER 25 Year Storm	DOUBLE RISER Emergency Hydrograph	DOUBLE RISER Freeboard Hydrograph	SINGLE RISER 25 Year Storm	SINGLE RISER Emergency Hydrograph	SINGLE RISER Freeboard Hydrograph
1. Normal Water Surface Elevation	670.1	670.1	670.1	670.8	670.8	670.8
2. Service Spillway						
a) Diameter of Conduit		30" i.d.			30" i.d.	
b) Spillway Weir Length		6.0'			3.0'	
c) Weir Elevation		669.0			669.0	
3. Emergency Spillway						
a) Crest Elevation		671.0			671.0	
b) Weir Length		15.0			15.0	
4. Maximum Inflow (cfs)	771.9	2134.2	4601.5	771.9	2134.2	4601.5
5. Maximum Outflow						
a) Service Spillway	47	94	149	37	63	110
b) Emergency	-0-	45	218	21*	99	293
6. Maximum Water Surface Elevation	670.9	672.0	673.8	671.6	672.7	674.4
7. Dam Crest Elevation		675.0			675.0	

*Should single riser operation be necessary, the service spillway weir crest elevation must be lowered to El. 668.0 to prevent discharge over the emergency spillway.

FIGURE 1

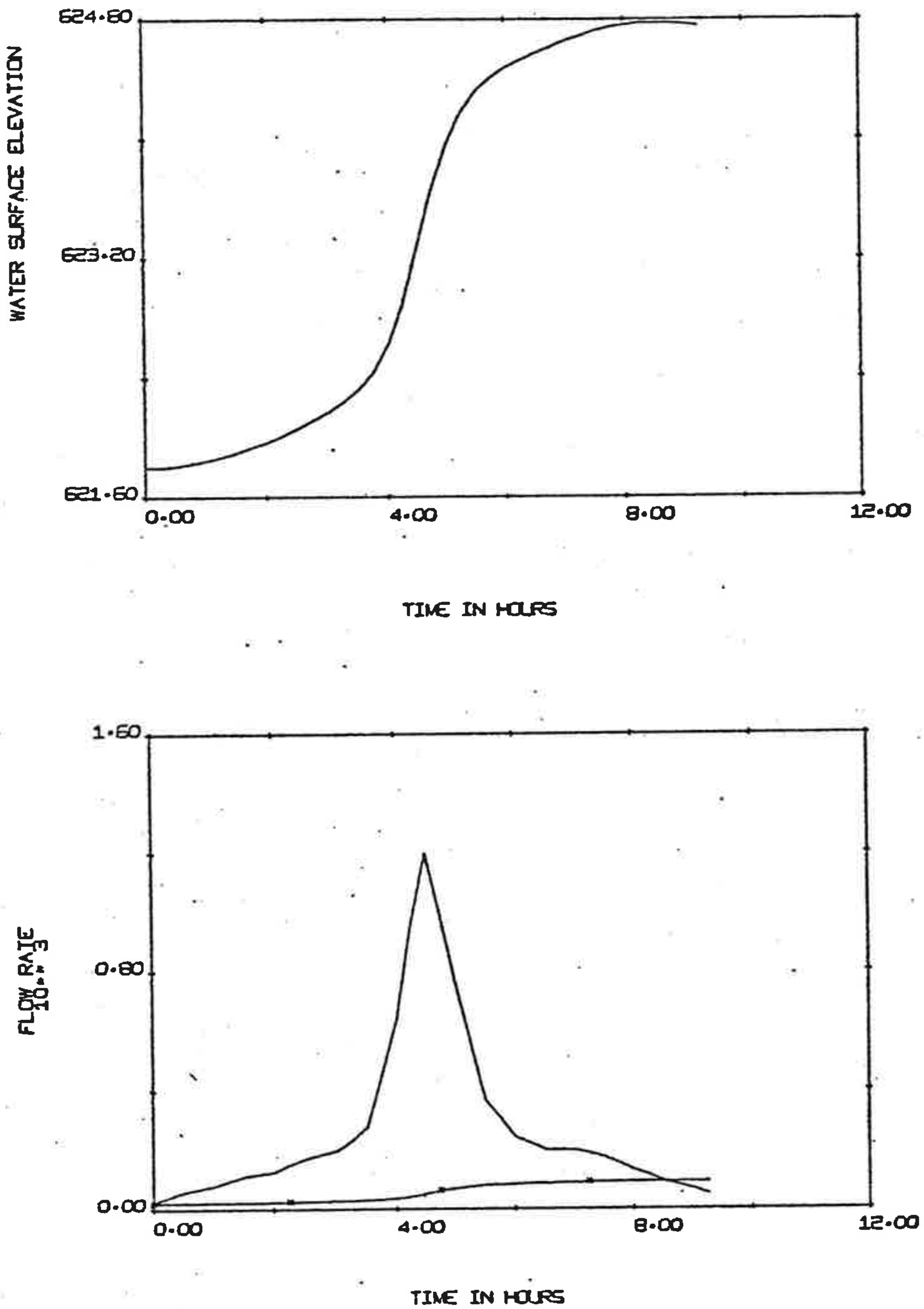


BIG SANDY - 25 YEAR FLOOD - EXISTING PRIN. SPWY. CREST 620. DAM CREST 625.

INFLOW HYDROGRAPH

OUTFLOW HYDROGRAPH

FIGURE 2



BIG SANDY - 100 YEAR FLOOD - EXISTING PRIN. SPWY. CREST 620, DAM CREST 625.

INFLOW HYDROGRAPH

OUTFLOW HYDROGRAPH

FIGURE 3

BIG SANDY PROJECT
SPILLWAY RATING CURVE
SERVICE SPILLWAY @MAX W.S.

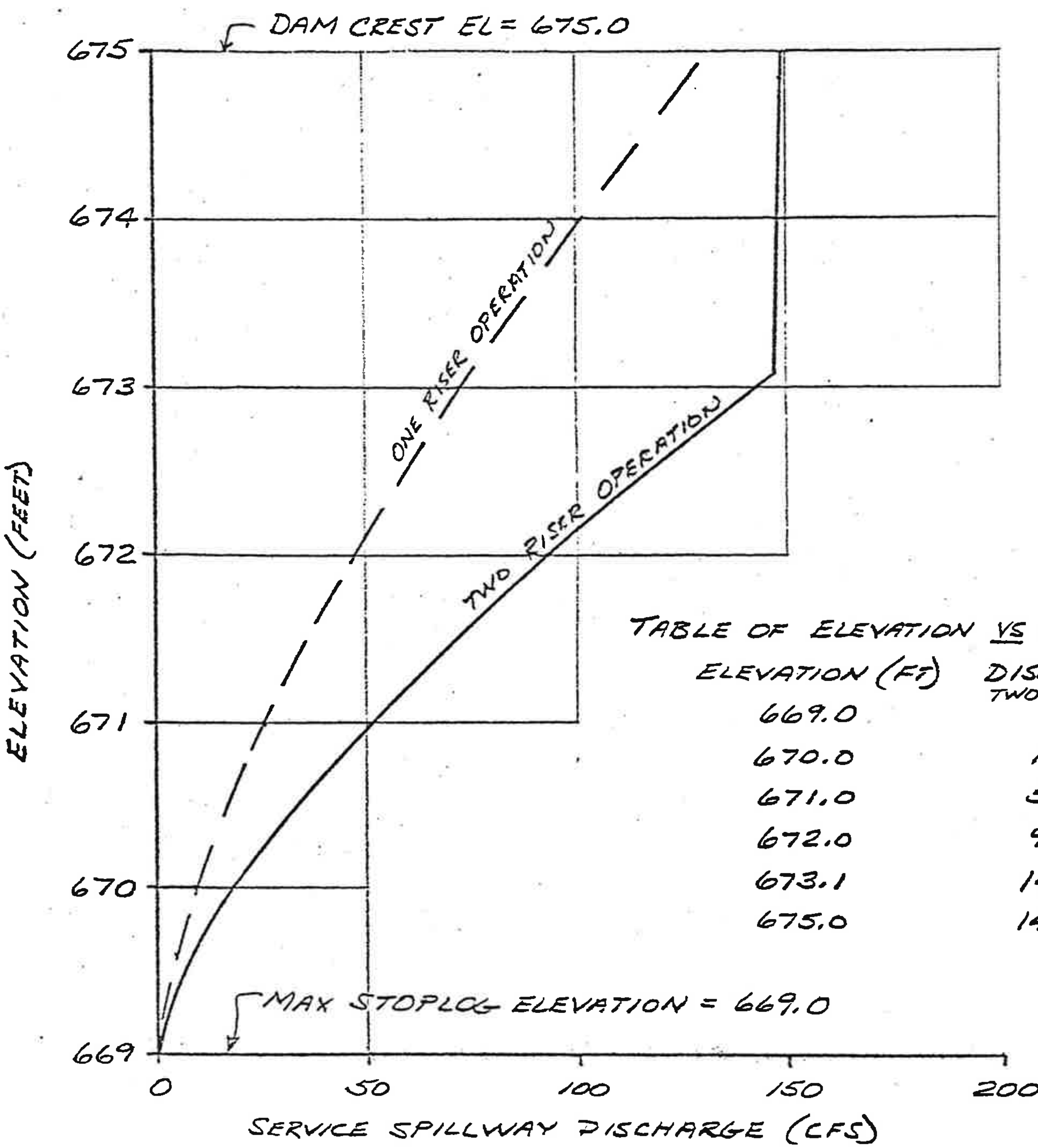


TABLE OF ELEVATION VS DISCHARGE

ELEVATION (FT)	DISCHARGE (CFS) TWO RISERS	ONE RISER
669.0	0.0	0.0
670.0	18.0	9.0
671.0	50.9	25.5
672.0	93.5	46.8
673.1	147.3	74.7
675.0	149.2	132.0

MARCH 9, 1977

FIGURE 4

BIG SANDY PROJECT
TOTAL SPILLWAY CAPACITY
RATING CURVE

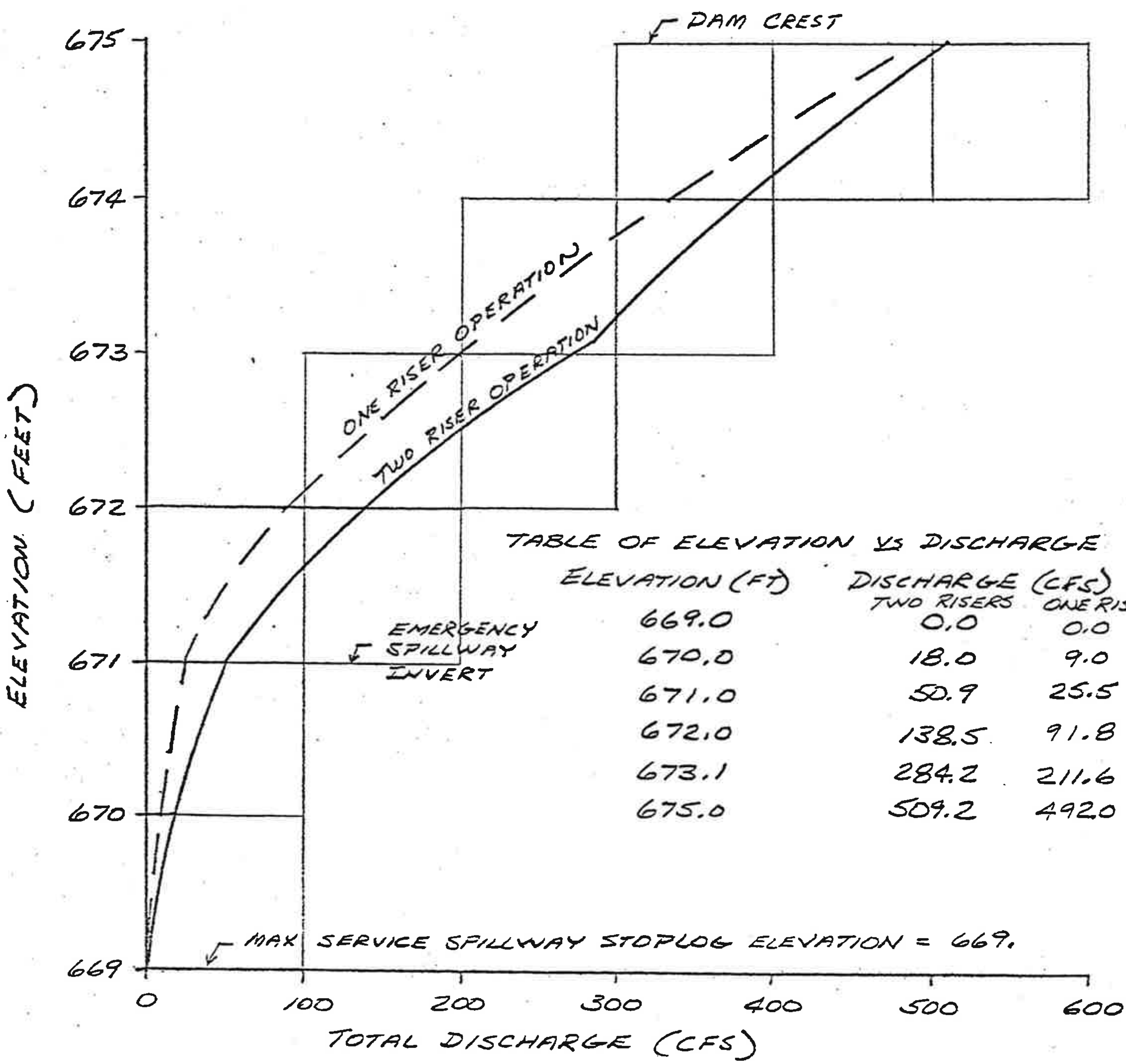
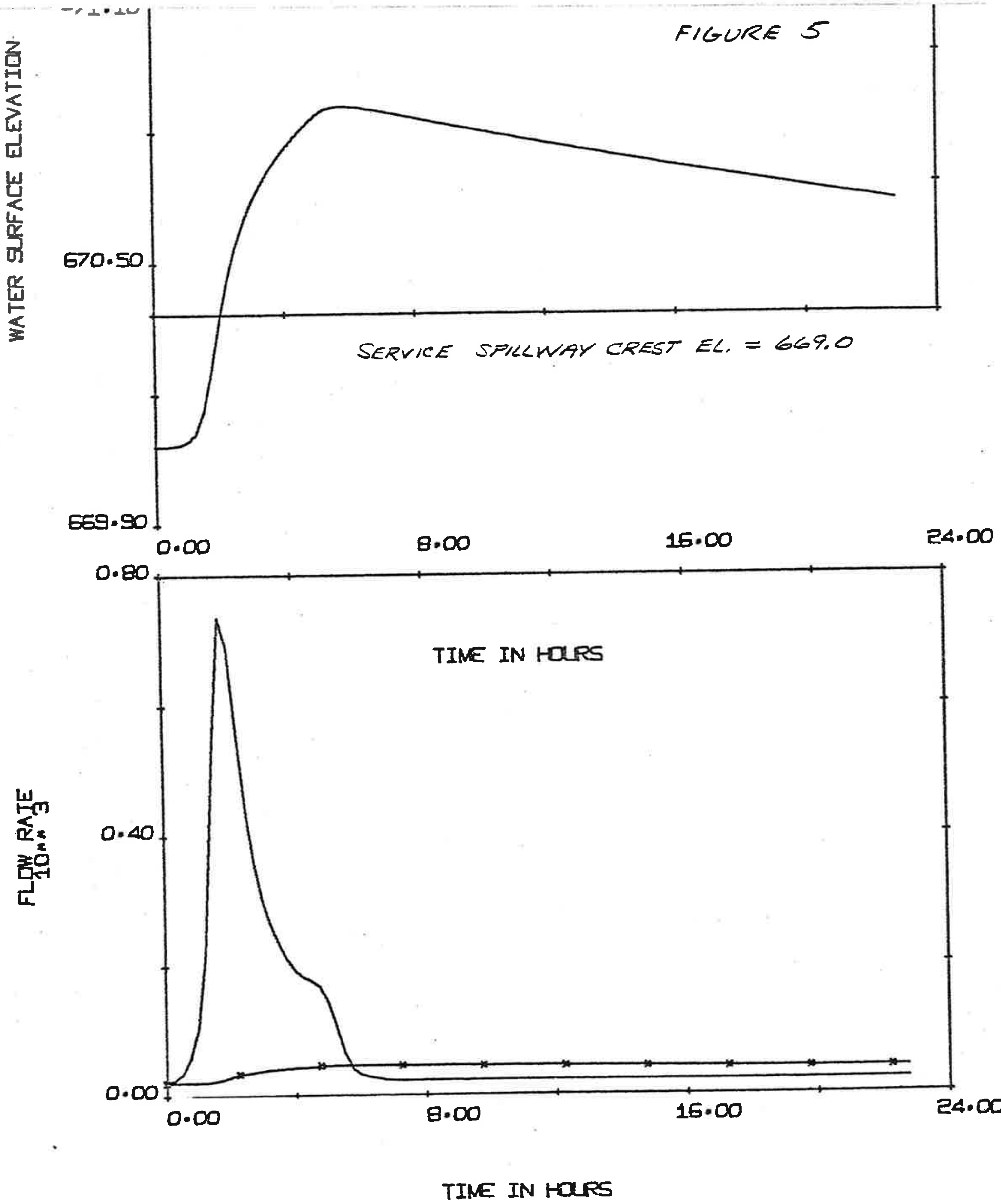


TABLE OF ELEVATION VS DISCHARGE

ELEVATION (FT)	DISCHARGE (CFS) TWO RISERS	ONE RISER
669.0	0.0	0.0
670.0	18.0	9.0
671.0	50.9	25.5
672.0	138.5	91.8
673.1	284.2	211.6
675.0	509.2	492.0

MARCH 9, 1977



BIB SANDY - 25 YEAR FLOOD - DOUBLE RISER, CREST EL.=669., L=6.0

INFLOW HYDROGRAPH

OUTFLOW HYDROGRAPH

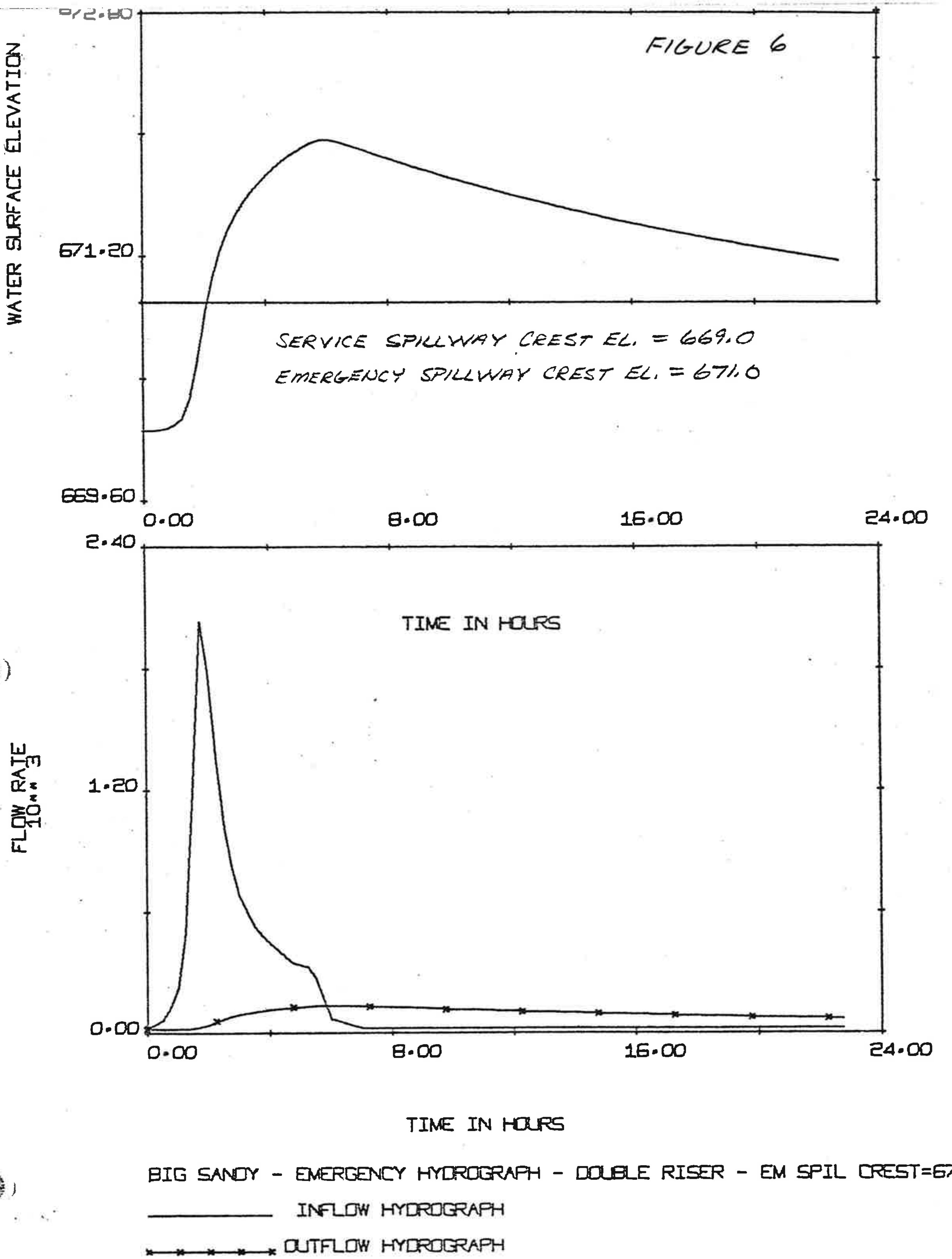
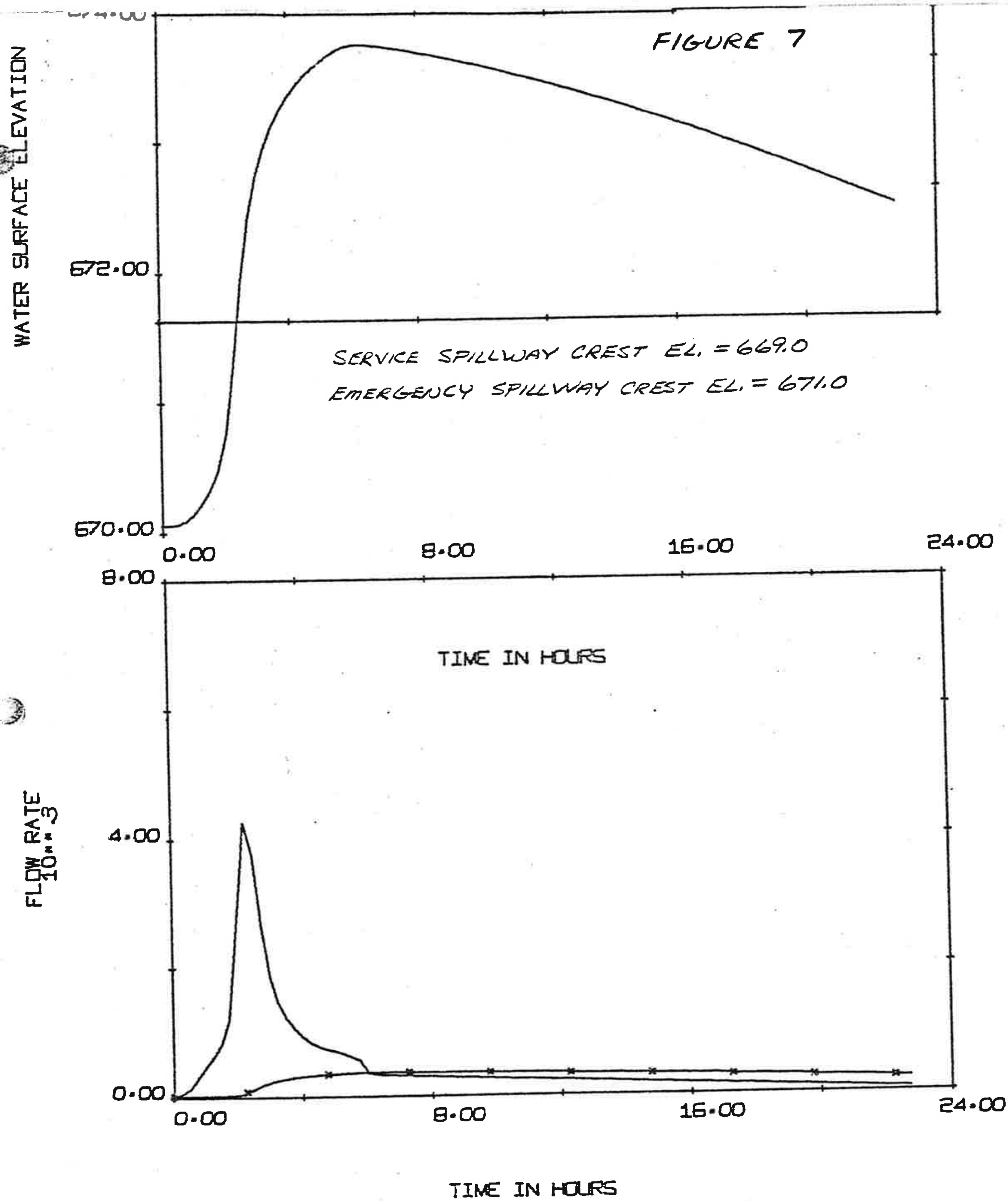


FIGURE 6
WATER SURFACE ELEVATION
672.80
671.20
669.60
SERVICE SPILLWAY CREST EL. = 669.0
EMERGENCY SPILLWAY CREST EL. = 671.0
0.00
8.00
16.00
24.00
TIME IN HOURS
FLOW RATE 10** 3
2.40
1.20
0.00
0.00
8.00
16.00
24.00
TIME IN HOURS
BIG SANDY - EMERGENCY HYDROGRAPH - DOUBLE RISER - EM SPIL CREST=67
INFLOW HYDROGRAPH
OUTFLOW HYDROGRAPH

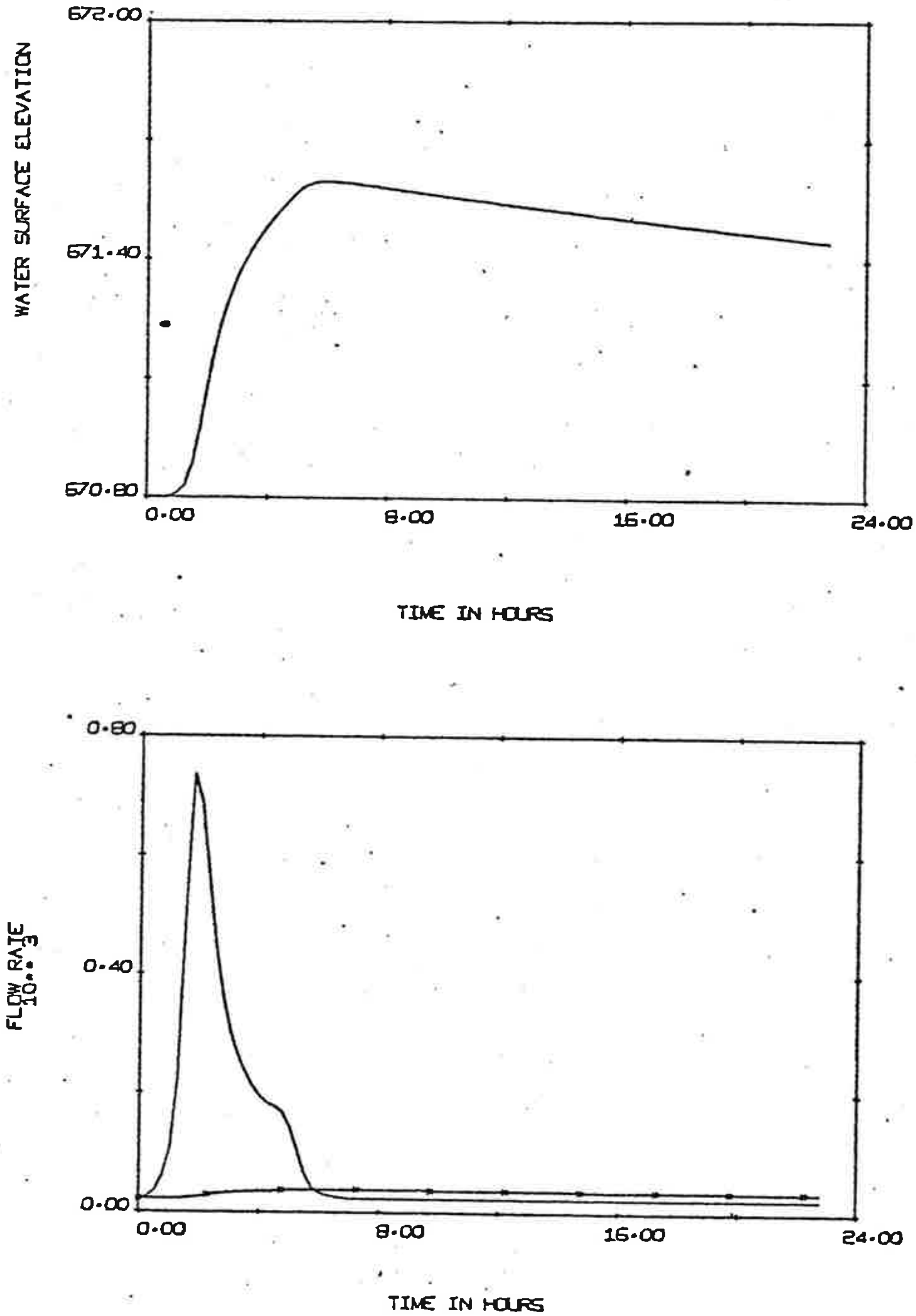


BIG SANDY - FREE BOARD HYDROGRAPH - DOUBLE RISER - EM SPIL CREST=67

INFLOW HYDROGRAPH

OUTFLOW HYDROGRAPH

FIGURE 8

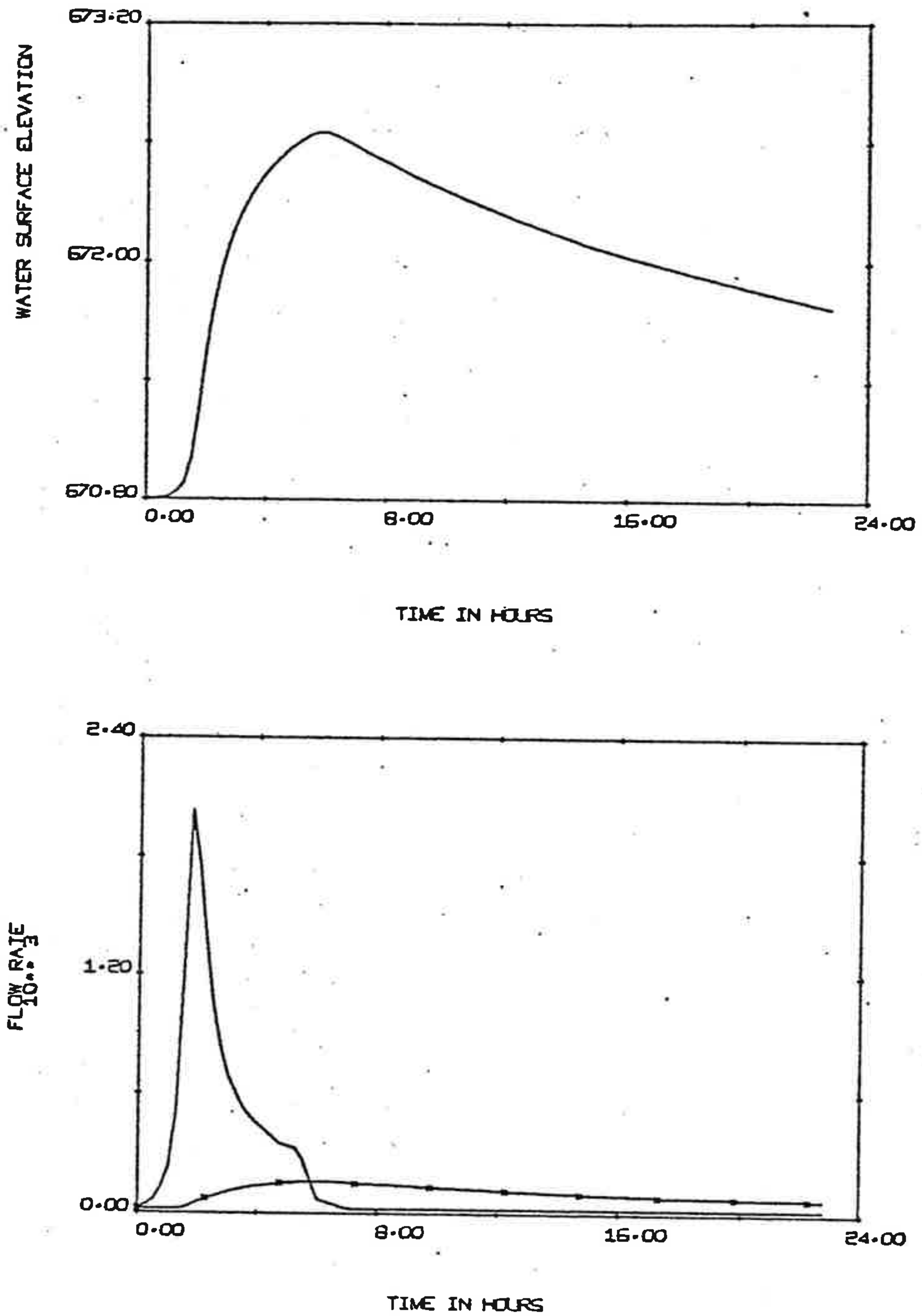


BIG SANDY - 25 YEAR FLOOD - SINGLE RISER, CREST EL.=669., L=3.0

——— INFLOW HYDROGRAPH

—•—•— OUTFLOW HYDROGRAPH

FIGURE 9



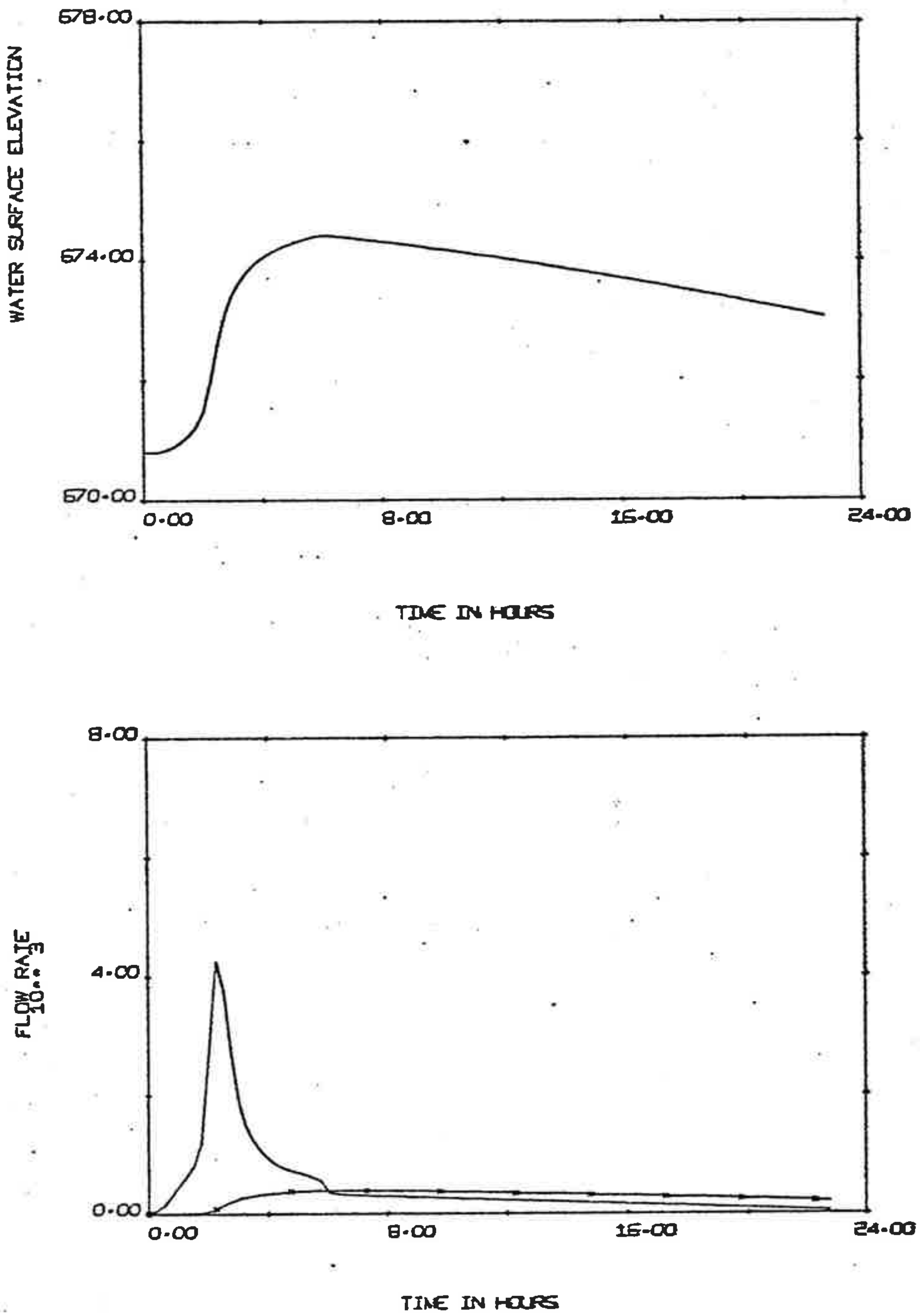
BIG SANDY - EMERGENCY HYDROGRAPH - SINGLE RISER - EM SPIL CREST=671

——— INFLOW HYDROGRAPH

—•—•— OUTFLOW HYDROGRAPH

SERVICE SPILLWAY CREST EL.= 669.0

FIGURE 10



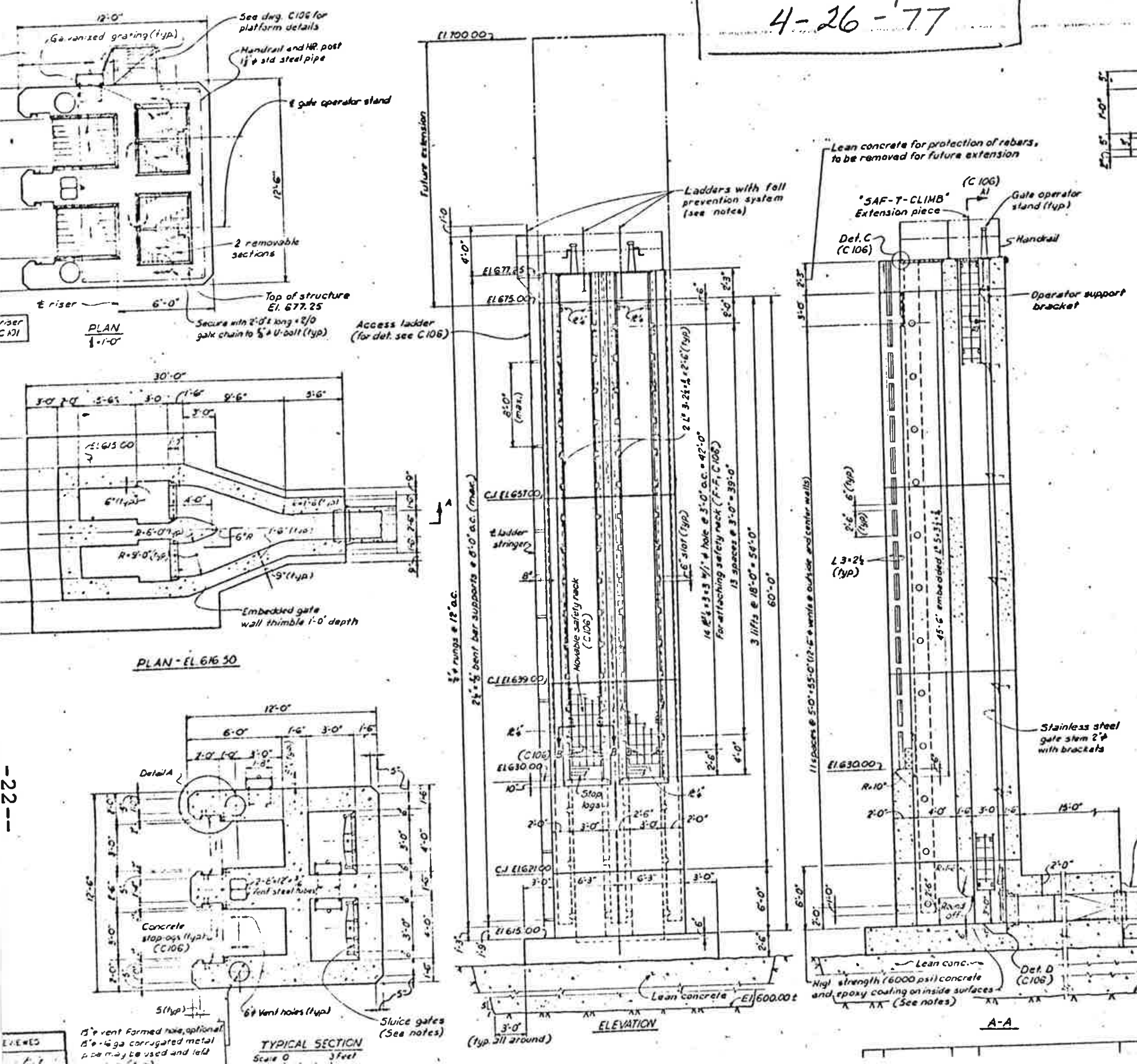
BIG SANDY - FREE BOARD HYDROGRAPH - SINGLE RISER - EM SPIL CREST=671·, L=15·

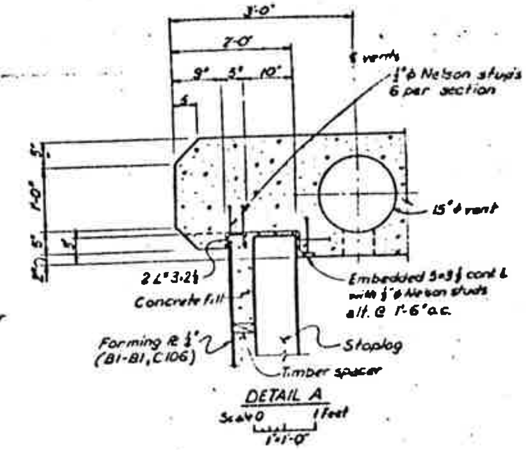
INFLOW HYDROGRAPH

OUTFLOW HYDROGRAPH

SERVICE SPILLWAY CREST EL. = 669.0

AS SENT TO AEP
4-26-'77





REFERERENCE DRAWINGS:

Work this drawing with 984 C101 and C106 - C109

NOTES:

1. All concrete shall be of 4000 psi strength except
   a. in tower between El. 615.00 to 621.00 and stop logs, f'c = 6000 psi.
   b. foundation slab, anti-seep collars, head-wall and conc. apron, f'c = 3000 psi.
   c. lean concrete under foundation slab and pipe cradle concrete, f'c = 2000 psi
2. All reinf. steel shall be deformed bars and shall conform to ASTM-A615-71 grade 60 specification.
3. The indicated concrete area shall be painted with two coats of "Sikagard 664, 9 Hi-Bild" epoxy resin or equal. The minimum thickness of each coat shall be 6 to 9 mils. Surface preparation and epoxy application shall be in accordance with manf. recommendations. No measurement for payment will be made for epoxy protective coating. The entire cost for all work and materials shall be included in the unit price for concrete 6000 psi.
4. All bolts, nuts and washers shall conform to ASTM-A307 specifications, and shall be galvanized.
5. All exposed corners of the structure shall have a ¾" chamfer (min) unless otherwise noted.
6. All exposed misc. metal including grating, ladders, handrails and angles shall be structural steel, conforming to ASTM-A36 specifications and shall be galvanized.
7. The sluice gate openings are 36"×30". Gates shall be of "Rodney Hunt HY-Q 240 H" series, or equal.
8. Provide "SAF-T-CLIMB" fall prevention systems for 3 ladders with one remov. exten. pc., 2 safety belts and 2 sleeves as supplied by Potter-Weil Co., Chicago, or equal.

FIGURE 11

Scale 1" = 1'-0"
Except as noted

KENTUCKY POWER COMPANY
BIG SANDY PLANT
BIG SANDY, KENTUCKY

BIG SANDY FLYASH DAM	SERVICE SPILLWAY

RISER
PLAN AND SECTIONS

CONSULTING ENGINEERS
HARZA ENGINEERING COMPANY

DWG NO. 984 C 105

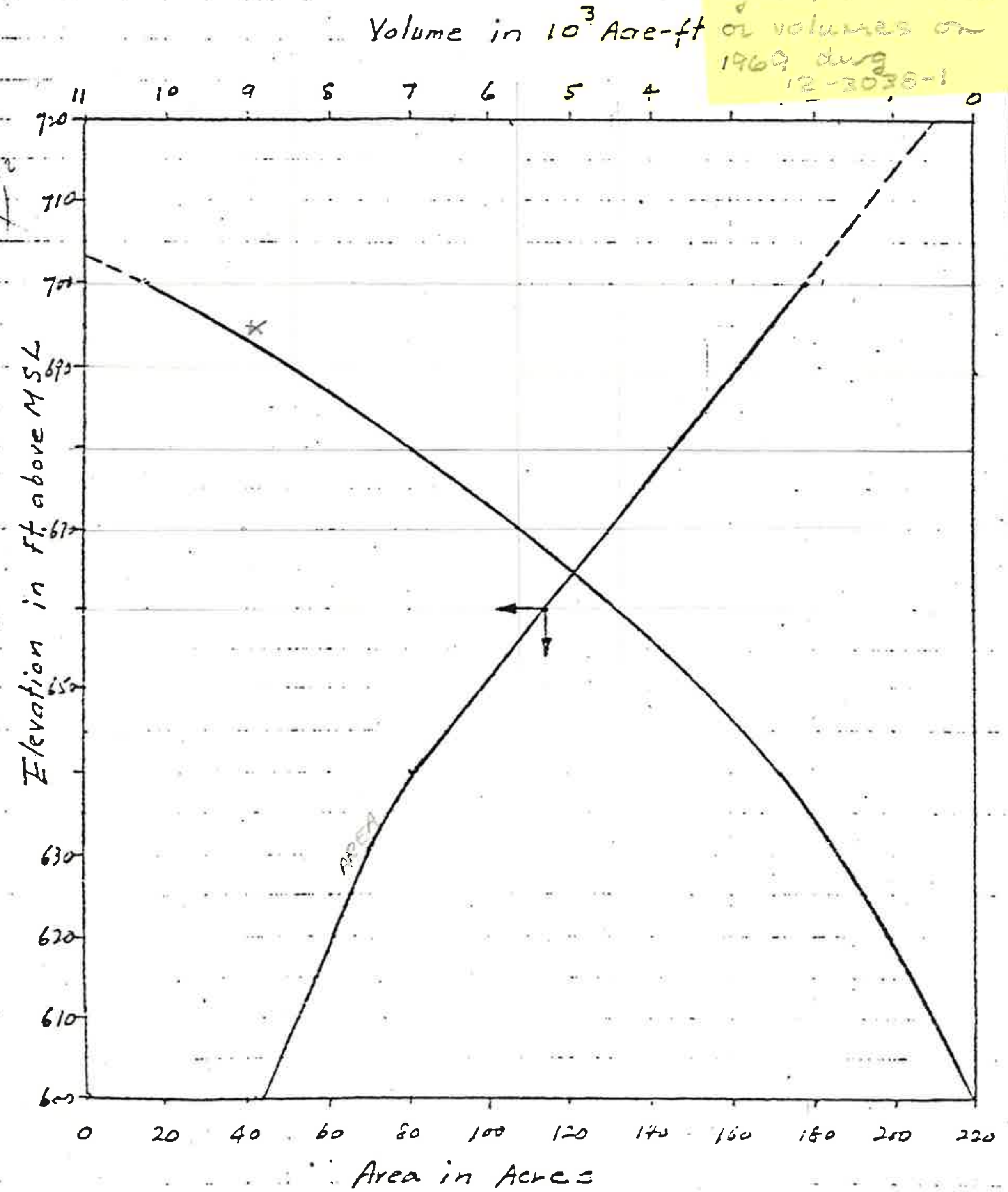
FIG. 12

ENGINEERING DEPT.
AMERICAN ELECTRIC POWER SERVICE CORP.
2 BROADWAY
NEW YORK

SHEET 9 OF
DATE BY ACL CK.
COMPANY Ky Power G.O.
PLANT

SUBJECT Big Sandy - Area - Capacity Curves

This curve does not agree with areas calculated by AP&M in 1986 or volumes on 1969 dwg 12-3038-1



Note: Volume below Ele. 600.0 is assumed zero.

APPENDIX A

BIG SANDY FLYASH DAM

DESIGN FLOODS

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

Summary

The Flyash Dam is classified as class B structure by the Kentucky Division of Water Resources, Department of Natural Resources and Environmental Protection (DWR), according to the downstream potential damages criteria in case of major failure. The emergency spillway and freeboard design floods were computed using the procedure outlined by the Kentucky DWR. Flood resulting from the 25-year 6-hour duration storm was also computed to determine the peak inflow at which the emergency spillway would start functioning. The resulting peak inflows are 770,2130 and 4600 cfs respectively for the 25-year 6-hour duration storm flood and the emergency spillway and the freeboard design floods.

An independent approach, utilizing synthetic triangular unit hydrograph, was used to develop the freeboard design flood from storms of various durations for comparison with the results by the Kentucky DWR procedures. The freeboard design flood resulting from the 24-hour storm has a peak inflow of 4,077 cfs, about 12 percent lower than the flood computed using the Kentucky DWR procedure; the volume, however, is about 20 percent greater.

Flood resulting from the 100-year 24-hour storm was also computed using synthetic triangular unit hydrograph to examine the operation of the existing service spillway. The peak inflow of this flood is about 1,230 cfs.

The Basin

The Flyash Dam is on Horseford Creek in Eastern Kentucky. The drainage area upstream of the dam is about 0.9 square miles, varying in elevation from 535 ft to 965 ft above mean sea level. The dam is classified as class B structure by the Kentucky DWR considering the downstream potential damages in case of major failure.

The proposed top of the dam is at elevation 675 ft. The emergency spillway crest level is El. 671. The reservoir lake area, at elevation 671 ft is about 0.195 square miles which is about 22 percent of the total drainage area.

Insufficient data are available to accurately determine the hydrologic soil group for the watershed. However, the areas about 6000 ft east (Catalpa) and 12,000 ft west (Fallsburg) of the project area, have predominantly Group C soils. Hence, the soils in the area are assumed to belong to Group C. The soil cover is woodland in good condition.

General Approach

Two independent approaches were used to derive the design floods. First, the procedure outlined by the Kentucky DRW (1)* for class B structure, was used to develop design flood hydrographs for the service and emergency spillways and the freeboard. The Kentucky DWR defines the emergency spillway hydrograph as the hydrograph used to establish the minimum dimensions of the emergency spillway. The freeboard hydrograph is defined as the hydrograph used to establish the minimum elevation of the top of the dam.

An other estimate of the freeboard design flood was made for comparison with that resulted by the method recommended by the

*Number in parenthesis indicate reference number.

Kentucky DWR. This method involves the application of synthetic triangular unit hydrograph and incremental rainfall excess. Storms of 6-, 12- and 24-hour durations were used for the freeboard design flood. The longer duration storm of 12- and 24-hours were analysed in case the spillway design was such that discharge of the flood runoff from the reservoir takes a relatively long period of time. In that case, flood volume may be more important than peak inflow.

Hydrologic Criteria

The Kentucky DWR recommends (1) as minimum criteria, the use of U.S. Soil Conservation Service (SCS) methods (2) for deriving the design floods. It also specifies the use of rainfall depth-duration-frequency data published by the U.S. Weather Bureau (3).

The recommended minimum design rainfall for class B structures are:

$$P_{ES} = P_{100} + 0.12\ (PMP - P_{100}) \text{ --- I}$$

$$P_{F} = P_{100} + 0.40\ (PMP - P_{100}) \text{ ---II}$$

in which

P_{ES} = emergency spillway design rainfall for storm duration, t hour

P_F = freeboard design rainfall for storm duration, t hour

P_{100} = 100-year rainfall for storm duration t, hour

PMP = probable maximum precipitation for storm duration, t hour.

Storm duration of 6-hours is recommended for design by the Kentucky DWR for watersheds having time of concentration equal to or less than 6 hours.

The recommended allowable frequency of activating emergency spillways on rock foundations is 25 years (Table F-1, reference (1)).

Computations of Design Floods

Kentucky Division of Water Resources Procedure

The time of concentration for the basin upstream of Flyash Dam was estimated to be 0.55 hours (see Figure 30, page 77 of reference (4)). Therefore, storm duration of 6-hour was used in equations I and II to compute the total rainfalls. The values of P_{100} and PMP were obtained from U.S. Weather Bureau TP 40 (3). These are shown in Table 1.

The watershed soils are of hydrologic group C and the land covers are woods in good conditions. The Kentucky DWR (1) recommends the use of antecedent moisture conditions (AMC) II or greater. AMC III was selected for conservative design. The run-off curve number was estimated as 85 (see Appendix A, tables A-4 and A-7 of reference (4)). This curve was used to determine the excess rainfalls. The reservoir area is about 22 percent of the total drainage area at the emergency spillway crest (671 ft msl). It was considered desirable to exclude reservoir area in calculating retention losses. Therefore, the computed excess rainfalls were adjusted accordingly. Table 1 shows the total and excess rainfall amounts.

The design floods were computed using the SCS method (2). The method essentially involves the selection of a family of curves based on the ratio To/Tp, where To is duration of excess rainfall and Tp is time to peak of the hydrograph estimated on the basis of time of concentration.

The quick return flow (see page 21-9 of reference 2) about 6.3 cfs, was computed using Table 21-4 of reference (2). The estimated flyash inflow is about 7100 gpm (16 cfs). Therefore, the total stream flow prior to a storm would be about 22 cfs.

Table 2 shows the service and emergency spillways, and the freeboard design flood hydrographs. The flood peaks are 770, 2130 and 4,600 cfs respectively.

Unit Hydrograph Approach

The freeboard design flood and the service spillway flood were derived using synthetic triangular unit hydrograph and incremental rainfall excesses.

The total rainfall amounts for 6-, 12- and 24-hour durations were computed using equation II for the freeboard design flood. The 6-hour rainfall was distributed in 0.5 hour increments using the rainfall distribution curve for Zone C figure 18, page 51 of reference (4). These incremental rainfalls were arranged as shown in table 3 for most critical flood conditions. The 12- and 24-hour rainfall amounts were also distributed into 0.5 hour increments and arranged in the pattern shown in table 3.

The 100-year 24-hour storm should, normally, be distributed according to a representative but less critical hourly rainfall pattern to produce a 100-year flood. Sufficient data are not available to develop such a pattern. Therefore, for conservative estimate, the incremental rainfall distribution pattern, adopted for the freeboard design storm was also used for the 100-year storm.

Runoff curve number 85 was estimated for the watershed as discussed previously. The rainfall excess for each time increment was determined using this curve. The minimum retention rate, for soils under hydrologic group C, ranges from 0.08 to 0.15 inch per hour. A retention rate of 0.10 inch per hour was assumed for this study. The computed excess rainfalls are shown in Table 1.

The drainage area above the dam is about 0.9 square miles. The time of concentration is 0.55 hours. A synthetic triangular unit hydrograph of 0.5 hour duration was derived using these data and the SCS method (see page 69 of reference (4)).

The derived unit hydrograph was used to determine flood hydrographs for the 6-, 12- and 24-hour freeboard design storms and the 100-year 24-hour storm. These hydrographs are given in tables 4 and 5 respectively. Figure 1 shows the freeboard flood hydrographs resulting from the Kentucky DWR procedure and the unit hydrograph approach.

Conclusion

A comparison of the freeboard design floods resulting from the Kentucky DWR procedure and the 24-hour storm hydrograph derived by the synthetic unit hydrograph indicates that the former has a higher peak inflow but a smaller volume. It is concluded that both hydrographs should be routed through the reservoir to determine the more critical one for use in the design.

Reference

1. Divison of water Resources, Hydrologic Criteria, Section C, Engineering Memorandum No. 5 (Preliminary), Department of Natural Resources and Environmental Protection, Frankfort, Kentucky, February 1, 1975.
2. Soil Conservation Service, "National Engineering Handbook," Section 4, Hydrology, Chapters 10 and 21, Department of Agriculture, U.S. Government Printing Office, Washington, D. C. August 1972.
3. U.S. Weather Bureau, "Rainfall Frequency Atlas of the United States for durations from 30 minutes to 24 hours and Returen Periods from 1 to 100 years," Technical Paper 40, Department of Commerce, U.S. Government Printing Office, Washington, D. C. May 1961.
4. U.S. Bureau of Reclamation, "Design of small Dams" Second Edition, Revised Reprint 1974.

Table 1

TOTAL AND EXCESS RAINFALL

(Depth, inches)

Duration	P_{100}		PMP	P_{ES}		P_F		P_{25}	
	(Total)	(Excess)	(Total)	(Total)	(Excess)	(Total)	(Excess)	(Total)	(Excess
6	4.50		27.0	7.20	5.83	13.50	12.01 11.75 1/	3.70	2.27
12	5.50		31.9			16.04	14.11 1/		
24	6.00	4.04 1/	34.6			17.42	15.21 1/		

P_{100} = 100-year rainfall

PMP = Probable maximum precipitation

P_{ES} = emergency spillway design storm

P_F = freeboard design storm

P_{25} = 25-year rainfall

1/ Rainfall values used in unit hydrograph approach

Table 2

DESIGN FLOOD HYDROGRAPHS USING
KENTUCKY DIVISION OF WATER RESOURCES PROCEDURES

Service Spillway		Emergency Spillway		Freeboard	
Hr	Disch;cfs	Hr	Disch;cfs	Hr	Disch;cfs
0	22	0	22	0	22
.30	26	.3	37	.23	37
.61	41	.6	76	.47	111
.91	78	.9	175	.70	244
1.21	152	1.2	304	.93	422
1.51	508	1.5	1151	1.17	570
1.81	772	1.8	2134	1.40	719
2.12	654	2.1	1654	1.63	941
2.42	488	2.4	1158	1.86	1386
2.72	376	2.7	876	2.10	3327
3.02	302	3.0	693	2.33	4602
3.33	255	3.3	579	2.56	3623
3.63	222	3.6	502	2.80	2556
3.93	197	3.9	449	3.03	1860
4.23	183	4.2	419	3.26	1460
4.54	174	4.5	388	3.50	1222
4.84	164	4.8	365	3.73	1060
5.14	131	5.1	358	3.96	926
5.44	74	5.4	342	4.19	837
5.75	41	5.7	198	4.43	763
6.05	30	6.0	68	4.66	719
6.35	26	6.3	45	4.89	689
6.65	24	6.6	30	5.13	674
6.95	22	6.9	22	5.36	659
				5.59	615
				5.83	526
				6.06	318
				6.29	141
				6.52	81
				6.76	52

Table 3

TOTAL INCREMENTAL RAINFALL

Hours	Freeboard Design Storm 6-hr	12-hr	24-hr	100-yr 24-hr storm					
(1)	(2)	(3)	(4)	(5)	(1)	(2)	(3)	(4)	(5)
.5	.52	.10	.02	.01	12.5			.56	.18
1.0	.54	.11	.02	.01	13.0			.64	.22
1.5	.56	.12	.03	.01	13.5			.97	.32
2.0	.64	.13	.03	.02	14.0			2.29	.76
2.5	.97	.16	.03	.02	14.5			4.19	1.40
3.0	2.29	.24	.04	.02	15.0			1.22	.41
3.5	4.19	.40	.04	.02	15.5			.92	.31
4.0	1.22	.52	.04	.02	16.0			.57	.18
4.5	.92	.54	.05	.02	16.5			.55	.18
5.0	.57	.56	.05	.02	17.0			.53	.18
5.5	.55	.64	.06	.02	17.5			.50	.16
6.0	.53	.97	.06	.02	18.0			.32	.12
6.5		2.29	.07	.02	18.5			.20	.09
7.0		4.19	.08	.02	19.0			.14	.07
7.5		1.22	.09	.03	19.5			.12	.05
8.0		.92	.10	.03	20.0			.11	.04
8.5		.57	.10	.04	20.5			.10	.03
9.0		.55	.12	.05	21.0			.09	.03
9.5		.53	.13	.06	21.5			.08	.03
10.0		.50	.16	.08	22.0			.07	.02
10.5		.32	.24	.10	22.5			.07	.02
11.0		.20	.40	.14	23.0			.06	.02
11.5		.14	.52	.18	23.5			.05	.02
12.0		.12	.54	.18	24.0			.05	.02

13.5

Table 4

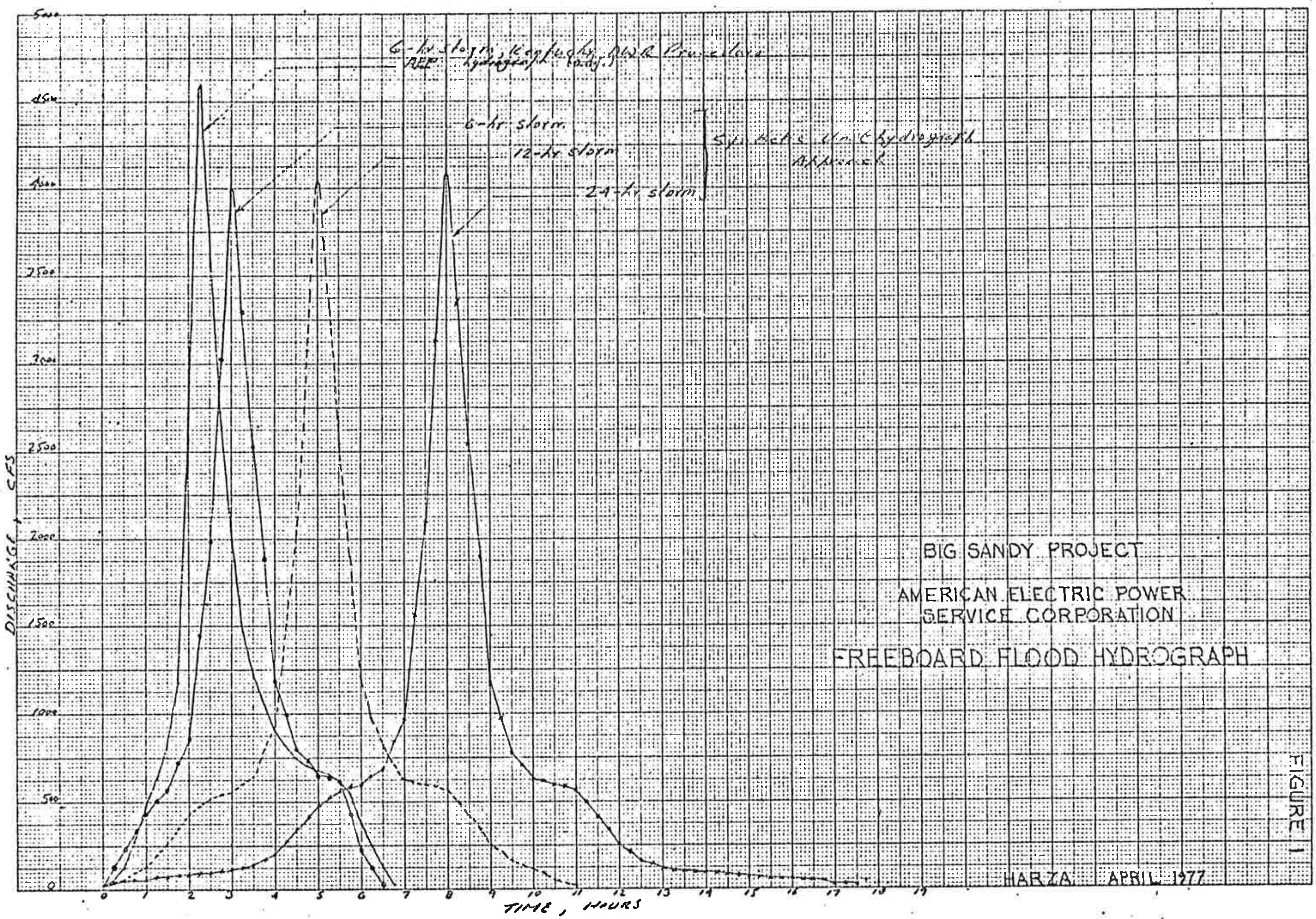
FREEBOARD DESIGN FLOODS
(Discharge in cfs)

Time Hours (1)	DWR 6-hr Storm (2)	Unit Hydrograph Aproach 6-hr Storm (3)	12-hr Storm (4)	24-hr Storm (5)
0	22	22	22	22
.25	50	129	46	38
.50	125	231	68	53
.75	275	334	101	61
1.00	475	429	130	69
1.25	630	506	201	73
1.50	825	569	266	77
1.75	1175	722	350	83
2.00	2575	856	425	88
2.25	4600	1441	476	95
2.50	3750	1983	515	99
2.75	2725	3026	543	111
3.00	2000	3984	558	119
3.25	1475	3281	610	133
3.50	1220	2515	646	142
3.75	1060	1870	802	175
4.00	910	1163	933	204
4.25	825	982	1510	275
4.50	750	775	2041	340
4.75	710	710	3080	415
5.00	680	624	4030	480
5.25	665	620	3313	524
5.50	625	600	2530	554
5.75	550	411	1878	585
6.00	370	213	1163	600
6.25	250	120	982	655
6.50	125	22	775	693
6.75	22		710	844
7.00			624	972
7.25			620	1560
7.50			600	2099
7.75			591	3132
8.00			569	4077
8.25			500	3344
8.50			418	2542
8.75			341	1884
9.00			255	1163
9.25			212	982
9.50			161	775
9.75			145	710
10.00			123	624
10.25			89	620
10.50			53	600
10.75			38	593
11.00			22	569
11.25				500
11.50				418
11.75				341
12.00				255
12.25				212
12.50				161
12.75				145
13.00				123
13.25				117
13.50				107
13.75				103
14.00				96
14.25				91
14.50				84
14.75				79
15.00				73
15.25				67
15.50				61
15.75				60
16.00				57
16.25				54
16.50				49
16.75				40
17.00				30
17.25				26
17.50				22

Table 5

100-YEAR 24-HOUR STORM FLOOD

Time Hour	Discharge cfs
0	22
.25	42
.50	61
.75	71
1.00	81
1.25	97
1.50	112
1.75	121
2.00	127
2.25	152
2.50	173
2.75	187
3.00	197
3.25	231
3.50	258
3.75	436
4.00	600
4.25	927
4.50	1228
4.75	1015
5.00	785
5.25	588
5.50	371
5.75	311
6.00	243
6.25	223
6.50	197
6.75	199
7.00	197
7.25	187
7.50	174
7.75	154
8.00	131
8.25	113
8.50	92
8.75	79
9.00	65
9.25	50
9.50	34
9.75	28
10.00	22


BIG SANDY PROJECT
AMERICAN ELECTRIC POWER
SERVICE CORPORATION
FREEBOARD FLOOD HYDROGRAPH
HARZA APRIL 1977
FIGURE 1
6-hr storm
12-hr storm
24-hr storm
DISCHARGE, CFS
TIME, HOURS

APPENDIX B

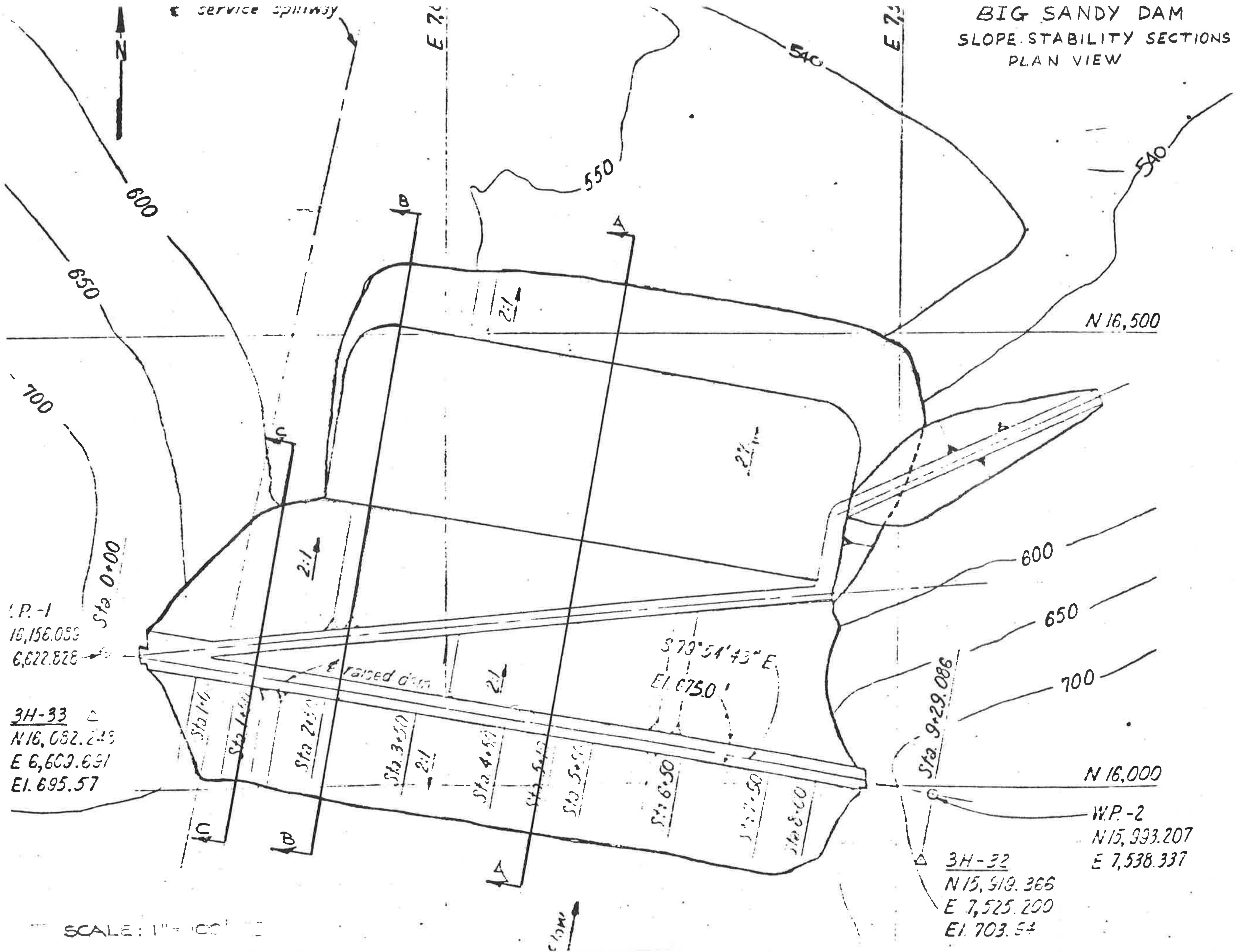
BIG SANDY FLYASH DAM

SLOPE STABILITY ANALYSIS SUMMARY SHEETS

AND

FLOW NET

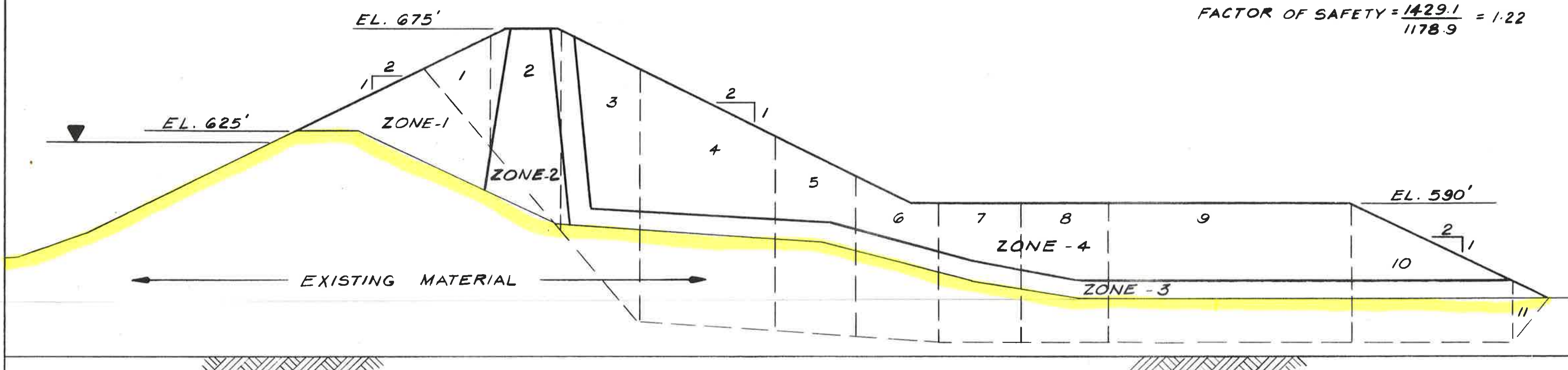
BIG SANDY DAM
SLOPE STABILITY SECTIONS
PLAN VIEW



SLICE NO.	α_i	$\tan \alpha_i$	Ft. AX_i	KSF P_i	KSF $\bar{C}$	K/Ft. B_{oi}	K/Ft. A''_{oi}	K/Ft. A_{oi}
1	49°	1.15	34	3.77	3.2	147.1	80.1	252.4
2	49°	1.15	35	10.40	3.2	417	110.6	256.6
3	49°	1.15	38	13.02	3.2	568.97	76	282.1
4	3°	.05	67	13.8	2.0	46.23	134	134
5	3°	.05	40	10.8	2.0	21.6	80	80
6	3°	.05	40	8.48	1.2	16.96	48	48
7	0	0	40	8.78	1.2	0	48	48
8	0	0	42	8.9	1.2	0	504	48
9	0	0	120	9.35	1.2	0	192	144
10	0	0	80	6.5	1.2	0	45.6	96
11	-45°	1	20	1.95	1.2	-39	43.7	48

$\Sigma = 1178.9$ $\quad \Sigma = 1429.1$

$$\text{FACTOR OF SAFETY} = \frac{1429.1}{1178.9} = 1.22$$



ZONE-1 - RANDOM ROCKFILL - γt = 135 PCF ; $\bar{C}$ = 3.2 KSF

ZONE-2 - CLAY CORE - γt = 130 PCF ; $\bar{C}$ = 3.2 KSF

ZONE-3 - DRAINAGE MATERIAL - γt = 70 PCF ; ϕ = 40°

ZONE-4 - SANDSTONE - γt = 135 PCF ; ϕ = 32°

EXISTING DAM AND FOUNDATION MATERIAL - γt = 130 PCF ; $\bar{C}$ = 1.2 KSF - 3.2 KSF

SECTION A - A

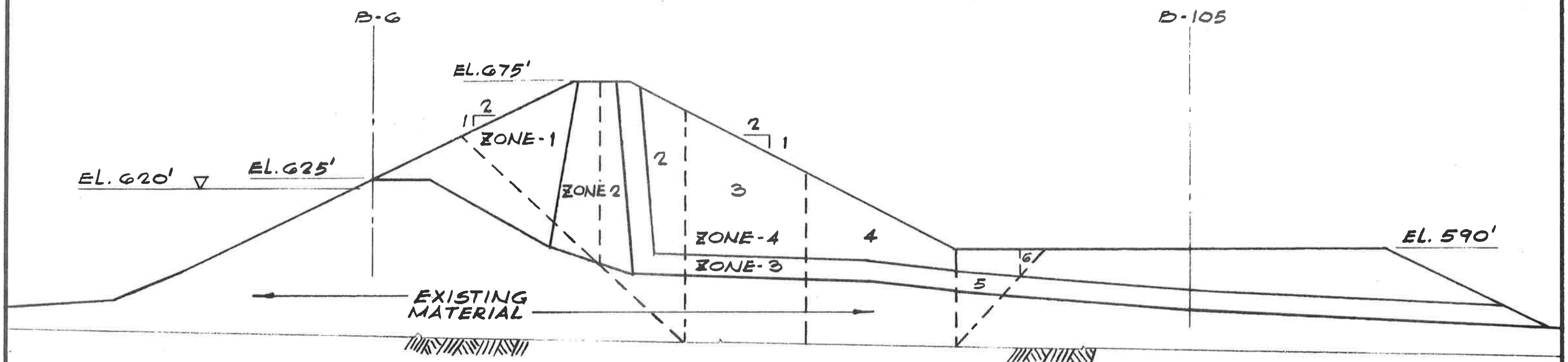
SCALE : 1" = 50'

KENTUCKY POWER CO.
BIG SANDY PLANT
LOUISA KENTUCKY

TOTAL STRESS ANALYSIS
SECTION A - A

SLICE #	α	TAN α	FT. Δx	KSF P	KSF C	K/FT. Bo	K/FT. Ao
1	43°	.93	71	6.6	3.2	432.5	422.59
2	43°	.93	40	14.2	2.0	526.5	148.8
3	0°	.0	61	12.9	1.2	0	73.4
4	0°	.0	88	8.9	1.2	0	99.6
5	40°	.84	40	4.81	1.2	-161.6	81.6
6	40°	.84	26	1.35	3.2	- 29.5	141.4
						Σ767.9	Σ967.39

$$\text{FACTOR OF SAFETY} = \frac{967.39}{767.9} = 1.26$$



SECTION B-B
1" = 50'

ZONE - 1 - RANDOM ROCKFILL - γt = 135 PCF, C = 3.2 KSF
ZONE - 2 - CLAY CORE ------ γt = 130 PCF, C = 3.2 KSF
ZONE - 3 - DRAINAGE MATERIAL - γt = 70 PCF, φ = 40°
ZONE - 4 - SANDSTONE ------ γt = 135 PCF, φ = 32°
EXISTING DAM & FOUNDATION -- γt = 130 PCF, C = 1.2 KSF-3.2 KSF

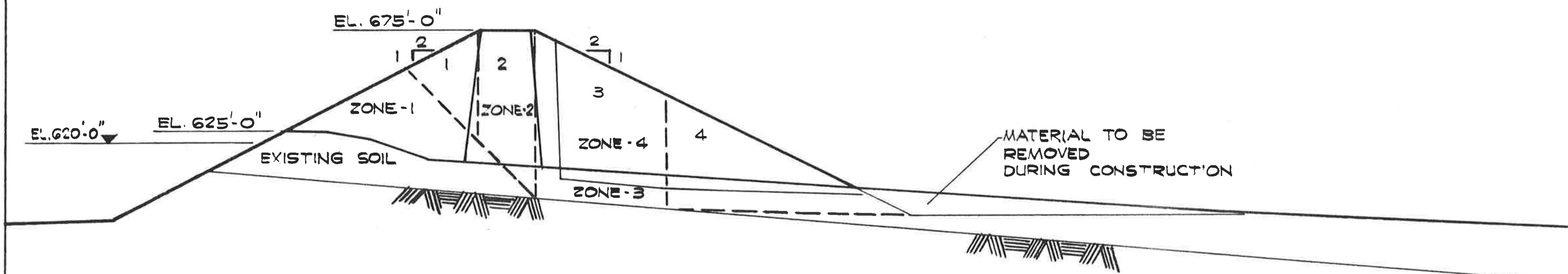
KENTUCKY POWER CO.
BIG SANDY PLANT
LOUISA KENTUCKY

TOTAL STRESS ANALYSIS
SECTION B-B

SLICE NO.	α	TANα	FT. ΔX	KSF P	KSF C	K/FT B	K/FT A
1.	45°	1.00	39	3.78	3.2	147.5	249.6
2	45°	1.00	25	8.84	3.2	221.0	160.0
3.	5°	.09	66	8.78	1.2	52.2	79.2
4	2°	.03	119	3.70	1.2	13.2	142.8
						Σ 433.8	631.6

FACTOR OF SAFETY =

$$\frac{631.6}{433.8} = 1.45$$



ZONE-1 – RANDOM ROCKFILL – γ_t = 135 PCF , C = 3.2 KSF
ZONE-2 – CLAY CORE – γ_t = 130 PCF , C = 3.2 KSF
ZONE-3 – DRAINAGE ZONE – γ_t = 70 PCF , ϕ = 40°
ZONE-4 – SANDSTONE – γ_t = 135 PCF , ϕ = 32°
EXISTING DAM & FOUNDATION – γ_t = 130 PCF , C = 1.2 KSF – 3.2 KSF

SECTION C-C

SCALE: 1" = 50'-0"

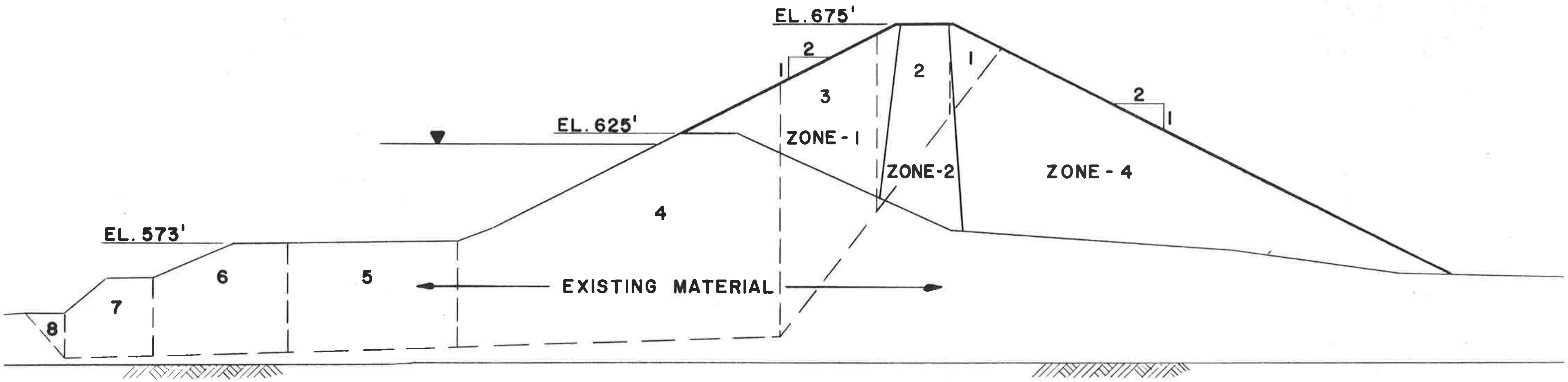
KENTUCKY POWER CO.
BIG SANDY PLANT
LOUISA KENTUCKY

TOTAL STRESS ANALYSIS
SECTION C-C

SLICE No.	∝	Tan∝	Ft. AX	KSF P	KSF C	Tan Ø	K/Ft. Bo	K/Ft. Ao
1	52°	1.28	24	2.73	0	32°/.63	83.9	66.7
2	52°	1.28	36	8.78	3.2	0	404.6	304.13
3	52°	1.28	47	13.33	3.0	0	801.9	372.24
4	1°	.02	153.5	11.05	3.0	0	29.6	460.5
5	1°	.02	80	6.5	2.0	0	10.4	160
6	0	0	62	6.13	2.0	0	0	124
7	0	0	42	4.3	2.0	0	0	84
8	45°	1	18	1.2	1.2	0	-21.6	21.6

$\Sigma = 1308.8$; $\Sigma = 1593.2$

Factor of Safety = 1593.2/1308.8
= 1.22



ZONE - 1 - RANDOM ROCKFILL - γt = 135 PCF ; C = 3.2 KSF
ZONE - 2 - CLAY CORE - γt = 130 PCF ; C = 3.2 KSF
ZONE - 4 - SANDSTONE - γt = 135 PCF ; Ø = 32°
EXISTING DAM AND FOUNDATION - γt = 130 PCF ; C = 1.2 KSF - 3.2 KSF

SECTION A' - A'

SCALE: 1" = 50'

KENTUCKY POWER CO.
BIG SANDY PLANT
LOUISA KENTUCKY

TOTAL STRESS ANALYSIS
UPSTREAM SLOPE
SECTION A' - A'

EARTHQUAKE CONDITION

SLICE #	α_i	TAN α	FT. $\Delta\ell_i$	KSF P_i	K/FT. ΔQ_i	KSF u_i	ϕ	TAN ϕ	K/FT. B_o	K/FT A_o'	N_o	K/FT. A_o
1	49°	1.15	34	3.77	12.8	2.17	32°	.62	160.3	34.	.66	51.5
2	49°	1.15	35	10.4	36.4	3.73	24°	.45	453.6	104.8	.6	174.6
3	49°	1.15	38	13.02	49.5	3.77	31°	.6	618.7	212.5	.65	326.9
4	3°	.05	67	13.8	92.5	3.26	31°	.6	138.7	423.7	1.	423.7
5	3°	.05	40	10.8	43.2	2.96	31°	.6	64.8	188.2	1.	188.2
6	3°	.05	40	8.48	33.9	2.55	31°	.6	50.9	142.3	1.	142.3
7	0	0	40	8.78	35.1	2.5	31°	.6	35.1	150.7	1.	150.7
8	0	0	42	8.9	37.3	2.5	31°	.6	37.3	160.8	1.	160.8
9	0	0	120	9.35	102.2	2.5	31°	.6	112.2	493.	1.	493.
10	0	0	80	6.5	52.	2.5	31°	.6	52.	192.	1.	192.
11	39°	.809	20	1.95	3.9	.94	31°	.6	35.1	11.16	.82	13.6
									Σ1758.7	Σ2113.1		Σ2317.

$$\frac{A_o'}{B_o} = \frac{2113.1}{1758.7} = 1.20$$

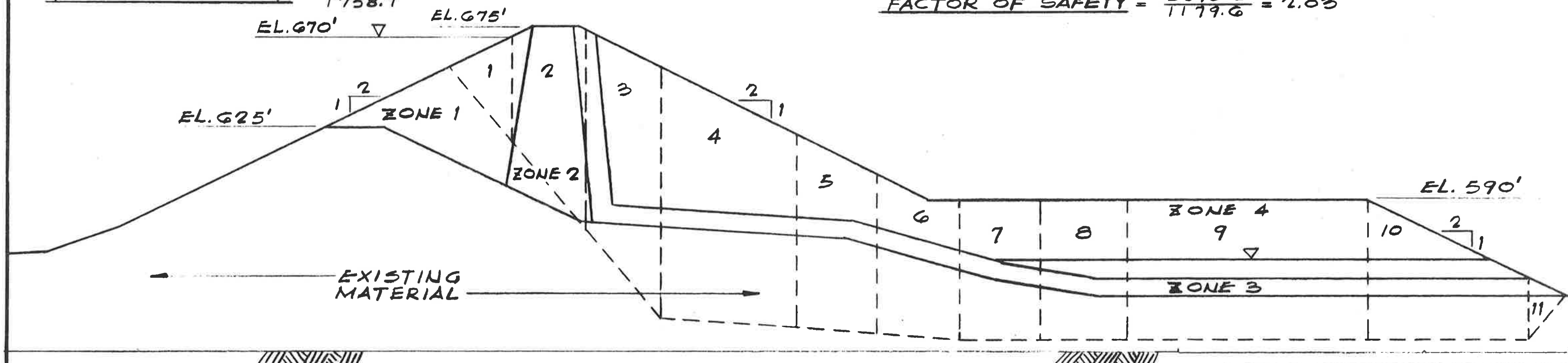
$$\text{FACTOR OF SAFETY} = \frac{2317}{1758.7} = 1.32$$

EFFECTIVE STRESS ANALYSIS

SLICE #	α_i	TAN α	FT. $\Delta\ell_i$	KSF P_i	KSF U_i	ϕ	TAN ϕ	K/FT. B_o	K/FT. A_o'	N_o	K/FT. A_o	K/FT. ΔE_o	K/FT. E_o
1	49°	1.15	34	3.77	2.17	32°	.62	147.5	34.	.62	58.6	118.3	118.3
2	49°	1.15	35	10.4	3.73	24°	.45	417.2	104.8	.56	194.	321.3	439.5
3	49°	1.15	38	13.02	3.71	31°	.6	569.2	212.5	.61	372.	335.4	825.
4	3°	.05	47	13.8	3.26	31°	.6	46.2	423.7	1	423.7	-162.3	662.7
5	3°	.05	40	10.8	2.96	31°	.6	21.6	188.2	1	188.2	-71.3	591.4
6	3°	.05	40	8.48	2.55	31°	.6	16.96	142.3	1	142.3	-53.3	538.1
7	0	0	40	8.78	2.5	31°	.6	0	150.7	1	150.7	-74.5	463.6
8	0	0	42	8.9	2.5	31°	.6	0	160.8	1	160.8	-79.5	389.1
9	0	0	120	9.35	2.5	31°	.6	0	493.	1	493.	-242.9	141.2
10	0	0	80	6.5	2.5	31°	.6	0	192.	1	192.	-94.9	461.5
11	39°	.809	20	1.95	.94	31°	.6	-39.	11.16	.78	14.9	-46.5	0
								Σ1179.6	Σ2113.2		Σ2390.2		

$$\frac{A_o'}{B_o} = \frac{2113.2}{1179.6} = 1.79$$

$$\text{FACTOR OF SAFETY} = \frac{2390.2}{1179.6} = 2.03$$



SECTION A-A

1" = 50'

ZONE - 1 - RANDOM ROCKFILL — — γ_t = 135 PCF, ϕ = 32°
ZONE - 2 - CLAY CORE — — — — γ_t = 130 PCF, ϕ = 24°
ZONE - 3 - DRAINAGE MATERIAL — γ_t = 70 PCF, ϕ = 40°
ZONE - 4 - SANDSTONE — — — — γ_t = 135 PCF, ϕ = 32°
EXISTING DAM & FOUNDATION — γ_t = 130 PCF, ϕ = 31°

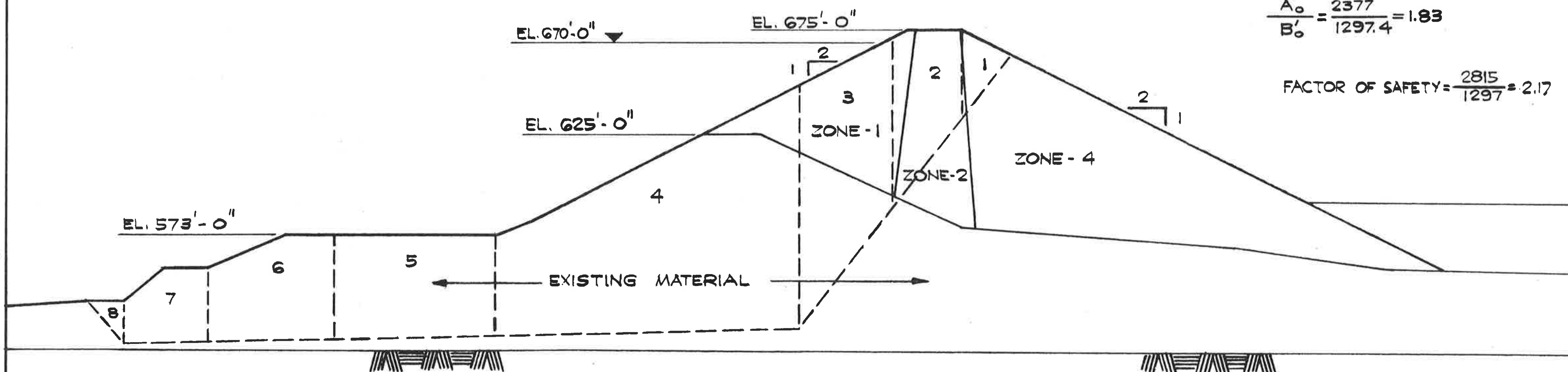
KENTUCKY POWER CO.
BIG SANDY PLANT
LOUISA KENTUCKY

EFFECTIVE STRESS
ANALYSIS
SECTION A-A

SLICE NO.	α	TANα	FT. ΔX	KSF P	ϕ	TANϕ	KSF U	K/FT. B_o	K/FT. A'_o	n_α	K/FT. A_o	K/FT. ΔE_o	K/FT E_o
1.	52°	1.28	24	2.73	32°	.62	0	83.9	41	.52	78.8	47.6	0 47.6
2	52°	1.28	36	8.78	31°	.60	0	404.6	189.7	.52	365.0	236.4	284.0
3	52°	1.28	47	13.33	31°	.60	5.01	801.9	234.6	.52	451.0	59.4	878.0
4	1°	.02	153.5	11.43	31°	.60	0	35	1052.7	1.00	1052.7	-450.1	427.9
5	1°	.02	80	7.25	31°	.60	0	11.6	348	1.00	348.0	-148.8	279.1
6	0	0	62	6.88	31°	.60	0	0	356	1.00	356.0	-164.0	115.1
7	0	0	42	5.20	31°	.60	0	0	131	1.00	131.0	-60.4	54.7
8	45°	1.00	18	2.20	31°	.60	0	-39.6	23.8	.74	32.2	-54.7	0
								Σ1297.4	Σ2377		Σ2815		

$$\frac{A'_o}{B'_o} = \frac{2377}{1297.4} = 1.83$$

$$\text{FACTOR OF SAFETY} = \frac{2815}{1297} = 2.17$$



ZONE - 1 - RANDOM ROCKFILL - γ_t = 135 PCF, ϕ = 32°
ZONE - 2 - CLAY CORE - γ_t = 130 PCF, ϕ = 31°
ZONE - 4 - SANDSTONE - γ_t = 135 PCF, ϕ = 32°
EXISTING MATERIAL - γ_t = 130 PCF, ϕ = 31°

SECTION A'-A'

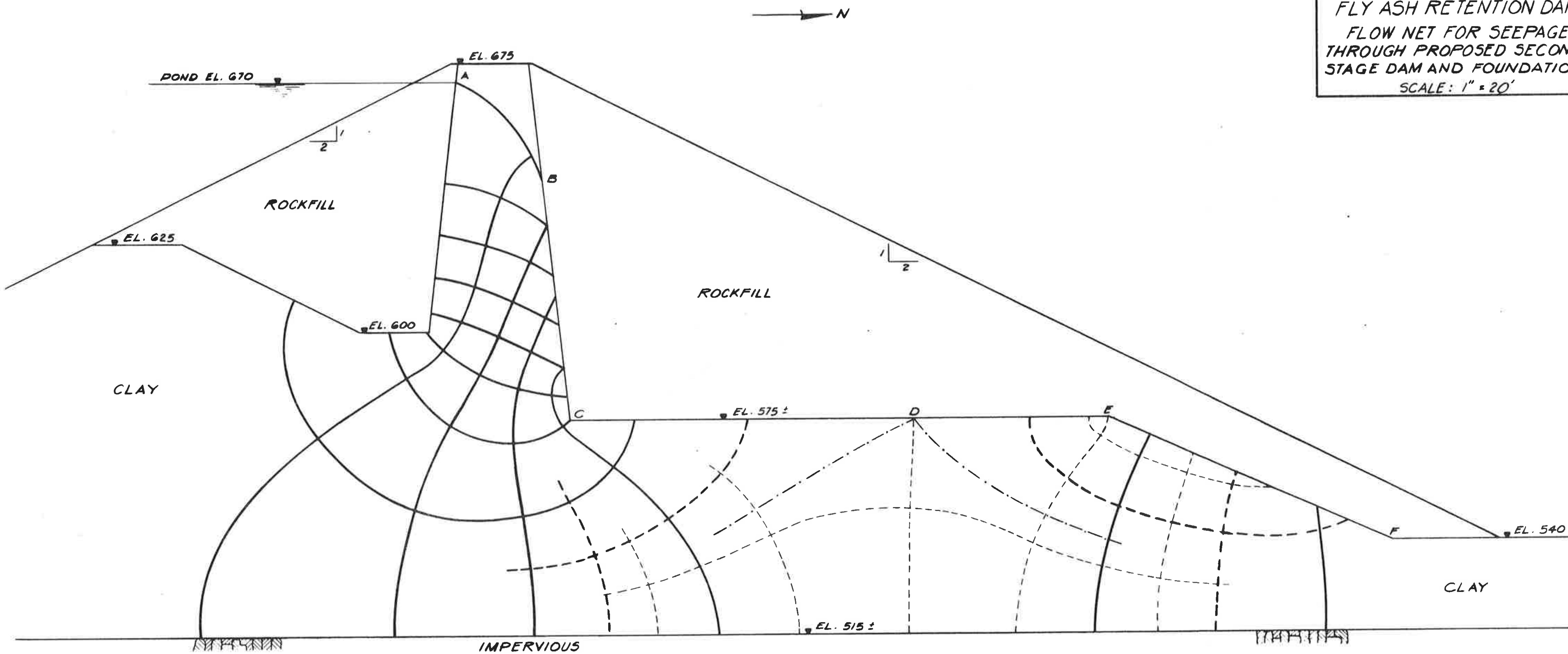
SCALE: 1" = 50'-0"

KENTUCKY POWER CO.
BIG SANDY PLANT
LOUISA KENTUCKY

UPSTREAM SLOPE
EFFECTIVE STRESS ANALYSIS
SECTION A'-A'

KENTUCKY POWER CO.
BIG SANDY PLANT
LOUISA KENTUCKY

FIG. 1
FLY ASH RETENTION DAM
FLOW NET FOR SEEPAGE THROUGH PROPOSED SECOND STAGE DAM AND FOUNDATION
SCALE: 1" = 20'



ATTACHMENT B-2

STAGE III Design Report

KENTUCKY POWER
BIG SANDY PLANT
FLY ASH RETENTION DAM
FINAL RAISING
DESIGN REPORT

PHILLIP J. SHEPHERD
SECRETARY

BRERETON C. JONES
GOVERNOR

COMMONWEALTH OF KENTUCKY
NATURAL RESOURCES AND ENVIRONMENTAL PROTECTION CABINET
DEPARTMENT FOR ENVIRONMENTAL PROTECTION
FRANKFORT OFFICE PARK
18 REILLY ROAD
FRANKFORT, KENTUCKY 40601

April 9, 1993

Russ Coburn
Kentucky Power Company
1701 Central Avenue
P. O. Box 1428
Ashland, Kentucky 41105-1428

Re: Modification to Horseford Creek Flyash Dam, Lawrence County.

Dear Mr. Coburn:

The Division of Water has reviewed the plans and application submitted for the above-referenced project and has approved those plans and application with respect to KRS 151.250.

The enclosed permit is issued from the standpoint of stream obstruction only and does not constitute certification of any other aspect of this project by the Commonwealth.

The permittee must notify this department in writing upon completion of this project.

Sincerely,

A. Leon Smothers

A. Leon Smothers, Manager
Water Resources Branch

ALS/WKC/dms

Enclosure

pc: Morehead Regional Office

RECEIVED
APR 20 1993

COMMONWEALTH OF KENTUCKY
NATURAL RESOURCES AND ENVIRONMENTAL PROTECTION CABINET
DEPARTMENT FOR ENVIRONMENTAL PROTECTION
DIVISION OF WATER
FRANKFORT, KENTUCKY 40601

Natural Resources and Environmental Protection Cabinet

Permit No. 3685(REVISED)
Expires if work is not begun by April 9, 1994

STREAM CONSTRUCTION PERMIT
For Construction In Or Along A Stream

Issued to: Kentucky Power Company

Address: P. O. Box 400
(Street)
Louisa — Kentucky — 41230
(City) (State) (Zip Code)

In accordance with KRS 151.250 and KRS 151.260, the Natural Resources and Environmental Protection Cabinet approves the application dated March 16, 1993 **, for** the modification of Horseford Creek Flyash Dam, including raising the crest to elevation 711' MSL and construction of a new saddle dam and emergency spillway, located at N 38 11 11, W 82 37 56, Lawrence County.

There shall be no deviation from the plans and specifications submitted and hereby approved unless the proposed change shall first have been submitted to and approved in writing by the Cabinet. This approval is subject to the following limitations.

(1) Upon completion of construction of this project, the permittee must notify this Cabinet in writing that the project has been completed.

2. This permit is issued from the standpoint of stream obstruction only and does not constitute certification of any other aspect of the proposed construction. The applicant is liable for any damage resulting from the construction, operation, or maintenance of this project. This permit has been issued under the provisions of KRS Chapter 151.250 and regulations promulgated pursuant thereto. Issuance of this permit does not relieve the permittee from the responsibility of obtaining any other permits or licenses required by this Cabinet and other state, federal and local agencies.
3. A copy of this permit must be posted at the construction site.

"SEE ADDITIONAL"
LIMITATIONS ON
REVERSE SIDE

This permit is nontransferable and is not valid unless actual construction of this authorized work is begun prior to the expiration date noted above. Any violation of the Water Resources Act of 1966 as amended is subject to penalties as set forth in KRS 151.990.

Issued this 9th **day of** April **, 19** 93 .

pc: Harold Jacobsen (KY #16578)
Engineer: Morehead Regional Office

By ____________
Division of Water

DEP7026

ADDITIONAL LIMITATIONS

4. Any design changes or amendments to the approved plans must be submitted to the Division of Water and approved in writing prior to implementation.
5. The design engineer will supervise and inspect construction and upon completion certify that the work is in accordance with approved plans and specifications.
6. The permittee shall notify the Dam Safety and Floodplain Compliance Section upon commencement of work on this project.

COMMONWEALTH OF KENTUCKY

NATURAL RESOURCES AND ENVIRONMENTAL PROTECTION CABINET

DEPARTMENT FOR ENVIRONMENTAL PROTECTION

DIVISION OF WATER

WATER RESOURCES BRANCH

18 Reilly Rd.
Frankfort, KY 40601

502/564-3410

FINAL CONSTRUCTION REPORT

PROJECT NAME: ____________________ PERMIT #: __________ DATE: __________

APPLICANT NAME:	AGENT NAME:
CORPORATE NAME:	
ADDRESS:	ADDRESS:
CITY | STATE | ZIP CODE	CITY | STATE | ZIP CODE
TELEPHONE NUMBER:	TELEPHONE NUMBER:
COUNTY:	LOCATION DESCRIPTION:
TOWN:	

Has all work on this project been completed according to plans and specifications on file with the Division of Water? ☐ Yes ☐ No If not, explain:

____

____

____

____

____

____

____

____

____

____

____

____

SIGNATURE: ____________________

WATER RESOURCES BRANCH
DIVISION OF WATER
FRANKFORT, KENTUCKY

MONTHLY CONSTRUCTION REPORT

Permit No. ____________________

Report No. ____________________

Date ____________________

Permit Holder: ____

Address: ____

Project Site: ____ (County)

Estimated percent of total work completed: ______________________________

Date construction started: ____

Estimated date of completion: ______________________________________

Is construction proceeding as scheduled in the application?

Yes______ No______. If not, why? ____

____

____

Has all work been completed in accordance with plans and specifications approved by this Division? Yes______ No______. If Not, describe the changes.______________

____

____

Briefly describe work completed to this date: ______________________________

____

____

____

I do, hereby, certify that the information given above is true and correct.

______________________________ (Date) ______________________________ (Signature)

AEPBSP-001247

PROJECT OVERVIEW

Kentucky Power Company (KPCo) has been slurrying fly ash produced at Big Sandy Electric Generating Plant into Horseford Creek fly ash pond since 1970. The estimated life of the fly ash pond, with the present dam crest at Elevation 675, is until the year 2000. It is anticipated that Unit 1 and 2 will be retired by January 1, 2024 and January 1, 2030, respectively. Therefore, it is necessary to increase the ash storage life of the pond until at least the year 2030.

Kentucky Power Company proposes to increase the available ash storage volume by raising the dam height to a crest elevation of 711 feet. The increased height will provide an ash storage capacity of approximately 12.9×10^6 cubic yards. The plan and sections for the proposed raising of the main dam and the construction of the saddle dam are presented in the accompanying set of design drawings. A complete listing of drawings is included in the drawing's cover sheet. The AEPSC Civil Engineering Division Technical Specifications for material and construction are attached under separate cover. The geotechnical aspects of the key components of the project are discussed below. This information is presented, in support of KPCo's application for an amendment to permit No. 3685 issued on September 20, 1988.

Main Dam

The extension of the core zone for the raising will be constructed from clays of low to medium plasticity obtained from the proposed borrow area. The downstream shell will be constructed using bottom ash. The faces of the dam will be lined with riprap to minimize erosion. The raising of the dam will be constructed at a maximum rate of 15 feet per year. Furthermore, a maximum of 5 feet of material may be placed in a given construction period. A minimum of 3 months should elapse between consecutive construction periods.

Cohesive soil will be placed to a final configuration of 2-3/4H:1V. Compaction will be accomplished at moisture contents ranging from -1% to +2% of the optimum moisture content. The soil shall be compacted in uniform, thin lifts to a unit dry weight equal to at least 95% of the maximum unit dry weight as determined in the laboratory by means of the Standard Proctor Compaction test. Bottom ash from Big Sandy Plant will continue to be used in the chimney drain and downstream shell to its final configuration of 1-3/4H:1V. This material will be compacted in uniform, thin lifts to at least 70% relative density determined by ASTM D 4253-83 and ASTM D4254-83. No moisture control will be required when compacting bottom ash. Results of the engineering and stability analyses indicated that the dam will perform within factors of safety commonly accepted for facilities of this type.

At locations where new construction meets the abutments or existing permanent features of the site, the new materials will be benched into the existing grade to allow placement in horizontal lifts and compaction control. Any feature of the site which may infringe into areas of proposed construction will be relocated as indicated on the drawings.

Saddle Dam

The impervious zone of the saddle dam will be constructed from clays of low to medium plasticity obtained from the proposed area. The downstream shell will be constructed using bottom ash. The faces of the dam will be lined with riprap. Construction of the saddle dam will require the filling of the existing emergency spillway which consists of a rock channel.

Cohesive soil will be placed to a final configuration of 2 3/4H:IV on the upstream slope. Compaction will be achieved at a moisture content ranging from -1% to +2% of the optimum moisture content. The soil shall be compacted in uniform thin lifts at a unit dry weight equal to at least 95% of the maximum unit dry weight as determined in the laboratory by means of the Standard Proctor Compaction tests. Bottom ash from Big Sandy Plant will be used in the chimney drain and downstream shell to its final configuration of 1-3/4H:1V. This material will be compacted in uniform thin lifts to at least 70% relative density as determined by ASTM D 4253-83 and ASTM D 4254-83. No moisture control will be required when compacting bottom ash. Results of the engineering and stability analyses indicate that the saddle dam will perform within factors of safety commonly accepted.

The existing emergency spillway will be filled with a fly ash/bottom ash stabilized mix, which is elastically compatible with the surrounding rock. This material will be placed to support the clay core on the area of the existing channel. The remaining of the channel will be filled with bottom ash as the downstream shell is constructed.

At locations where new construction meet the abutments, the new materials will be benched into the existing grade to allow placement in horizontal lifts and compaction control. Any feature of the site which may infringe into areas of proposed construction will be relocated as indicated on the drawings.

Emergency Spillway

The proposed emergency spillway will be constructed on the left abutment of the proposed saddle dam a few feet south of the existing emergency spillway. The spillway will consist of a rock cut channel approximately 100 foot wide and having a bottom graded at 2% slope. The walls of the rock cut will be graded at a slope equal to 1H:6V.

Rock excavated from the construction of the emergency spillway may be used as riprap provided it meets or exceeds commonly accepted criteria.

Mr. J. W. Marchant
Division of Water
Flood Plain Management Section
18 Reilly Road
Frankfort, Kentucky 40601

April 2, 1993

Dear Mr. Marchant:

Re: Big Sandy Plant Fly Ash Dam Inspections

In reference to our recent discussion with you about Big Sandy Plant fly ash dam inspections, please find enclosed the 1992 inspection report which was prepared by Woodward-Clyde Consultants. Notice that the next inspection is scheduled for May 1993.

If you have any questions concerning the enclosed information or the next scheduled inspection, please call (606) 327-1279.

Sincerely,

Russ Coburn
Environmental Affairs Director

RC/skd

Attachment

bcc: Pedro Amaya

BIG SANDY PLANT
HORSEFORD CREEK FLY ASH DAM

HYDROLOGY

General

The watershed of the Horseford Creek Dam is depicted on Fig. 6.1. The major drainage feature, Horseford Creek, drains into Blaine Creek which flows into the Big Sandy River at river mile 19.6.

Presently, the crest of the existing dam is at el. 678±. Discharge is regulated by the intake tower of the principal spillway which is connected to a 30-inch diameter reinforced concrete pipe. The concrete pipe terminates into a riprap-lined channel. The intake tower consists of twin shafts with side openings for decanting effluent. Concrete stoplogs are placed in the side openings as necessary to maintain settling action of the fly ash and act as an overflow weir to establish operating levels. During most operating conditions, discharge through the principal spillway will be controlled by weir flow over both stoplogs. The current emergency spillway consists of a 50-foot wide open channel excavated in rock on the left abutment of the saddle dam.

The proposed raising of the main dam will necessitate filling in the existing emergency spillway, raising of the saddle dam, and constructing a new emergency spillway.

The following sections present the hydrologic considerations and analyses performed during the design phase of this project.

Basin Characteristics

Fig. 6.1 depicts the limits of the watershed boundary for the fly ash dam. A review of available topographic maps and aerial photographs was made to determine essential basin characteristics. Such characteristics include the drainage boundaries, areas, slopes, soil types, land use and time of concentration. The time of concentration is defined as the elapsed time for runoff to travel from the hydraulically most distant part of the watershed to some reference point downstream.

Present land use within the watershed is limited to the fly ash pond proper and the adjacent wooded hillslopes. Raising of the dam will only alter the total acreage for both land uses.

A detailed soil survey for Lawrence County has not been published. Previous reports on the Phase 1 and 2 designs classified the soils such that they fall under the hydrologic soil group C as defined by the Soil Conservation Service

(SCS) of the U.S. Department of Agriculture. The table below lists the basin characteristics used for this project.

Drainage Area	675 acres
Average Land Slope	28%
Hydrologic Soil Group	C
SCS Curve Number (weighted)	73
Time of Concentration	0.25 hour

Based on Drawing No. 12-30030, the fly ash pond will have the following surface areas and storage capacities above elevation 650 feet.

Elevation	Area (ac)	Storage (ac-ft)
650	87	0
660	103	945
680	145	3406
700	169	6540
710	184	8302

Fig. 6.2 illustrates the stage-area-volume relationship for the fly ash pond.

Assumptions and Design Requirements

No rainfall-runoff data were available for the site. Therefore, runoff hydrographs were generated using the SCS method as described in the National Engineering Handbook, Section 4 - Hydrology. Since the raised pool levels will inundate a significant portion of the watershed, the hydrograph analysis for the emergency spillway was divided in two subwatersheds - the pond area and the remaining drainage area. The hydrograph for the pond area was developed by converting the precipitation hyetograph into an inflow hydrograph since there is 100 percent instantaneous runoff. The inflow hydrograph for the remaining area was determined by the referenced method. These two hydrographs were combined to define the design flood hydrographs. The general design requirements for the principal and emergency spillways have been established by the Kentucky Department for Environmental Protection, Division of Water.

Principal Spillway

According to Division of Water, Engineering Memorandum No. 5, design of the principal (service) spillway for Class C dams must be such that the average frequency of use of the emergency spillway cut in rock is predicted to be less than once in fifty years. To meet this criteria, the 10-day principal spillway hydrograph (PSH) must be developed based on the procedures in the SCS National Engineering Handbook. The principal spillway shall also have the capacity to release 80% of the maximum flood waters retained in the reservoir within ten days.

It was assumed that a 50-year rainfall event would produce the 50-year flood. Antecedent soil moisture condition II was assumed for the hydrograph development. Precipitation values were obtained from the Division of Water, Engineering Memorandum No. 2 (rev. 1979). For the project site located in Lawrence County, the rainfall values are 5.1 and 8.9 inches for the 50-year, 1 day and 10 day storms, respectively.

Emergency Spillway

The emergency spillway shall pass the design emergency spillway hydrograph (ESH) and freeboard hydrograph (FH) without overtopping the dam. For Class C dams, the minimum hydrologic criteria are:

ESH design rainfall, P = P100 + 0.26 (PMP-P100)
FH design rainfall, P = PMP

For this project, the design rainfalls have a duration of 6 hours. The PMP, probable maximum precipitation, is defined as the greatest depth of precipitation for a given duration that is meteorologically possible for a given basin at a particular time of year. The probable maximum flood (PMF) is the result of the PMP. For the project site, the rainfall values are 4.3 and 28.1 inches for the 100-year P100 and PMP storms, respectively. Antecedent moisture condition II was assumed for both storms.

Hydrologic Analysis

Most rainfall - runoff computations and all reservoir flood routings were conducted using the U.S. Army Corps of Engineers HEC-1 computer program. Basin characteristics, stage-storage, and discharge rating curves are input data supplied by the user.

Principal Spillway

Daily discharges from the reservoir will continue to be regulated by the existing principal spillway structure with no modifications. The principal spillway hydrograph (PSH) was manually calculated following the procedures in the SCS Handbook. Ash sluice waters (estimated to be 10 cfs) were added to the PSH to develop a total inflow hydrograph. Flood routings were conducted to establish a maximum operating pool level and the invert elevation of the emergency spillway.

Emergency Spillway

Flood routings of the freeboard hydrograph were conducted to determine the size of the emergency spillway necessary to pass the probable maximum flood without overtopping the dam. The initial water surface in the reservoir was assumed to be at the maximum operating level at the beginning of the storm. Flood routings were conducted for two situations (a) the principal spillway discharging and (b) neglecting those discharges.

Based on the principal spillway hydrograph flood routings, the 10-day drawdown level was at the maximum operating level. Therefore, the freeboard hydrograph was the controlling flood to set the emergency spillway dimensions and minimum top of dam.

In addition to the 6-hour PMF, a 24-hour PMF was developed and routed through the facility. This was done to verify the pond's capacity to handle a larger volume storm over a longer duration. The 24-hour PMP has a value of 35.5 inches.

Results

Principal Spillway

The development of the 50-year principal spillway hydrograph (PSH) indicates a peak inflow of 357 cfs which includes 10 cfs of ash sluice waters and 5 cfs of minimum quick return flow. The runoff volume over the 10 day duration is 610 acre-feet.

A maximum operating pool level has been set at elevation 705. This corresponds to a maximum stop-log elevation of 704. Final flood routings of the PSH started at an initial pool level equivalent to the maximum operating level. The reservoir rises to a maximum elevation of 706.0 feet. The peak outflow through the principal spillway is 54 cfs. Fig. 6.3 is a plot of the inflow and outflow principal spillway hydrographs. The receding flood pool is drained within eight days and six hours after the peak pool level is attained. Therefore, the requirements of Section E-I of Engineering Memorandum No. 5 are satisfied.

Emergency Spillway

The development of the 6-hour PMF hydrograph indicates a peak inflow of 10,610 cfs and a runoff volume of 1159 acre-feet.

The invert of the emergency spillway has been set at elevation 706.25 which is above the 50-year PSH flood elevation. The reservoir will rise to elevation 709.4 feet during passage of the PMF. This is based on an initial pool level of 705 feet and a 100 foot wide emergency spillway section. The flood routing was based on the principal spillway being plugged, a conservative condition. The peak outflow from the facility is 1672 cfs. Fig. 6.4 is a plot of the inflow and outflow emergency spillway hydrographs.

The 24-hour PMF generates 1564 acre-feet of runoff and has a peak inflow of 15,687 cfs. The pond will rise to elevation 710.5 feet and the peak outflow is 2433 cfs.

Summary and Conclusions

The 50-year, 10-day principal spillway hydrograph and the 6-hour probable maximum flood hydrograph were generated for the proposed raising of the Horseford Creek Dam. The SCS procedures for hydrologic design were followed with the use of the U.S. Army Corps of Engineers computer programs HEC-1 and HEC-2. Table No. 6.1 gives a summary of the study.

The existing principal system will be used without any modifications. It has the capacity to safely discharge the design flood without engaging the emergency spillway. The total spillway system (principal and emergency) has enough capacity to pass the probable maximum flood without overtopping the crest of the dam. There is sufficient freeboard for wind and wave height calculated to be 1.39 feet.

TABLE NO. 6.1
SUMMARY OF HYDROLOGIC/HYDRAULIC DATA

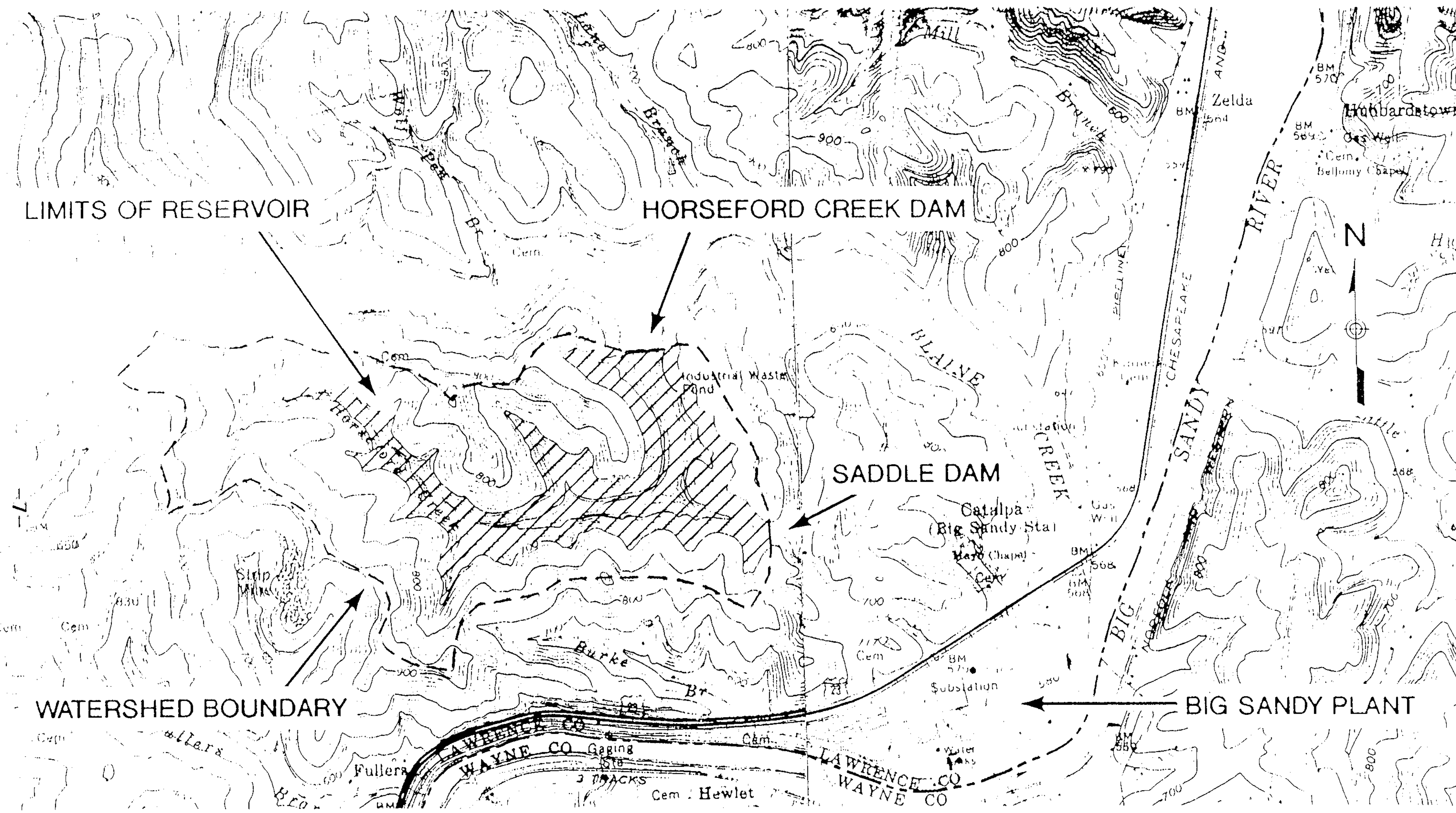
DRAINAGE AREA	0.9 SQ. MI.
DESIGN INFLOW FLOODS	
50-YR, 10-DAY PEAK	357 cfs
6-YR PMF PEAK	10,610 cfs
24 HR PMF	15,687 cfs
PEAK DISCHARGE	
50-YR, 10-DAY	54 cfs
6-HR PMF	1,672 cfs
24 HR PMF	2,433
MAXIMUM POOL ELEVATION	
50-YR, 10-DAY	706.0 FT
6-HR PMF	709.4 FT
24 HR PMF	710.5
DAM CREST ELEVATION	711.0 FT
MAXIMUM OPERATING POOL ELEVATION	705.0 FT

PRINCIPAL SPILLWAY:

CONCRETE RISER TOWER AND CONDUIT
SHAFT OPENING 2 @ 3.0' X 4.0'
30" DIAMETER REINFORCED CONCRETE PIPE

EMERGENCY SPILLWAY:

EXCAVATED CHANNEL IN ROCK	
CREST ELEVATION	706.25 FT
BOTTOM WIDTH	100 FT
SIDE SLOPES	1H:6V



EFERENCE: USGS QUADRANGLE MAPS
FALLSBURG, KY.-W.VA.
PRICHARD, W.VA.-KY.

SCALE:
0 2000 4000 FEET

Big Sandy Plant
Horseford Creek Fly Ash Dam
Watershed Map

FIGURE 6.1

Big Sandy Plant
Horseford Creek Fly Ash Dam
Stage-Area-Volume Curves

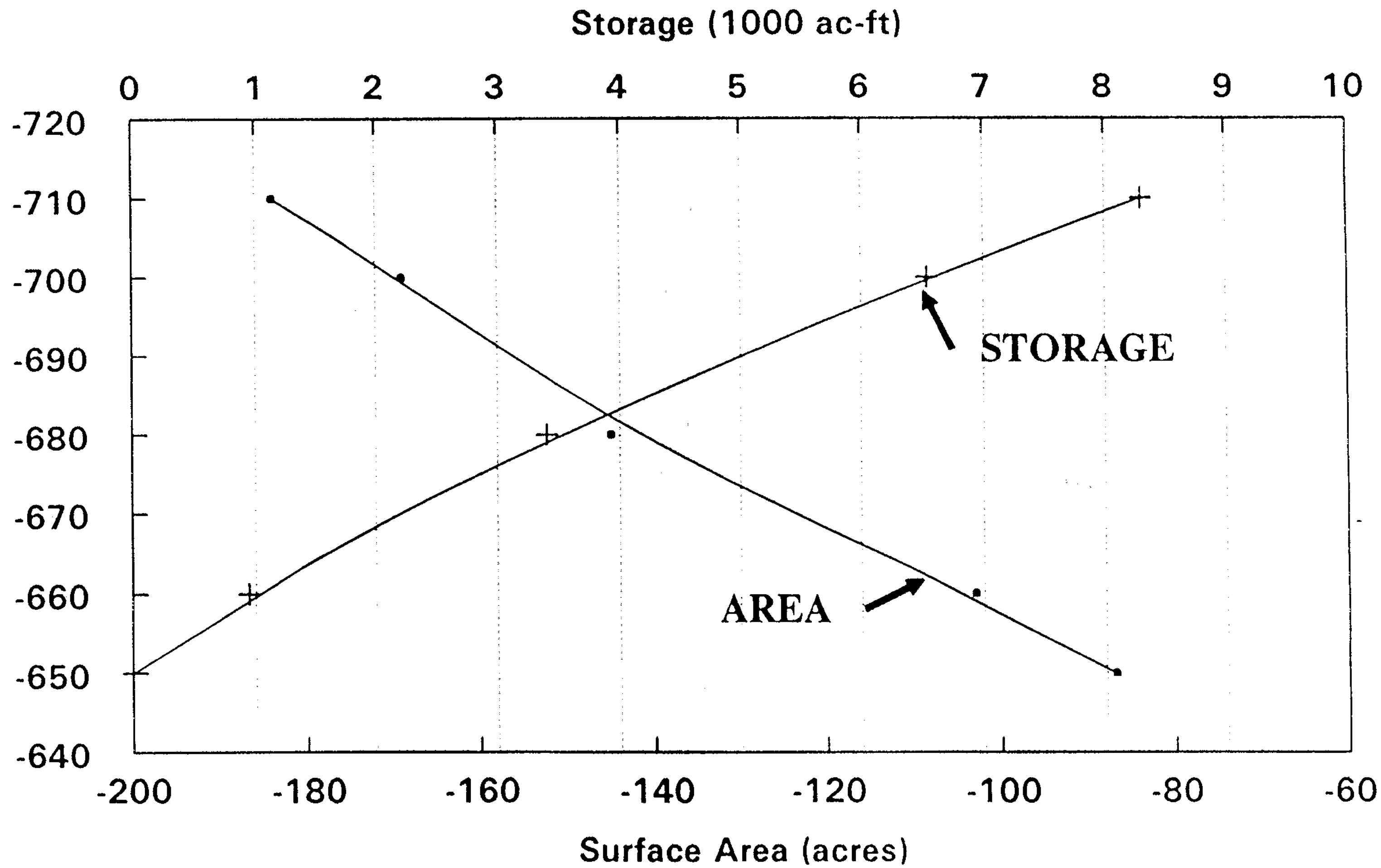


FIGURE 6.2

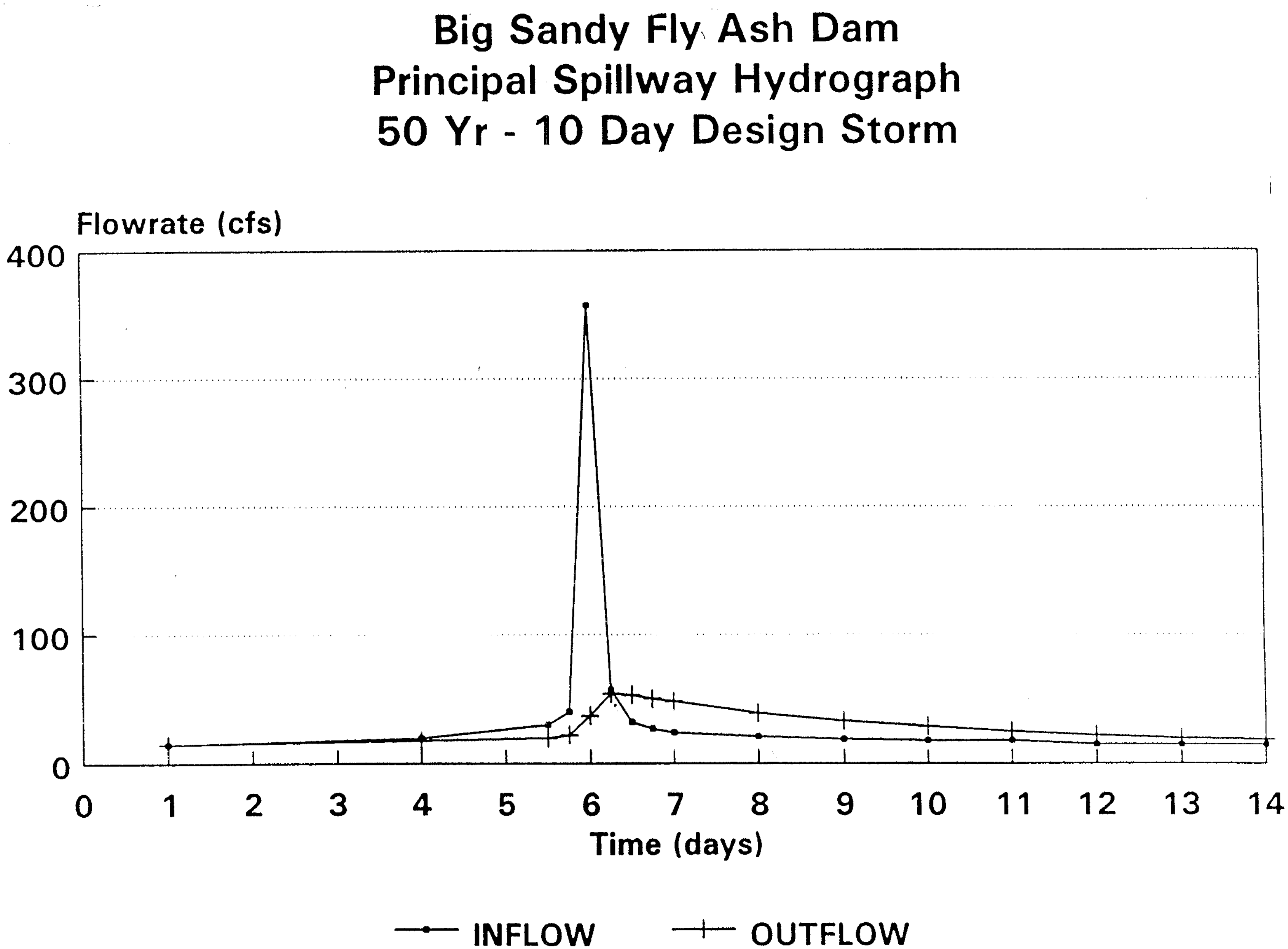

Big Sandy Fly Ash Dam
Principal Spillway Hydrograph
50 Yr - 10 Day Design Storm
Flowrate (cfs)
400
300
200
100
0
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
Time (days)
INFLOW
OUTFLOW

FIGURE 6.3

Big Sandy Fly Ash Dam
Emergency Spillway
6 Hr PMF Design Flood

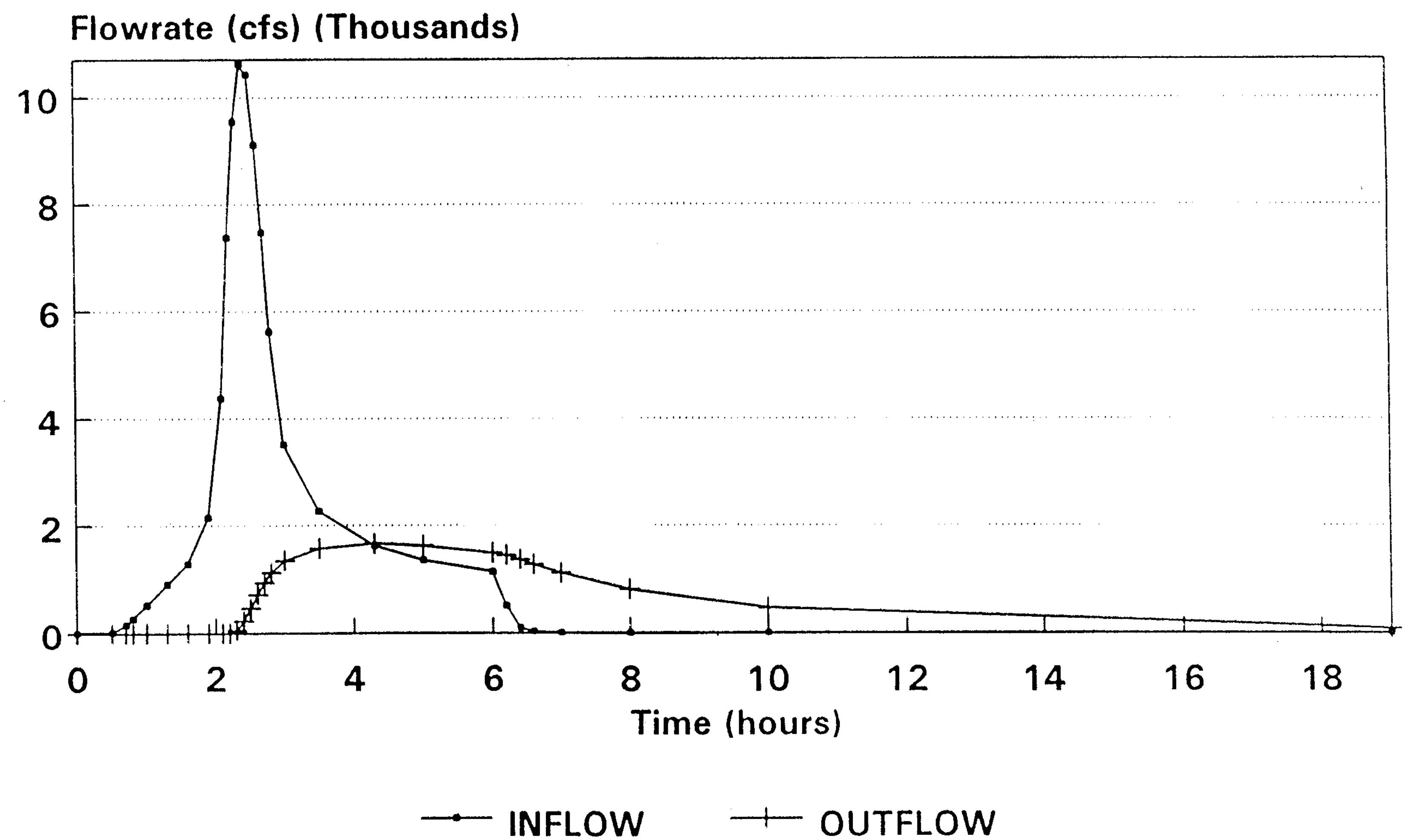


FIGURE 6.4

KENTUCKY POWER COMPANY
BIG SANDY PLANT
FLY ASH RETENTION DAM
STAGE 3 RAISING
ENGINEERING REPORT

Prepared by
American Electric Power Service Corporation
Civil Engineering Department
Geotechnical Section

March 1993

PROJECT OVERVIEW

Kentucky Power Company (KPCo) has been slurrying fly ash produced at Big Sandy Electric Generating Plant into Horseford Creek fly ash pond since 1970. The estimated life of the fly ash pond, with the present dam crest at Elevation 675, is until the year 2000. It is anticipated that Unit 1 and 2 will be retired by January 1, 2024 and January 1, 2030, respectively. Therefore, it is necessary to increase the ash storage life of the pond until at least the year 2030.

Kentucky Power Company proposes to increase the available ash storage volume by raising the dam height to a crest elevation of 711 feet. The increased height will provide an ash storage capacity of approximately 12.9×10^6 cubic yards. The plan and sections for the proposed raising of the main dam and the construction of the saddle dam are presented in the accompanying set of design drawings. A complete listing of drawings is included in the drawing's cover sheet. The AEPSC Civil Engineering Division Technical Specifications for material and construction are attached under separate cover. The geotechnical aspects of the key components of the project are discussed below. This information is presented, in support of KPCo's application for an amendment to permit No. 3685 issued on September 20, 1988.

Main Dam

The extension of the core zone for the raising will be constructed from clays of low to medium plasticity obtained from the proposed borrow area. The downstream shell will be constructed using bottom ash. The faces of the dam will be lined with riprap to minimize erosion. The raising of the dam will be constructed at a maximum rate of 15 feet per year. Furthermore, a maximum of 5 feet of material may be placed in a given construction period. A minimum of 3 months should elapse between consecutive construction periods.

Cohesive soil will be placed to a final configuration of 2-3/4H:1V. Compaction will be accomplished at moisture contents ranging from -1% to +2% of the optimum moisture content. The soil shall be compacted in uniform, thin lifts to a unit dry weight equal to at least 95% of the maximum unit dry weight as determined in the laboratory by means of the Standard Proctor Compaction test. Bottom ash from Big Sandy Plant will continue to be used in the chimney drain and downstream shell to its final configuration of 1-3/4H:1V. This material will be compacted in uniform, thin lifts to at least 70% relative density determined by ASTM D 4253-83 and ASTM D4254-83. No moisture control will be required when compacting bottom ash. Results of the engineering and stability analyses indicated that the dam will perform within factors of safety commonly accepted for facilities of this type.

At locations where new construction meets the abutments or existing permanent features of the site, the new materials will be benched into the existing grade to allow placement in horizontal lifts and compaction control. Any feature of the site which may infringe into areas of proposed construction will be relocated as indicated on the drawings.

Saddle Dam

The impervious zone of the saddle dam will be constructed from clays of low to medium plasticity obtained from the proposed area. The downstream shell will be constructed using bottom ash. The faces of the dam will be lined with riprap. Construction of the saddle dam will require the filling of the existing emergency spillway which consists of a rock channel.

Cohesive soil will be placed to a final configuration of 2 3/4H:IV on the upstream slope. Compaction will be achieved at a moisture content ranging from -1% to +2% of the optimum moisture content. The soil shall be compacted in uniform thin lifts at a unit dry weight equal to at least 95% of the maximum unit dry weight as determined in the laboratory by means of the Standard Proctor Compaction tests. Bottom ash from Big Sandy Plant will be used in the chimney drain and downstream shell to its final configuration of 1-3/4H:1V. This material will be compacted in uniform thin lifts to at least 70% relative density as determined by ASTM D 4253-83 and ASTM D 4254-83. No moisture control will be required when compacting bottom ash. Results of the engineering and stability analyses indicate that the saddle dam will perform within factors of safety commonly accepted.

The existing emergency spillway will be filled with a fly ash/bottom ash stabilized mix, which is elastically compatible with the surrounding rock. This material will be placed to support the clay core on the area of the existing channel. The remaining of the channel will be filled with bottom ash as the downstream shell is constructed.

At locations where new construction meet the abutments, the new materials will be benched into the existing grade to allow placement in horizontal lifts and compaction control. Any feature of the site which may infringe into areas of proposed construction will be relocated as indicated on the drawings.

Emergency Spillway

The proposed emergency spillway will be constructed on the left abutment of the proposed saddle dam a few feet south of the existing emergency spillway. The spillway will consist of a rock cut channel approximately 100 foot wide and having a bottom graded at 2% slope. The walls of the rock cut will be graded at a slope equal to 1H:6V.

Rock excavated from the construction of the emergency spillway may be used as riprap provided it meets or exceeds commonly accepted criteria.

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1.0 INTRODUCTION

Kentucky Power Company (KPCo) has been slurrying fly ash produced at Big Sandy Electric Generating Plant into Horseford Creek fly ash pond since 1970. The estimated life of the fly ash pond, with the present dam crest at elevation 675, is until the year 2000. It is anticipated that the plant Units 1 and 2 will be retired by January 1, 2024 and January 1, 2030, respectively. Therefore, it is necessary to increase the ash storage life of the pond until at least the year 2030.

Kentucky Power Company proposes to increase the available ash storage volume by raising the dam height to a crest elevation of 711 feet. The increased height will provide an ash storage capacity of approximately 12.9 x 10^6 cubic yards,(1). This engineering report and the accompanying set of drawings contain the engineering analyses and design of the proposed raising in support of KPCo's application for an amendment to permit No. 3685 issued on September 20, 1988.

1.1 Existing Conditions and Dam History

The Horseford Creek Dam (also known as the Big Sandy Dam) was constructed to create a pond for storing fly ash from the Big Sandy Plant. It is located just south of Blaine Creek near Louisa, Kentucky, approximately 1.3 miles from the plant. The topography of the pond area, before construction of the dam, is shown in Fig. A.1., in Appendix A to this report.

Construction of the dam was started late in 1968 and continued into early 1970. In January 1969, when the dam was 70 ft. high (crest el. 598±), a 500 ft. long central section of the embankment began spreading in both upstream and downstream directions. The dam was inspected and it was recommended that rockfill berms be constructed without delay on both the upstream and downstream sides of the dam. Additional borings indicate that the deformation possibly was caused by a layer of soft clay beneath the dam and/or by some embankment materials being placed too wet.

By the end of February 1969, the rate of deformation in the central section had decreased substantially and construction was resumed. The embankment was raised by only 9.5 ft. (el. 607.5) before construction was again stopped because of excessive deformations.

In mid-May, the rate of deformation again decreased and construction was resumed. In an effort to control the deformation the operation was modified so that the dam was raised at a rate of only 1.5 feet per week. However, in mid-June deformations started again and construction was halted. The deformations were no longer just limited to the central section but now included deformation in both the upstream and downstream berms.

By October 1969, the rate of deformation had subsided and it was possible to resume construction. However, prior to continuing construction of the dam, piezometers were installed in the embankment and foundation materials and both the upstream and downstream berms were enlarged. Construction of the dam was then continued at a very slow rate until the final design crest elevation of 625 was reached in mid-February 1970.

Settlements and horizontal deflections for the central upper portion of the embankment slopes during construction are as follows:

Settlement from Jan. 17 to Aug. 15, 1969:
DS Slope = 14 In.
US Slope = 16 In.

Horiz. Deflection from Jan. 30 to Oct. 6, 1969:
DS Slope = 22 In.
US Slope = 29 In.

Deformation records are not complete as the measurements were started after the initial deformations had occurred and other data were not maintained to the end of construction in January 1970. Thus, the total settlement during construction may have been as much as two feet and the maximum horizontal deflection, which was in upstream direction, may have been as much as three feet.

In April, 1976, the site investigation for the second stage dam raising was initiated by Casagrande Consultants. The American Electric Power Service Corporation developed the designs necessary to raise the crest elevation of the existing dam to elevation 675. During the second stage construction of the main embankment, an area near the west abutment of the original dam sloughed. The vegetation of the downstream slope was stripped immediately and a stabilizing berm (which later became part of the clay core) was constructed utilizing a 24-hour work schedule. The presence of this berm controlled the movement and no significant movement or excessive pore pressures have been recorded since that time. A discussion of instrumentation observation since the completion of the dam to its current elevation is presented in Section 2.0 "Performance Evaluation" of this report.

The dam construction was completed in 1979 without additional delays. A description of the features of the dam subsequent to the completion of the second stage is presented in the following paragraphs.

The crest of the embankment after the second stage raising is approximately 800 ft. long and 30 ft wide. The total height of the dam at the maximum section is approximately 135 ft., as measured from the crest of the embankment to the toe of the stabilizing berm. The main section of the embankment has an upstream slope of 1V:2H. The stabilizing berm on the upstream slope has a top width of about 70 ft., and a top elevation at approximate el. 635. The general slope of the downstream face is 1V:2H, but has been broken into several segments by an access road and the stabilizing berm. The stabilizing berm is approximately 190 ft. wide and its top elevation is el. 590. The dam was constructed with a clay core, a chimney drain and a drainage blanket beneath the downstream slope.

The outlet works consist of a reinforced concrete, drop-inlet type spillway located on the left abutment of the dam. The intake is an 85 ft. high, vertical, decanting riser structure with concrete stoplogs. The stoplogs provide a mechanisum to raise the overflow weir crest elevation concurrently with the rising (impounded) fly ash elevation. The riser consists of a double weir and operates using both weirs under all normal and flood conditions. In the unlikely event that a stoplog should break, a slide gate can be operated to terminate flow through either bay of the riser. The slide gate will prevent fly ash from being discharged without affecting the function of the other cell.

The water discharged over the stoplogs flows through a 30 in. diameter prestressed concrete cylinder pipe. The conduit terminates in a riprap lined plunge pool and stilling basin. A flow measurement device is located near the outlet of the 30 in. conduit.

The emergency spillway, located in a saddle near the southeastern portion of the reservoir, consists of a 50 ft. wide channel at el. 671.0. The invert of the emergency spillway channel is founded in rock and has been cut to a 2 percent slope. The rock lined channel below the emergency spillway has side slopes of 1V:0.5H.

1.2 Proposed Construction

The proposed construction consists of (1) raising the crest of the dam to an elevation of 711, and (2) constructing a new saddle dam and emergency spillway. The proposed raising of the main dam is to be constructed by widening the existing downstream shell of the dam over the existing berm and then adding a maximum of 15 feet of material per year across the entire dam until the proposed elevation is reached. A maximum of 5 feet of material may be placed in a given construction period. A minimum of 3 months should elapse between consecutive construction periods. Bottom ash will be used for the shell of the dam and clay will be used for the core material. The raising of the dam to a crest elevation 711 is expected to extend from 1993 until the year 2000.

The saddle dam and the new emergency spillway will be constructed beginning in 1998 and completed by the year 2000. The saddle dam will be constructed of bottom ash and be similar in cross section to the main dam; a new emergency spillway will be excavated in the rock of the left abutment of the saddle dam with crest elevation 706.

1.3 Hazard Classification

The hazard classification for the dam was evaluated in accordance with the guidelines set forth by the Natural Resources and Environmental Protection Cabinet, Department for Environmental Protection, Division of Water of the Commonwealth of Kentucky. Based upon review of the USGS quadrangle maps of the area, it appears that there are a few homes within 1 mile of the downstream toe. At a point about 1.5 miles downstream, the downstream channel (Blaine Creek) discharges into the Big Sandy River. At the intersection of Blaine Creek and the Big Sandy River, a highway, a power station and several additional homes are located at elevations well below the normal pool level of the impoundment. A failure of the dam causing a sudden and uncontrolled discharge of water would, therefore, result in loss of human life, damage to homes and a main highway, and contamination of a main waterway. Based on this information, the Horseford Creek Dam has been classified as a Class C Dam "High Hazard".

1.4 Normal Operating Procedure

Fly ash is pumped to the upper end of the reservoir. The concrete stoplogs in the riser structure are raised periodically to maintain approximately 10 feet of clear water above the fly ash level at the upstream slope of the dam. Raising of the water level is dictated by the influence of the fly ash in the amount of suspended solids in the water flowing through the service spillway.

2.0 PERFORMANCE EVALUATION

The instrumentation installed, during stage 2, to monitor the performance of the fly ash dam consisted of deformation monuments, piezometers, observation wells, and flow measurement weirs. The approximate locations of these instruments are shown in Fig. A.2 and A.3. in Appendix A. In 1990, three new pneumatic piezometer arrays (designated as PZ-4A, 4B and 4C; PZ-5A, 5B and 5C; and, PZ-6A, 6B and 6C) were installed along the downstream edge of the El. 590 berm. In addition, two inclinometer casings were installed in 1991, and 1992 to monitor movements of the slope during the placement of bottom ash fill for the proposed construction.

In addition, inspections of the existing dam are conducted routinely as follows:

1. Once a quarter by plant personnel.
2. At least once a year by a representative of the Civil Engineering Department of the American Electric Power Service Corporation.
3. Every two years by Woodward-Clyde Consultants, Inc.

Copies of recent reports prepared under each of the above inspection programs are included in Appendix A. A summary of the most recent observations is presented in the following sections.

2.1 Piezometers/Observation Wells

Summaries of the water level data obtained from the monitoring instruments through 1992 are presented as graphs in Fig. A.4 through A.8. Piezometer locations are presented on Fig. A-13 in Appendix A.

During the period of January 1989 and December 1992, the level of the pond rose by 6.5 ft. Except for the new instruments installed at the el. 590 berm, the peaks in piezometer readings P-124, P-127 and PR-21 probably are caused by high tailwater due to the flood stage in Blaine Creek, Big Sandy and Ohio River. Similar peaks occurred during the Winters of 84, 86, 88 and 90. Discounting several isolated anomalous readings, the remaining instruments showed little variation during this reporting period.

Measurements of water levels in the new piezometers installed at the el. 590 berm were initiated on January 31, 1990. In general, it is probably that the water levels are controlled by tailwater levels instead of reservoir levels.

2.2 Deformation Monuments

Eleven deformation monuments were originally installed in 1978 on the surface of the Stage 2 dam to permit monitoring of post-construction vertical and horizontal deformations. As shown in Figure A.2 and A.3 (Appendix A), five monuments were installed on the crest (denoted as SM-1, SM-3, SM-4, SM-6 and SM-7); four on the downstream slope (SM-4-3, SM-6-5, SM-4-1 and SM-6-2) and two at the downstream toe (SM-6-6 and SM-10). As a result of the new construction to raise the dam, monuments SM-7, SM-7-3, and SM-6-5 were reported to be destroyed as of December 5, 1990. Monument SM-6 was destroyed back in 1986. Three surveys have been performed in the last four years since May of 1988. The last survey was completed on April 2, 1991.

a. Vertical Movements

A summary of the settlement (or heave) measured at each of the monuments during the period from March 15, 1985 to April 2, 1991 is given in Table A.1 (Appendix A). Except for monument SM-3, the average rate of settlement (less than about 0.2 in. per year) of the crest monuments during each of the last two 3-year periods has not changed significantly. The maximum settlement since 1978 (8.16 in. at SM-3) is equivalent to about 0.53 percent of the current structural height (135-ft.) of the dam.

Monument SM-4-1 on the el. 590 berm underwent approximately 0.3 in. of additional settlement during the last 3-year period. The remaining monuments continued to show heave at a nearly constant rate. Some of these movements are likely due to the effects of the construction activities started in the spring of 1989.

b. Horizontal Movements

A summary of the approximate horizontal movements during each of the last two 3-year periods is given in Table A.2 (Appendix A). In general, the movements are in the downstream direction, trending toward the center of the valley.

2.3 Flow Weir

The Plant installed a V-notch weir to measure the seepage from the toe area at the right side of the dam. Flow measurements are obtained periodically by Plant personnel. A summary of the available flow data during the period January 5, 1989 to March 17, 1992 is given in Table A.3 (Appendix A). Also shown for comparison purposes are several readings obtained during the preceding four years. Based on these data it appears that the flow through the weir has diminished over the last seven years, despite an increase in pond elevation of nearly 8-ft. Either the seepage out of the toe of the dam has actually been reduced, or more of it is bypassing the channel leading to the weir.

2.4 Slope Inclinometer

Two slope inclinometer casings were installed at the el. 590 berm in June of 1991 and at elevation of 610.6, to monitor lateral movements of the embankment during placement of new fill for raising the dam. The bottom of the casings were set at approximately el. 495. This is about 5-ft. below bedrock. Locations of inclinometers are presented on Fig. A-13 in Appendix A.

Periodic measurements have been made since the installation. Fig. A.11 and A.12 (Appendix A) depict the data recorded with these instruments. Data from these instruments indicate that the horizontal movements have been less than 0.2 in. For comparison, a best fit curve through the Survey data collected indicates a maximum horizontal movement of 0.2 ft. in 5 years or 0.48 inches per year.

2.5 Conclusions

Available data from surveys of the deformation monuments indicate that the magnitude of settlement along the crest is still within the range normally expected for dams of this height. The maximum settlement as of April 2, 1991 for monument SM-3 near the maximum section of the dam was 8.16 in., or about 0.53 percent of the height. The rate of settlement between 1988 and 1991 was somewhat lower than that for the previous three-year period.

Based on the survey data as of April 2, 1991, an increase in the horizontal movement in the downstream direction has occurred. However, data from the inclinometer installed in the area of the berm subsequent to the last survey does not indicate any significant movement either at the surface or at depth. On this basis, it is concluded that the dam is performing satisfactorily.

3.0 FIELD INVESTIGATION

A detailed description of the field investigations and performance evaluation for the first two dam stages (stages 1 and 2 with corresponding crest elevations of 625 and 675 feet) are presented in the Casagrande December 1976 report(2).

During the period of July 1989 through February 1990, a total of sixteen, 16, borings were drilled to investigate the subsurface conditions at both the main dam and the saddle dam. In addition, seven, 7, test trenches were excavated at the clay borrow area for the proposed raising. Drawing Nos. 12-30030, 12-30031 and 12-30032 depict boring and test trench location. The borings were advanced using a 6-1/2 inch diameter hollow stem auger from ground surface to depths in the range of 6.2 to 166.1 ft. All borings were advanced to final depths in the range of 15 to 210 feet using an NX-diamond rock bit and NXM double-tube core barrel sampler, with circulating water used to cool the bit and flush the hole of cuttings. Disturbed, but representative, soil samples were obtained by driving a 2-inch O.D. split barrel sampler in accordance with ASTM D 1586. Undisturbed soil samples were procured by hydraulically pushing a 3-inch diameter Shelby tube at a constant rate of penetration. Disturbed samples were examined immediately after recovery and representative portions were preserved in airtight glass jars. Undisturbed samples were preserved in the tubes by cleaning the ends of cuttings and sealing the ends of the tubes with wax. Rock core was preserved in carefully marked and identified compartmented boxes.

In addition, during August 4 through 12, 1992, Boring SI-2 was drilled to obtain additional undisturbed samples and to install a second slope indicator. These undisturbed samples were procured by hydraulically pushing a 4-1/2 inch diameter shelby tube at a constant rate of penetration.

All borings and trenches were either grouted or backfilled at the completion of the site investigation. All samples were transported to an independent laboratory for further identification and testing.

3.1 Regional Geology

This dam is located in the Kentucky Physiographic Region known as the Eastern Coal Field, which is typified by narrow valleys and high relief.

The soils present at the dam are primarily residual and colluvial in nature with minor amounts of alluvium found in the valley bottom.

The bedrock present beneath the embankment and in the abutments is the Pennsylvanian Age Breathitt Formation which consists primarily of sandstone, siltstone and shale with lesser amounts of underclay and coal. The Princess No. 7 Coal outcrops in this valley at about el. 600 and at one time was mined (as indicated by a caved mine adit) about 500 ft. south of the dam. The bedrock units dip, in general, to the north at about 40 ft. per mile into the Appalachian Basin, the regional structural feature(3).

According to the Seismic Zone Map, this dam is in Zone 1 which indicates that minor damage would result from the expected seismic activity in this area.

3.2 Site Geology

A geologic cross section of the site of the main dam was obtained during the inspection of the clearing of the east abutment, performed on June 12, 1989. The following paragraphs describe the work that was performed on the east abutment and the abutment's appearance and condition.

The area which was cleaned for the dam abutment extends from the existing dam's east abutment, approximately 175 feet north along the existing hillside. The height of the cleaning extends from the edge of the existing haul road up the slope of the hillside approximately 125 feet to elevation 614.0, Fig. 3.1.

Between 3.0 and 4.5 feet of material was removed from the area to expose the rock. The surface of the cleaned slope is primarily a siltstone/clayshale material. Two sandstone units and a thin coal seam were also located, Fig. 3.1 and 3.2.

A measurable flow of water was located below the groin of the existing dam. The water appeared to be flowing along the top of a sandstone unit at elevation 600. The seep was estimated to be flowing at one gallon per 6 minutes or 1.6×10^{-1} GPM. The location of the seep is indicated on Fig. 3.2 and labeled as S-1.

3.3 Water Pressure Tests

In order to assess the potential for seepage through the abutments and underlying rock formation, double packer water pressure tests were performed at 10 foot intervals in several of the borings. The testing procedure used closely follows the procedure outlined in Chapter 10 of the groundwater manual published by the U.S. Department of Interior. Results of these tests are presented in the Appendix C to this report.

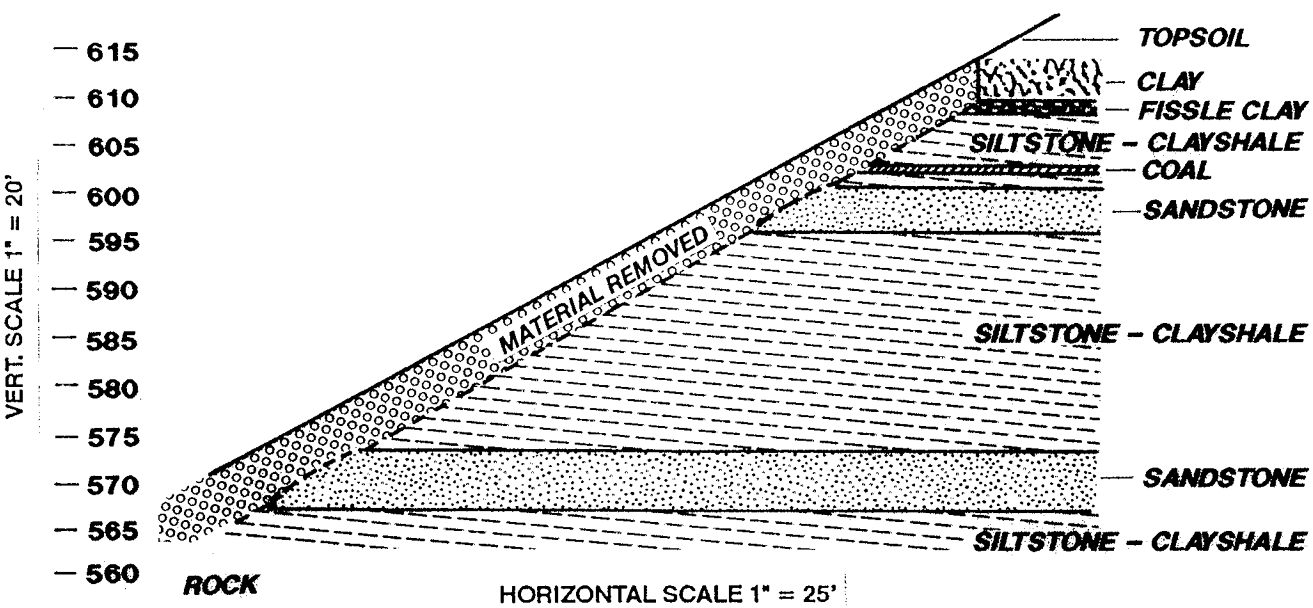
Rock permeabilities are summarized in Table No. 3.1

TABLE NO. 3.1
SUMMARY OF ROCK PERMEABILITIES

Boring No.	Depth	Permeability K
BSFD-1	94.6'-150.0'	$2.5x10^{-4}$-$2.2x10^{-6}$ cm/sc
BSFD-3A	106.0'-165.0'	$2.1x10^{-4}$-$2.5x10^{-6}$ cm/sc
BSFD-7	9.4'-165.0'	$1.1x10^{-3}$-$3.0x10^{-7}$ cm/sc
BSFD-8	8.2'-185.3'	$4.6x10^{-4}$-$1.6x10^{-6}$ cm/sc
BSFD-8A	8.5'-45.9'	$4.9x10^{-4}$-$2.3x10^{-4}$ cm/sc

No apparent correlation between lithology and permeability can be detected from these results. In general, however, permeabilities tend to decrease with depth ranging from 10^{-3} to 10^{-7} cm/sc. This trend is believed to be the result of a decreasing number of open joints and junctures with the increasing depth of overburden.

CROSS SECTIONAL VIEW OF EXCAVATION AND GEOLOGY



GEOLOGICAL LOG OF ABUTMENT:

615 -614	*TOPSOIL*, ORGANIC, VEGATATION, ROOFS, SOME CLAY
614 -609	(HIGHWALL) *CLAY*, TAN, LT BROWN, SOME SAND, SOME SANDSTONE FRAGMENTS, MOIST
609 -608	*CLAY*, SLIGHTLY FISSLE, SLIGHTLY WEATHERED, TAN, LT BROWN, GRAY, SOME SAND
608 -603	*SILTSTONE - CLAYSHALE* SHALEY, GRAY, LT BROWN, TAN, SOME IRON, PRECIPITATOR
603 -602.2	*COAL*, BLACK, SOFT
602.2 -600	*SILTSTONE - CLAYSHALE*, SHALEY, FISSLE GRAY, TAN, LT BROWN
600 -596	*SANDSTONE*, TAN, LT BROWN, HARD, MEDIUM GRAINED
596 -573	*SILTSTONE - CLAYSHALE*, SHALEY, FISSLE GRAY, LT BROWN, MODERATLY HARD
573 -567	*SANDSTONE*, TAN, LT BROWN, RED GIA
567	*SILTSTONE - CLAYSHALE*, TAN,LT BROWN, FISSLE

FIG 3.1 SITE GEOLOGY

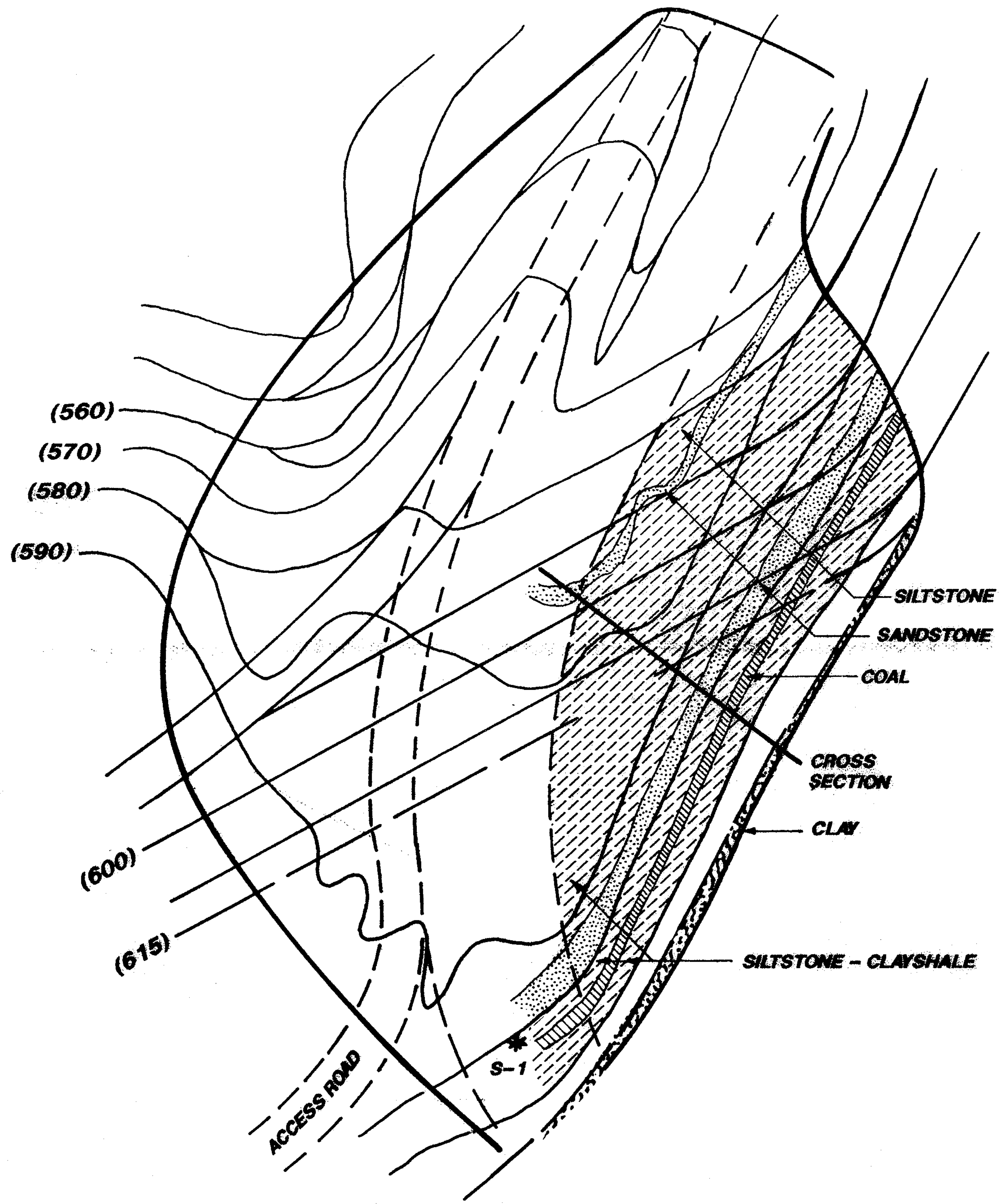
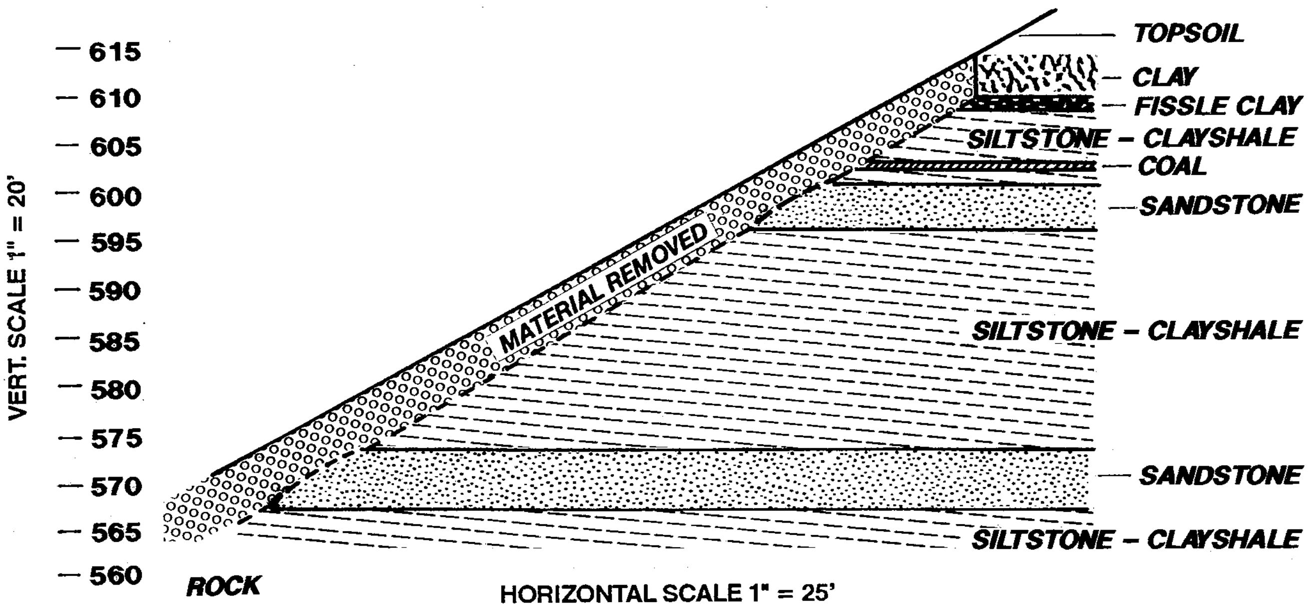


FIG. 3.2 PLANVIEW OF EXCAVATION AREA

CROSS SECTIONAL VIEW OF EXCAVATION AND GEOLOGY



GEOLOGICAL LOG OF ABUTMENT:		
	615 -614	*TOPSOIL*, ORGANIC, VEGATATION, ROOFS, SOME CLAY
	614 -609	(HIGHWALL) *CLAY*, TAN, LT BROWN, SOME SAND, SOME SANDSTONE FRAGMENTS, MOIST
	609 -608	*CLAY*, SLIGHTLY FISSLE, SLIGHTLY WEATHERED, TAN, LT BROWN, GRAY, SOME SAND
	608 -603	*SILTSTONE - CLAYSHALE* SHALEY, GRAY, LT BROWN, TAN, SOME IRON, PRECIPITATOR
	603 -602.2	*COAL*, BLACK, SOFT
	602.2 -600	*SILTSTONE - CLAYSHALE*, SHALEY, FISSLE GRAY, TAN, LT BROWN
	600 -596	*SANDSTONE*, TAN, LT BROWN, HARD, MEDIUM GRAINED
	596 -573	*SILTSTONE - CLAYSHALE*, SHALEY, FISSLE GRAY, LT BROWN, MODERATLY HARD
	573 -567	*SANDSTONE*, TAN, LT BROWN, RED GIA
	567	*SILTSTONE - CLAYSHALE*, TAN,LT BROWN, FISSLE

FIG 3.1 SITE GEOLOGY

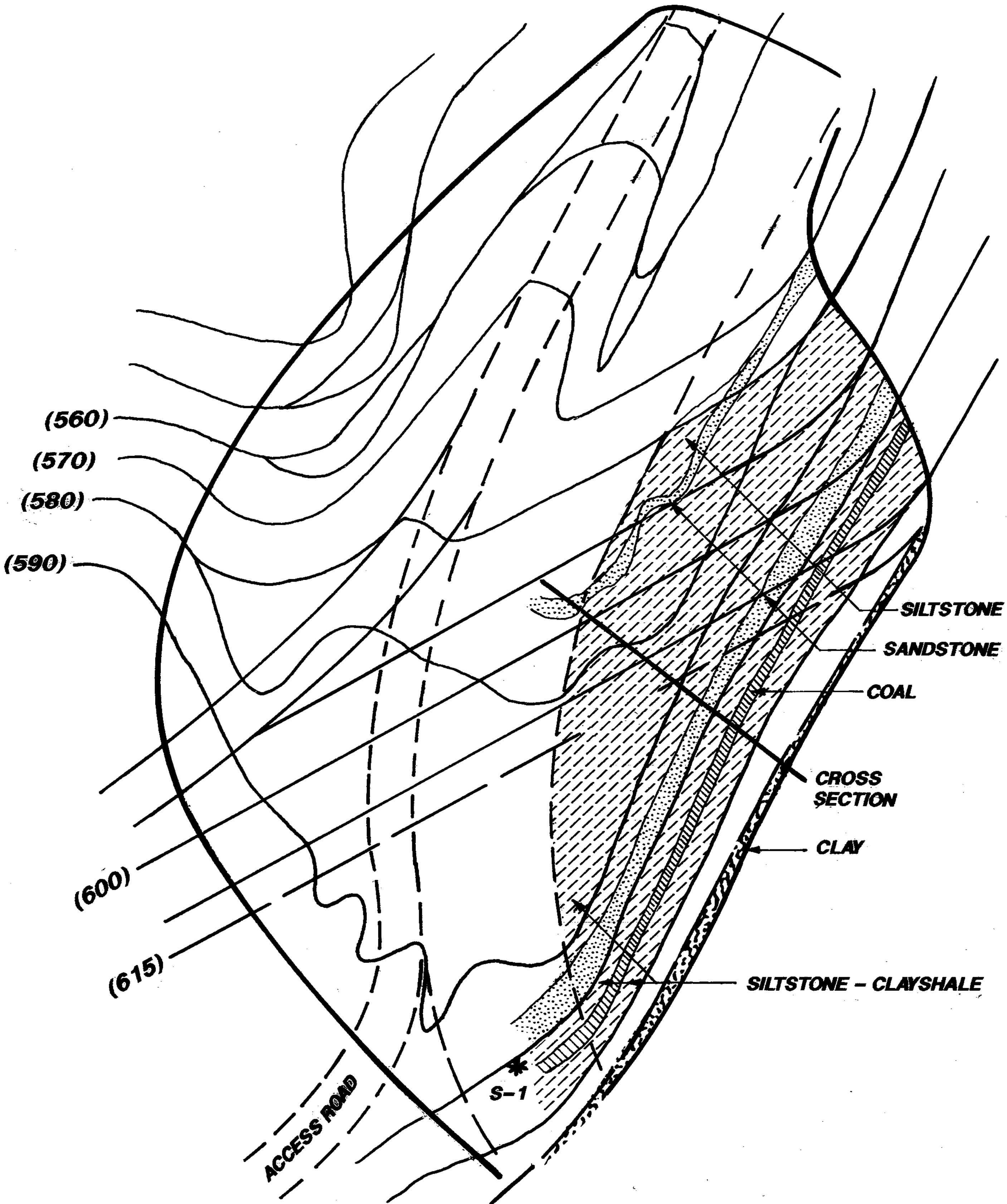


FIG. 3.2 PLANVIEW OF EXCAVATION AREA

4.0 SUBSURFACE CONDITIONS

Drawing No. 12-30041 depicts three schematic subsurface sections taken at the existing main dam parallel to the crest, parallel to the existing El. 590 berm; and; along the center of the dam. Drawing No. 12-30034 depicts a schematic section taken parallel to the crest of the existing saddle dam and proposed emergency spillway. Sections chosen in this manner depict the typical stratigraphy at the main and saddle dam sites. Except for minor variations in strata thickness and in the location and size of discontinuous deposits, the explorations not shown in the sections do not reveal significantly different conditions from those shown on the sections. In order to simplify the section, the soil descriptions have been presented in a shortened form. If more detailed descriptions of these soils are desired at a particular location, the logs of the individual borings (Appendix B) should be examined in conjunction with the sections. Subsurface conditions at the main dam and saddle dam are described in general, in the following sections.

4.1 Main Dam

Materials in Existing Embankment

The original design of the dam (crest el. 625) consisted of a homogeneous clay section. The second stage (crest el. 675) consisted of a clay core and upstream and downstream rockfill shells. A cross section of the existing dam as completed is shown on Fig. B.1, in Appendix B to this report.

According to the samples obtained from the borings made through the core and original embankment, the clay portion was constructed primarily of sandy clays containing zones which included weathered shale particles. Some of the samples, however, consisted of clay only. The consistency of most of the samples range from stiff to very stiff, but isolated zones of soft, wet, clays were also encountered.

Overburden Soils

The borings show that the overburden in the bottom of the valley, within the foundation area of the existing dam and the proposed enlargement, consist of clayey soils, and contain particles of weathered rock. The clayey soils are generally of medium plasticity.

According to the 1968 borings, the original thickness of overburden in the bottom of the valley, within the foundation of the original dam, ranged from about 5 ft. at the east side of the valley to 36 ft. on the west side. The thickness in the center of the valley was about 28 ft. Fig. B.2, in Appendix B, shows a cross section of the valley along the centerline of the dam.

The overburden on the east abutment slope was 5 to 10 ft. thick. On the west abutment slope, it was slightly thicker, with the thickness increasing to about 20 ft. near the bottom of the slope. There are outcrops of bedrock on both abutments. The overburden consists typically of sandy clays containing particles of weathered sandstone. Near the bedrock surface the overburden generally reflected the bedrock type; e.g., the overburden graded into weathered sandstone if the underlying rock was sandstone.

Bedrock Conditions

The rock formations at this site are all of sedimentary origin, with approximately horizontal bedding. The major bedrock type is sandstone of various gradations and degrees of cementation. The thickness of the sandstone strata varies from a few inches to over 100 ft. Within the upper portion of the abutment slopes, the sandstone alternates with layers of siltstone, cemented shale and clay shale ranging in thickness from a fraction of an inch to about 20 ft. There is also a 1 ft. layer of coal at about el. 600.

The bedrock in the bottom of the valley is sandstone which, beneath the center of the dam, extends to a depth of about 19 ft. below the lowest point in the bedrock valley. This sandstone bed is underlain by cemented shale and clay shale.

4.2 Saddle Dam

The soils within the vicinity of the saddle dam are primarily composed of a thin veneer of interbedded deposits underlain by bedrock. The upper most soil layer within the center portion of the dam is a zone of bottom ash mixed with sand, approximately three feet thick.

The layers underneath the surface zone are glacial outwash deposits and can be characterized by randomly alternating zones of sandy silt, sandy clay, and clayey sand. Each individual zone does not exceed three feet in thickness but the interbedded layers ranges in total thickness from a few feet to a maximum of 25 feet near the east abutment of the dam. The soil covering at both abutments is extremely shallow, in most places the underlying bedrock lies exposed at the surface.

Within the dam, bedrock lies directly below the alternating layers of glacial outwash deposits, or is exposed in the vicinity of the abutments. The bedrock consists primarily of thin alternating layers of sandstone, claystone and silty shale. One of the sandstone layers appears to exceed 10 feet in thickness, but on average most of the layers are approximately three to five feet thick.

5.0 LABORATORY TESTING

In the laboratory all of the soil samples were visually identified, and natural moisture content and liquid and plastic limit determinations were performed on selected representative specimens. In addition, a number of consolidation, direct shear and triaxial compression tests were performed on selected undisturbed samples to determine the strength and compressibility characteristics of the soils encountered at the sites of the proposed dams. The types and numbers of tests performed are listed as follows:

Visual Identification	241
Natural Moisture Content	107
Liquid and Plastic Limits	86
Grainsize Analysis	85
Consolidation Tests	2
Direct Shear	16
Triaxial Compression	64
Unit Dry Weights	84
Specific Gravity	3
Relative Density	1
Permeability	4
Standard Compaction	1

All test results are submitted in Appendix D to this report.

5.1 Main Dam

Because it has not been possible to determine the actual dimensions of the cut-off trench, which was excavated in the foundation of the original dam, nor the character and method of placement of the backfill, it will be assumed that all samples from below the original ground surface are original foundation materials. Therefore, samples from below about el. 540 in borings BSFD-1, BSFD-2 and BSFD 3 and 3A, and below about el. 538 in borings BSFD-4, BSFD-5 and BSFD-6 and SI-2 are considered foundation soils.

5.1.1 Strength Tests

Because of the history of significant deformations observed at the dam, strength parameters associated with the foundation soils and existing materials have been evaluated at 15% and 20% strains. In addition, undisturbed specimens from foundation soils recovered in Boring SI-2, were used to obtain undisturbed samples at angles of 30° and 45° with the horizontal to evaluate the shear strength of these soils on horizontal shear planes. These samples were consolidated under consolidation ratios ($Kc = \sigma_{1c}/\ \sigma_{3c}$) equal to 1; and 1.75.

Strength of Foundation Soils:

Four triaxial compression tests under unconsolidated undrained, UU, conditions were performed on samples from Borings BSFD-4 and BSFD-5 from depths below the elevation of the original ground surface. The soils consisted of stiff to hard brown mottled with gray silty clay, little to some fine to coarse sand (lean clay). The total stress circles for the four tests are presented in Fig. D.1 and D.2, in Appendix D, which also show strength envelopes.

Six triaxial compression tests under consolidated undrained conditions with back pressure saturation and pore-pressure measurements, CU, were performed on samples from Borings BSFD-4 and BSFD-5 from depths below the elevation of the original ground surface. The soils consisted of very stiff to hard gray mottled with brown clayey silt, "and" fine to coarse sand (lean clay). The total and effective stress circles for the six tests are presented in Fig. D.3, D.3A, D.4, and D.4A which also show the associated strength envelopes. These tests were performed on vertical samples and were consolidated isotropically.

Eight direct shear tests, DS, were performed on samples from Borings BSFD-5 and BSFD-6 from depths below the elevation of the original ground surface. The soils consisted of very stiff gray clayey silt, some fine to coarse sand, trace gravel (lean clay). Strength envelopes associated with the tests results are presented in Fig. D.5 and D.6.

Isotropically and anisotropically consolidated, CU, tests were performed on samples obtained at angles of 30° and 45° from shelby tubes recovered in Boring SI-2. All samples tested were representative of the soils located below the elevation of the original ground surface. Six isotropically consolidated CU tests with back pressure saturation and pore-pressure measurements, were performed. The samples consisted of very stiff to hard gray clayey silt, some fine to coarse sand (lean clay). The total and effective stress circles for these tests are presented in Fig. D.7, D.7A, D.8 and D.8A. Also shown are the associated strength envelopes. In addition, five anisotropically consolidated, Kc=1.75, CU tests with back pressure saturation and pore pressure measurements were also performed. These samples also consisted of very stiff to hard brown mottled with gray clayey silt (lean clay). The total and effective stress circles for these tests are presented in Fig. D.9, D.9A, D.10, and D.11. Also shown are the associated strength envelopes.

A summary of the shear strength obtained from each of these tests is presented on Table No. 5.1.

TABLE NO. 5.1

SUMMARY OF SHEAR STRENGTH OF FOUNDATION SOILS AT MAIN DAM

α	Con. Rat. Kc	Test	Shear Strength (TSF) Total Stress 20% Strain	Total Stress 15% Strain	Effec. Stress 20% Strain	Effec. Stress 15% Strain
90°	--	UU (Q)	$\sigma \le 2.5$ $S=\sigma \tan 16°$ $2.5 < \sigma < 5.0$ $S=0.4+\sigma \tan 6°$ $5.0 \le \sigma$ $S = 1.0$	$S = \sigma \tan 14°$ $S=0.35+\sigma \tan 7°$ $S = 0.85$	---	---
90°	--	UU (Q)	$5.0 \le \sigma$ $S = 1.1$	$S = 1.0$	---	---
90°	1	Cu (R)	$1.1+\sigma \tan 16°$	$0.8+\sigma \tan 17°$	$\sigma' \tan 28°$	$\sigma' \tan 31°$
90°	1	Cu (R)	$0.6+\sigma \tan 18°$	$0.8+\sigma \tan 19°$	$\sigma' \tan 25°$	$\sigma' \tan 27°$
90°	1	DS	---	---	$\sigma' \tan 26°$	$\sigma' \tan 27°$
90°	1	DS	---	---	$\sigma' \tan 35°$+	$\sigma' \tan 37°$+
30°	1	Cu (R)	$1.1+\sigma \tan 12°$	$1.3+\sigma \tan 12°$	$\sigma' \tan 23°$*	$\sigma' \tan 25°$
45°	1	Cu (R)	$0.65+\sigma \tan 18°$	$1.0+\sigma \tan 15°$	$\sigma' \tan 27°$	$\sigma' \tan 25°$
45°	1.75	Cu (R)	$4.8+\sigma \tan 6°$**	---	$\sigma' \tan 34°$**+	---
45°	1.75	Cu (R)	$1.9+\sigma \tan 12°$**	---	$\sigma' \tan 30°$**	---

** At approximately 6% strain.
* Appears to be low.
+ Appears to be high.

<u>Strength of Clay in Existing Dam</u>:

Five triaxial compression tests under unconsolidated undrained, UU, Conditions were performed on samples from Borings BSFD-1, BSFD-2, BSFD-3 and BSFD-3A from depths above the elevation of the original ground surface. These soils consisted of very stiff to hard brown silty clay. Total stress circles for these tests are presented in Fig. D.12 and D.13, which also show strength envelopes.

Six triaxial compression tests, under consolidated undrained conditions with back pressure saturation and pore-pressure measurements, CU, were performed on samples from Borings BSFD-1 and BSFD-2 from depths above the elevation of the original ground surface. The samples consisted of very-stiff to hard brown silty clay. The total and effective stress circles for these tests are presented in Fig. D.14, D.14A, D.15 and D.15A. These tests were performed on vertical samples and were consolidated isotropically.

Six direct shear tests, DS, were performed on samples from Borings BSFD-1 and BSFD-2 from depths above the elevation of the original ground surface. The samples consisted of stiff to very-stiff brown mottled with gray silty clay, some fine to coarse sand; and of hard brown silty clay, little fine to medium sand. Strength envelopes associated with test results are presented in Fig. D.16 and D.17.

A summary of the shear strength obtained from these tests is presented on Table No. 5.2.

TABLE NO. 5.2

SUMMARY OF SHEAR STRENGTH OF CLAY IN EXISTING MAIN DAM

TEST	SHEAR STRENGTH (TSF)			
	TOTAL STRESS		EFFECTIVE STRESS	
	20% STRAIN	15% STRAIN	20% STRAIN	15% STRAIN
UU (Q)	$\sigma \le 3.0$ tsf $S=\sigma \tan 28^\circ$+ $3.0<\sigma<6.0$ $S=0.4+\sigma \tan 24^\circ$ $6.0 \le \sigma$ $S = 3.0$	$S=\sigma \tan 22^\circ$ $S=\sigma \tan 22^\circ$ $S=2.7$	---	---
UU (Q)	$\sigma \le 4$+sf $S=\sigma \tan 20^\circ$ $4<\sigma<8.3$tsf $S=0.6+\sigma \tan 11^\circ$ $8.3<\sigma<15$tsf $S=0.7+\sigma \tan 10^\circ$ $\sigma<15$tsf $S=3.5$	$S=\sigma \tan 20^\circ$ $S=0.6+\sigma \tan 11^\circ$ $S=0.6+\sigma \tan 11^\circ$ $S=3.8$	---	---
Cu (R)	$0.8+\sigma \tan 20^\circ$	$0.5+\sigma \tan 23^\circ$	$\sigma' \tan 25^\circ$	$\sigma' \tan 27^\circ$
Cu (R)	$1.1+\sigma \tan 18^\circ$	$1.2+\sigma \tan 16^\circ$	$\sigma' \tan 26^\circ$	$\sigma' \tan 29^\circ$
DS	---	---	$\sigma' \tan 23^\circ$	$\sigma' \tan 27^\circ$
DS	---	---	$1.7+\sigma' \tan 17^\circ$	$1.6+\sigma' \tan 17^\circ$

+ Appears to be high.

<u>Strength of Clay from Proposed Borrow</u>:

The samples tested came from combining samples obtained from the seven test trenches dug at the location of the proposed borrow site. The samples consisted of brown silty clay, "and" fine to coarse sand, trace fine to coarse gravel. Before testing, all composite samples were remolded by compacting the soil to a dry unit weight equal to 95% of the maximum dry unit weight as obtained in accordance with ASTM D698. Strength tests were conducted on remolded samples having moisture contents from about -1% to +3% of the optimum moisture content.

Six triaxial compression tests under unconsolidated undrained, UU, conditions, were performed on samples remolded at moisture contents of -1% and +3% of the optimum moisture content. Total stress circles for these tests are presented on Fig. D.18 and D.19, which also show strength envelopes.

Twelve triaxial compression tests under consolidated undrained conditions with back pressure saturation and pore-pressure measurements, CU, were performed on samples remolded at moisture contents of -1%, optimum, +2% and +3%. Total and effective stress circles for these tests are presented on Fig. D.20 and D.23.

A summary of the shear strength obtained from these tests is presented on Table No. 5.3.

TABLE No. 5.3

SUMMARY OF SHEAR STRENGTH OF CLAY FROM BORROW AREA

Moisture Content	Tests	Shear Strength (TSF)	
		Total Stress	Effective Stress
-1%	UU(Q)	$\sigma \leq 0.85$+tsf $S=0.5+\sigma \tan 27^\circ$ $0.85<\sigma<1.3$tsf $S=0.4+\sigma \tan 30^\circ$ $1.3\leq\sigma$ $S=1.2$	--
+3%	UU(Q)	$\sigma \leq 0.55$tsf $S=0.42$ $0.55<\sigma<0.86$ $S=0.26+\sigma \tan 14^\circ$ $0.86\leq\sigma$ $S=0.5$	--
-1%	Cu(R)	$0.25+\sigma \tan 15^\circ$	$\sigma' \tan 27^\circ$
Opt.	Cu(R)	$0.3+\sigma \tan 19^\circ$	$\sigma' \tan 33^\circ$
+2%	Cu(R)	$0.2+\sigma \tan 20^\circ$	$\sigma' \tan 28^\circ$
+3%	Cu(R)	$0.38+\sigma \tan 16^\circ$	$\sigma' \tan 25^\circ$

Strength of Bottom Ash:

It was assumed that the lowest strength of the bottom ash would be represented by the sample containing the highest percentage of material passing the No. 200 sieve. The minus No. 4 sieve fraction from this sample was used in four triaxial tests under Consolidated Drained Conditions, CD. This fraction contained 12% passing the No. 200 sieve. Sieve analysis is included in Appendix D of this report. For these tests, the wet ash was compacted to an approximate relative density equal to 70%, and the samples were then saturated. The effective stress circle for the CD tests on this specimen are shown in Fig. D.24, and the tangent to the circles through the origin indicates a friction angle of 38 degrees.

<u>Strength of Existing Rockfill</u>:

No samples of rockfill material for testing were obtained during the current investigation and design. However, presented in Table No. 5.4 are the results of Casagrande Consultant's triaxial tests on 1.4 in. diameter specimens of rockfill from the Amos Dam site, where the bedrock, in general, is probably weaker than at the Big Sandy Dam site. (The Amos Dam is owned and operated by Appalachian Power as a fly ash retention facility similar to the Big Sandy Dam.)

TABLE NO. 5.4

SUMMARY OF SHEAR STRENGTH OF EXISTING ROCKFILL

		Angle of Internal Friction with confining Pressure in Ton/sq. ft.		
Material	Type of Test	1	4	8
Compacted Clay Shale	R (total stress circles)	23.0°-28.5°	17.5°-19.6°	16.2°-16.8°
	S			23.0°-24.7°
Compacted Sandstone	S	41.4°	32.8°-36.5°	30.5°-34.6°

Triaxial tests on 6 in. diameter compacted specimens of sandstone from the Amos Dam site, made by Thurber Consultants Ltd. (Edmonton, Alberta, Canada), yielded effective angles of internal friction that are near the upper end of the ranges listed above.

5.1.2 Consolidation Tests

One consolidation test was performed on samples of the existing dam clays from above the original ground surface. The sample tested was from Boring BSFD-1. Another consolidation test was performed on samples of the overburden soils from beneath the existing dam berm. The sample tested was from boring BSFD-6. The laboratory data for these tests are included in Appendix D, the void ratio vs. pressure curves are also included in the appendix. Shown on each of these figures is an estimate of the preconsolidation pressure, Pc, which is estimated from the shape of the void ratio - pressure curve Fig. F-3 in Appendix F. In the following Table No. 5.5, the preconsolidation pressures are compared with the maximum stress corresponding to the existing height of the dam. Fig. F-1 and F-2 depict the location of selected points for which the existing state of stresses have been estimated.

TABLE NO. 5.5
PRECONSOLIDATION PRESSURE VS. EXISTING OVERBURDEN PRESSURE

1-5	Foundation 70'	3.2	6.0	1.9
6	Foundation 83'	3.3	6.0	1.8
7	Foundation 104	4.2	6.0	1.4
8	Foundation 117	4.8	6.0	1.3
9	Foundation 123'	5.2	6.0	1.2
10	Foundation 138'	5.8	6.0	1.0
11	Foundation 142'	5.9	6.0	1.0
12	Foundation 155'	5.3	6.0	1.1
13	Foundation 155'	5.6	6.0	1.1
14	Foundation 155'	5.6	6.0	1.1
A	Ext. Dam Clay 99'	4.2	2.8	0.7
B	Ext. Dam Clay 101'	4.4	2.8	0.6
C	Ext. Dam Clay 113'	5.0	2.8	0.6
D	Ext. Dam Clay 125'	4.3	2.8	0.7
E	Ext. Dam Clay 72'	3.1	2.8	0.9
F	Ext. Dam Clay 72'	3.1	2.8	0.9

NOTE: Consolidation tests performed by Casagrande Consultants for the design of Stage 2 dam. Revealed similar preconsolidation pressures for the foundation soils.

It is evident from this tabulation that the foundation soils present underneath the existing dam are overconsolidated at the present time.

5.1.3 Permeability Tests

Clay from Proposed Borrow Area:

To estimate the magnitude of the coefficient of permeability of the clay soils at this site, two flex-wall permeability tests were performed on compacted specimens from each one of the seven test pits dug within the Borrow Site. In addition, four flex-wall permeability tests were performed on composite samples from all test pits compacted to a unit dry weight equal to 95% of the maximum unit dry weight obtained in accordance to ASTM D698. The samples were compacted at moisture contents ranging from -1% to +3% of the optimum moisture content. A summary of the results is presented on Table No. 5.6. Data from the tests are contained in Appendix D to this report.

TABLE NO. 5.6
SUMMARY OF PERMEABILITY TEST RESULTS

TEST PIT NO.	PERMEABILITY CM/SEC.	MOISTURE CONTENT %	
		Optimum	Tested
TP-1	8.1 E-8 to 0.9 E-8	14.2	-2.2 to +1.2
TP-2	1.3 E-7 to 1.6 E-8	15.2	-2.4 to +2.4
TP-3	2.3 E-7 to 1.7 E-8	17.3	-1.8 to +2.0
TP-4	9.5 E-7 to 9.9 E-8	12.8	+0.8 to +1.3
TP-5	9.9 E-9 to 4.2 E-9	15.4	-1.7 to +2.9
TP-6	3.9 E-7 to 1.4 E-8	14.8	-2.1 to +2.6
TP-7	3.4 E-7 to 2.4 E-8	13.0	-1.6 to +2.1
Composite	2.0 E-7	13.5	--
Composite	1.0 E-7	13.5	+2
Composite	3.1 E-8	13.5	+3
Composite	1.8 E-7	13.5	-1

Bottom Ash

Because bottom ash from the Big Sandy Plant has been used for the internal drainage zone of the dam and performing well no permeability tests were performed on samples from this material during the present work. However, the permeability of several samples of the ash with different gradations was studied by Casagrande Consultants to determine what effect the percentage passing the No. 200 sieve has on the permeability of this material. All pertinent data is included in Casagrande Consultant's report dated 1976, (2). Excerpts from their discussion are presented in the following paragraphs.

Bottom ash samples consisting of material passing the 1/2 inch sieve were tested in a 4-inch diameter permeameter. Wet ash was compacted into the permeameter in layers with sufficient tamping effort to give approximately maximum density, but being careful not to cause excessive breakage of the particles during compaction. Before testing, the specimens were saturated from the bottom up. The tests were run using tap water and the constant head method.

The results show that the coefficient of permeability (at 20°C) decreases with increasing percentage of fines. With 2% passing the No. 200 sieve, the coefficient of permeability is about 1.7×10^{-2} cm/sec; and with 10% passing No. 200 sieve, it is about 1.4×10^{-3} cm/sec.

5.1.4 Compaction Tests

Clays from proposed Borrow:

Seven samples of clay were taken from Test Pits TP-1 to TP-7. The descriptions of these samples and the results of classification tests are presented in Appendix D, to this report.

ASTM D698 (Standard Proctor Compaction) tests were performed on all seven samples; and the laboratory data for these tests are included in Appendix D. The first compaction test for each sample was performed at the natural water content and, because all samples had natural water contents wet of optimum, they were allowed to dry progressively to obtain additional points for the compaction curves.

In addition, a Standard Proctor compaction test was performed on a composite sample from all seven test pits. Data from this test is also included in Appendix D.

A summary of the results of all these tests is presented in Table No. 5.7.

TABLE NO. 5.7

SUMMARY OF COMPACTION TESTS

SAMPLE	MAXIMUM DRY UNIT WEIGHT	OPTIMUM MOISTURE CONTENT
TP-1	116 PCF	14.2%
TP-2	114.6 PCF	15.2%
TP-3	109.1 PCF	17.3%
TP-4	118.5 PCF	12.8%
TP-5	114.6 PCF	15.4%
TP-6	114.9 PCF	14.8%
TP-7	120.1 PCF	13.0%
COMPOSITE	119.2 PCF	13.4%

5.1.5 Density Tests

Bottom Ash:

Maximum and Minimum density tests were performed in accordance with the ASTM D4254-83 method. The results of the density tests indicate that the minimum dry unit weight was 42.0 pcf, and the maximum was 84 pcf.

To determine the degree of particle breakdown during compaction, the gradation of the sample was determined before and after compaction on the vibratory table. The grain size curves before and after vibration were practically identical; i.e., no significant particle breakdown had occurred during the test.

It was also determined that the bottom ash currently used in connection with the construction of the dam has an average specific gravity equal to 2.32.

5.2 Saddle Dam

Construction of the existing Saddle Dam consisted of building a key into the existing rock foundation and to install a bottom ash blanket and chimney drain downstream of the core. The subsurface investigation at this site revealed the presence of an overburden on top of the rock. Thus, all soil samples obtained are considered to be from the overburden soils. With the exception of the strength of overburden soils, all other soil types and characteristics associated with the proposed construction of the Saddle Dam have been presented in Section 5.1., Main Dam, of this report.

5.2.1 Strength Tests

Strength of Foundation Soils:

Three triaxial compression tests under unconsolidated undrained, UU, conditions were performed on samples from boring BSFD-12 from a depth of 14.7 feet below the existing ground surface. The samples consisted of hard brown mottled with gray lean clay. The total stress circles for the three circles are presented in Fig. D.25, which also shows the strength envelope.

Three triaxial compression tests under consolidated undrained conditions with back pressure saturation pore-pressure measurements, CU, were performed on samples from Boring BSFD-14. The samples consisted of brown mottled with gray lean clay. The total and effective stress circles for the tests are presented in Fig. D.26 and D.26A respectively, which also show the associated strength envelopes.

A summary of the shear strength obtained from each of the UU and CU tests is presented on Table No. 5.8.

TABLE NO. 5.8

SUMMARY OF SHEAR STRENGTH OF FOUNDATION SOILS

TEST	Shear Strength (TSF) Total Stress	Effective Stress
UU (Q)	$\sigma \leq 3.0$ tsf $S=0.6+\sigma \tan 33^\circ$+ $3.0 \leq \sigma$ $S = 2.7$ tsf	--
CU (R)	$0.4 + \sigma \tan 25^\circ$	$\sigma' \tan 34^\circ$

+ Appears to be high.

6.0 HYDROLOGY

6.1 General

The watershed of the Horseford Creek Dam is depicted on Fig. 6.1. The major drainage feature, Horseford Creek, drains into Blaine Creek which flows into the Big Sandy River at river mile 19.6.

Presently, the crest of the existing dam is at el. 678±. Discharge is regulated by the intake tower of the principal spillway which is connected to a 30-inch diameter reinforced concrete pipe. The concrete pipe terminates into a riprap-lined channel. The intake tower consists of twin shafts with side openings for decanting effluent. Concrete stoplogs are placed in the side openings as necessary to maintain settling action of the fly ash and act as an overflow weir to establish operating levels. During most operating conditions, discharge through the principal spillway will be controlled by weir flow over both stoplogs. The current emergency spillway consists of a 50-foot wide open channel excavated in rock on the left abutment of the saddle dam.

The proposed raising of the main dam will necessitate filling in the existing emergency spillway, raising of the saddle dam, and constructing a new emergency spillway.

The following sections present the hydrologic considerations and analyses performed during the design phase of this project.

6.2 Basin Characteristics

Fig. 6.1 depicts the limits of the watershed boundary for the fly ash dam. A review of available topographic maps and aerial photographs was made to determine essential basin characteristics. Such characteristics include the drainage boundaries, areas, slopes, soil types, land use and time of concentration. The time of concentration is defined as the elapsed time for runoff to travel from the hydraulically most distant part of the watershed to some reference point downstream.

Present land use within the watershed is limited to the fly ash pond proper and the adjacent wooded hillslopes. Raising of the dam will only alter the total acreage for both land uses.

A detailed soil survey for Lawrence County has not been published. Previous reports on the Phase 1 and 2 designs classified the soils such that they fall under the hydrologic soil group C as defined by the Soil Conservation Service

(SCS) of the U.S. Department of Agriculture. The table below lists the basin characteristics used for this project.

Drainage Area	675 acres
Average Land Slope	28%
Hydrologic Soil Group	C
SCS Curve Number (weighted)	73
Time of Concentration	0.25 hour

Based on Drawing No. 12-30030, the fly ash pond will have the following surface areas and storage capacities above elevation 650 feet.

Elevation	Area (ac)	Storage (ac-ft)
650	87	0
660	103	945
680	145	3406
700	169	6540
710	184	8302

Fig. 6.2 illustrates the stage-area-volume relationship for the fly ash pond.

6.3 Assumptions and Design Requirements

No rainfall-runoff data were available for the site. Therefore, runoff hydrographs were generated using the SCS method as described in the National Engineering Handbook, Section 4 - Hydrology. Since the raised pool levels will inundate a significant portion of the watershed, the hydrograph analysis for the emergency spillway was divided in two subwatersheds - the pond area and the remaining drainage area. The hydrograph for the pond area was developed by converting the precipitation hyetograph into an inflow hydrograph since there is 100 percent instantaneous runoff. The inflow hydrograph for the remaining area was determined by the referenced method. These two hydrographs were combined to define the design flood hydrographs. The general design requirements for the principal and emergency spillways have been established by the Kentucky Department for Environmental Protection, Division of Water.

6.3.1 Principal Spillway

According to Division of Water, Engineering Memorandum No. 5, design of the principal (service) spillway for Class C dams must be such that the average frequency of use of the emergency spillway cut in rock is predicted to be less than once in fifty years. To meet this criteria, the 10-day principal spillway hydrograph (PSH) must be developed based on the procedures in the SCS National Engineering Handbook. The principal spillway shall also have the capacity to release 80% of the maximum flood waters retained in the reservoir within ten days.

It was assumed that a 50-year rainfall event would produce the 50-year flood. Antecedent soil moisture condition II was assumed for the hydrograph development. Precipitation values were obtained from the Division of Water, Engineering Memorandum No. 2 (rev. 1979). For the project site located in Lawrence County, the rainfall values are 5.1 and 8.9 inches for the 50-year, 1 day and 10 day storms, respectively.

6.3.2 Emergency Spillway

The emergency spillway shall pass the design emergency spillway hydrograph (ESH) and freeboard hydrograph (FH) without overtopping the dam. For Class C dams, the minimum hydrologic criteria are:

ESH design rainfall, $P = P100 + 0.26\ (PMP - P100)$
FH design rainfall, $P = PMP$

For this project, the design rainfalls have a duration of 6 hours. The PMP, probable maximum precipitation, is defined as the greatest depth of precipitation for a given duration that is meteorologically possible for a given basin at a particular time of year. The probable maximum flood (PMF) is the result of the PMP. For the project site, the rainfall values are 4.3 and 28.1 inches for the 100-year P100 and PMP storms, respectively. Antecedent moisture condition II was assumed for both storms.

6.4 Hydrologic Analysis

Most rainfall - runoff computations and all reservoir flood routings were conducted using the U.S. Army Corps of Engineers HEC-1 computer program. Basin characteristics, stage-storage, and discharge rating curves are input data supplied by the user.

6.4.1 Principal Spillway

Daily discharges from the reservoir will continue to be regulated by the existing principal spillway structure with no modifications. The principal spillway hydrograph (PSH) was manually calculated following the procedures in the SCS Handbook. Ash sluice waters (estimated to be 10 cfs) were added to the PSH to develop a total inflow hydrograph. Flood routings were conducted to establish a maximum operating pool level and the invert elevation of the emergency spillway.

6.4.2 Emergency Spillway

Flood routings of the freeboard hydrograph were conducted to determine the size of the emergency spillway necessary to pass the probable maximum flood without overtopping the dam. The initial water surface in the reservoir was assumed to be at the maximum operating level at the beginning of the storm. Flood routings were conducted for two situations (a) the principal spillway discharging and (b) neglecting those discharges.

Based on the principal spillway hydrograph flood routings, the 10-day drawdown level was at the maximum operating level. Therefore, the freeboard hydrograph was the controlling flood to set the emergency spillway dimensions and minimum top of dam.

In addition to the 6-hour PMF, a 24-hour PMF was developed and routed through the facility. This was done to verify the pond's capacity to handle a larger volume storm over a longer duration. The 24-hour PMP has a value of 35.5 inches.

6.5 Results

6.5.1 Principal Spillway

The development of the 50-year principal spillway hydrograph (PSH) indicates a peak inflow of 357 cfs which includes 10 cfs of ash sluice waters and 5 cfs of minimum quick return flow. The runoff volume over the 10 day duration is 610 acre-feet.

A maximum operating pool level has been set at elevation 705. This corresponds to a maximum stop-log elevation of 704. Final flood routings of the PSH started at an initial pool level equivalent to the maximum operating level. The reservoir rises to a maximum elevation of 706.0 feet. The peak outflow through the principal spillway is 54 cfs. Fig. 6.3 is a plot of the inflow and outflow principal spillway hydrographs. The receding flood pool is drained within eight days and six hours after the peak pool level is attained. Therefore, the requirements of Section E-I of Engineering Memorandum No. 5 are satisfied.

6.5.2 Emergency Spillway

The development of the 6-hour PMF hydrograph indicates a peak inflow of 10,610 cfs and a runoff volume of 1159 acre-feet.

The invert of the emergency spillway has been set at elevation 706.25 which is above the 50-year PSH flood elevation. The reservoir will rise to elevation 709.4 feet during passage of the PMF. This is based on an initial pool level of 705 feet and a 100 foot wide emergency spillway section. The flood routing was based on the principal spillway being plugged, a conservative condition. The peak outflow from the facility is 1672 cfs. Fig. 6.4 is a plot of the inflow and outflow emergency spillway hydrographs.

The 24-hour PMF generates 1564 acre-feet of runoff and has a peak inflow of 15,687 cfs. The pond will rise to elevation 710.5 feet and the peak outflow is 2433 cfs.

6.6 Summary and Conclusions

The 50-year, 10-day principal spillway hydrograph and the 6-hour probable maximum flood hydrograph were generated for the proposed raising of the Horseford Creek Dam. The SCS procedures for hydrologic design were followed with the use of the U.S. Army Corps of Engineers computer programs HEC-1 and HEC-2. Table No. 6.1 gives a summary of the study.

The existing principal system will be used without any modifications. It has the capacity to safely discharge the design flood without engaging the emergency spillway. The total spillway system (principal and emergency) has enough capacity to pass the probable maximum flood without overtopping the crest of the dam. There is sufficient freeboard for wind and wave height calculated to be 1.39 feet.

TABLE NO. 6.1
SUMMARY OF HYDROLOGIC/HYDRAULIC DATA

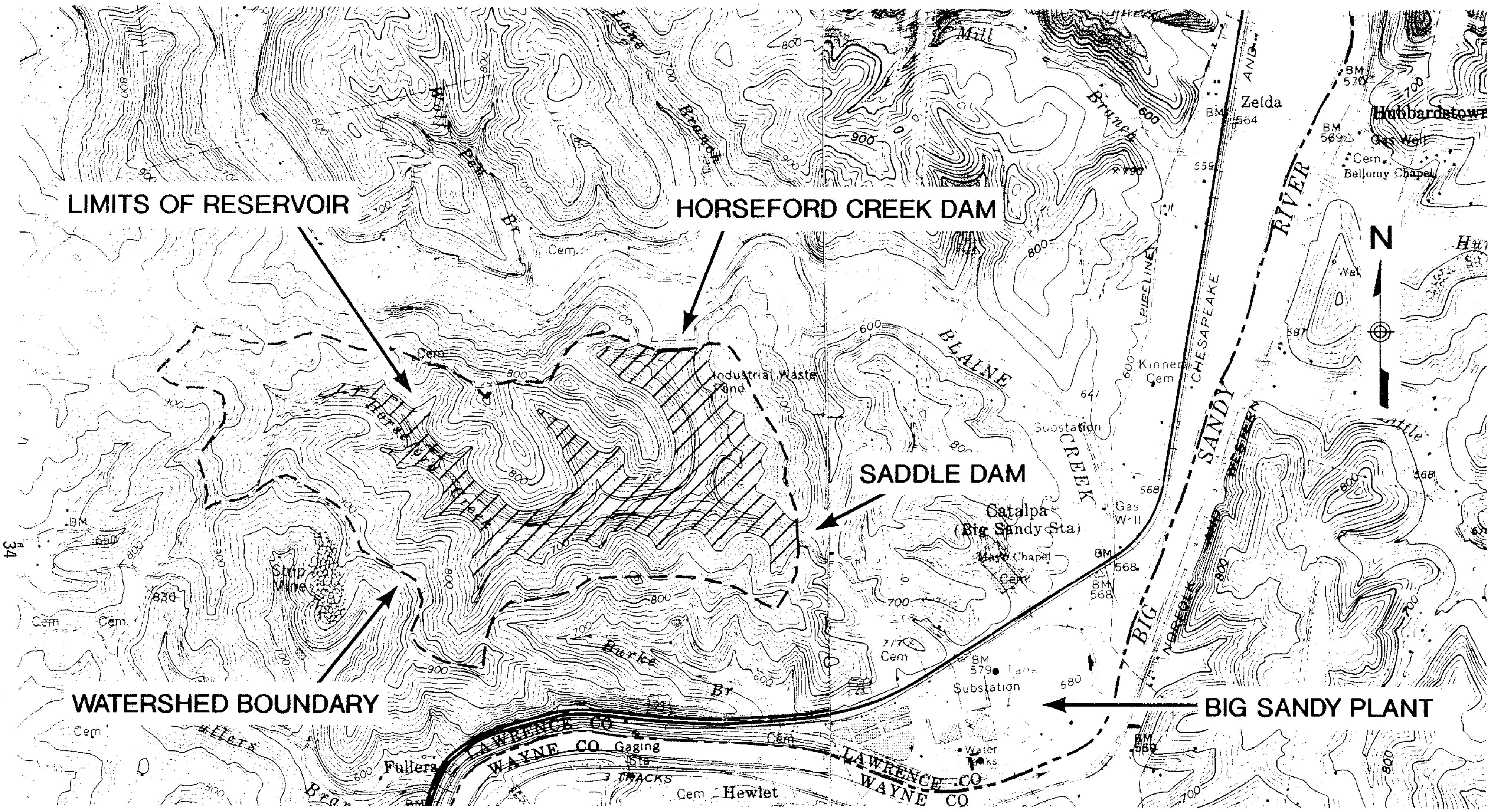
DRAINAGE AREA	0.9 SQ. MI.
DESIGN INFLOW FLOODS	
50-YR, 10-DAY PEAK	357 cfs
6-YR PMF PEAK	10,610 cfs
24 HR PMF	15,687 cfs
PEAK DISCHARGE	
50-YR, 10-DAY	54 cfs
6-HR PMF	1,672 cfs
24 HR PMF	2,433
MAXIMUM POOL ELEVATION	
50-YR, 10-DAY	706.0 FT
6-HR PMF	709.4 FT
24 HR PMF	710.5
DAM CREST ELEVATION	711.0 FT
MAXIMUM OPERATING POOL ELEVATION	705.0 FT

PRINCIPAL SPILLWAY:

CONCRETE RISER TOWER AND CONDUIT
SHAFT OPENING 2 @ 3.0' X 4.0'
30" DIAMETER REINFORCED CONCRETE PIPE

EMERGENCY SPILLWAY:

EXCAVATED CHANNEL IN ROCK	
CREST ELEVATION	706.25 FT
BOTTOM WIDTH	100 FT
SIDE SLOPES	1H:6V



REFERENCE: USGS QUADRANGLE MAPS
FALLSBURG, KY.-W.VA.
PRICHARD, W.VA.-KY.

SCALE:

0 2000 4000 FEET

Big Sandy Plant
Horseford Creek Fly Ash Dam
Watershed Map

FIGURE 6.1

Big Sandy Plant
Horseford Creek Fly Ash Dam
Stage-Area-Volume Curves

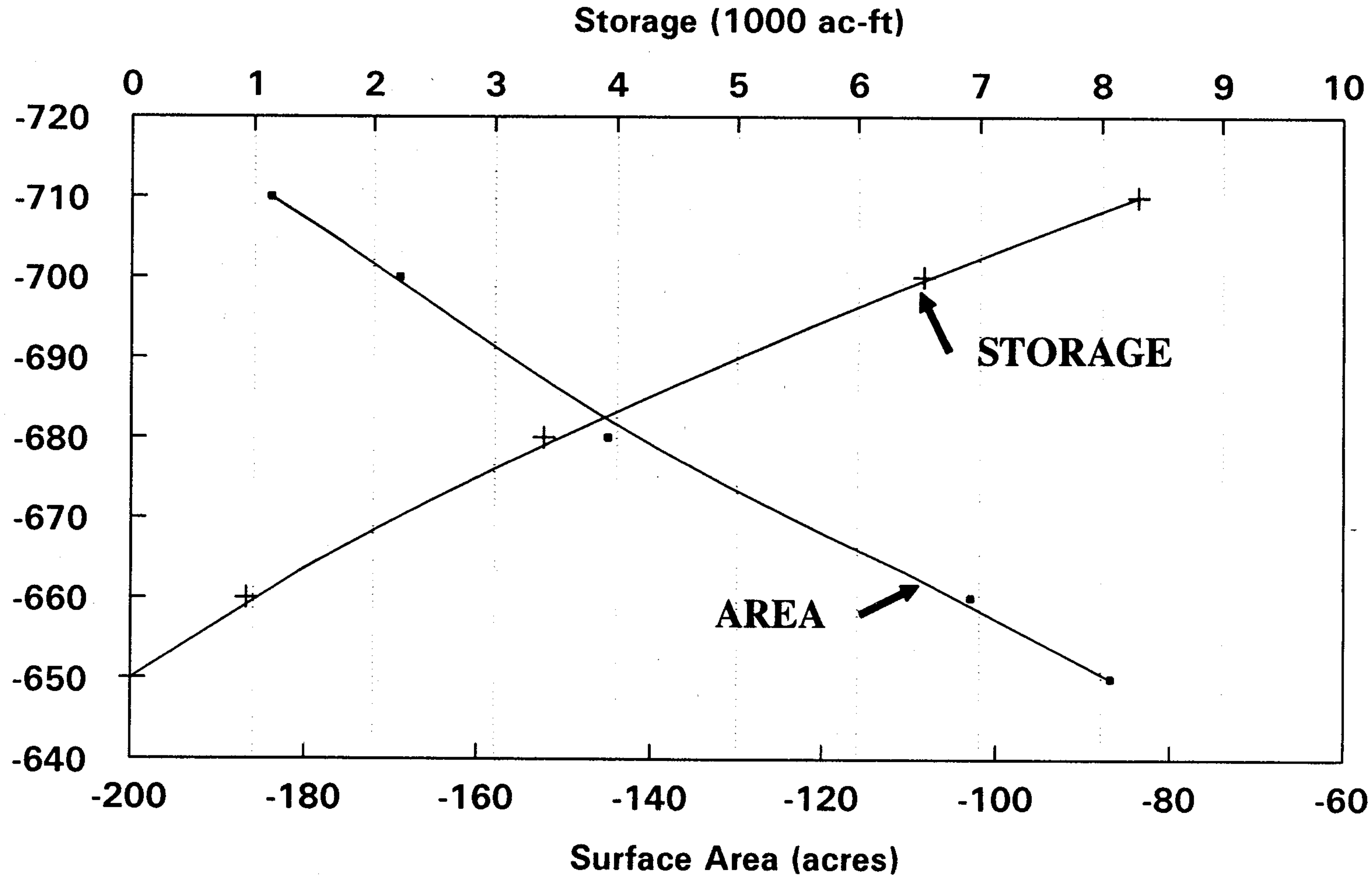


FIGURE 6.2

Big Sandy Fly Ash Dam
Principal Spillway Hydrograph
50 Yr - 10 Day Design Storm

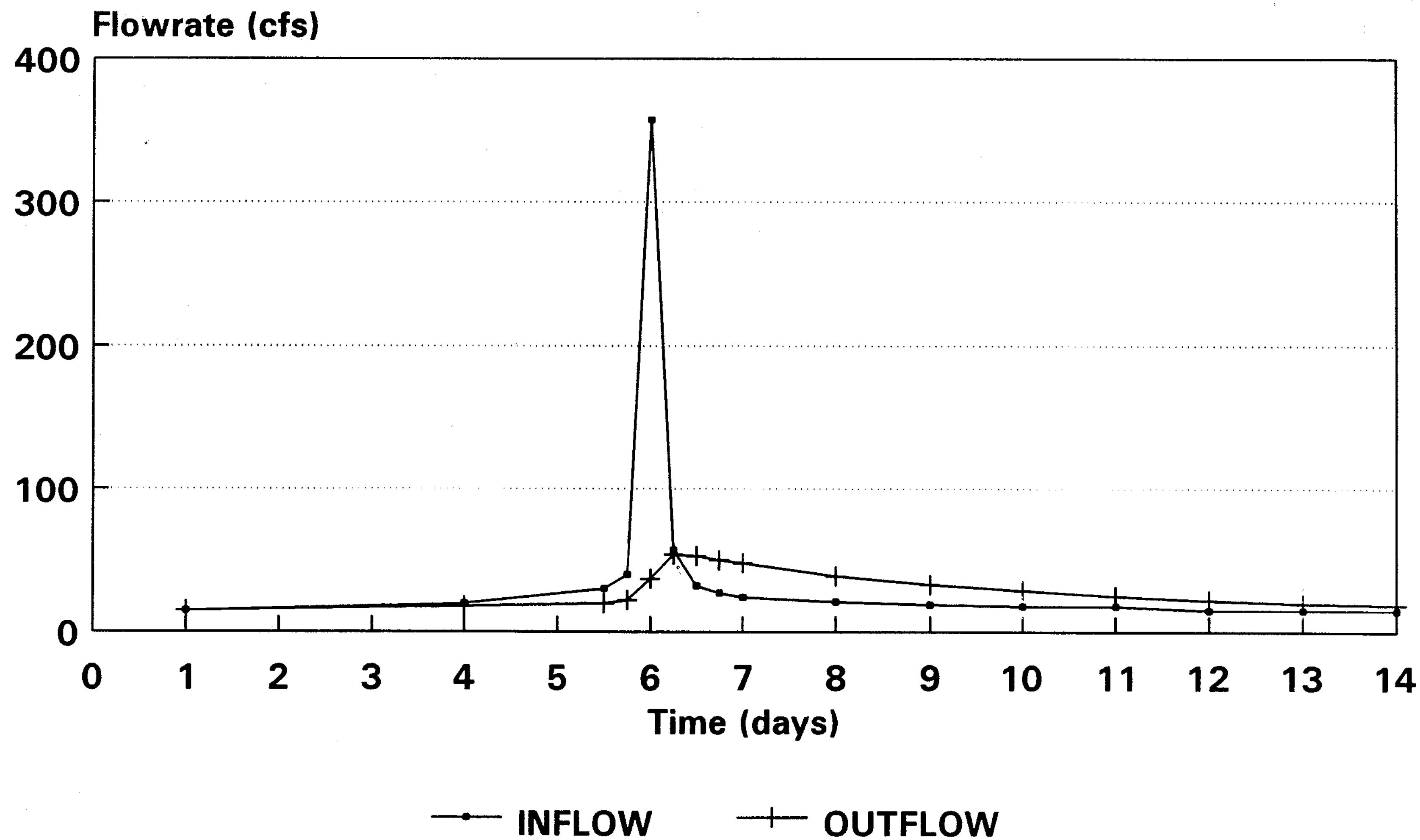


FIGURE 6.3

Big Sandy Fly Ash Dam
Emergency Spillway
6 Hr PMF Design Flood

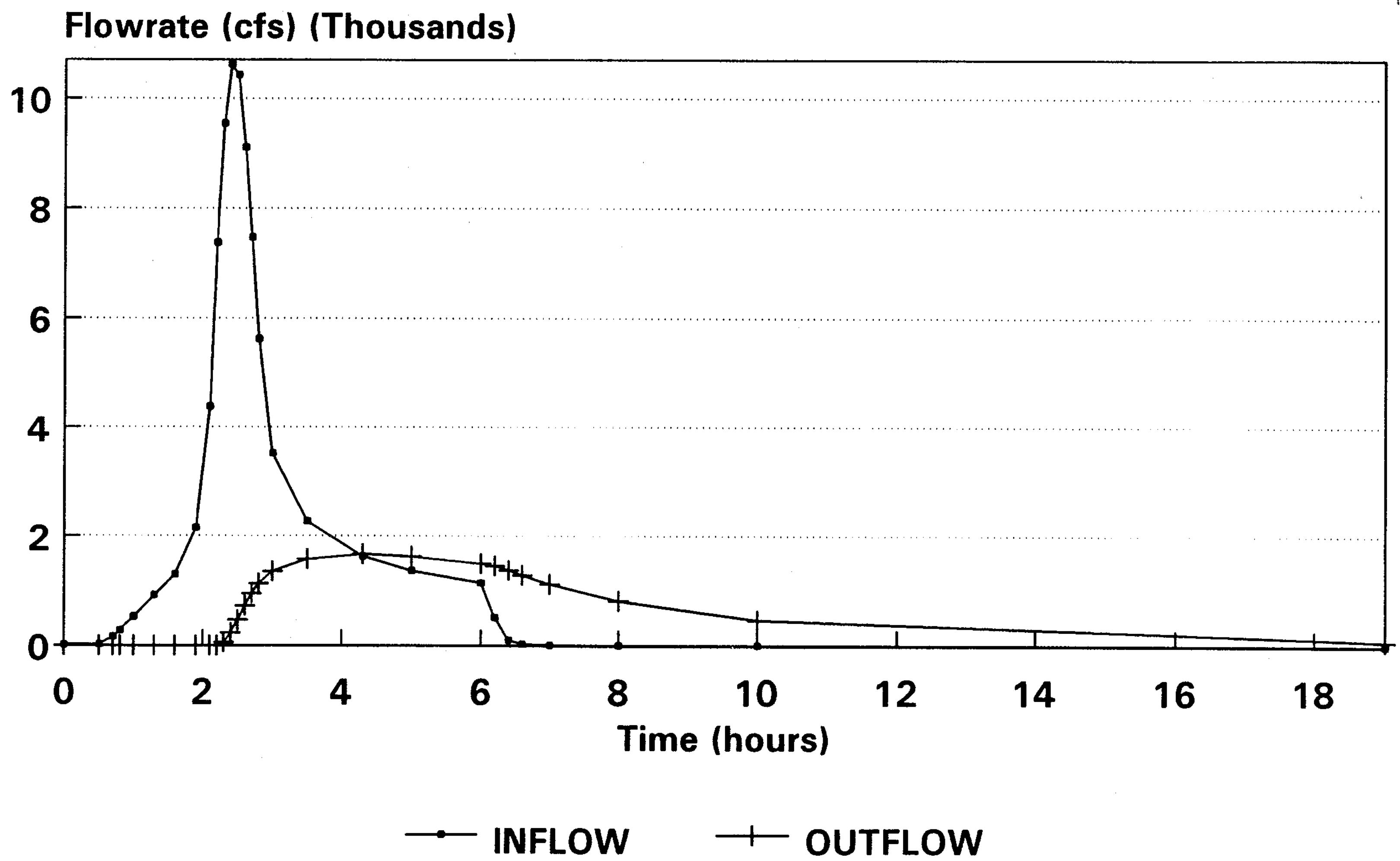


FIGURE 6.4

7.0 GEOTECHNICAL ANALYSIS

Seepage, settlement and stability analyses for the main dam and saddle dam were conducted. These analyses and the selection of design parameters are discussed in this section of this report. Also a discussion of the effect of the proposed construction on the existing spillway pipe as well as the criteria for filter design are presented.

7.1 Main Dam

The geotechnical analyses considered the location of the seepage line at different water elevations in the reservoir, the effects of the proposed loading on the settlement behavior of both the foundation soils and the clay zones of the existing dam and, the results of the strength tests in the selection of the design parameters associated with the stability both the upstream and downstream slopes. A discussion of these items is presented in the following paragraphs.

7.1.1 Seepage

In the seepage through an earth dam, the upper boundary or upper most flow line (known as the line of seepage) is not known, but must be found. Among the available solutions for seepage with a free surface, it has been decided to follow Casagrande's solution for the different levels of the water throughout the construction and operation of this dam. In this way, two conditions have been considered in connection with the proposed construction. (1) As long as the water elevation inside the reservoir remains below elevation 675, the seepage line will meet a vertical chimney drain ($\alpha = 90°$); (2) After the water elevation inside the reservoir exceeds elevation 675, the seepage line will meet an overhanging discharge surface ($90° < \alpha < 180°$).

In the first case, the line of seepage is estimated using:
$a = 3/4Yo$; $Yo = (h^2 + d^2)^{1/2} - d$.

For the second condition, the line of seepage was estimated using:

$C = \Delta a/a+\Delta a$;
$Yo = (d^2 + h^2)^{1/2} - d$, where C is a function of α.

Fig. F.1 and F.2 (Appendix F) illustrate the location of the seepage lines for several water levels of the reservoir as estimated using the above mentioned solutions. Calculations associated with the solution of these equations are included in Appendix F to this report.

7.1.2 Settlement

Since it is planned to achieve the final crest elevation of the dam over a minimum period of seven years, it is concluded that the time required to increase the load to its final value is a significant part of the time required for consolidation. Hence, there is an interval during which consolidation occurs simultaneously with the increase of the load. This initial interval is followed by an ordinary consolidation process.

The consolidation due to every load increment proceeds independently of the consolidation due to the preceding and succeeding load increments. Therefore, the rate of consolidation during the period of transition can be computed by a process of superposition. Based upon the results of the consolidation tests, settlements at selected locations in the foundation soils and within the clay of the existing dam were calculated using the approximate method proposed by Taylor, 1948 as outlined by T. W. Lambe & R. V. Whitman (6). Fig. F.4 and F.5 illustrate the results of the analyses. Calculations are included in Appendix F to this report. In general, it is concluded that during the construction period of Stage 3,70% of the total settlement expected to occur in the foundation soil; and, 30% of the total settlement expected to occur in the clay of the existing dam will have taken place.

In addition to settlements of the foundation and existing dam clay, there are settlements associated with the proposed embankment itself. In this case, it is anticipated that settlements of the bottom ash shell will take place during construction. Settlements of the upstream shell or clay core are estimated to be approximately 2½ inches. Literture references (10) indicate that for this type of fill about 15% of the total settlement will occur as the fill is being placed; and, that the time for all of the settlement to occur will be approximately 1½ years.

7.1.3 Design Shear Strengths

The selection of the design shear strength of the different types of material follows, in general, the procedures and terminology outlined in the Department of the Army Corps of Engineers, Engineer Manual (7).

During the selection of the design shear strength the shape of the stress-strain curves as well as the stress-strain history of the soils have been considered. Since undisturbed foundation soils and compacted soils do not show a significant drop in shear or deviator stress after peak stresses are reached, the design shear strength have been chosen as (a) the peak shear stress in CD tests, (b) the peak deviator stress, or (c) the deviator stress at 20 percent strain where large deformation of the soils have taken place. While design shear strengths will generally correspond to either UU or CU and CD test conditions, intermediate strength values were selected where appropriate. For each embankment zone and foundation layer, design shear strengths were selected such that test values may exceed the design values. In some cases, however, the design shear strength for the various zone and layer was greater than the lowest test value for the zone being considered.

Intermediate design shear strengths were estimated by interpolating between the envelopes on the basis of the estimated degree of consolidation where the degree of consolidation is expected to be intermediate between the UU and CU test results. Results of the settlement analyses were used to assist in determining the degree of consolidation.

An embankment and its foundation are subjected to shear stresses imposed by the weight of the embankment and by pool fluctuations, seepage, or earthquake forces. The cases for which stability analyses was performed are presented as follows:

Case I: End of Construction. In an embankment composed partially of impervious soils placed at water contents higher than those corresponding to ultimate water contents after complete consolidation under the imposed loading, pore pressure will be induced because the soil cannot consolidate readily during the construction period. Where this is indicated, the use of UU shear strengths implies that pore water pressures occurring in laboratory tests satisfactorily approximate field pore water pressures. Where consolidation during construction is significant, however, its effect can be estimated by selecting strength values intermediate between UU and CU. This condition is evaluated for both the upstream and downstream slopes.

Design Shear Strengths for Case I are presented on Table No. 7.1. Fig. F.6 through F.12 illustrate the selection of the design shear strengths.

TABLE 7.1

SUMMARY OF DESIGN SHEAR STRENGTHS AT MAIN DAM
CASE I: END OF CONSTRUCTION

DAM ZONE	DESIGN SHEAR STRENGTH	COMMENTS
* FOUNDATION SOIL UNDER EXISTING BERM UNDER EXISTING DAM*	 $0.7 + \sigma \tan 13^\circ$ $1.14 + \sigma \tan 11^\circ$	Kc=1;α = 90^0
* FOUNDATION SOIL UNDER EXISTING BERM UNDER EXISTING DAM*	 $0.7 + \sigma \tan 13^\circ$ $0.82 + \sigma \tan 12^\circ$	Kc=1;α = 45°
* FOUNDATION SOIL UNDER EXISTING BERM UNDER EXISTING DAM*	 $1.55 + \sigma \tan 9^\circ$ $1.65 + \sigma \tan 9^\circ$	Kc=1.75;α = 45°
* CLAY IN EXISTING DAM	$1.4 + \sigma \tan 6.0^\circ$	Kc=1;α = 90°
* RANDOM ROCKFILL	σ' tan 32°	--
* BOTTOM ASH	σ' tan 38°	Kc=1
* COMPACTED CLAY FROM PROPOSED BORROW	0.48	Mc= +3%

* (confining pressure $\sigma \geq 5$ tsf)

Case II and III: Sudden Drawdown

Embankments may become saturated by seepage during prolonged high reservoir stages. If subsequently, the reservoir pool is drawn down faster than pore water can escape, excess pore water pressures and unbalanced seepage forces result. Shear strengths to be used in Cases II and III shall be based on the minimum of the combined CU and CD envelopes. In general, analyses for these cases are based on the conservative assumptions that (1) pore pressure dissipation does not occur during drawdown and (2) the water surface is lowered instantaneously from maximum pool (Case II) or spillway crest elevation (Case III) to the minimum pool elevation. Because of the use and operation of Horseford Creek fly ash pond,this condition will not be present. Therefore, stability of the dam under cases II and III has not been considered.

Case IV: Partial Pool. Analyses of the upstream slope for intermediate reservoir stages assume that a condition of steady seepage has developed at these intermediate stages. This condition is examined for the upstream slope only.

Partial pool is critical only for cases where submerging the toe portion of a failure surface reduces F.S. due to reducing the effective weight and, thereby, shear resistance of that portion. Generally, critical surface for upstream slope failure is along the upstream slope face of an impervious sloping core.

Since at Horseford Creek Dam water elevation is only increased following an increase in the elevation of fly ash deposited on the face of the embankment, partial pool is not expected to be a critical condition.

Case V: Steady Seepage with Maximum Storage Pool. A condition of steady seepage from the maximum water storage level that can be maintained sufficiently long to produce a condition of steady seepage throughout an embankment is critical for downstream slope stability.

Case VI: Steady Seepage with Surcharge Pool. The case where a steady seepage condition exists in an embankment and an additional horizontal thrust is imposed by a surcharge pool should also be examined for downstream slope stability.

Design shear strengths for cases IV, V and VI are presented on Table No. 7.2. Fig. F.12 through F.19 illustrate the selection of the design shear strengths.

TABLE 7.2

SUMMARY OF DESIGN SHEAR STRENGTHS AT MAIN DAM

CASE IV: PARTIAL POOL
CASE V: STEADY SEEPAGE WITH MAXIMUM STORAGE POOL
CASE V1: STEADY SEEPAGE WITH SURCHARGE POOL

DAM ZONE	DESIGN SHEAR STRENGTH (TSF)	COMMENTS
* FOUNDATION SOIL: UNDER EXISTING BERM UNDER EXISTING DAM*	σ' tan 25° 0.5 + σ' tan 23°	Kc=1; α = 90°
* FOUNDATION SOIL: UNDER EXISTING BERM UNDER EXISTING DAM*	σ' tan 27° 0.4 + σ' tan 23°	Kc=1; α = 45°
* FOUNDATION SOIL: UNDER EXISTING BERM UNDER EXISTING DAM*	σ' tan 30° σ' tan 30°	Kc=1.75;α=45°
* CLAY IN EXISTING DAM	σ' tan 25°	
* RANDOM ROCKFILL	σ' tan 24°	
* BOTTOM ASH	σ' tan 38°	
* COMPACTED CLAY	σ' tan 27°	Mc= -1% to+2%

* (Confining pressure $\sigma \geq 5$ tsf)

<u>Case VII: Earthquake</u>. Much research is in progress on the behavior of earth dams subjected to earthquake shocks, and new analytical methods for evaluating seismic effects are being developed. However, for this design, the traditional approach was used. This assumes that the earthquake imparts an additional horizontal force F_h acting in the direction of potential failure. The arc or set of planes found to be critical without earthquake loading is used with this added driving force to determine the factor of safety for Cases I, VI, V, VI. The horizontal seismic force is equal to the mass involved times the horizontal acceleration, i.e.

$$F_h = \frac{W}{g} a_h$$

The total weight of the sliding soil mass W should be based on saturated unit weights below the saturation line and moist unit weights above the line. Selection of the seismic coefficient should be based on the degree of seismic activity in the region in which the dam is to be built. Based upon the classification of the different regions in the USA, the location of this dam is placed within Zone One. Seismic loading coefficient generally associated with Zone One is 0.05g.

7.1.4 Stability Analyses

The methods of analyzing the stability of earth and rock fill embankments, outlined in Appendix G are of the circular arc and sliding wedge methods. The circular arc method is generally more applicable for analyzing essentially homogenous earth dams and dams on thick deposits of fine grained materials, whereas the wedge method is generally more applicable to rock fill dams on firm foundations and to earth dams on foundation containing one or more weak layers. In addition, the infinite slope method is used to some extent to supplement the results of the circular arc or wedge method. These methods provide a uniform basis for evaluating alternative designs, and the sensitivity of the Factor of Safety to variations on the shear strength.

<u>Downstream Stability</u>:

The downstream shell of the proposed raising will be constructed of bottom ash and graded to a slope of 1-3/4H:1V. The stability of the downstream shell was evaluated for the applicable conditions using the computer program, known commercially as XSTABL, developed by Interactive Software Designs Inc. This program allows for a systematic search for the minimum factor of safety. Wedge type foundation failure surfaces were analyzed using a random technique for generating sliding block surfaces. The active and passive portions of the sliding surfaces were generated according to the Rankine Theory. In addition, an "infinite slope" type of analyses was performed to evaluate the stability of the shell with regard to shallow failures within the bottom ash material. A summary of the factors of safety associated with the different design conditions considered, is presented on Table 7.3. Summaries of the analyses are presented on Fig. F.20 through F.25. Data associated with the complete stability analyses of the downstream shell is presented in Appendix G of this report. It should be pointed out that for the end of construction case, water levels in the reservoir, of 650, 660 and 670 were considered as the possible water level at the time of completion of the proposed construction.

TABLE 7.3

SUMMARY OF MINIMUM FACTORS OF SAFETY FOR DOWNSTREAM SLOPE
MAIN DAM

DESIGN CONDITION	MINIMUM FACTOR OF SAFETY		COMMENTS
	STATIC	EARTHQKE	
END OF CONSTRUCTION	1.6	1.4	STRENGTH PARAMETERS BASED UPON Kc=1;α = 90°
	1.6	1.4	STRENGTH PARAMETERS BASED UPON Kc=1; α = 45°
	2.0	1.7	STRENGTH PARAMETERS BASED UPON Kc=1.75; α = 45°
STEADY SEEPAGE WITH MAXIMUM STORAGE POOL	1.5	1.2	STRENGTH PARAMETERS BASED UPON Kc=1; α = 90°
	1.5	1.3	STRENGTH PARAMETERS BASED UPON Kc=1; α = 45°
	1.7	1.4	STRENGTH PARAMETERS BASED UPON Kc=1.75; α = 45°
STEADY SEEPAGE WITH SURCHARGE POOL	1.4	1.1	STRENGTH PARAMETERS BASED UPON Kc=1; α = 90
	1.5	1.2	STRENGTH PARAMETERS BASED UPON Kc=1; α = 45°
	1.6	1.3	STRENGTH PARAMETERS BASED UPON KC=1.75; α = 45°
INFINITE SLOPE	1.4	1.2	BASED UPON F.S.=$\frac{\tan \phi}{\tan \theta}=\frac{0.781}{1/1.75}$

Upstream Stability

The upstream shell for the Stage 3 raising will be constructed using clay from the proposed borrow site and graded at a slope of 2-3/4H:1V. The upstream shell will also perform as the impervious core of the dam from its present crest, elevation 675, to elevation 711. The stability of the downstream slope of the dam was evaluated for the applicable conditions using the XSTABL computer program. Circular type failure surfaces were analyzed using Janbu's Method and a random technique for generating sliding surfaces. In addition, an "infinite slope" type of analysis was performed to evaluate the stability of the clay core with regard to shallow failures within the compacted clay material. A summary of the factors of safety, associated with the different design conditions considered, is presented on Table No. 7.4. Summaries of the analysis are presented on Fig. F.26 through F.35. Data associated with the presented stability analyses of the upstream slope is submitted in Appendix G of this report. It should be pointed out that for the end of construction case water levels, is the reservoir, of 650, 660, and 670 were considered similarly for the partial pool case water levels, in the reservoir, of 680, 690 and 700 were studied.

TABLE 7.4

SUMMARY OF MINIMUM FACTORS OF SAFETY FOR UPSTREAM SLOPE
MAIN DAM

DESIGN CONDITION	MINIMUM OF FACTOR SAFETY		COMMENTS
	STATIC	EARTHQKE	
END OF CONSTRUCTION	2.4	2.1	STRENGTH PARAMETERS BASED UPON Kc=1; α = 90°
	2.4	2.1	STRENGTH PARAMETERS BASED UPON Kc=1; α =45°
	2.4	2.1	STRENGTH PARAMETERS BASED UPON Kc=1.75; σ = 45
PARTIAL POOL	1.5	1.1	STRENGTH PARAMETERS BASED UPON Kc=1; α =90°
	1.5	1.1	STRENGTH PARAMETERS BASED UPON Kc=1; α =45
	1.5	1.1	STRENGTH PARAMETERS BASED UPON Kc=1.75; σ = 45
INFINITE SLOPE	1.4	1.1	BASED UPON F.S. $\frac{\tan \phi'}{\tan \theta} = \frac{0.51}{1/2.75}$

7.2 Saddle Dam

The Geotechnical analyses considered the location of the seepage line at different water elevations in the reservoir, settlement, and the results of the strength tests in the selection of the design parameters associated with the different stability conditions for both the upstream and downstream slopes. A discussion of these items is presented in the following paragraphs.

7.2.1 Seepage

In the seepage through an earth dam, the upper boundary or uppermost flow line (known as the line of seepage) is not known, but must be found. Among the available solutions for seepage with a free surface, it has been decided to follow asagrande's solution (5) for the different levels of water during the operation of the dam. In this way, the seepage line will meet a vertical chimney drain ($\alpha = 90°$). The line of seepage, in this case, was found by using a = 3/4 Yo; Yo = $(h^2 + d^2)^{1/2}-d$. Fig. F.36 illustrates the location of the seepage lines for several water levels of the reservoir as estimated using the above mentioned solution. Calculations associated with the solution of these equations are included in Appendix G of this report.

7.2.2 Settlement

A majority of the downstream shell of the saddle dam is founded in rock. The upstream clay core of this dam is founded on as much as 17 feet of very stiff to hard sandy clay. Settlement of the foundation soil due to the weight of the embankment have been estimated to be one inch. It is believed, therefore, that settlements associated with the proposed embankment are mostly limited to those settlements occurring within itself. In this case it is anticipated that settlements of the bottom ash shell will take place during the construction. Settlements of the upstream shell or clay core are estimated to be approximately 14 inches. Literature references (10) indicate that for this type of fill about 15% of the total settlement will occur as the fill is being placed; and that the time for all of the settlement to occur will be approximately 6½ years.

7.2.3 Design Shear Strengths

The materials used in the construction of the saddle dam will consist of bottom ash and clay from the proposed borrow area. The local overburden soil will provide the foundation soil. With the exception of the overburden soil, the design shear strength of the construction materials for the saddle dam have been presented in Section 7.1.3. Therefore, the discussion in this section will only refer to the overburden soil.

The procedures to select the design shear strength of the overburden soil at the saddle dam closely followed the procedures discussed in Section 7.1.3 for the soils at the main dam. For the saddle dam, as for the main dam, the stability analyses considered cases I, IV, V, VI and VII. The design shear strengths of the overburden soils associated with each of these cases is presented as follows:

CASE I: END OF CONSTRUCTION - S = 1.4 tsf

CASE IV: PARTIAL POOL ELEVATION - $S = \sigma' \tan 34^\circ$

CASE V: STEADY SEEPAGE WITH MAXIMUM STORAGE POOL
$S = \sigma' \text{ Tan } 34^\circ$

CASE VI: STEADY SEEPAGE WITH SURCHARGE POOL-$S = \sigma' \text{Tan } 34^\circ$

Fig. F.12 and F.19 illustrate the selection of the design shear strengths.

7.2.4 Stability Analyses

Downstream Stability

Similar to the main dam, the downstream shell of the saddle dam will be constructed using bottom ash and graded at a slope of 1-3/4 H:1V. The stability of the downstream shell was evaluated for the applicable conditions using the computer program XSTABLE. Wedge type foundation failure surfaces were analyzed using a random technique for generating sliding block failures. The active and passive portions of the sliding surfaces were generated according to the Rankine Theory. In addition, an "infinite slope" type analysis was performed to evaluate the stability of the shell with regard to shallow failures within the bottom ash material. A summary of the factors of safety associated with the different design conditions expected, is presented on Table No. 7.5. Summaries of the analyses are presented on Fig. F.38 through F.45. Data associated with the complete stability analyses of the downstream shell is submitted in Appendix F of this report.

TABLE No. 7.5

SUMMARY OF MINIMUM FACTOR OF SAFETY FOR DOWNSTREAM SLOPE
SADDLE DAM

DESIGN CONDITIONS	STATIC	EARTHQUAKE	COMMENTS
END OF CONSTRUCTION	1.8	1.6	
STEADY SEEPAGE MAXIMUM STORAGE POOL	1.8	1.5	
STEADY SEEPAGE SURCAHRGE POOL	1.7	1.4	
INFINITE SLOPE SOLUTION	1.4	1.2	$F.S. = \frac{\tan\phi}{\tan\theta} = \frac{0.70}{1/1.75}$

Upstream Stability

The upstream shell for the saddle dam will be constructed using clay from the proposed borrow site and graded to a slope of 2-3/4H:1V. The upstream shell will also perform as the impervious core of the dam from to the present ground surface elevation of 675 to elevation 711. The stability of the downstream slope of the dam was evaluated for the applicable conditions using the XSTABLE computer program. Circular type failure surfaces were analyzed using a random technique for generating sliding surfaces. In addition, an "infinite slope" type analysis was performed to evaluate the stability of the clay core with regard to shallow failures within the compacted clay. A summary of the factors of safety, associated with the different design conditions is presented on Table 7.6. Summaries of the analyses are depicted on Fig. F.46 through F.53. Data associated with the complete stability analyses of the upstream slope is submitted in Appendix G of this report. It should be pointed out that for the partial pool case water levels in the reservoir of 680, 690 and 700 were studied.

TABLE 7.6

SUMMARY OF MINIMUM FACTOR OF SAFETY FOR UPSTREAM SLOPE
SADDLE DAM

DESIGN CRITERIA	STATIC	EARTHQUAKE	COMMENTS
END OF CONSTRUCTION	2.2	1.9	
PARTIAL POOL	1.5	1.1	
INFINITE SLOPE SOLUTION	1.4	1.1	$F.S. = \frac{\tan\phi}{\tan\theta} = \frac{0.51}{1/2.75}$

7.3 Spillway Pipe Overburden Analyses

A Spangler type analysis for the spillway pipe has been performed for the proposed Stage 3 raising. These results are shown in Appendix F. The analyses considered only the portion of the pipe which will support an overburden load greater than it has previously experienced, since the pipe construction and placement was consistent throughout its total length. The results of the analysis indicate that for the Stage 3 rising, the pipe will acceptably support the additional overburden load.

Hydraulic analysis for this pipe shows that it will not flow under a pressurized condition. An inspection program to ensure that the pipe continues to perform satisfactorily will be performed as part of the general inspection of the dam on an annual basis. The inspection program is part of the operations and maintenance procedures for the dam.

7.4 Filter Criteria

Many of the problems associated with the design of adequate filters and drains stems from the needs for satisfying conflicting requirements (9).

1. Piping Requirements: The pore spaces in drains that are in contact with erodible soils and rocks must be small enough to prevent particles from being washed in or through them.

2. Permeability requirement: The pore spaces in drains must be large enough to impart sufficient permeability to permit seepage to flow freely and thus provide a high degree of control over seepage forces and hydrostatic pressures.

To comply with the piping requirements, it is intended to verify the relation between the poreholes of the proposed clay core materials and of the filter material (bottom ash). Grain distribution analyses of both the clay and bottom ash are presented on Fig. F.46. On the basis of these gradations compliance with Bertram, (1940), criteria will be investigated.

Criterion 1: The 15% size (D_{15}) of a filter material must be not more than four or five times the 85% size (D_{85}) of a protected soil. The ratio of D_{15} of a filter to D_{85} of a soil is called the piping ratio.

$$\frac{D_{15}\text{ (of filter)}}{D_{85}\text{ (of Soil)}} = \frac{0.12\text{ mm}}{1.3\text{ mm}} = 0.09 < 4\text{ or }5$$

Criterion 2: The 15% size (D_{15}) of a filter material should be at least four or five times the 15% size (D_{15}) of the protected soil.

$$\frac{D_{15}\text{ (of filter)}}{D_{15}\text{ (of Soil)}} = \frac{0.12\text{ mm}}{0.0013\text{ mm}} = 92 > 4\text{ or }5$$

To comply with the permeability requirements, it is intended to verify the relation between the permeability of the materials. As Taylor (1948) pointed out, when:

$\frac{K_f}{K_s} > 20$ to 25, the filter will discharge seepage freely.

where: K_f = Permeability of Filter Material
K_s = Permeability of Soil

Thus:

$$K_f = 10^{-3},\ K_s = 10^{-7} \Rightarrow \frac{K_f}{K_s} = \frac{10^{-3}}{10^{-7}} = 10^4 > 20\text{ to }25$$

It is therefore concluded that the bottom ash will satisfy all requirements to perform as a filter.

8.0 GEOTECHNICAL DESIGN

The plan and sections for the proposed raising of the main dam and the construction of the saddle dam are presented in the accompanying set of design drawings. A complete listing of drawings is included in the drawing's cover sheet. The AEPSC Civil Engineering Division Technical Specifications for material and construction are attached under separate cover. The geotechnical aspects of the key components of the project are discussed below.

8.1 Main Dam

The extension of the core zone for the raising will be constructed from clays of low to medium plasticity obtained from the proposed borrow area. The downstream shell and chimney drain will be constructed using bottom ash. The faces of the dam will be lined with riprap to minimize erosion. The raising of the dam will be constructed at a maximum rate of 15 feet per year. Furthermore, a maximum of 5 feet of material may be placed in a given construction period.

Cohesive soil will be placed to a final configuration of 2-3/4H:1V. Compaction will be accomplished at moisture contents ranging from -1% to +2% of the optimum moisture content. The soil shall be compacted in uniform, thin lifts to a unit dry weight equal to at least 95% of the maximum unit dry weight as determined in the laboratory by means of ASTM D698 (Standard Proctor Compaction Test). Bottom ash from Big Sandy Plant will continue to be used as the chimney drain and downstream shell to its final configuration of 1-3/4H:1V. This material will be compacted in uniform, thin lifts to at least 70% relative density determined by ASTM D 4253-83 and ASTM D4254-83. No moisture control will be required when compacting bottom ash.

At the locations where new construction meets the abutments or existing permanent features of the site, the new materials will be benched into the existing grade to allow placement in horizontal lifts and compaction control. Any feature of the site which may infringe into areas of proposed construction will be relocated as indicated on the drawings.

8.2 Saddle Dam

The impervious zone of the saddle dam will be constructed from clays of low to medium plasticity obtained from the proposed area. The downstream shell will be constructed using bottom ash. The faces of the dam will be lined with riprap. Construction of the saddle dam will require the filling of the existing emergency spillway which consists of a rock channel.

Cohesive soil will be placed to a final configuration of 2 3/4H:1V on the upstream slope. Compaction will be achieved at a moisture content ranging from -1% to +2% of the optimum moisture content. The soil shall be compacted in uniform thin lifts at a unit dry weight equal to at least 95% of the maximum unit dry weight as determined in the laboratory by means of ASTM D698 (Standard Proctor Compaction Test). Bottom ash from Big Sandy Plant will be used as the chimney drain and downstream shell to a final configuration of 1-3/4H:1V. This material will be compacted in uniform thin lifts to at least 70% relative density as determined by ASTM D 4253-83 and ASTM D 4254-83. No moisture control will be required when compacting bottom ash.

The existing emergency spillway will be filled with a fly ash/bottom ash stabilized mix, which is elastically compatible with the surrounding rock. This material will be placed to support the clay core on the area of the existing channel. The remaining of the channel will be filled with bottom ash as the downstream shell is constructed.

At the location where new construction meet the abutments, the new materials will be benched into the existing grade to allow placement in horizontal lifts and compaction control. Any feature of the site which may infringe into areas of proposed construction will be relocated as indicated on the drawings.

8.3 Emergency Spillway

The proposed emergency spillway will be constructed on the left abutment of the proposed saddle dam a few feet south of the existing emergency spillway. The spillway will consist of a rock cut channel approximately 100 foot wide and having a bottom graded at 2% slope. The walls of the rock cut will be graded at a slope equal to 1H:6V.

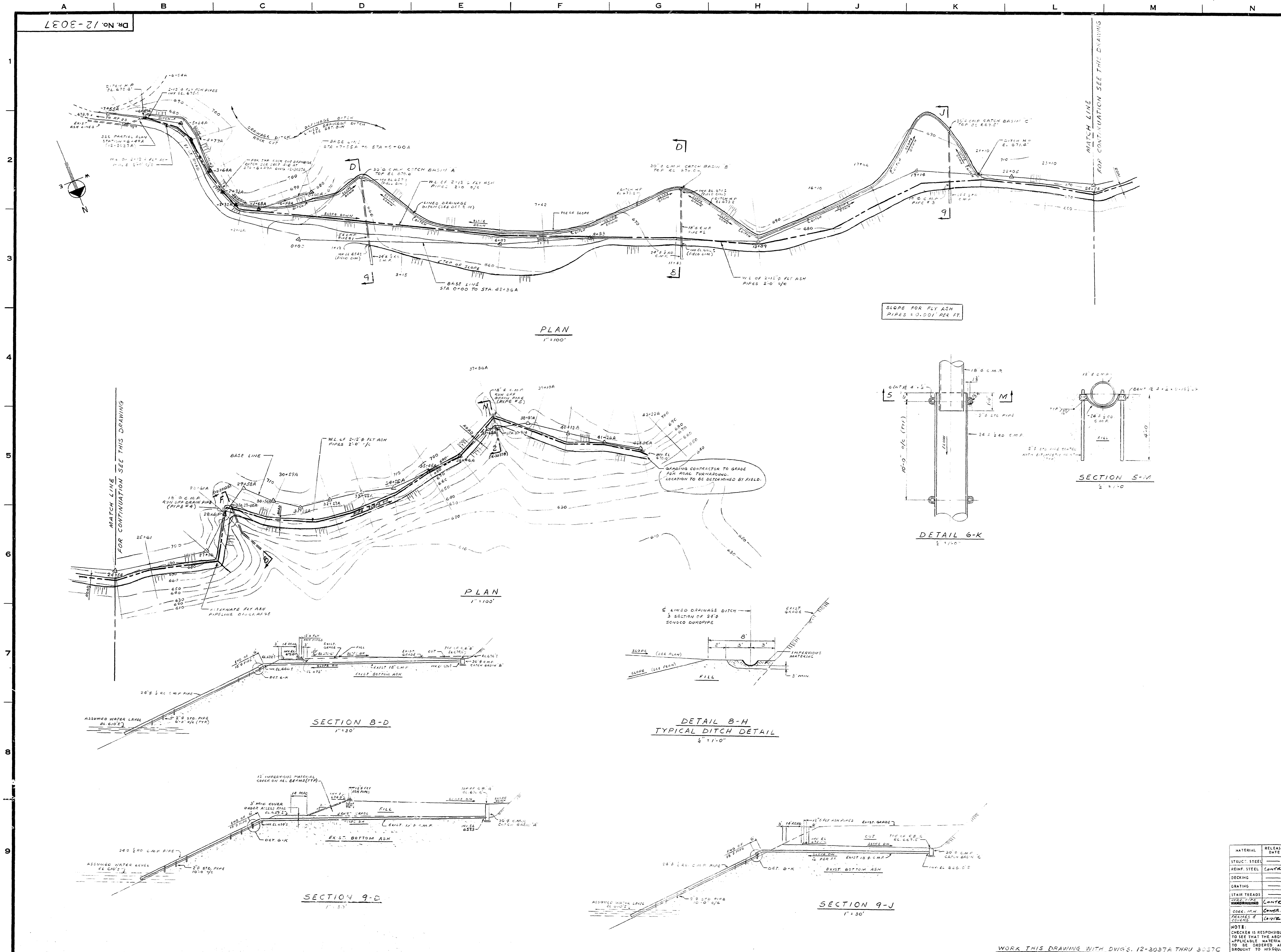
Rock excavated from the construction of the emergency spillway may be used as riprap provided it meets or exceeds commonly accepted criteria.

<u>ATTACHMENT C</u>

DESIGN DRAWINGS

ATTACHMENT C-1

STAGE I DRAWINGS



DATE	NO.	DESCRIPTION	APPD.
5/18/76	1	ADDED NOTE FOR ROAD TURNAROUND	

REVISIONS

KENTUCKY POWER CO.
BIG SANDY PLANT
BIG SANDY, KENTUCKY

FLY ASH STORAGE AREA
GRADING FOR EXTENSION
OF PIPE LINES

DR. NO. 12-3037-1

MATERIAL	RELEASE DATE
STRUCT. STEEL	—
REINF. STEEL	CONTR.
DECKING	—
GRATING	—
STAIR TREADS	—
CORR. PIPE ~~HANDRAILING~~	CONTR.
CORR. M.H.	CONTR.
FRAMES & COVERS	CONTR.

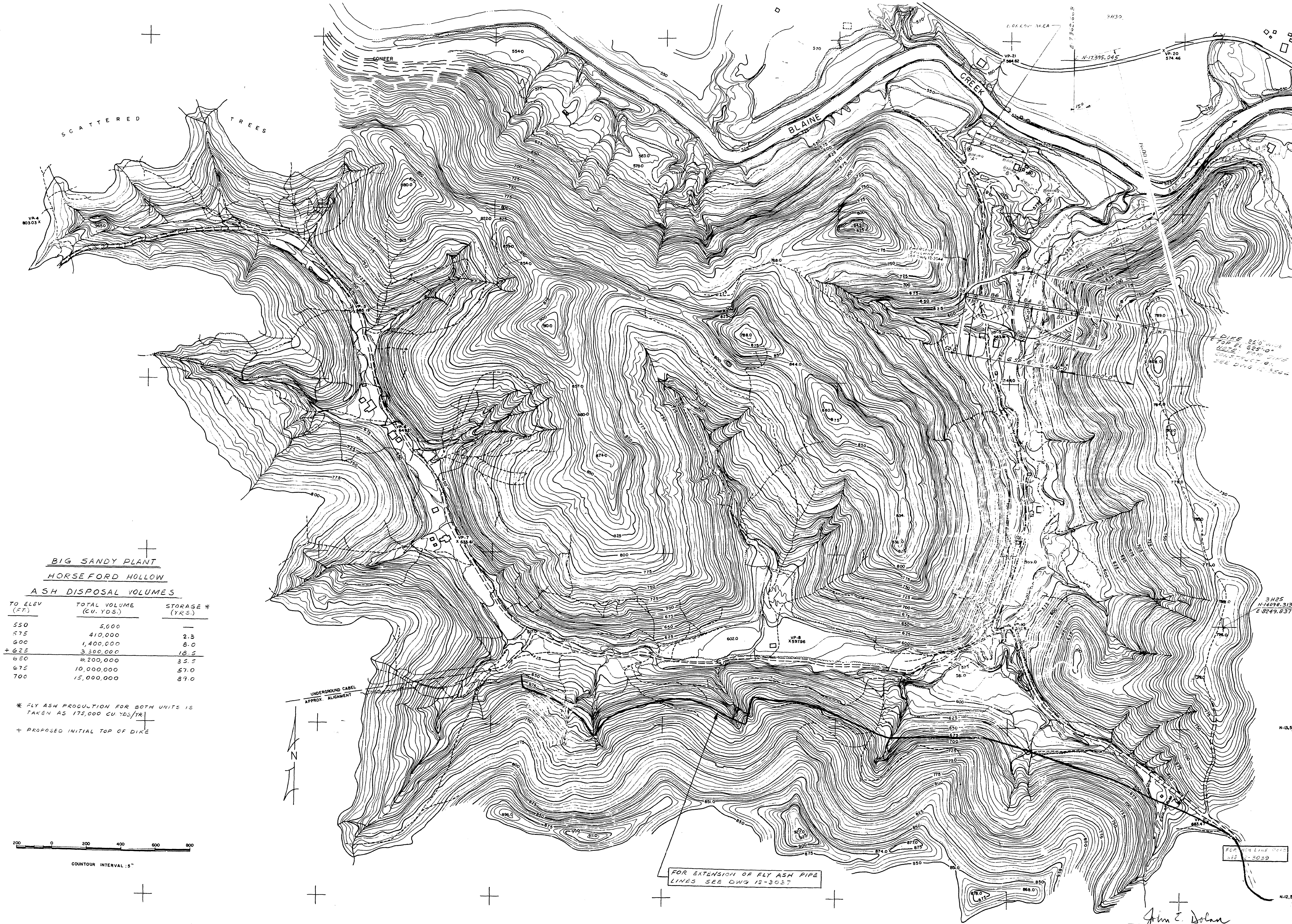
NOTE: CHECKER IS RESPONSIBLE TO SEE THAT THE ABOVE APPLICABLE MATERIALS TO BE ORDERED ARE BROUGHT TO HIS SQUAD LEADERS' ATTENTION.

ARCH. | ELEC | MECH. | STR.

SCALE: AS NOTED
DR. J.M.
CH.
SQ. LDR. C.H.M.
DATE: 3/10/76
APPROVED BY: ENGINEERING DIV; DESIGN DIV

AMERICAN ELECTRIC POWER SERVICE CORP.

DR. NO. 12-3038



BIG SANDY PLANT
HORSEFORD HOLLOW
ASH DISPOSAL VOLUMES

TO ELEV (FT.)	TOTAL VOLUME (CU. YDS.)	STORAGE * (YRS.)
550	5,000	—
575	410,000	2.3
600	1,400,000	8.0
+ 625	3,300,000	18.5
650	6,200,000	35.5
675	10,000,000	57.0
700	15,000,000	89.0

* FLY ASH PRODUCTION FOR BOTH UNITS IS TAKEN AS 175,000 CU. YDS./YR.

+ PROPOSED INITIAL TOP OF DIKE

PREPARED BY PHOTOGRAMMETRIC METHODS FROM AERIAL PHOTOGRAPHY TAKEN MARCH 30, 1967

VERTICAL AND HORIZONTAL CONTROL FURNISHED BY APPALACHIAN POWER CO.

DATE		DESCRIPTION	APPD.
9-24-68	1	Released for Construction	US

REVISIONS

KENTUCKY POWER CO.
BIG SANDY PLANT
BIG SANDY, KENTUCKY

FLY ASH STORAGE AREA
HORSEFORD CREEK
SH. 1 OF 3

DR. NO. 12-3038-1

ARCH. | ELEC. | MECH. | STR.

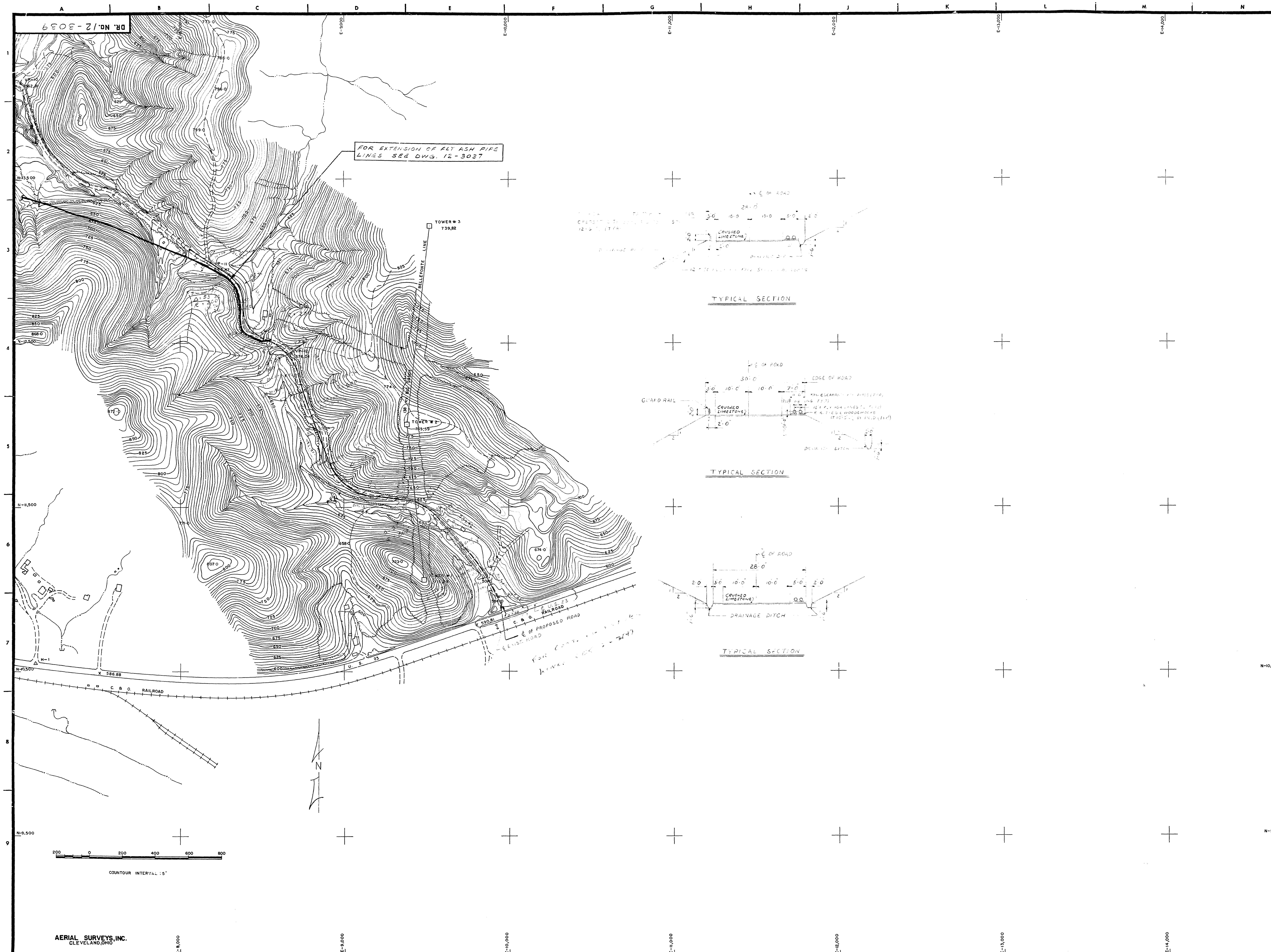
SCALE: 1" = 200'
DR. A.P./J.K.
CH. NB
SQ. LDR. M.E.
DATE: AUG. - 1967

APPROVED BY
ENGINEERING DIV.
DESIGN DIV.

John E. Dolan
JOHN E. DOLAN
KENTUCKY P.E. LICENSE #4801

AERIAL SURVEYS, INC.
CLEVELAND, OHIO

AMERICAN ELECTRIC POWER SERVICE CORP.
2 BROADWAY NEW YORK



PREPARED BY PHOTOGRAMMETRIC METHODS FROM AERIAL PHOTOGRAPHY TAKEN MARCH 30, 1967

VERTICAL AND HORIZONTAL CONTROL FURNISHED BY APPALACHIAN POWER CO.

DATE		DESCRIPTION	APPD.
9/24/68	1	Released for Construction	LWB

REVISIONS

KENTUCKY POWER CO.
BIG SANDY PLANT
BIG SANDY, KENTUCKY

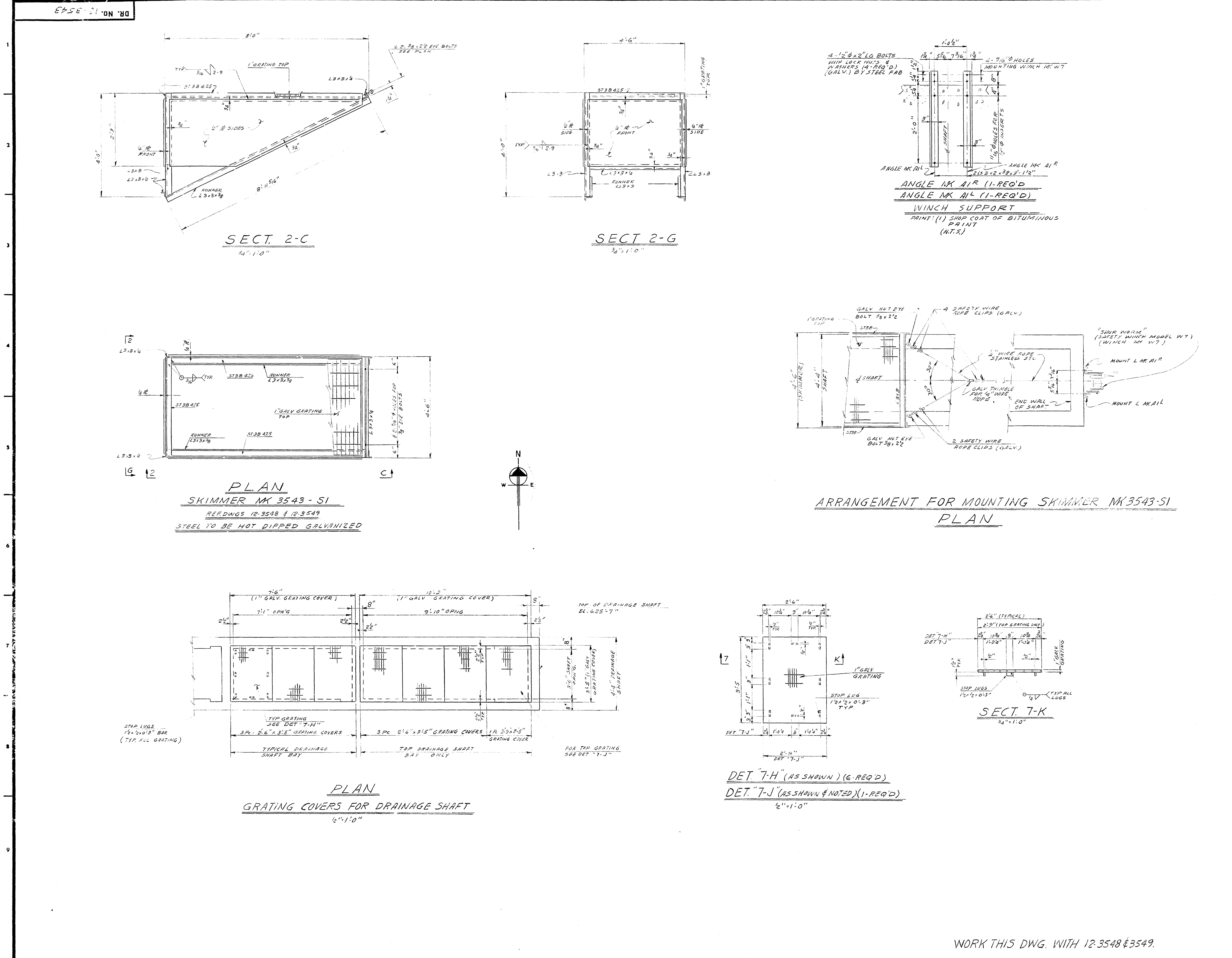
FLY ASH STORAGE AREA
HORSEFORD CREEK
SH. 2 OF 3

DR. No. 12-3039-1

ARCH. FAA | ELEC. FAJ | MECH. | STR.

SCALE: 1" = 200'
DR. A. PIAS.
CH. ME
SQ. LDR. ME
DATE: OCT. - 1967
APPROVED BY L. F. Smith LWB

AMERICAN ELECTRIC POWER SERVICE CORP.
2 BROADWAY NEW YORK



GENERAL NOTES

DO NOT SCALE THIS DWG.

ALL STEEL A-36
STEEL TO BE HOT DIPPED GALVANIZED

MATERIAL

STEEL BY: Huntington Steel & Supply
ORDER NO. 8609 Ox-5
1 - "SHUR WORM"
SAFETY WINCH, MODEL W-7
NUTEYE BOLTS, & 1/4" STAINLESS
STEEL WIRE ROPE, CLIPS, &
THIMBLE - BY: Madison Tool & Equip
ORDER NO: 10211 Ox-3
1" GALV. GRATING BY [illegible]
(PLAIN) Products Co

REFERENCE DRAWINGS

AS NOTED

DATE	NO.	DESCRIPTION	APPD.

REVISIONS

NOTE:
GRADE ELEVATION HAS BEEN ESTABLISHED AS ELEVATION 0'-0". TO CONVERT TO PLANT DATUM ADD 568'-0" TO ELEVATIONS SHOWN.

MATERIAL	RELEASE DATE
STRUCT. STEEL	11/27/68
REINF. STEEL	—
DECKING	—
GRATING	11/27/68
STAIR TREADS	—
HANDRAILING	—
WINCH, TIMBLE, ETC.	11/27/68

NOTE:
CHECKER IS RESPONSIBLE TO SEE THAT THE ABOVE APPLICABLE MATERIALS TO BE ORDERED ARE BROUGHT TO HIS SQUAD LEADERS' ATTENTION.

KENTUCKY POWER CO.
BIG SANDY PLANT
BIG SANDY, KENTUCKY

FLY ASH STORAGE AREA
DRAINAGE SHAFT
SKIMMER

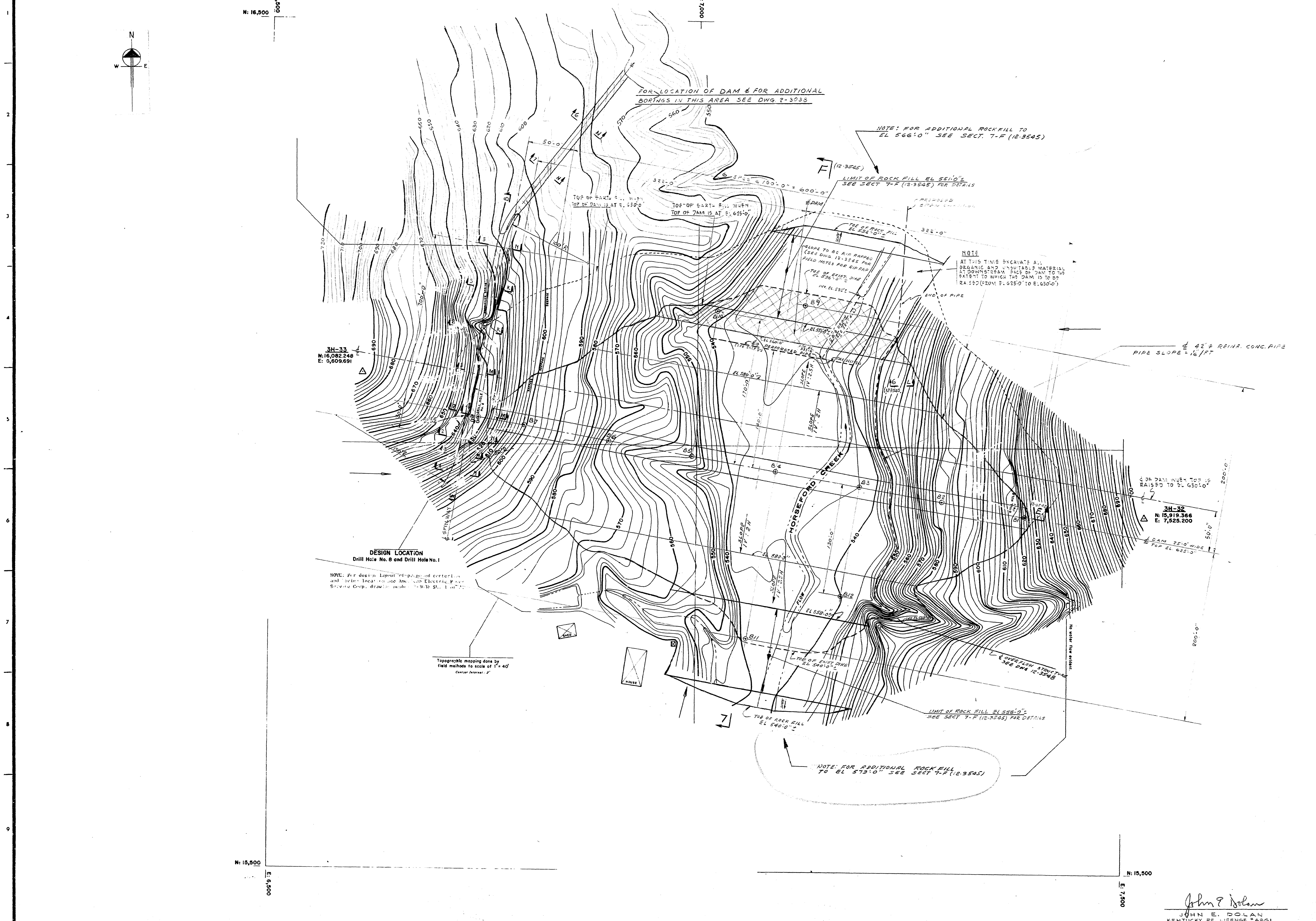
DR. No. 12 - 3543

ARCH. FWA ELEC. FAJ MECH. [illegible] STR.

SCALE: AS NOTED
DR. J.R.
CH. I.T.
SQ. LDR. M.E.
DATE: 11/27/68
APPROVED BY — ENGINEERING DIV.; DESIGN DIV.

AMERICAN ELECTRIC POWER SERVICE CORP.
2 BROADWAY NEW YORK

DR. NO. 12-3544



GENERAL NOTES

STRIP ABUTMENT AREAS TO BEDROCK.

QUANTITY OF MATERIAL

CUT — DETERMINE IN FIELD
FILL + 12% COMPACTION = 500,000 C.Y.

DATE	NO.	DESCRIPTION	APPD.
5/20/70	3	ADDED ROCKFILL NOTE	W.S.
2/5/69	2	~~SPILLWAY LENGTHENED~~ ADDED ROCK FILL @ TOE OF DIKE	W.S.
11/1/68	1	SPILLWAY RELOCATED	W.S.

REVISIONS

KENTUCKY POWER CO.
BIG SANDY PLANT
BIG SANDY, KENTUCKY

FLY ASH STORAGE AREA
DAM

DR. NO. 12-3544-3

ARCH. | ELEC. | MECH. | STR.

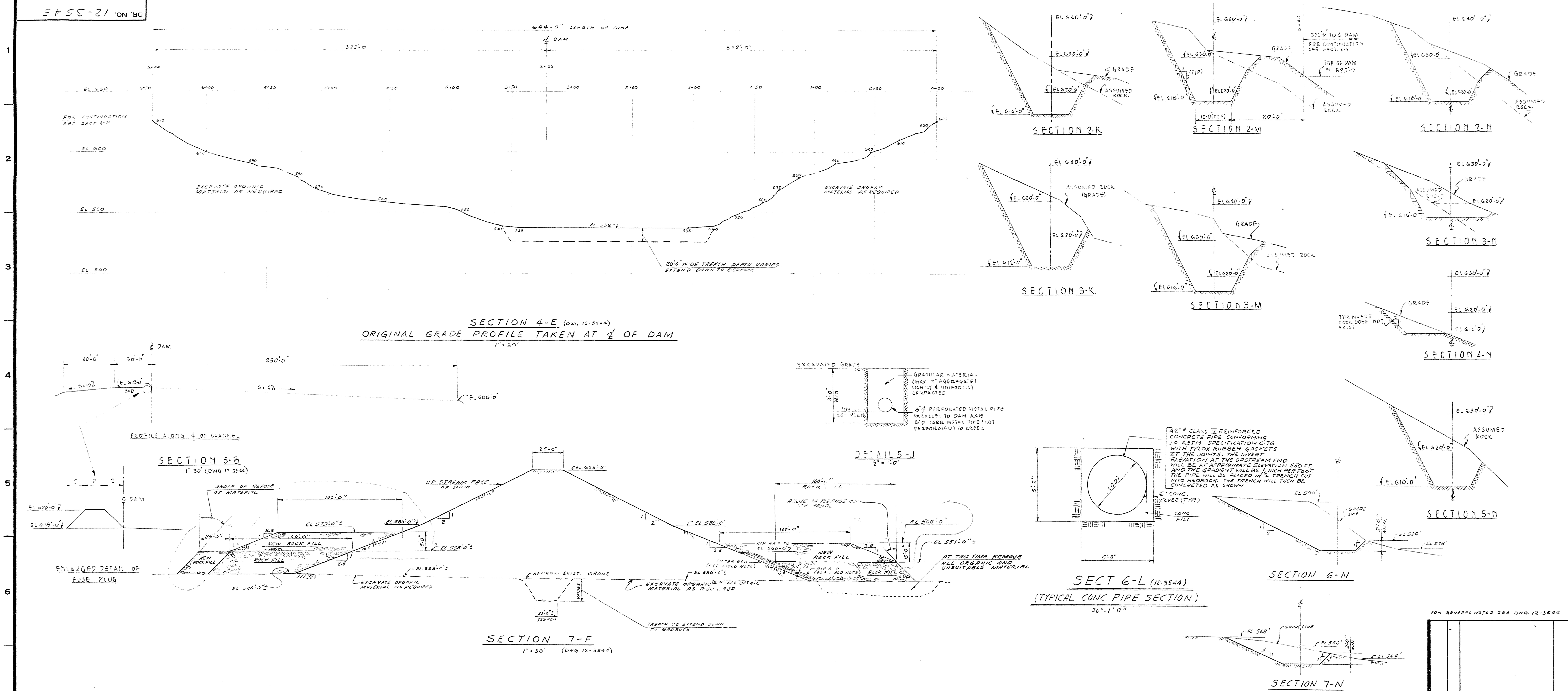
SCALE: 1" = 40'
DR. J. M.
CH. N. B.
SQ. LDR. M. E.
DATE: 9-24-68

APPROVED BY
ENGINEERING DIV. J.R. Strangle
DESIGN DIV. L. Smith

AMERICAN ELECTRIC POWER SERVICE CORP.
2 BROADWAY NEW YORK

John E. Dolan
JOHN E. DOLAN
KENTUCKY P.E. LICENSE #4891

WORK THIS DWG. WITH DWG 12-3545



FIELD NOTES - RIP RAP

RIP RAP SHALL SHALL BE PLACED AS SHOWN. A 9" GRAVEL BLANKET SHALL BE PLACED AS BASE FOR THE RIP RAP. THIS FILTER SHOULD BE REASONABLY WELL GRADED WITHIN THE FOLLOWING LIMITS:

SIEVE SIZE	PERCENT PASSING
2"	100
1½"	90 - 100
¾"	70 - 90
No. 4	35 - 60
No. 16	15 - 35
No. 100	0 - 5

THE RIP-RAP SHALL BE 18" IN THICKNESS & BE REASONABLY WELL GRADED FROM THE MINIMUM SIZE STONE PERMITTED WEIGHING APPROXIMATELY 50 LBS. TO THE MAXIMUM SIZE WEIGHING APPROXIMATELY 150 LBS. NEITHER THE BREADTH OR THE THICKNESS OF ANY PIECE OF RIP RAP SHALL BE LESS THAN ONE THIRD OF ITS LENGTH.
THE RIP RAP SHALL BE PLACED ON THE FILTER MATERIAL IN SUCH A MANNER AS TO PRODUCE A REASONABLY WELL GRADED MASS OF ROCK WITH A MINIMUM PRACTICAL PERCENTAGE OF VOIDS.
RIP RAP SHALL BE PLACED TO ITS FULL COURSE THICKNESS IN ONE OPERATION AND IN SUCH A MANNER SO AS TO AVOID DISPLACING THE GRAVEL BLANKET.

COMPACTION SPECIFICATIONS

THE FILL MATERIAL SHALL BE PLACED IN 9" THICK LAYERS BEFORE COMPACTION. EACH LAYER SHALL BE COMPACTED BY SIX COMPLETE COVERAGES OF THE SURFACE WITH SHEEPSFOOT ROLLERS. IN NO CASE, HOWEVER, SHALL THE COMPACTION ACHIEVED BE LESS THAN 95% OF "STANDARD PROCTOR" AS DETERMINED BY FIELD DENSITY TESTS.

SETTLEMENT RECORD

SETT POINT	DATE	ELEV.	DATE	ELEV	SETT.	DATE	ELEV	SETT	DATE	ELEV	SETT	DATE	ELEV.	SETT.	DATE	ELEV.	SETT.	DATE	ELEV.	SETT.

John E. Dolan
JOHN E. DOLAN
KENTUCKY P.E. LICENSE #4861

DATE	NO.	DESCRIPTION	APPD.
5/29/70	3	ADDED ROCK FILL TO SECT 7-F	W.S.
2/5/69	2	ADDED: ROCK FILL @ TOE OF DIKE; ADDED: SECT. 6-N & SECT. 7-N	W.S.
11/1/68	1	SECTIONS ADDED	W.S.

REVISIONS

KENTUCKY POWER CO.
BIG SANDY PLANT
BIG SANDY, KENTUCKY

FLY ASH STORAGE AREA
DAM
SECTION & PROFILE & SETTLING RECORD

DR. NO. 12-3545-3

ARCH. | ELEC | MECH. | STR.

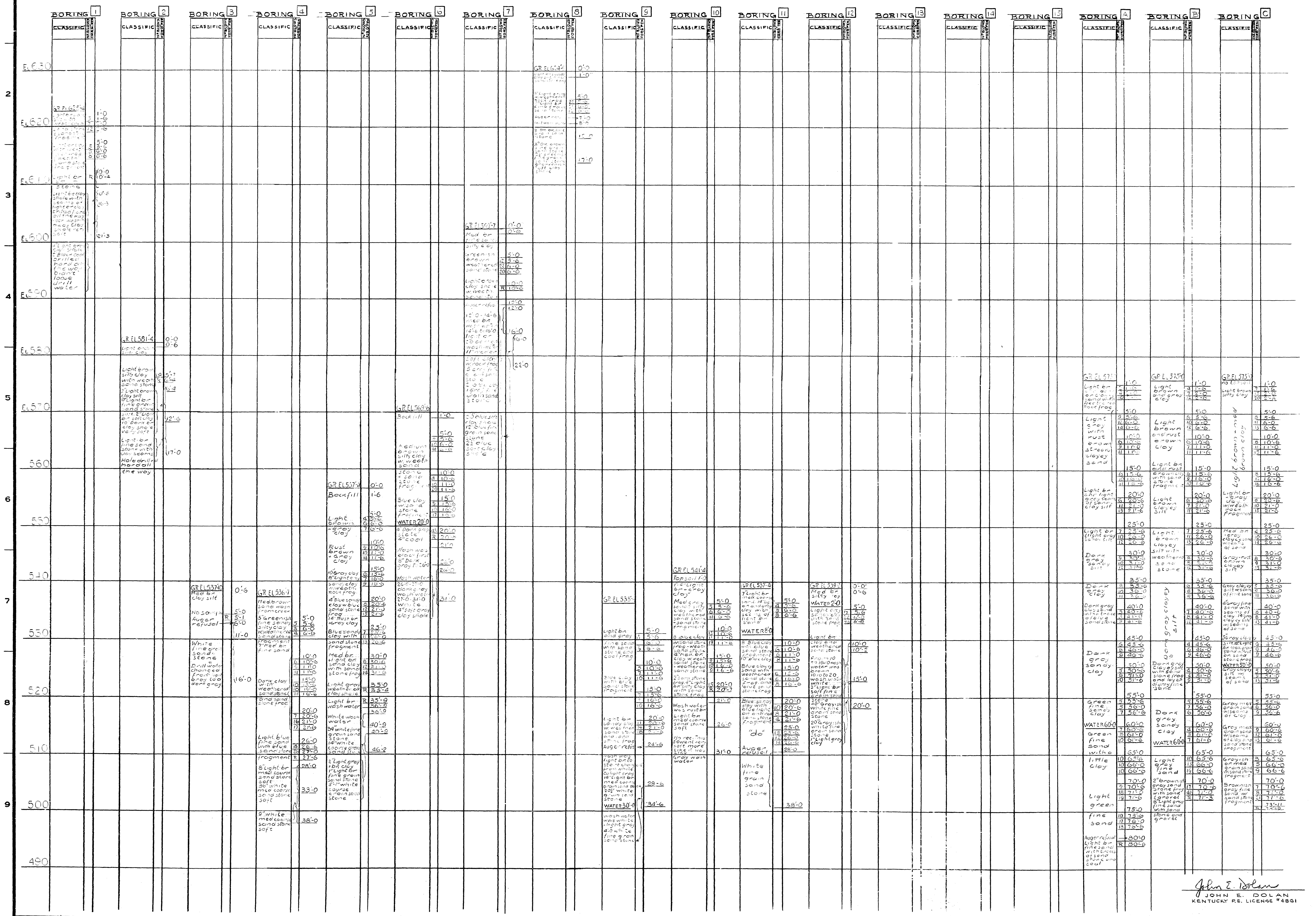
SCALE: 1" = 30'
DR. J.M.
CH. M.B.
SQ. LDR. M.E.
DATE: 9-24-68

APPROVED BY
ENGINEERING DIV
DESIGN DIV

AMERICAN ELECTRIC POWER SERVICE CORP.
2 BROADWAY NEW YORK

DR. NO. 12-3546

AMERICAN ELECTRIC POWER SERVICE CORPORATION BORING CLASSIFICATION



John E. Dolan
JOHN E. DOLAN
KENTUCKY P.E. LICENSE #4861

DATE	NO.	DESCRIPTION	APPD.
9-24-68	1	Released to Field	WS

REVISIONS

KENTUCKY POWER Co
BIG SANDY PLANT
BIG SANDY KENTUCKY

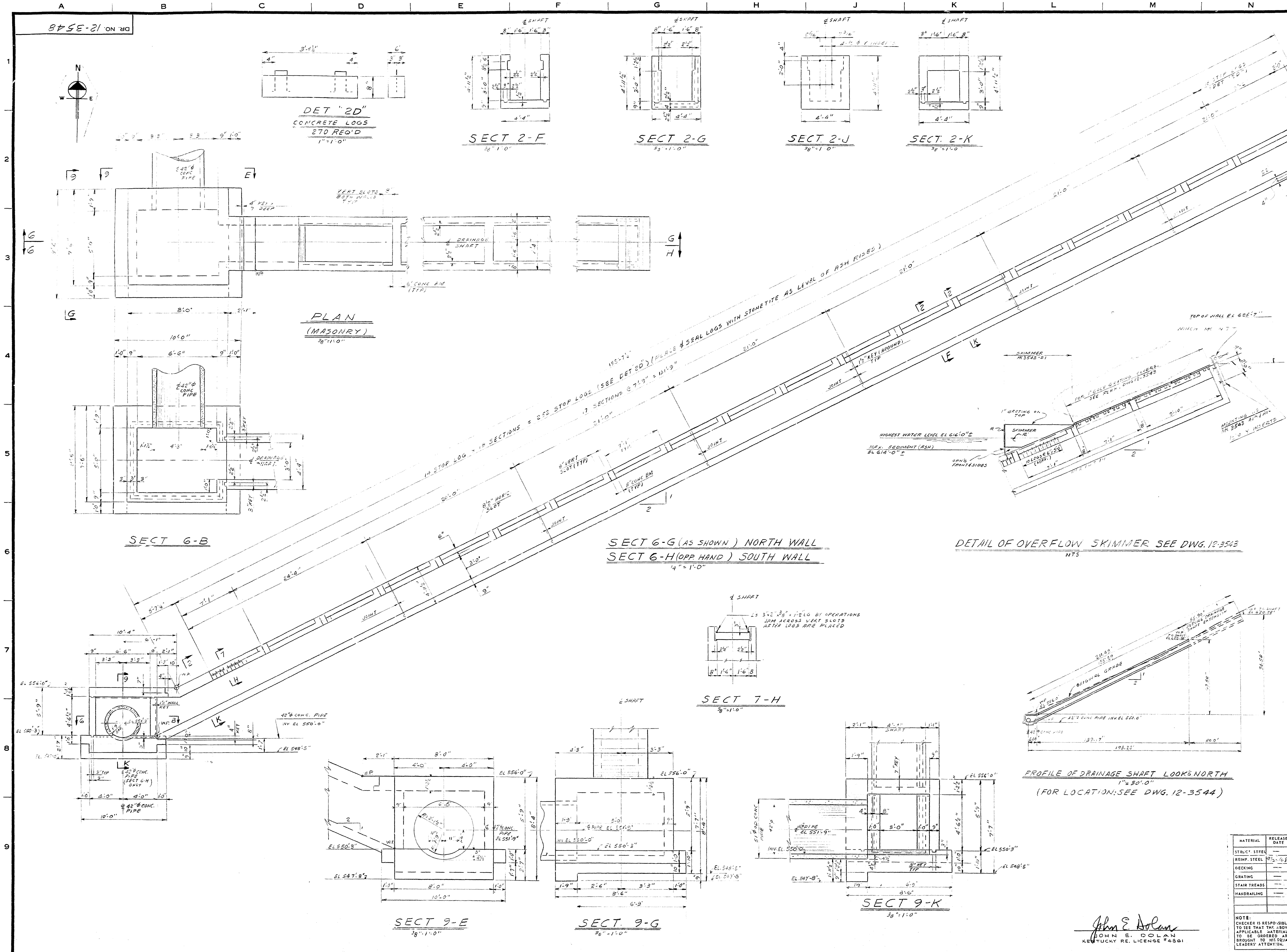
FLY ASH STORAGE AREA
LOG OF BORINGS

DR. NO. 12-3546

ARCH.		ELEC.		MECH.		STR.	

SCALE: —
DR. T.S.
CH. C.H.M.
SQ. LDR. M.E.
DATE: 5/21/68
APPROVED BY: E. F. Smith, WS

AMERICAN ELECTRIC POWER SERVICE CORP.
2 BROADWAY NEW YORK



MATERIAL	RELEASE DATE
STRUCT. STEEL	—
REINF. STEEL	9/12/68
DECKING	—
GRATING	—
STAIR TREADS	—
HANDRAILING	—

NOTE: CHECKER IS RESPONSIBLE TO SEE THAT THE ABOVE APPLICABLE MATERIALS TO BE ORDERED ARE BROUGHT TO HIS SQUAD LEADERS' ATTENTION.

DATE	NO.	DESCRIPTION	APPD.
10/14/68	1	DRAWING COVERS NOTE ADDED	

REVISIONS

KENTUCKY POWER CO.
BIG SANDY PLANT
BIG SANDY, KENTUCKY

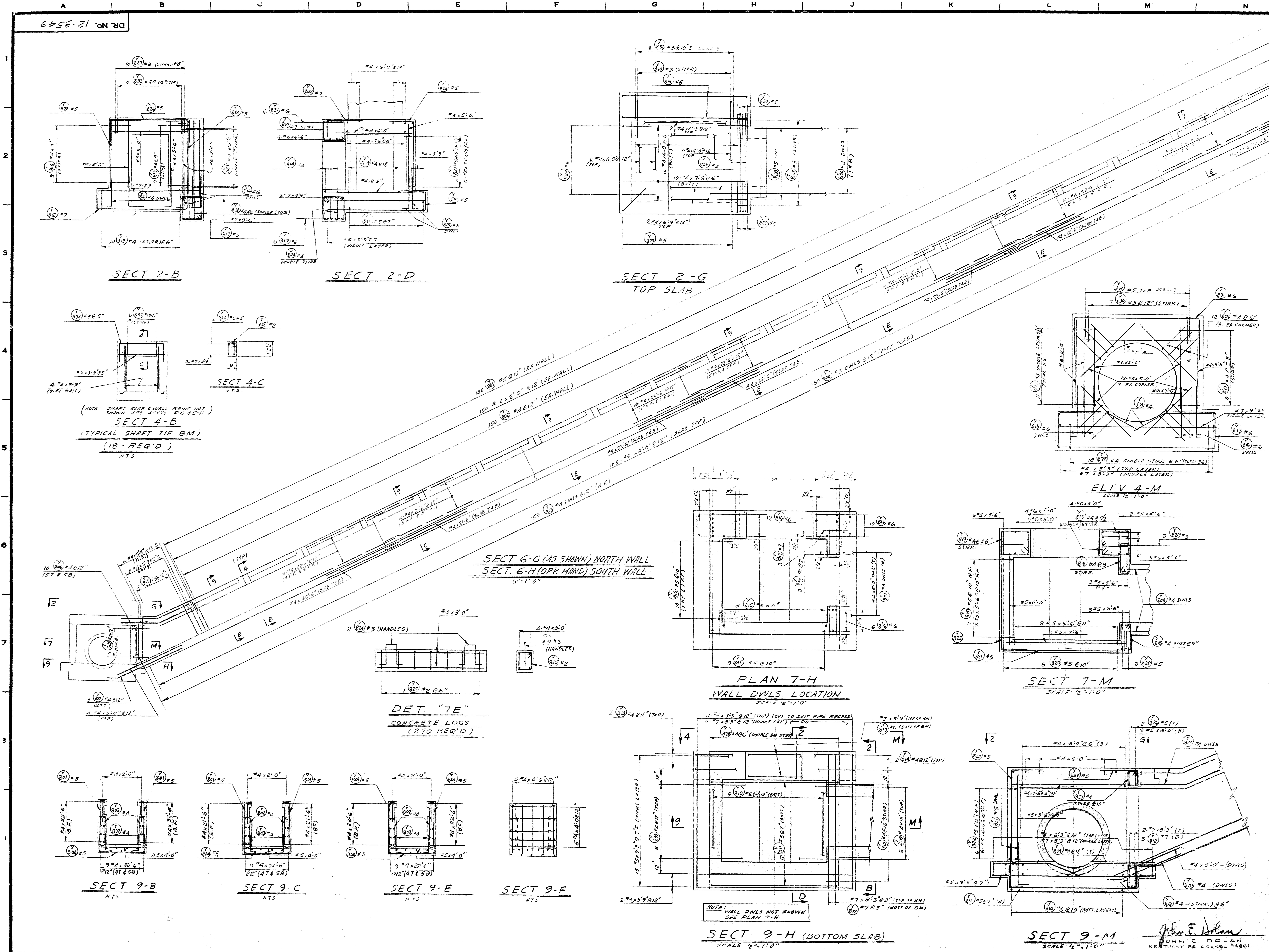
FLY ASH STORAGE AREA
DRAINAGE SHAFT
MASONRY

DR. NO. 12-3548-1

ARCH. | ELEC. | MECH. | STR.

SCALE: AS NOTED
DR. J.F.
ENGINEERING DIV.
DESIGN DIV.

AMERICAN ELECTRIC POWER SERVICE CORP.
2 BROADWAY NEW YORK



DATE	NO.	DESCRIPTION	APPD.

REVISIONS

NOTE:
GRADE ELEVATION HAS BEEN ESTABLISHED AS ELEVATION 0'-0". TO CONVERT TO PLANT DATUM ADD 568'-0" TO ELEVATIONS SHOWN.

KENTUCKY POWER CO.
BIG SANDY PLANT
BIG SANDY, KENTUCKY

FLY ASH STORAGE AREA
DRAINAGE SHAFT
REINFORCING

DR. No. 12-3549

ARCH. | ELEC. | MECH. | STR.

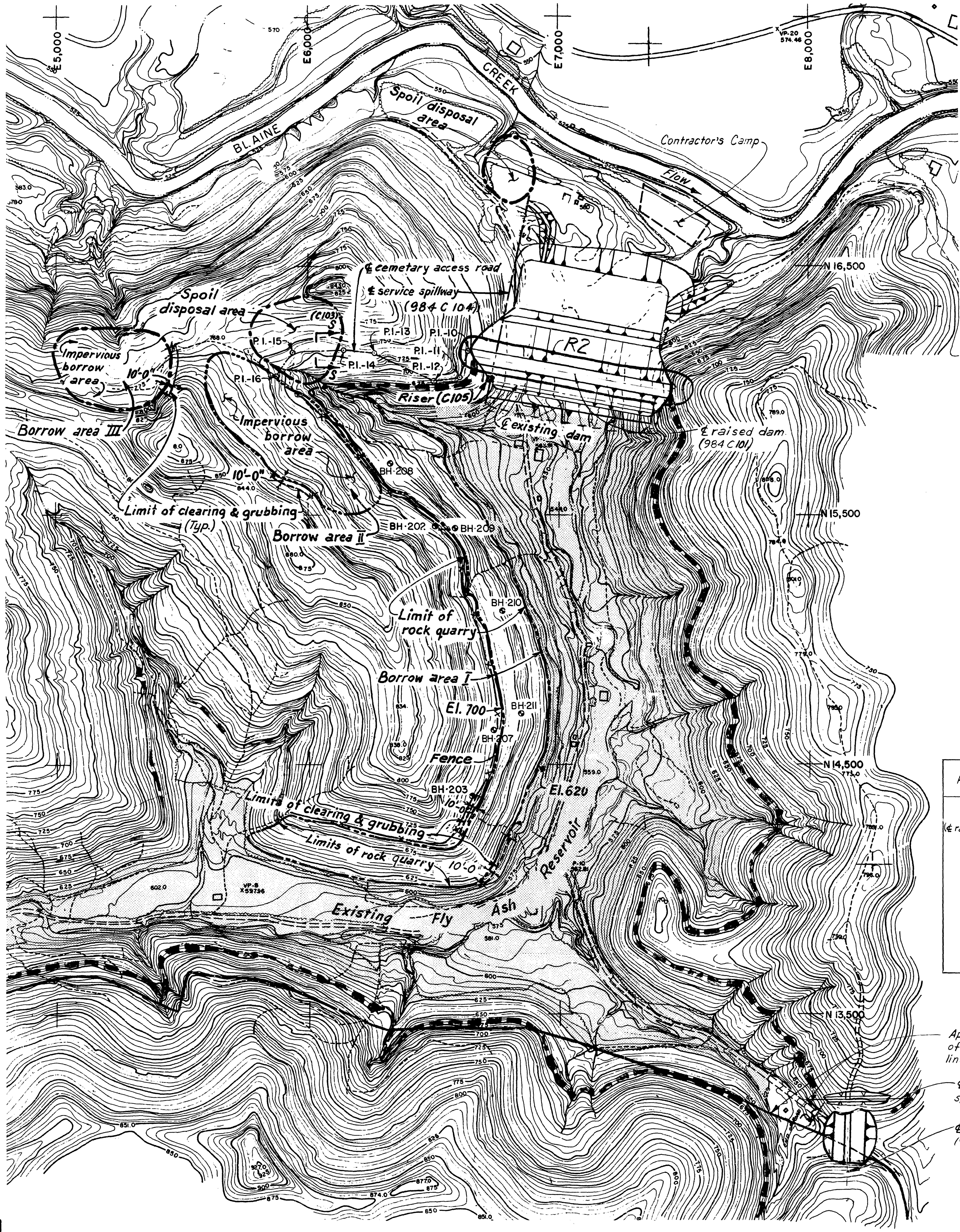
SCALE: AS NOTED
DR. J.P.
CH. I.T.
SQ. LDR. M.E.
DATE: 9/26/68

APPROVED BY
ENGINEERING DIV. HAF J. R. Stumph
DESIGN DIV. L. F. Smith

AMERICAN ELECTRIC POWER SERVICE CORP.
2 BROADWAY NEW YORK

ATTACHMENT C-2

STAGE II DRAWINGS

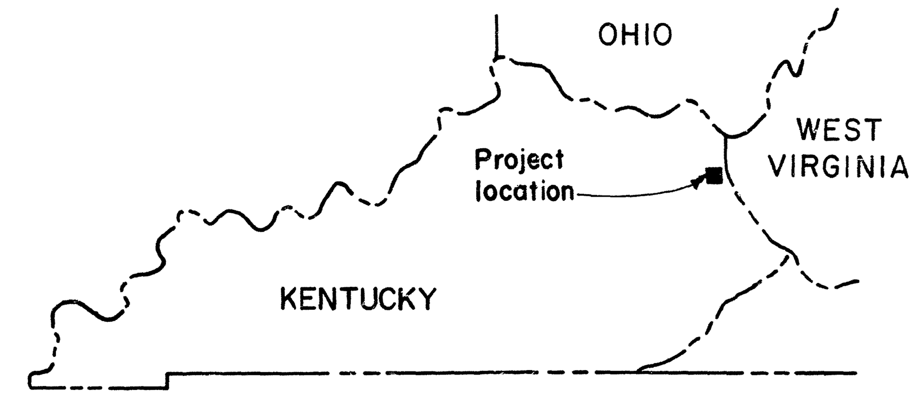


PLAN

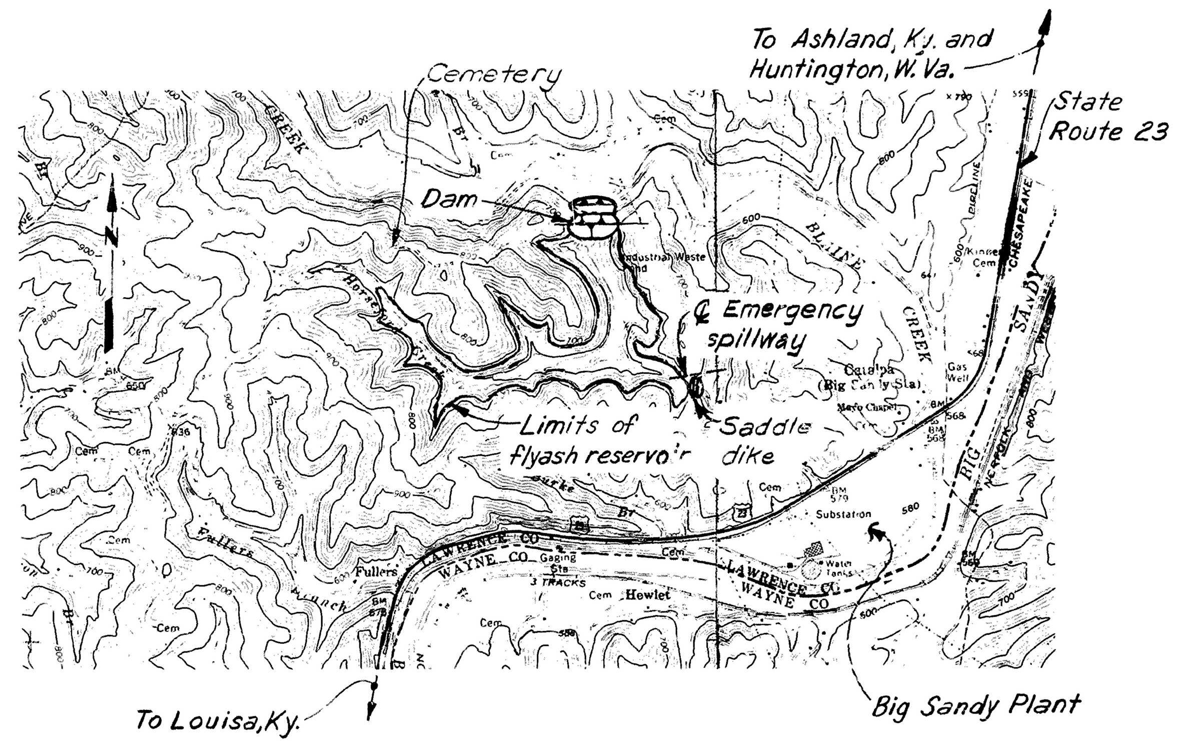
Approximate route of existing flyash lines. (To be relocated by others)

℄ emergency spillway (984 C103)

℄ saddle dike (984 C 103)



KEY MAP
N.T.S.



LOCATION MAP
1000 0 3000
1" = 2000'

Cemetary Access Road Alignment *

P. I.	Coordinates N	Coordinates E	Distance (Ft)	Bearing Angle
10 (℄ raised dam)	16,150.0	6,662.0		
			102.22	S 30° 34' 45" W
11	16,062.0	6,610.0		
			110.89	S 82° 44' 48" W
12	16,048.0	6,500.0		
			256.26	N 63° 50' 06" W
13	16,161.0	6,270.0		
			127.32	N 85° 56' 47" W
14	16,170.0	6,143.0		
			201.16	S 87° 43' 15" W
15	16,162.0	5,942.0		
			106.51	S 34° 17' 13" W
16	16,074.0	5,882.0		

* Note : For profile and typical section of road, see C103

REFERENCE DRAWINGS:

Service spillway Riser C105
Conduit Details C109
Abandonment of existing spillway C110

NOTES:

1. Topography supplied by Kentucky Power Co. Date of mapping pre-dates original project construction and is shown for schematic purposes only.
2. Grid system refers to Kentucky Power Co. plant grid system.
3. Limits of spoil, borrow, and quarry are approximate.
4. Payment for stripping borrow areas will be to a maximum depth of 6" measured on the basis of horizontal projection.
5. Payment for clearing and grubbing will be based on horizontal projection.

Scale 0 200 400 Feet
Except as noted

KENTUCKY POWER COMPANY
BIG SANDY PLANT
BIG SANDY, KENTUCKY

BIG SANDY FLYASH DAM | DAM RAISING

PROJECT LOCATION AND BORROW AREA LOCATION MAP

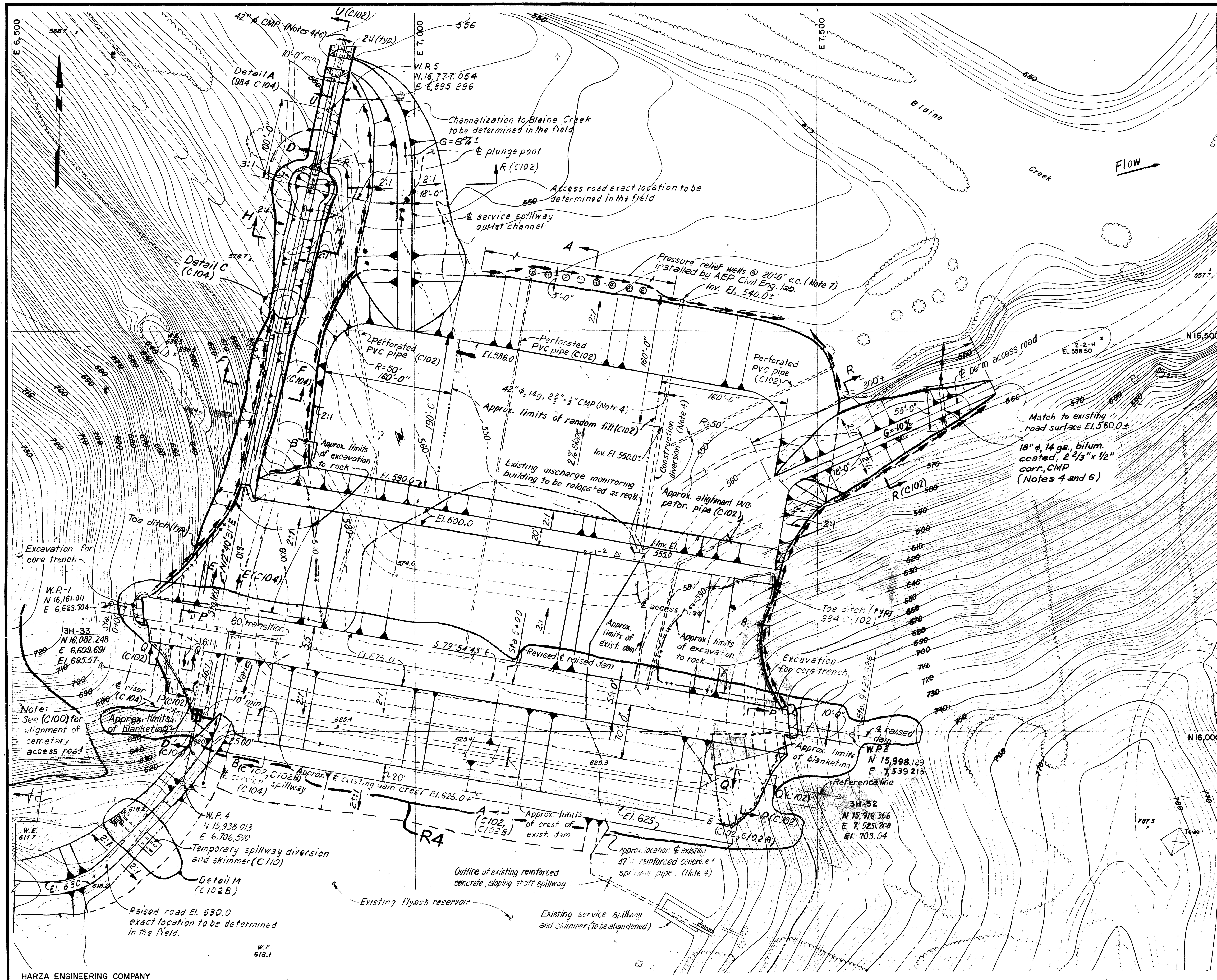
AMERICAN ELECTRIC POWER SERVICE CORP.
APPROVED

NEW YORK, NEW YORK | DATE APRIL 1977 | DWG. NO. 984 C 100R2

REV. NO.	DWG. TRANSMITTAL LETTER NO.	DATE	NATURE OF REVISION	BY	CHKD.	APPD.
2		5/16/78	Revised crest width	C.W	WAR	WAR
1		4/21/78	Changes as shown	J.V.	J.L.M	RMO

HARZA ENGINEERING COMPANY
CHICAGO, ILLINOIS

		REVIEWED	
DSGN.		CIVIL	[illegible signature]
CHKD.		MECH.	
DWN.	T.G.	ELECT.	
CHKD.	WAR		
SUBM.			



PLAN

LEGEND:

- △ Kentucky Power Co. benchmark
- Fill slope
- Cut slope
- Limits of max. normal water surface El. 670.0
- Toe ditch (C102)
- ◎ Pressure relief well

NOTES:

1. Work this drawing with C102, C102A, and C104.
2. Topography supplied by Kentucky Power Co. Date of mapping 1976.
3. All exploration performed by Kentucky Power Co.
4. Corrugated metal pipe as manufactured by Armco or equal.
5. Both existing concrete spillway pipe and construction diversion to be concreted upon completion of construction as directed
6. A minimum 2'-0" cover shall be placed over top of pipe. Location of CMP shall be determined in the field.
7. Pressure relief wells to extend 25'-0" into rock. Number of wells to be determined in the field.

Scale 0 50 100 Feet

HARZA ENGINEERING COMPANY
CHICAGO, ILLINOIS

DSGN.		REVIEWED
CHKD.		CIVIL
DWN.		MECH.
CHKD.		ELECT.
SUBM.		

REV NO.	DWG TRANSMITTAL LETTER NO.	DATE	NATURE OF REVISION	BY	CHKD.	APPD.
R4		5/16/78	Changed crest width & U/S slope as shown	JW	WAR	WAR
R3		4/28/77	Changes as shown	JV	JLM	
R2		6/23/77	Crest width changed to 35'	TEG	WAR	
R1		5/16/77	Service Spillway Conduit Extension		WAR	

KENTUCKY POWER COMPANY
BIG SANDY PLANT
BIG SANDY, KENTUCKY

BIG SANDY FLYASH DAM — DAM RAISING

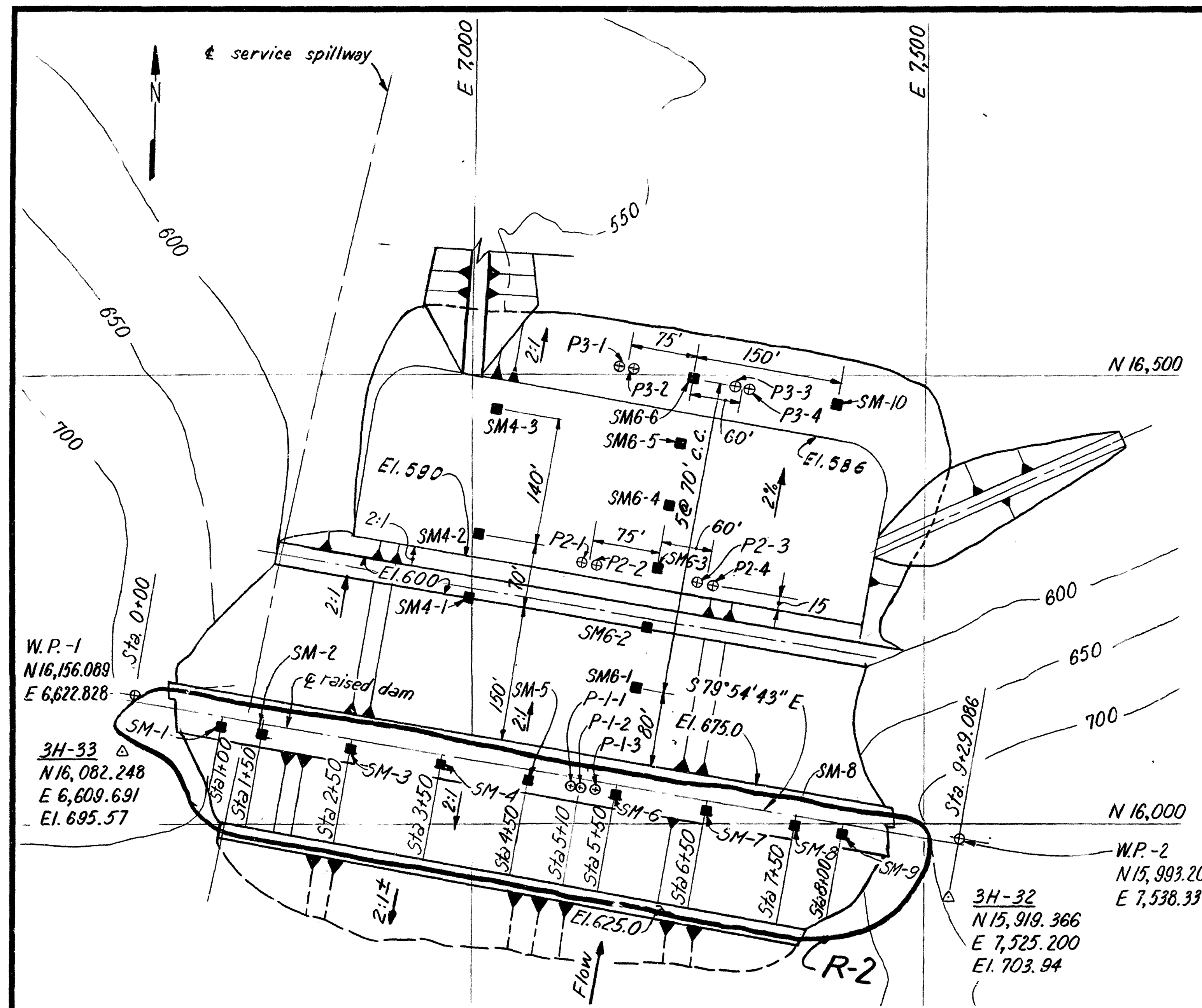
DAM

SHEET 1 OF 4

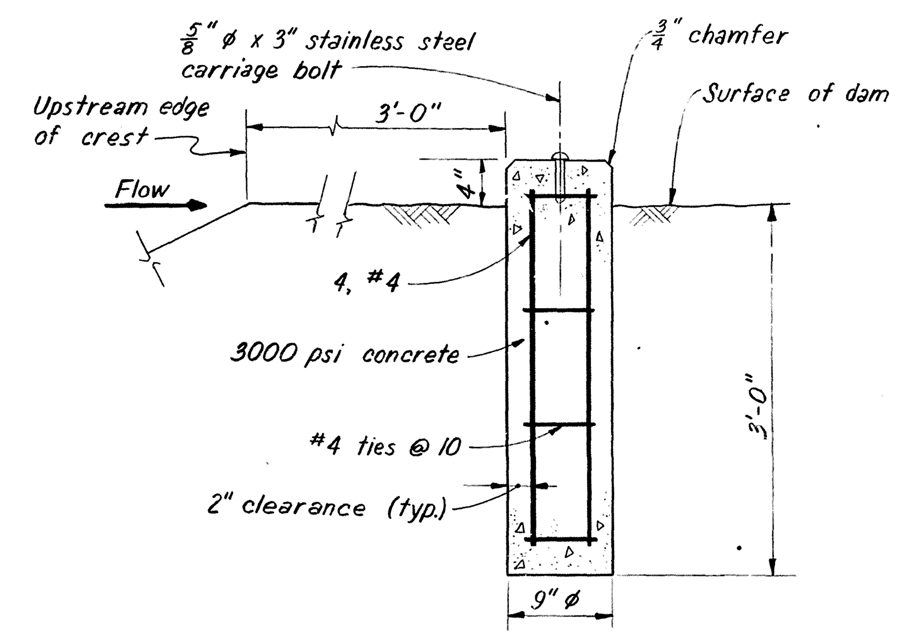
AMERICAN ELECTRIC POWER SERVICE CORP.

APPROVED ________________

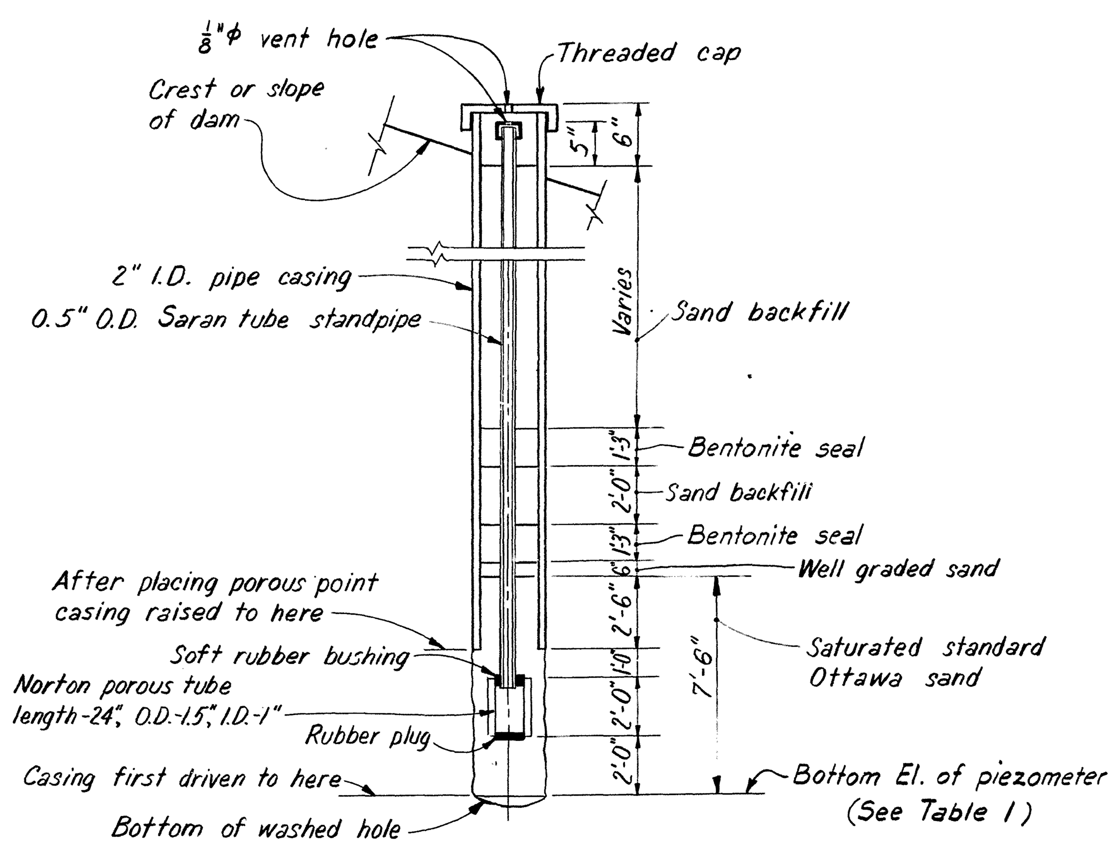
NEW YORK, NEW YORK	DATE APRIL, 1977	DWG.NO. 984 C 101R4



INSTRUMENTATION LOCATION PLAN



SURFACE SETTLEMENT MONUMENT (19 REQ'D)
(SEE C 100)
N.T.S.



TYPICAL PIEZOMETER DETAIL
N.T.S.

TABLE 1
BOTTOM ELEVATION OF PIEZOMETER

Piezometer No.	Bottom Elevation
P1-1	*
P1-2	559
P1-3	625
P2-1	*
P2-2	530
P2-3	*
P2-4	530
P3-1	*
P3-2	530
P3-3	*
P3-4	530

* 15' below top of rock

LEGEND:
- ■ SM-2 - Surface settlement movement.
- ⊕ P3-2 - Piezometer

NOTES:
1. Work this drawing with C 101

Scale 0 — 100 Feet
Except as noted

KENTUCKY POWER COMPANY
BIG SANDY PLANT
BIG SANDY, KENTUCKY

BIG SANDY FLYASH DAM	INSTRUMENTATION

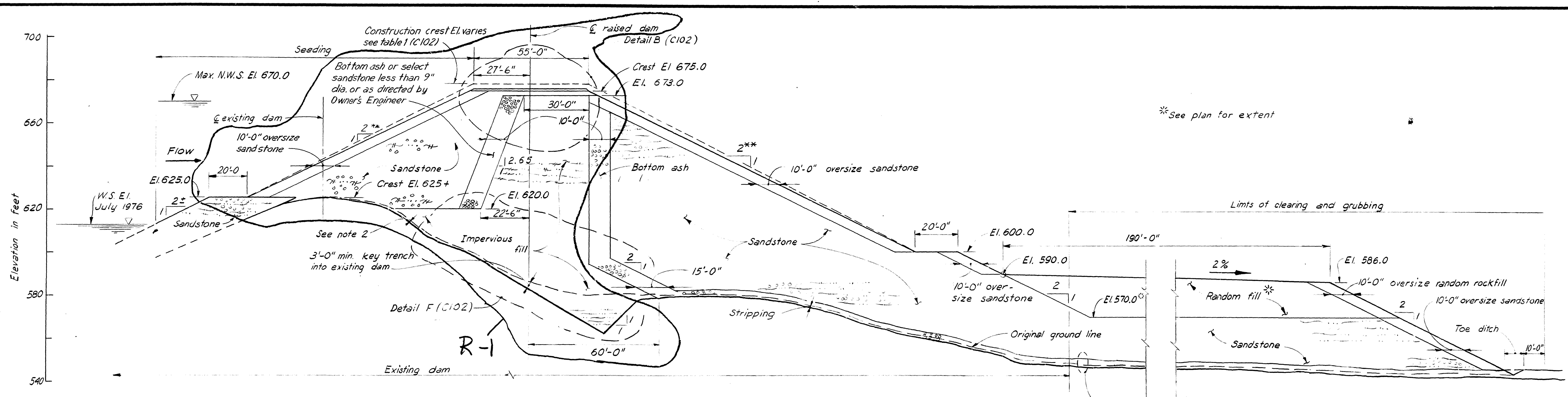
DAM
SHEET 3 OF 4

AMERICAN ELECTRIC POWER SERVICE CORP.
APPROVED ____________
NEW YORK, NEW YORK — DATE APRIL, 1977 — DWG.NO. 984 C 102A R2

REV. NO.	DWG. TRANSMITTAL LETTER NO.	DATE	NATURE OF REVISION	BY	CHKD.	APPD.
2		7/14/78	Revised crest width	J.V.	WAR	WAR
1		4/29/78	Changes as shown	J.V.	J.L.M.	RO

HARZA ENGINEERING COMPANY
CHICAGO, ILLINOIS

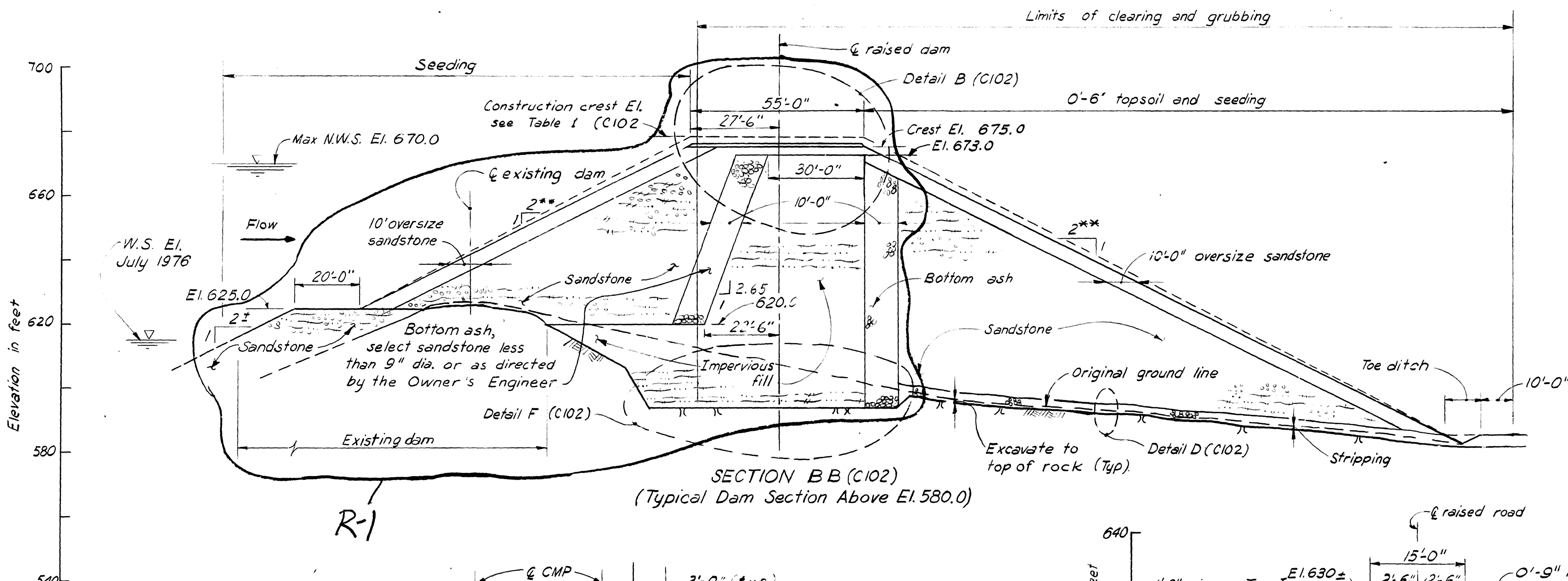
		REVIEWED	
DSGN.			
CHKD.		CIVIL	
DWN.		MECH.	
CHKD.	WAR		
SUBM.		ELECT.	



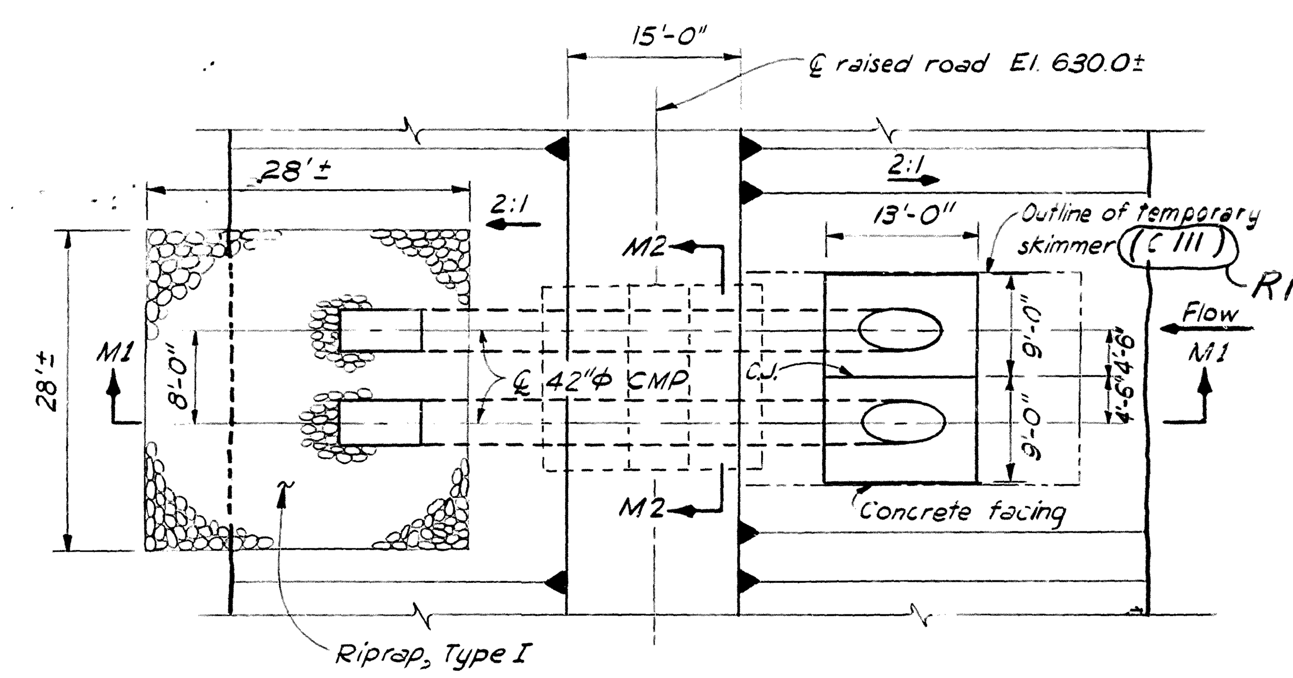
** See note 4 (C102)

SECTION AA (C101)
(Typical Dam Section)

Detail D (C102)

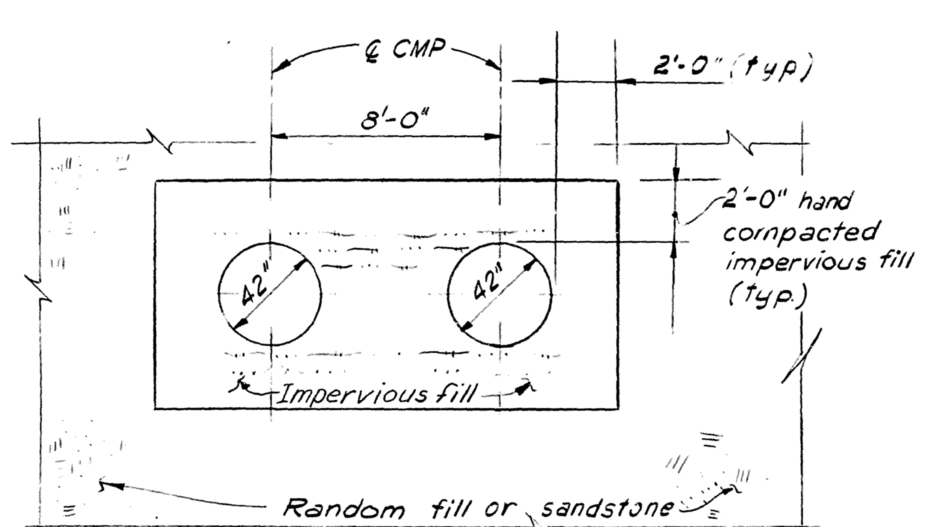


SECTION BB (C102)
(Typical Dam Section Above El. 580.0)



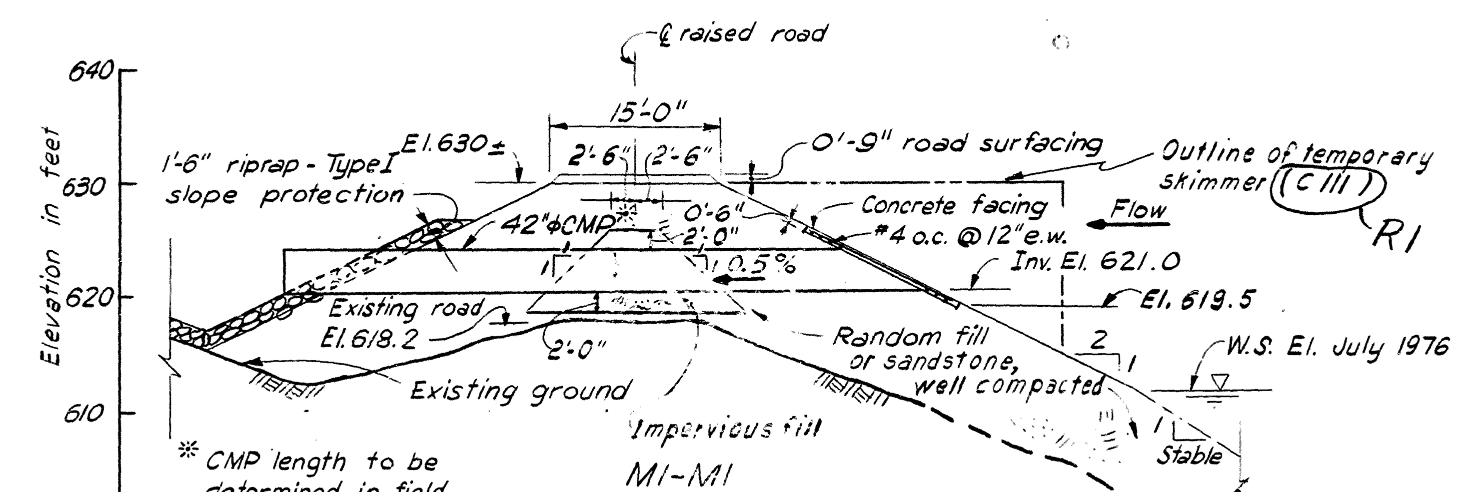
TEMPORARY SPILLWAY DIVERSION & SKIMMER
DETAIL M (C101)

Scale 0 10 Feet



M2-M2

Scale 0 4 Feet



M1-M1

Scale 0 10 Feet

NOTE:

Work this drawing with C101, C102.

Scale 0 20 40 Feet
Unless Noted.

HARZA ENGINEERING COMPANY
CHICAGO, ILLINOIS

DSGN.		REVIEWED	
CHKD.		CIVIL	
DWN.		MECH.	
CHKD.		ELECT.	
SUBM.			

REV. NO.	DWG. TRANSMITTAL LETTER NO.	DATE	NATURE OF REVISION	BY	CHKD.	APPD.
R1		5/16/78	Revised crest width	CW	WAR	WAR

KENTUCKY POWER COMPANY
BIG SANDY PLANT
BIG SANDY, KENTUCKY

BIG SANDY FLYASH DAM — DAM RAISING

DAM
SHEET 4 OF 4

AMERICAN ELECTRIC POWER SERVICE CORP.
APPROVED

NEW YORK, NEW YORK — DATE MAY, 1978 — DWG. NO. 984 C102BR1

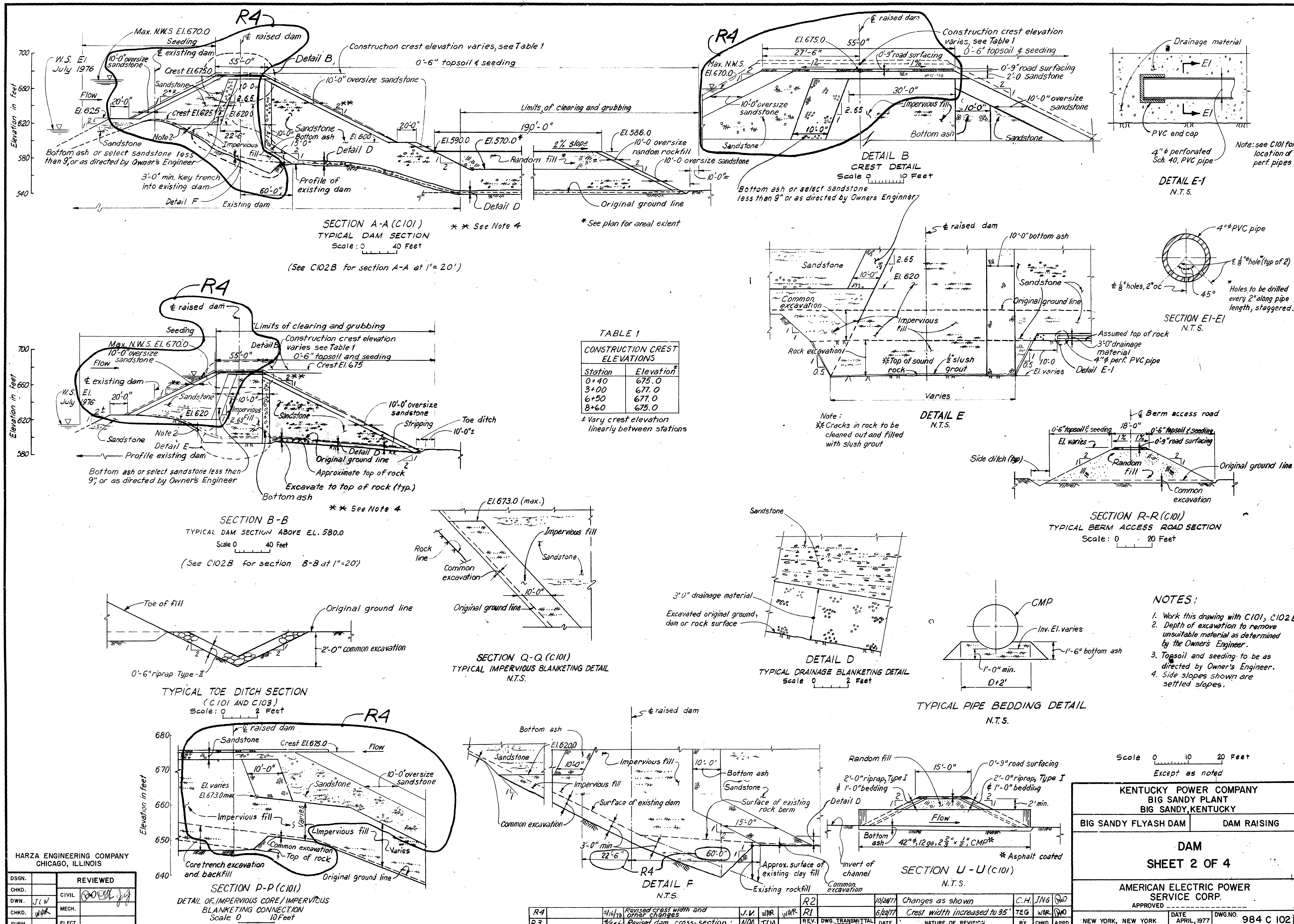


TABLE 1

CONSTRUCTION CREST ELEVATIONS	
Station	Elevation*
0+40	675.0
3+00	677.0
6+50	677.0
8+60	675.0

* Vary crest elevation linearly between stations

NOTES:

1. Work this drawing with C101, C102B
2. Depth of excavation to remove unsuitable material as determined by the Owners Engineer.
3. Topsoil and seeding to be as directed by Owner's Engineer.
4. Side slopes shown are settled slopes.

Scale 0 10 20 Feet
Except as noted

REV. NO.	DATE	NATURE OF REVISION	BY	CHKD.	APPD.
R4	5/16/78	Revised crest width and other changes	J.V.	WDR	WAR
R3	4/21/78	Revised dam cross-section	NDA	JLM	
R2	10/24/77	Changes as shown	C.H.	JNG	
R1	6/29/77	Crest width increased to 35'	TEG	WDR	

HARZA ENGINEERING COMPANY
CHICAGO, ILLINOIS

		REVIEWED	
DSGN.			
CHKD.			
DWN.	JLW	CIVIL	
CHKD.	WDR	MECH.	
SUBM.		ELECT.	

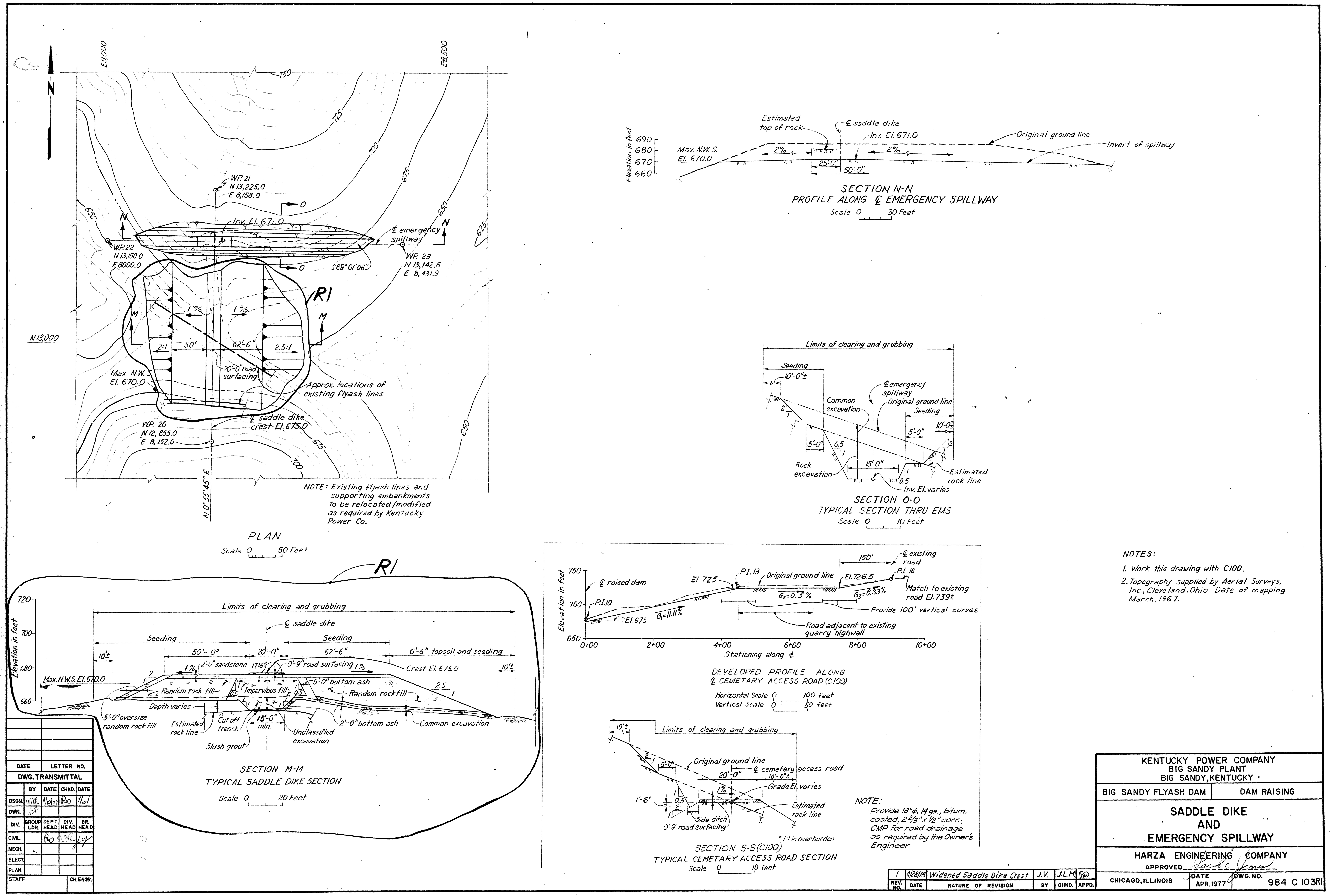
KENTUCKY POWER COMPANY
BIG SANDY PLANT
BIG SANDY, KENTUCKY

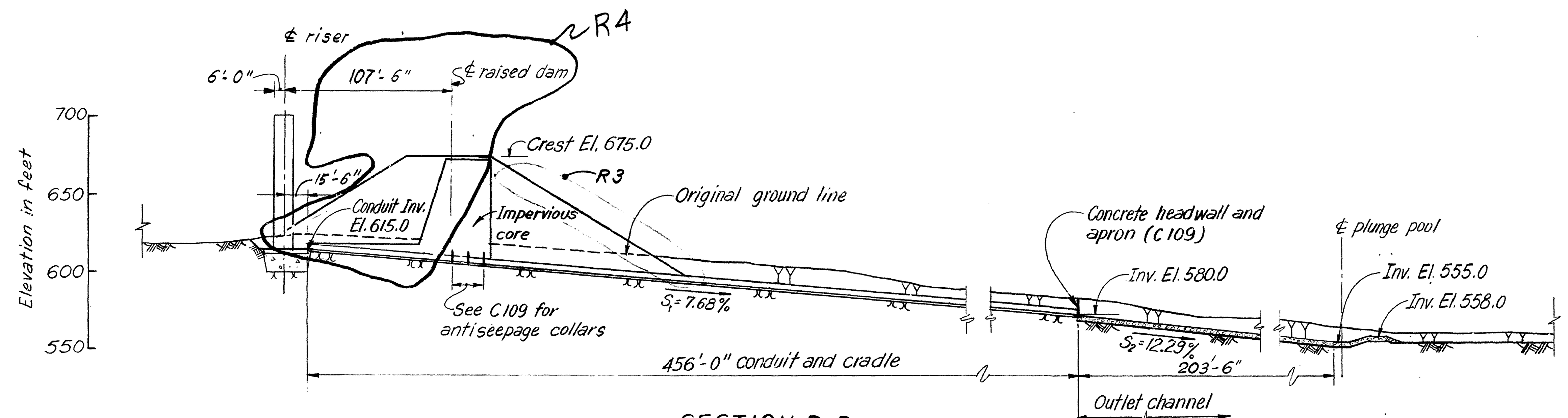
BIG SANDY FLYASH DAM — DAM RAISING

DAM
SHEET 2 OF 4

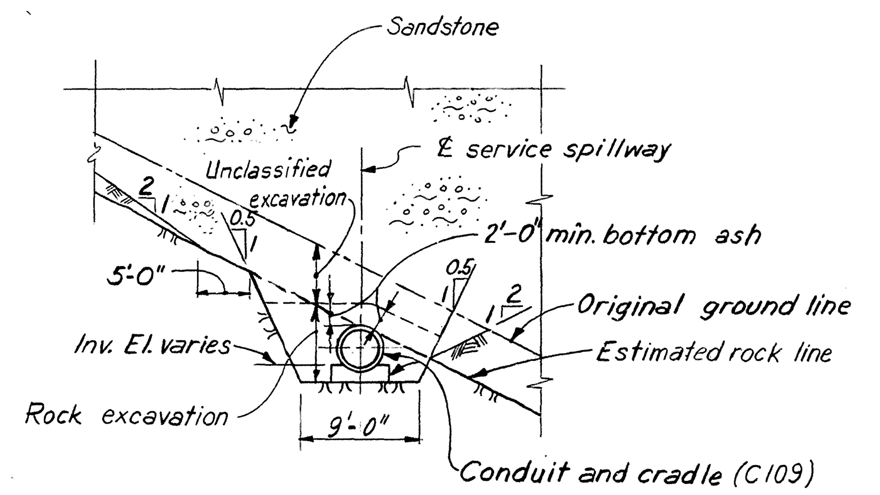
AMERICAN ELECTRIC POWER SERVICE CORP.
APPROVED

NEW YORK, NEW YORK — DATE APRIL, 1977 — DWG. NO. 984 C 102 R4

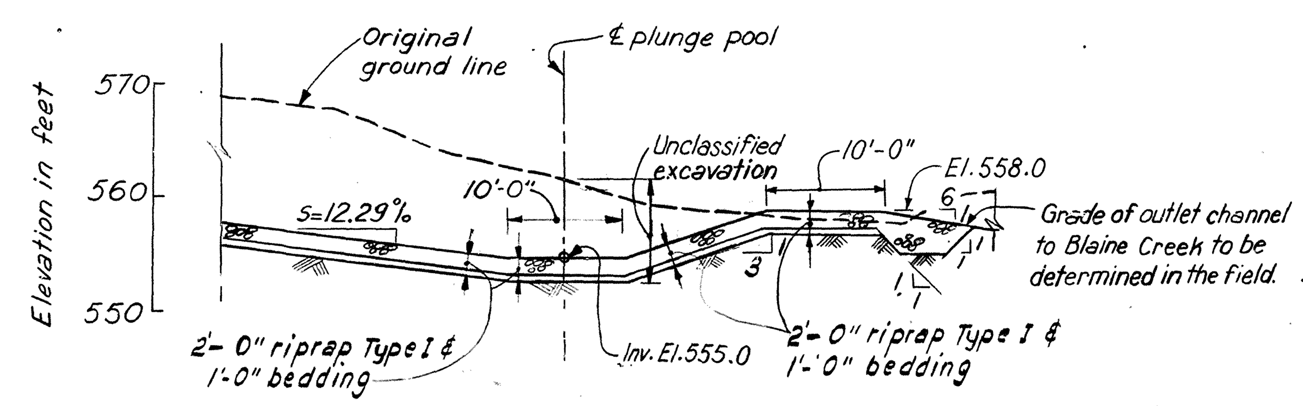

PLAN
Scale 0 50 Feet
E8,000
E8,500
N13,000
W.P. 21
N 13,225.0
E 8,158.0
W.P. 22
N 13,150.0
E 8,000.0
W.P. 23
N 13,142.6
E 8,431.9
W.P. 20
N 12,855.0
E 8,152.0
Inv. El. 671.0
℄ emergency spillway
S89°01'06"
N0°55'45"E
Max. N.W.S. El. 670.0
20'-0" road surfacing
Approx. locations of existing flyash lines
℄ saddle dike crest El. 675.0
NOTE: Existing flyash lines and supporting embankments to be relocated/modified as required by Kentucky Power Co.
R1
SECTION N-N
PROFILE ALONG ℄ EMERGENCY SPILLWAY
Scale 0 30 Feet
Elevation in feet
Estimated top of rock
℄ saddle dike
Inv. El. 671.0
Original ground line
Invert of spillway
Max. N.W.S. El. 670.0
SECTION O-O
TYPICAL SECTION THRU EMS
Scale 0 10 Feet
Limits of clearing and grubbing
Seeding
Common excavation
℄ emergency spillway
Original ground line
Rock excavation
Estimated rock line
Inv. El. varies
SECTION M-M
TYPICAL SADDLE DIKE SECTION
Scale 0 20 Feet
Limits of clearing and grubbing
℄ saddle dike
Seeding
50'-0"
20'-0"
62'-6"
0'-6" topsoil and seeding
Max. N.W.S. El. 670.0
2'-0" sandstone
0'-9" road surfacing
Crest El. 675.0
Random rock fill
Impervious fill
5'-0" bottom ash
Random rock fill
Depth varies
5'-0" oversize random rock fill
Estimated rock line
Cut off trench
15'-0" min.
2'-0" bottom ash
Common excavation
Slush grout
Unclassified excavation
DEVELOPED PROFILE ALONG ℄ CEMETARY ACCESS ROAD (C100)
Horizontal Scale 0 100 feet
Vertical Scale 0 50 feet
℄ raised dam
P.I. 10
El. 675
El. 725
P.I. 13
Original ground line
El. 726.5
℄ existing road
P.I. 16
Match to existing road El. 739±
Provide 100' vertical curves
Road adjacent to existing quarry highwall
Stationing along ℄
SECTION S-S (C100)
TYPICAL CEMETARY ACCESS ROAD SECTION
Scale 0 10 feet
Limits of clearing and grubbing
Original ground line
℄ cemetary access road
Grade El. varies
Estimated rock line
Side ditch
0'-9" road surfacing
* 1:1 in overburden
NOTE:
Provide 18"ϕ, 14 ga., bitum. coated, 2 2/3" x 1/2" corr., CMP for road drainage as required by the Owner's Engineer
NOTES:
1. Work this drawing with C100.
2. Topography supplied by Aerial Surveys, Inc., Cleveland, Ohio. Date of mapping March, 1967.
KENTUCKY POWER COMPANY
BIG SANDY PLANT
BIG SANDY, KENTUCKY
BIG SANDY FLYASH DAM
DAM RAISING
SADDLE DIKE AND EMERGENCY SPILLWAY
HARZA ENGINEERING COMPANY
CHICAGO, ILLINOIS
DATE APR. 1977
DWG. NO. 984 C 103R1
1 4/28/78 Widened Saddle Dike Crest J.V. J.L.M.
REV. NO. DATE NATURE OF REVISION BY CHKD. APPD.
DWG. TRANSMITTAL
DSGN. DWN. CIVIL MECH. ELECT. PLAN. STAFF CH. ENGR.



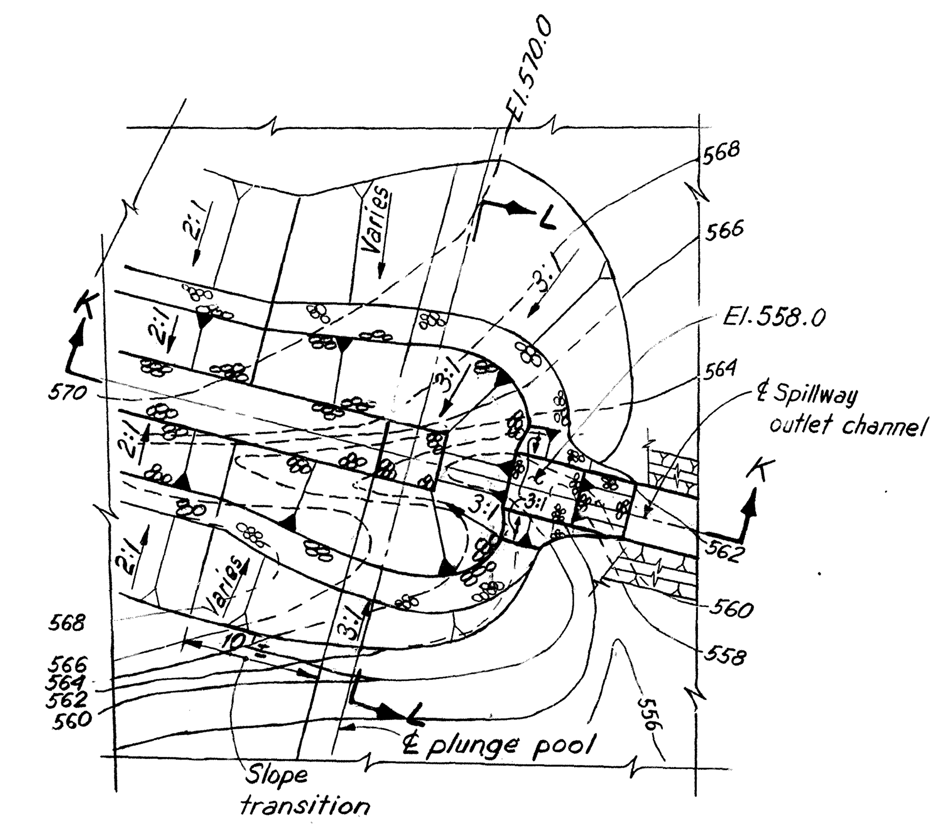
SECTION D-D
DEVELOPED PROFILE ALONG PRINCIPAL SPILLWAY CENTERLINE
Scale 0 50 Feet



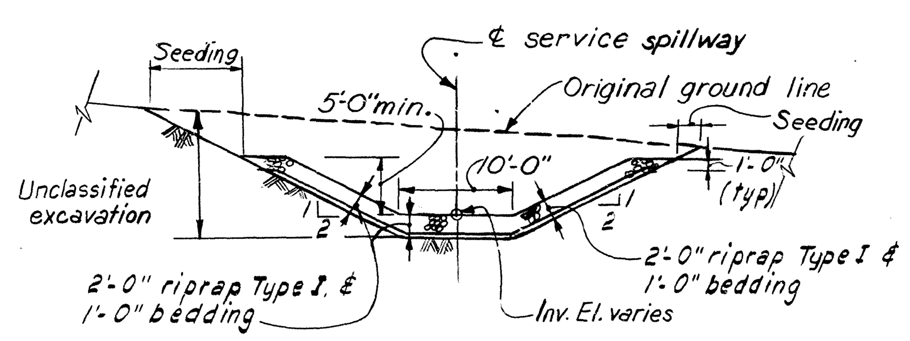
SECTION E-E
(984 C 101)
Scale 0 10 Feet



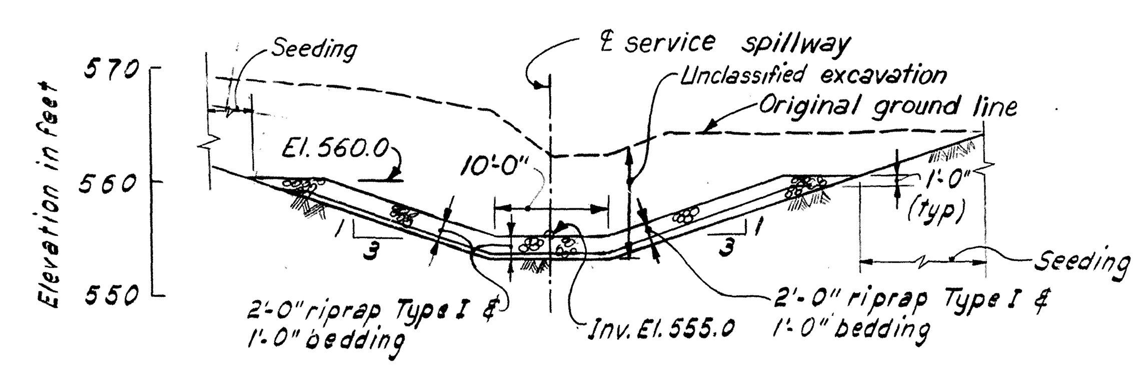
SECTION K-K
Scale 0 10 Feet



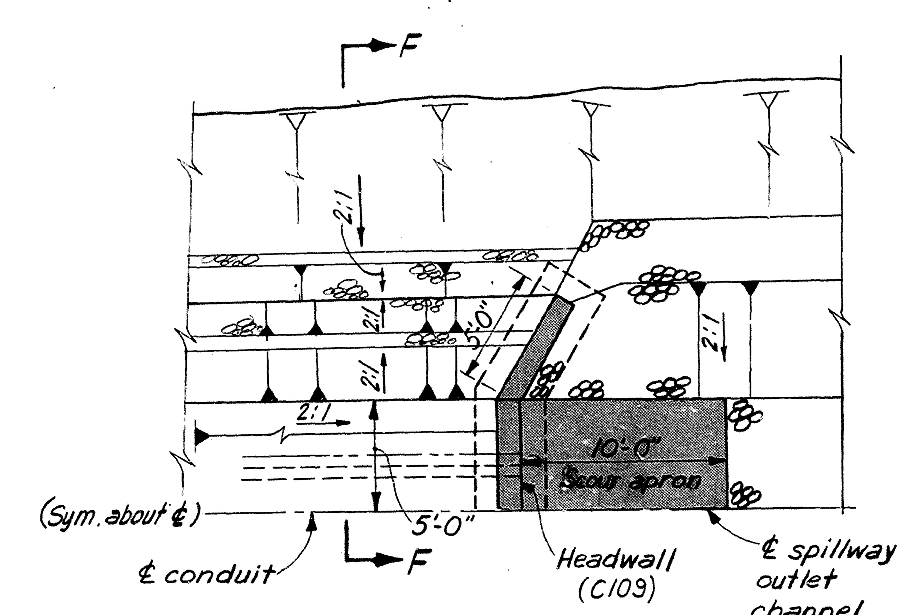
DETAIL A
(984 C 101)
PLAN - PLUNGE POOL
Scale 0 20 Feet



SECTION H-H
(984 C 101)
TYPICAL OUTLET CHANNEL SECTION
Scale 0 10 Feet



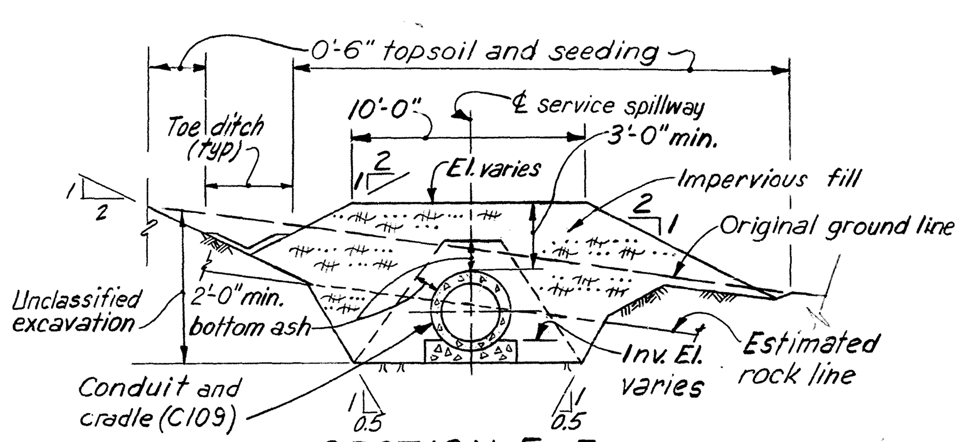
SECTION L-L
Scale 0 10 Feet



DETAIL C
(984 C 101)
CONDUIT OUTLET DETAIL
N.T.S.

REFERENCE DRAWINGS:
RISER C105 - C108
CONDUIT C109

NOTES:
1. Work this drawing with C101.
2. All riprap for the service spillway outlet channel and plunge pool is Riprap - Type I.
3. Topsoil and seeding to be at the direction of Owner's Engineer.



SECTION F-F
(984 C 101)
Scale 0 10 Feet

DSGN.	WDR	REVIEWED	
CHKD.		CIVIL	
DWN.	L.L.	MECH.	
CHKD.	WDR	ELECT.	
SUBM.			

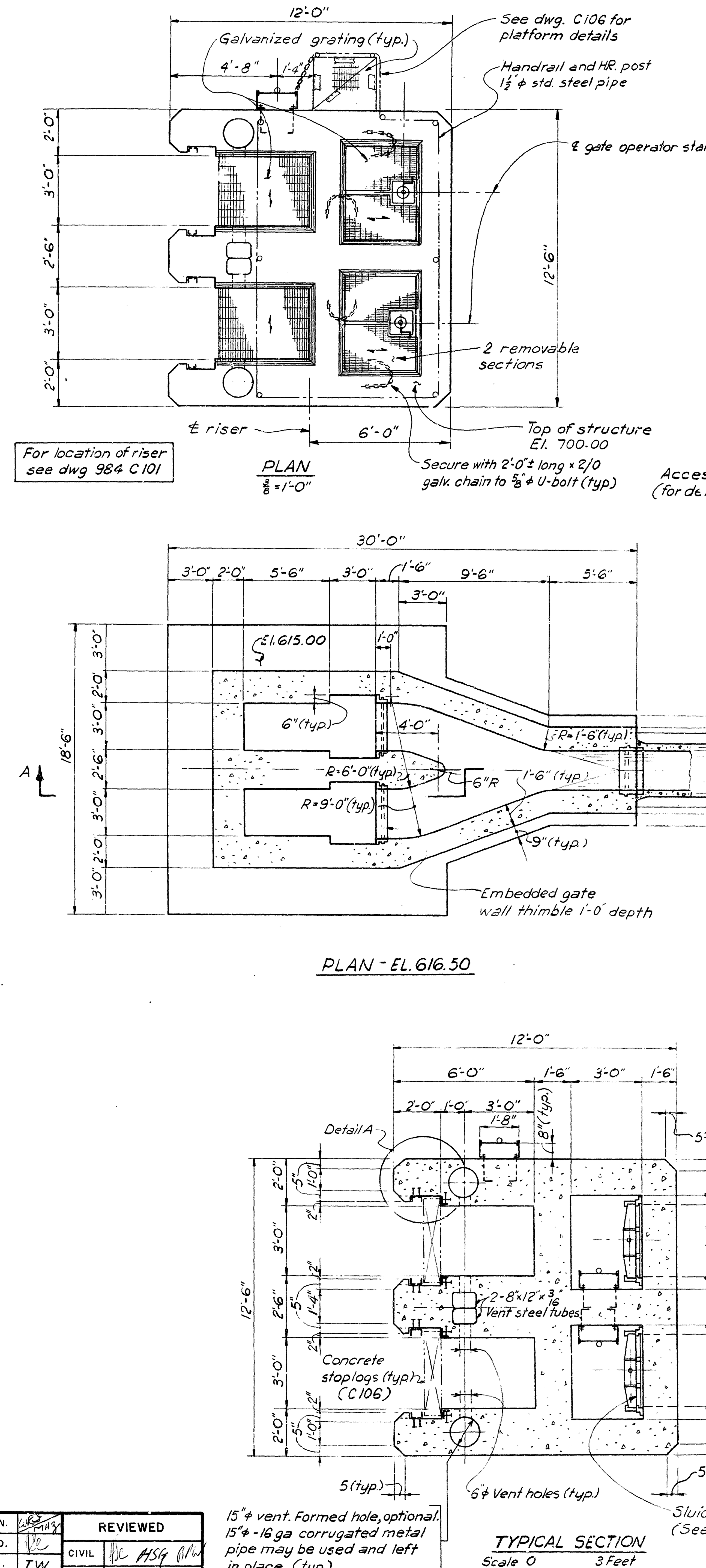
REV. NO.	DWG. TRANSMITTAL LETTER NO.	DATE	NATURE OF REVISION	BY	CHKD.	APPD.
R4		5/16/78	Revised crest width	J.L.W.	WAR	WAR
R3		4/29/78	Changes as shown	J.V.	J.L.H.	
R2		6/23/77	Revised inv. El. and length of conduit		WAR	
R1		5/6/77	Service Spillway Conduit Extension		WAR	

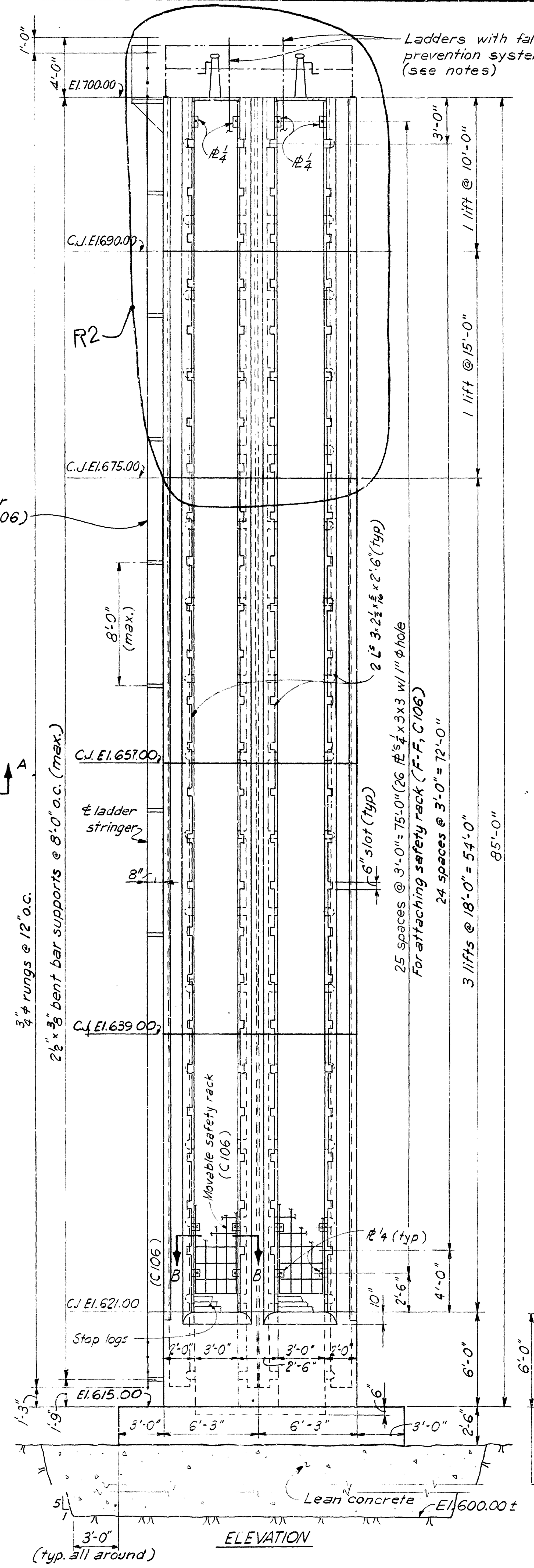
KENTUCKY POWER COMPANY
BIG SANDY PLANT
BIG SANDY, KENTUCKY

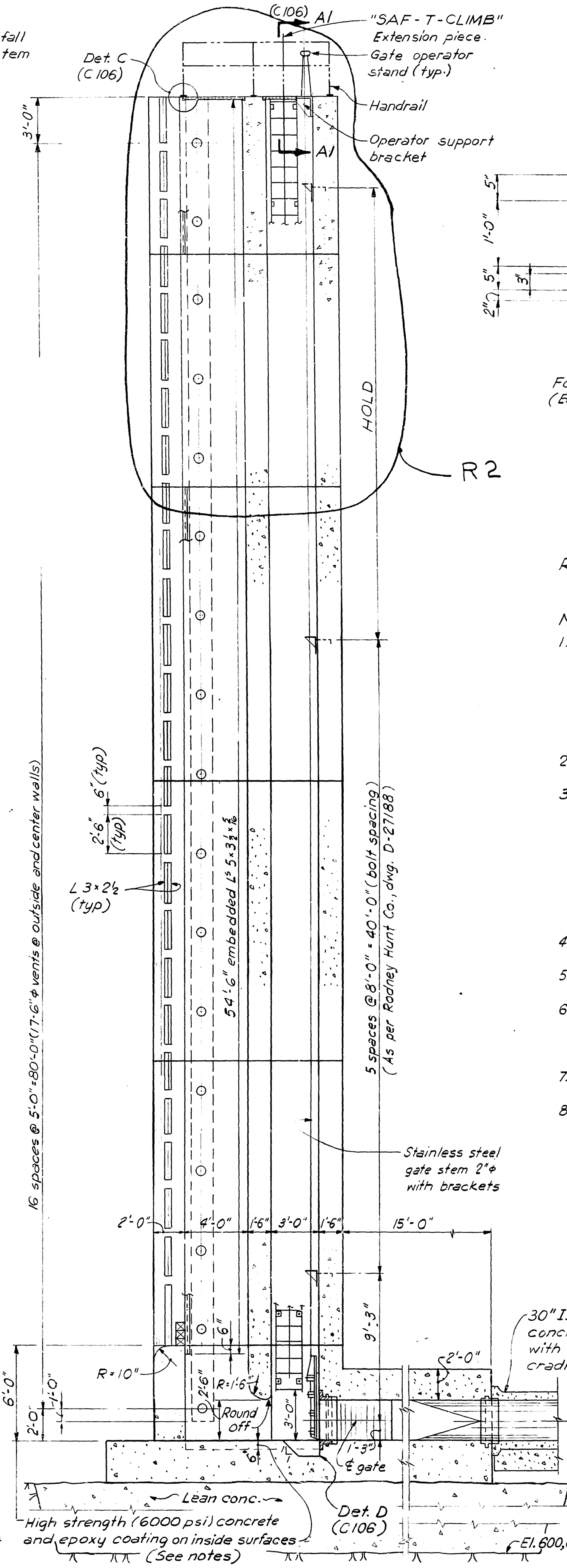
BIG SANDY FLYASH DAM | DAM RAISING

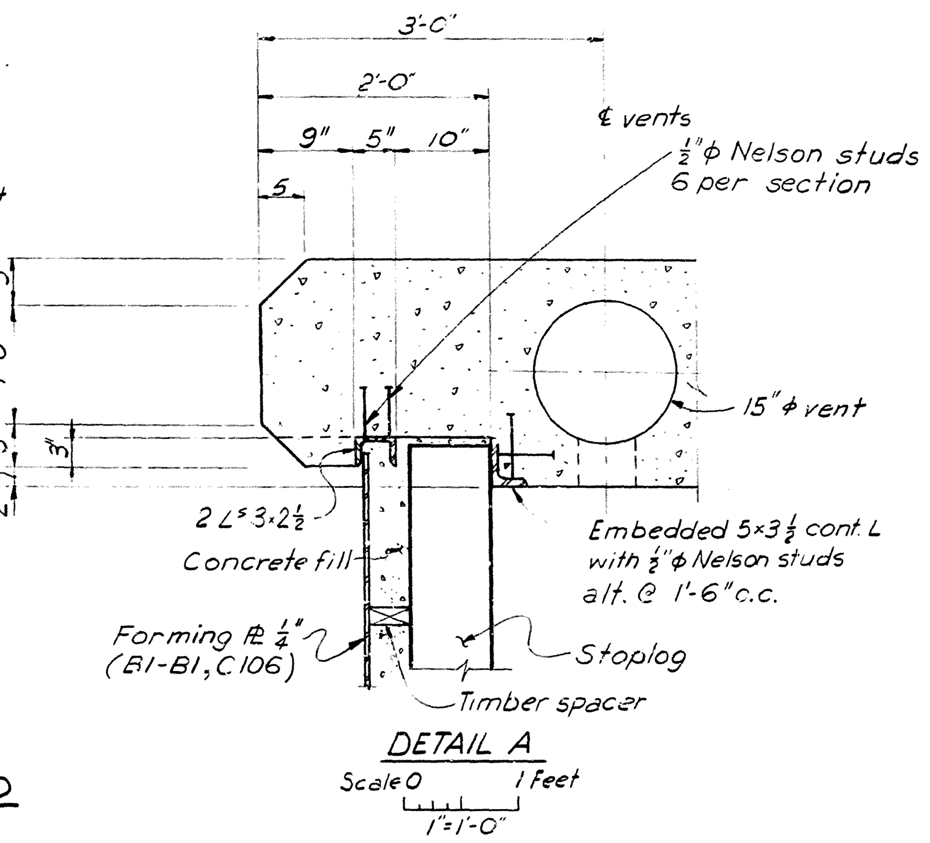
SERVICE SPILLWAY
EARTHWORK DETAILS

CONSULTING ENGINEERS
HARZA ENGINEERING COMPANY
APPROVED
CHICAGO, ILLINOIS | DATE April, 1977 | DWG. NO. 984 C 104R4









REFERERENCE DRAWINGS :

Work this drawing with 984 C101 and C106 - C109

NOTES :

1. All concrete shall be of 4000 psi strength except
   a. in tower between El. 615.00 to 621.00 and stop logs, f'c = 6000 psi.
   b. foundation slab, anti-seep collars, head-wall and conc. apron, f'c = 3000 psi.
   c. lean concrete under foundation slab and pipe cradle concrete, f'c = 2000 psi
2. All reinf. steel shall be deformed bars and shall conform to ASTM-A 615-71 grade 60 specification.
3. The indicated concrete area shall be painted with two coats of "Sikagard 664/9 Hi-Bild" epoxy resin or equal. The minimum thickness of each coat shall be 6 to 9 mils. Surface preparation and epoxy application shall be in accordance with manf. recommendations. No measurement for payment will be made for epoxy protective coating. The entire cost for all work and materials shall be included in the unit price for concrete 6000 psi.
4. All bolts, nuts and washers shall conform to ASTM-A 307 specifications, and shall be galvanized.
5. All exposed corners of the structure shall have a $\frac{3}{4}$" chamfer (min) unless otherwise noted.
6. All exposed misc. metal including: grating, ladders, handrails and angles shall be structural steel, conforming to ASTM-A 36 specifications and shall be galvanized.
7. The sluice gate openings are 36"×30". Gates shall be of "Rodney Hunt HY-Q 240 H" series, or equal.
8. Provide "SAF-T-CLIMB" fall prevention systems for 3 ladders with one remov. exten. pc., 2 safety belts and 2 sleeves as supplied by Potter-Weil Co., Chicago, or equal.

Scale 0 4 8 Feet
$\frac{1}{4}$"=1'-0"
Except as noted

DSGN.		REVIEWED	
CHKD.		CIVIL	
DWN.	T.W.	MECH.	
CHKD.	MH?	ELECT.	
SUBM.			

REV. NO.	DWG. TRANSMITTAL LETTER NO.	DATE	NATURE OF REVISION	BY	CHKD.	APPD.
2		May 1978	25 Ft. extension above El. 675.0 added			
1		Nov. 1977	Revise crest elevation	HHL	MH?	

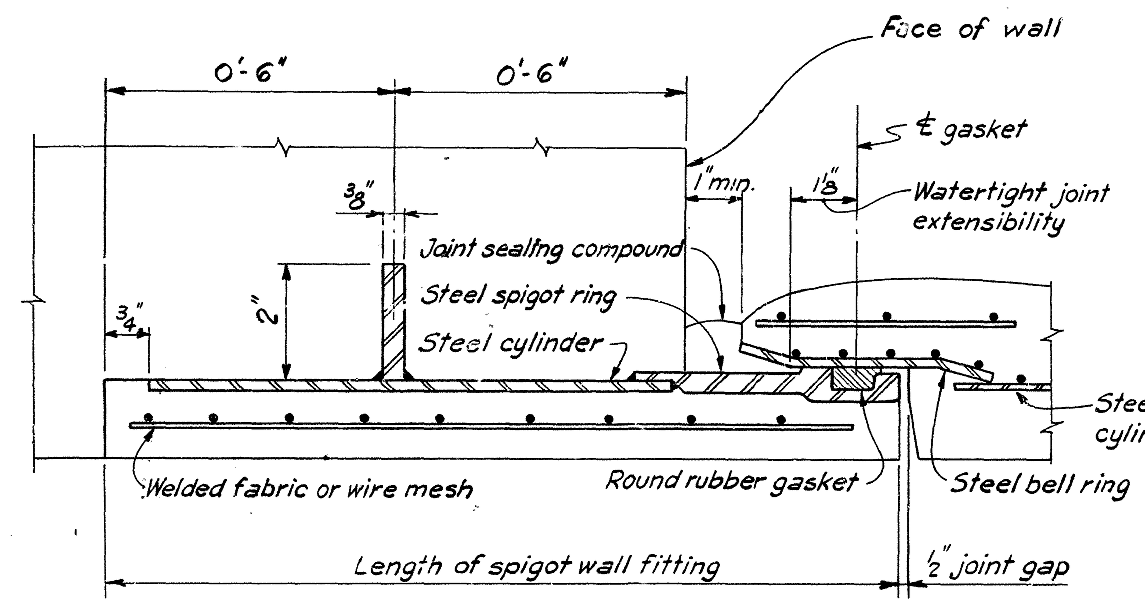
KENTUCKY POWER COMPANY
BIG SANDY PLANT
BIG SANDY, KENTUCKY

BIG SANDY FLYASH DAM — SERVICE SPILLWAY

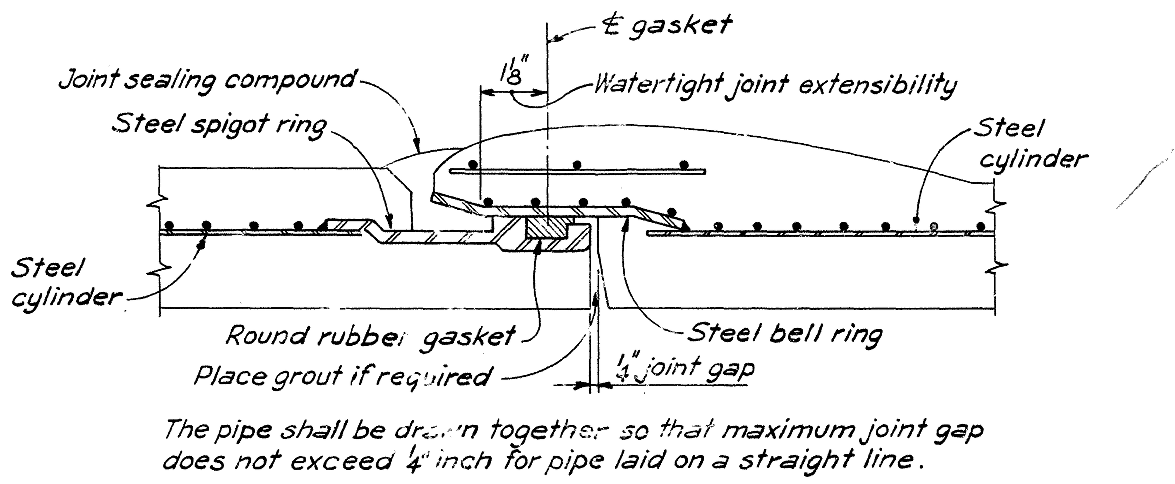
RISER
PLAN AND SECTIONS

CONSULTING ENGINEERS
HARZA ENGINEERING COMPANY
APPROVED
CHICAGO, ILLINOIS

DATE 4-25-77 — DWG. NO. 984 C 105 R2

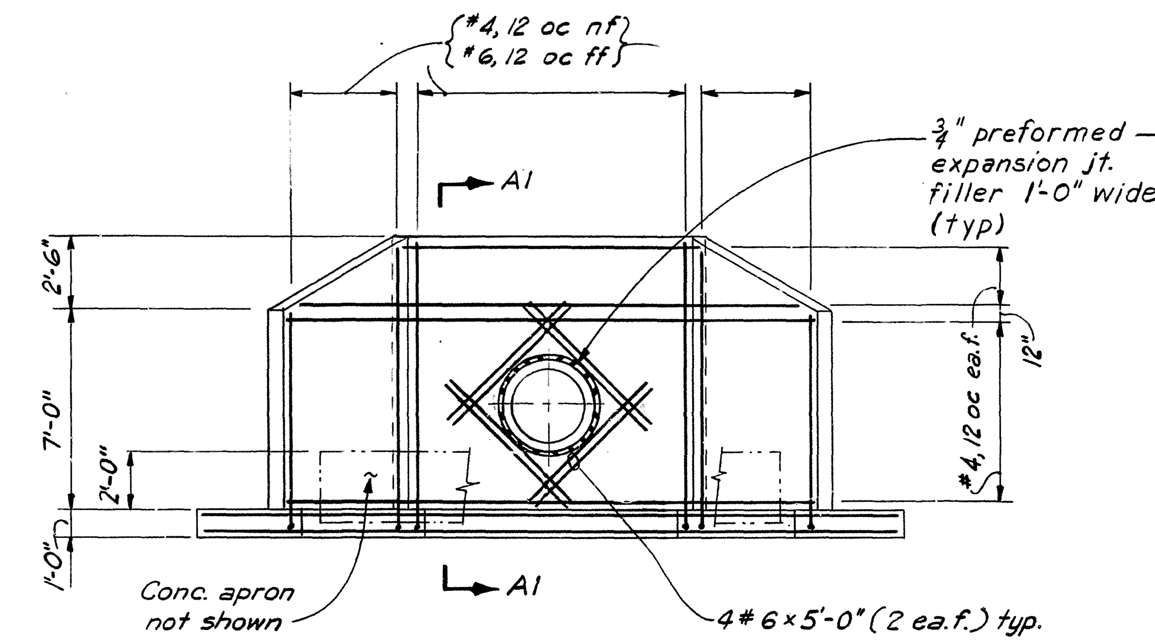


DETAIL A
N.T.S.

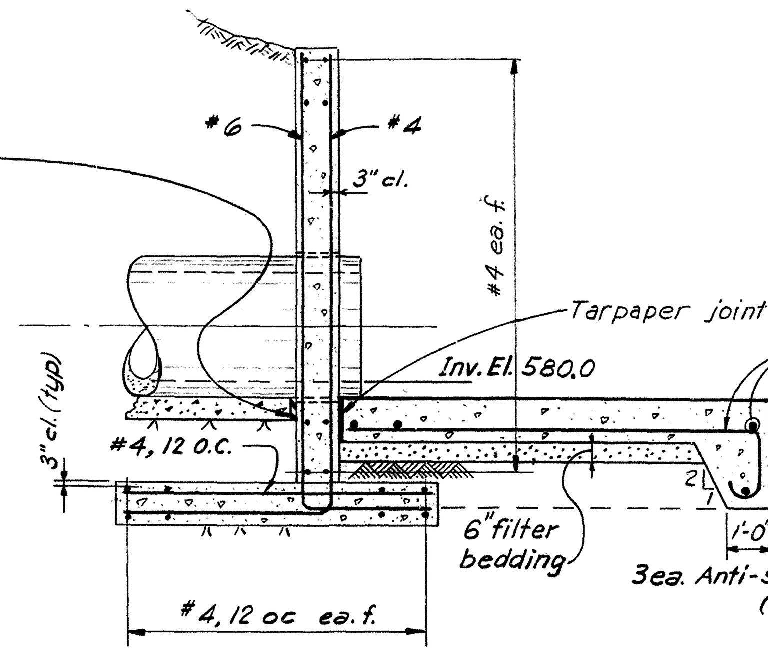


The pipe shall be drawn together so that maximum joint gap does not exceed 1/2 inch for pipe laid on a straight line.

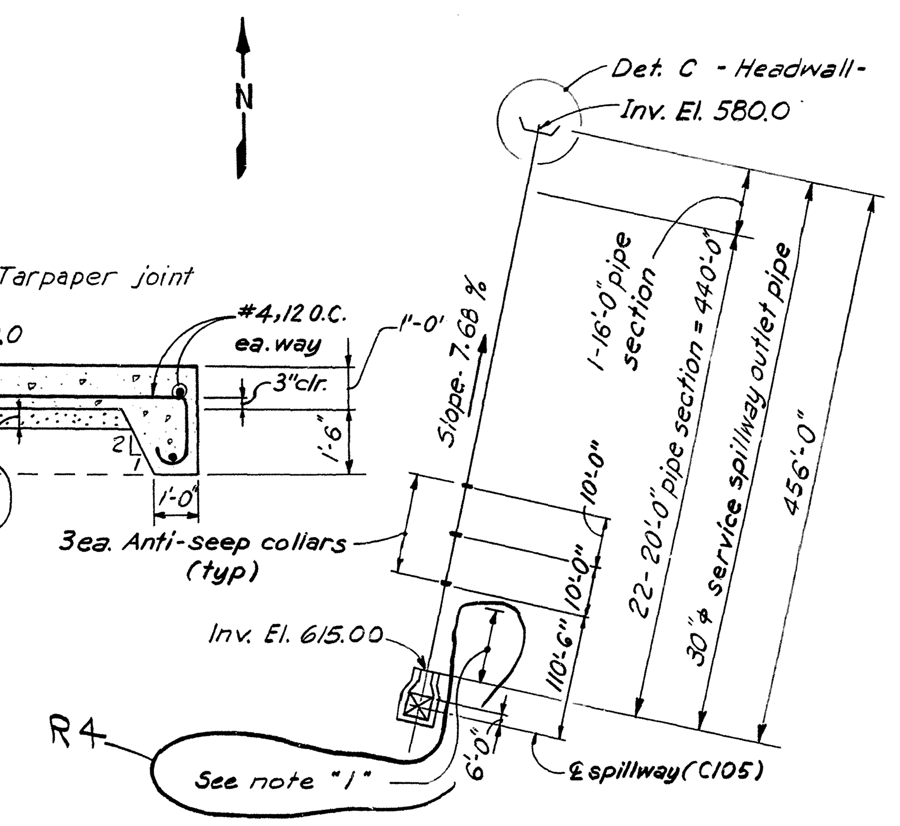
DETAIL B
N.T.S.



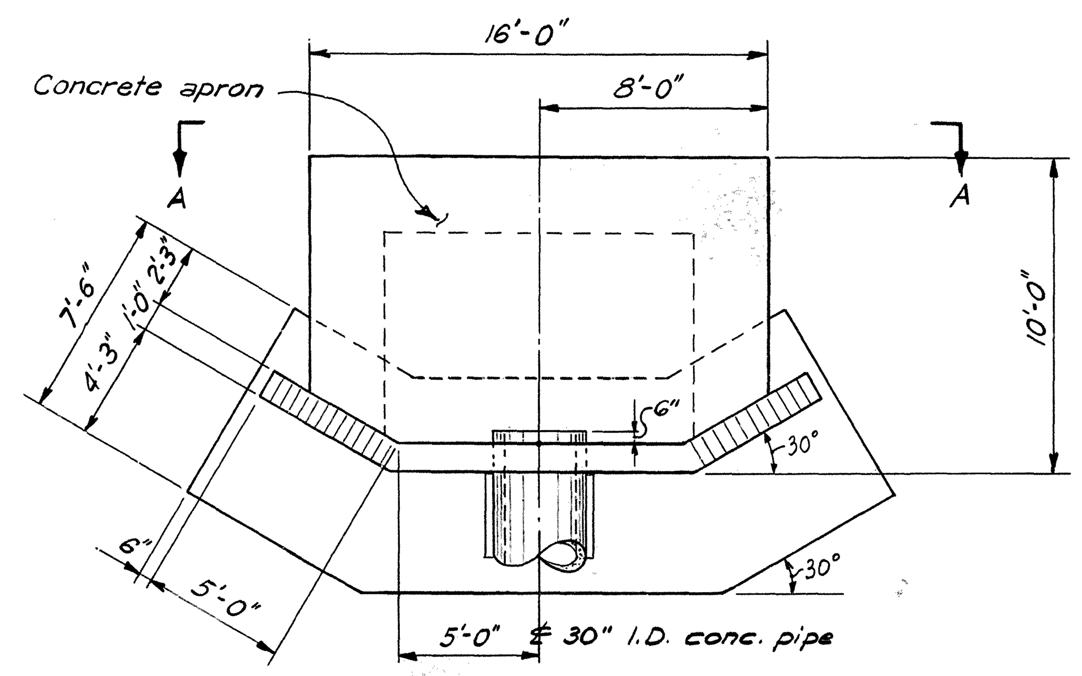
ELEVATION A-A



A1 - A1
Scale 0 3 Feet
3/8" = 1'-0"



CONDUIT INSTALLATION LAYOUT
N.T.S.



DETAIL C - PLAN, HEADWALL

JOINT REQUIREMENTS			
Length of Pipe section	Minimum Joint Length	Minimum Joint Limiting Angle	
feet	inches	radians	degrees
20	3¼	——	——

For pipe length other than shown, joint requirements will be determined by the Engineer.

Prior to delivery of pipe, the pipe joint detail proposed for use shall be submitted to the Engineer for approval.

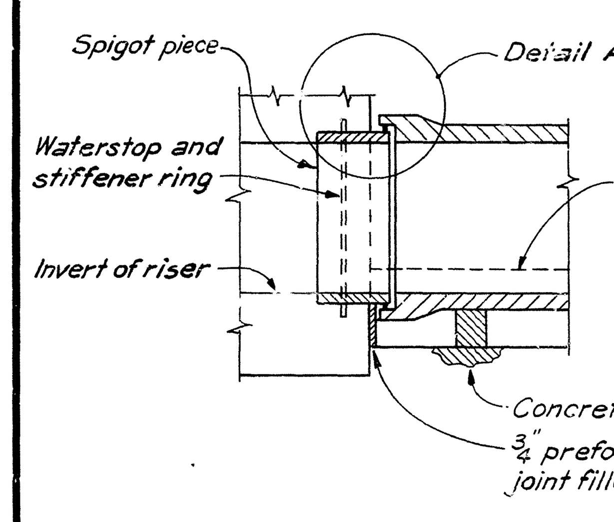
Minimum Joint Limiting Angle should be used as recommended by Pipe Manufacturer.

STRENGTH REQUIREMENTS			
Inside Diameter of pipe	Internal Load	External Load	
	Hydrostatic Pressure	Minimum 3-Edge Bearing Strength in Pounds per Lineal Foot of Pipe	
		Applicable Standard Specification	
		AWWA C-301	AWWA C-300
	Head of Water	Load to produce 0.001 inch crack one foot long	Load to produce 0.01 inch crack one foot long
inches	feet		
30	85	17,000	Not applicable

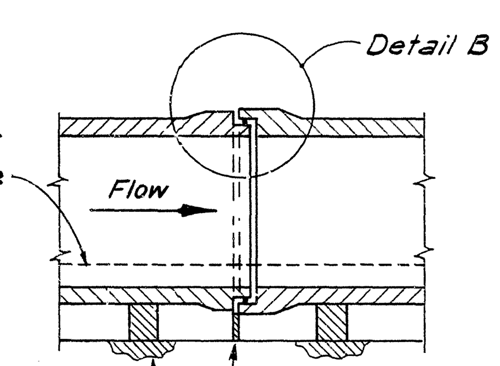
The outside diameter of pipe assumed in design is 36 inches.

Joint sealing compound shall be "Ram-Nek" as manufactured by K.T. Snyder Co., Inc., Houston, Tx. or equal.

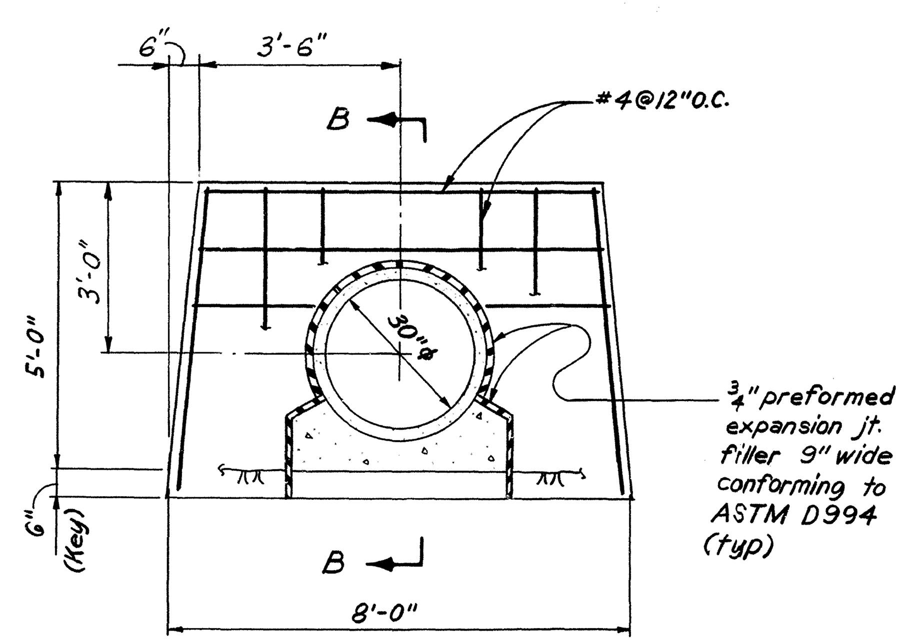
PIPE SPECIFICATIONS



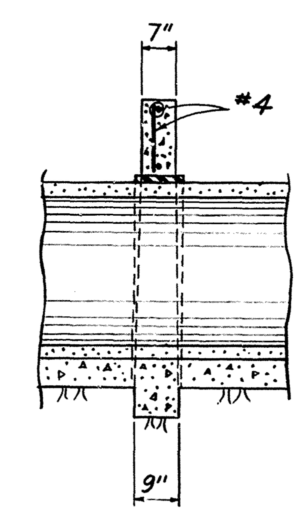
DETAIL OF SPIGOT WALL FITTING
N.T.S.



DETAIL OF PIPE JOINT
N.T.S.



ANTI SEEP COLLAR DETAIL
Scale 0 2 Feet
1/2" = 1'-0"

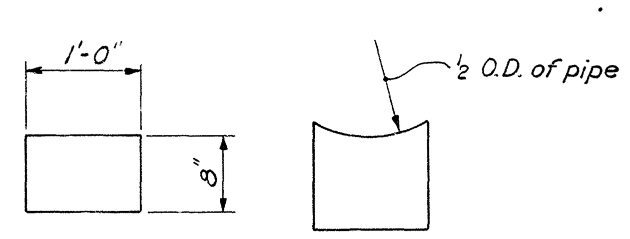


B - B
1/2" = 1'-0"

R4

NOTES:
1. 1/2" max. joint opening at top of pipe (0" opening at bottom) to be allowed on first five lengths of pipe to deflect pipe to design slope.

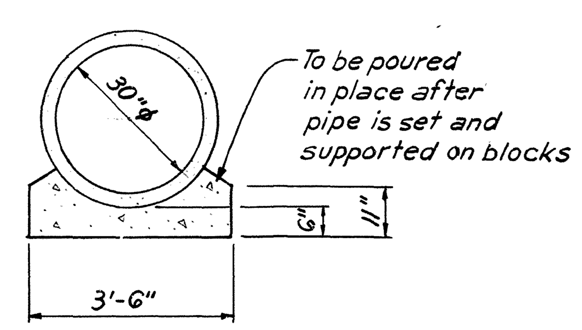
REFERENCE DRAWINGS:
Work this dwg with C105



PLAN FRONT ELEVATION

Sufficient blocks shall be provided to support the pipe to the required line and grade. The contractor shall determine the number and size of blocks required. Wedges may be used as an alternate.

SUGGESTED SUPPORT BLOCKS
N.T.S.



PIPE CRADLE DETAIL
1/2" = 1'-0"
(lean concrete)

Scale 0 4 8 Feet
1/4" = 1'-0"
Unless noted otherwise

DSGN.	MHZ	REVIEWED
CHKD.		CIVIL H&G
DWN.	B.K.	MECH.
CHKD.	MHZ	ELECT.
SUBM.		

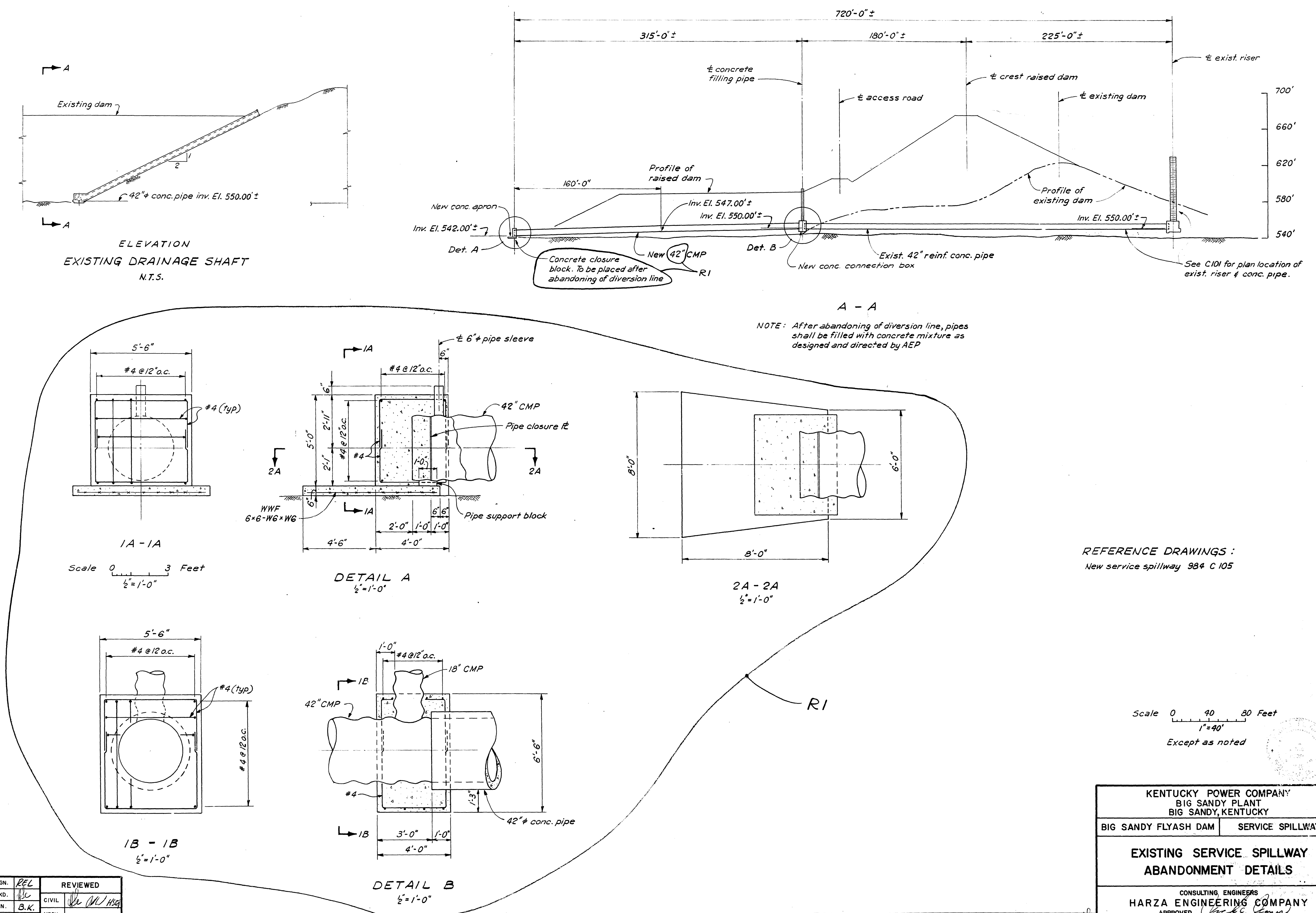
REV. NO.	DWG. TRANSMITTAL LETTER NO.	DATE	NATURE OF REVISION	BY	CHKD.	APPD.
R4		Sept. 1977	Pipe deflection note	PJ	MHZ	
R3		JULY 1977	Revised Det. A	MHZ	PJ	
R2		June 1977	Revised Inv. El. and length of outlet pipe	T.W.	PJ	REL
R1		5-17-77	Revised Inv. El. and length of outlet pipe	B.K.	MHZ	

KENTUCKY POWER COMPANY
BIG SANDY PLANT
BIG SANDY, KENTUCKY

BIG SANDY FLYASH DAM | SERVICE SPILLWAY

SERVICE SPILLWAY CONDUIT
SECTIONS & DETAILS

CONSULTING ENGINEERS
HARZA ENGINEERING COMPANY
APPROVED Jack Jones
CHICAGO, ILLINOIS
DATE 4-25-77
DWG.NO. 984 C 109 R4



DSGN.	REL	REVIEWED	
CHKD.		CIVIL	
DWN.	B.K.	MECH.	
CHKD.	M.H.	ELECT.	
SUBM.			

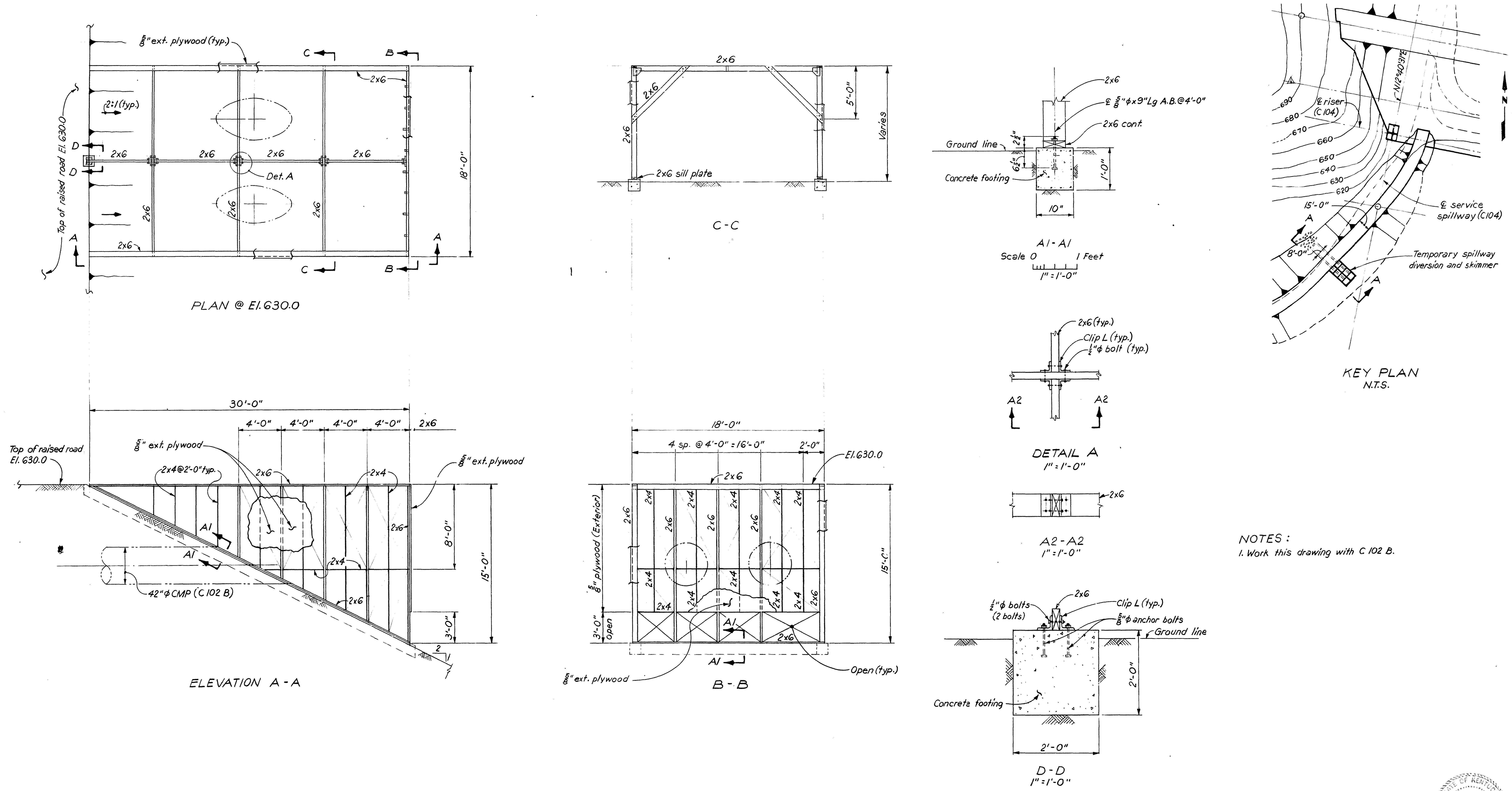
REV. NO.	DWG. TRANSMITTAL LETTER NO.	DATE	NATURE OF REVISION	BY	CHKD.	APPD.
R1		5-18-77	Revised details A ⁊ B	B.K.	M.H.	

KENTUCKY POWER COMPANY
BIG SANDY PLANT
BIG SANDY, KENTUCKY

BIG SANDY FLYASH DAM | SERVICE SPILLWAY

EXISTING SERVICE SPILLWAY ABANDONMENT DETAILS

CONSULTING ENGINEERS
HARZA ENGINEERING COMPANY
APPROVED
CHICAGO, ILLINOIS DATE 4-25-77 DWG. NO. 984 C 110 R1



NOTES:

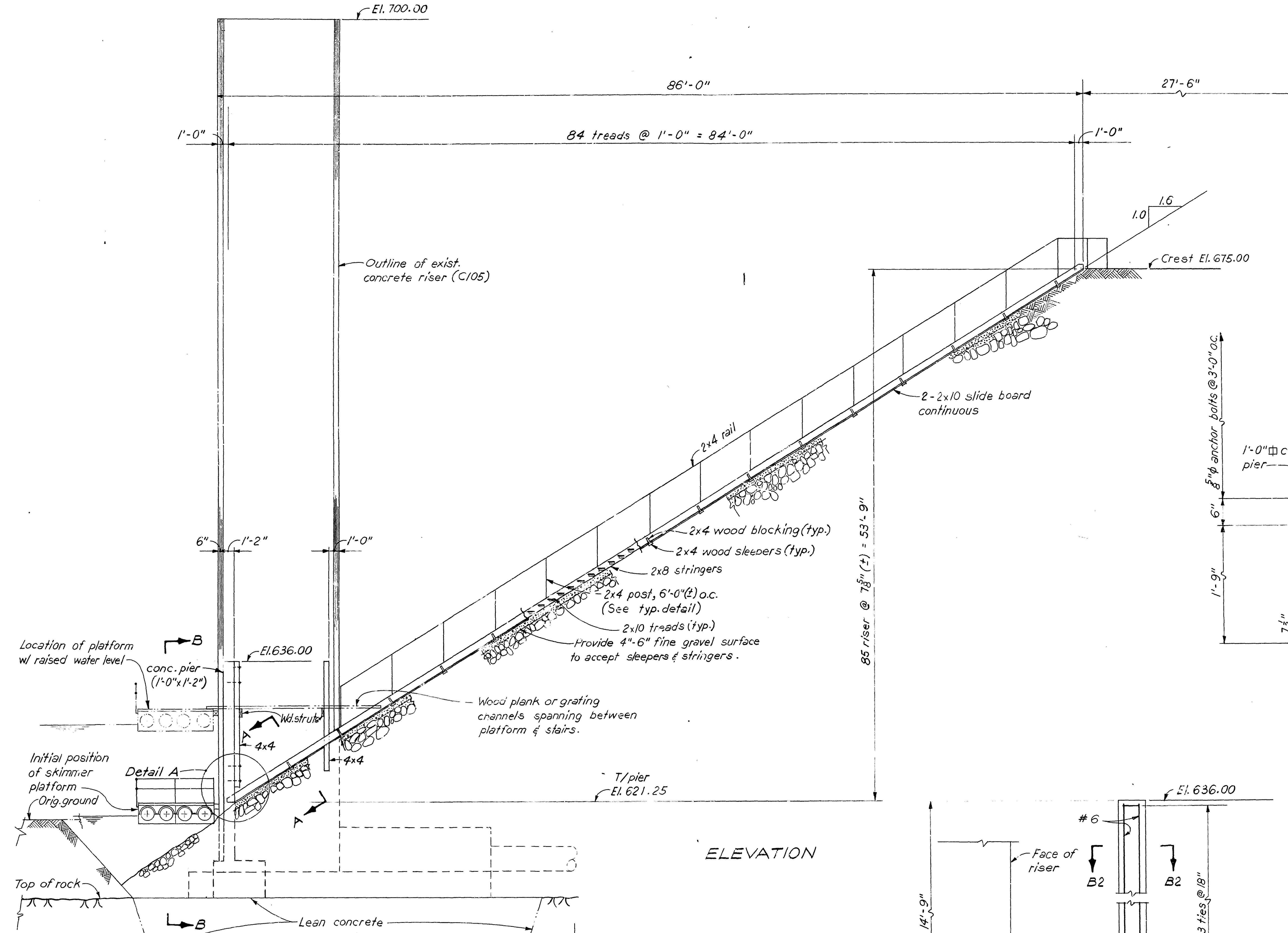
1. Work this drawing with C 102 B.

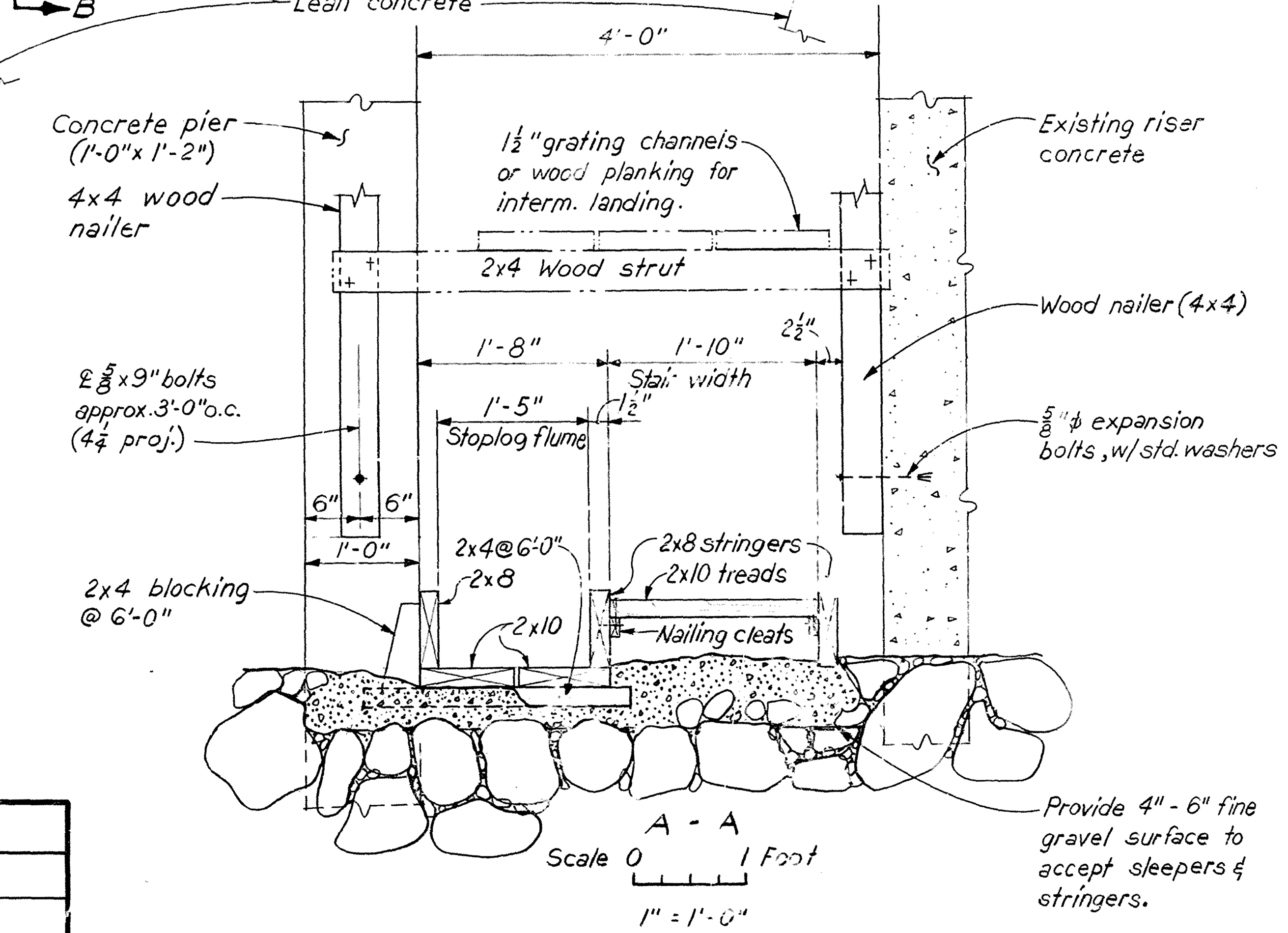
Scale 0 4 8 Feet
1/4" = 1'-0"
Except as noted

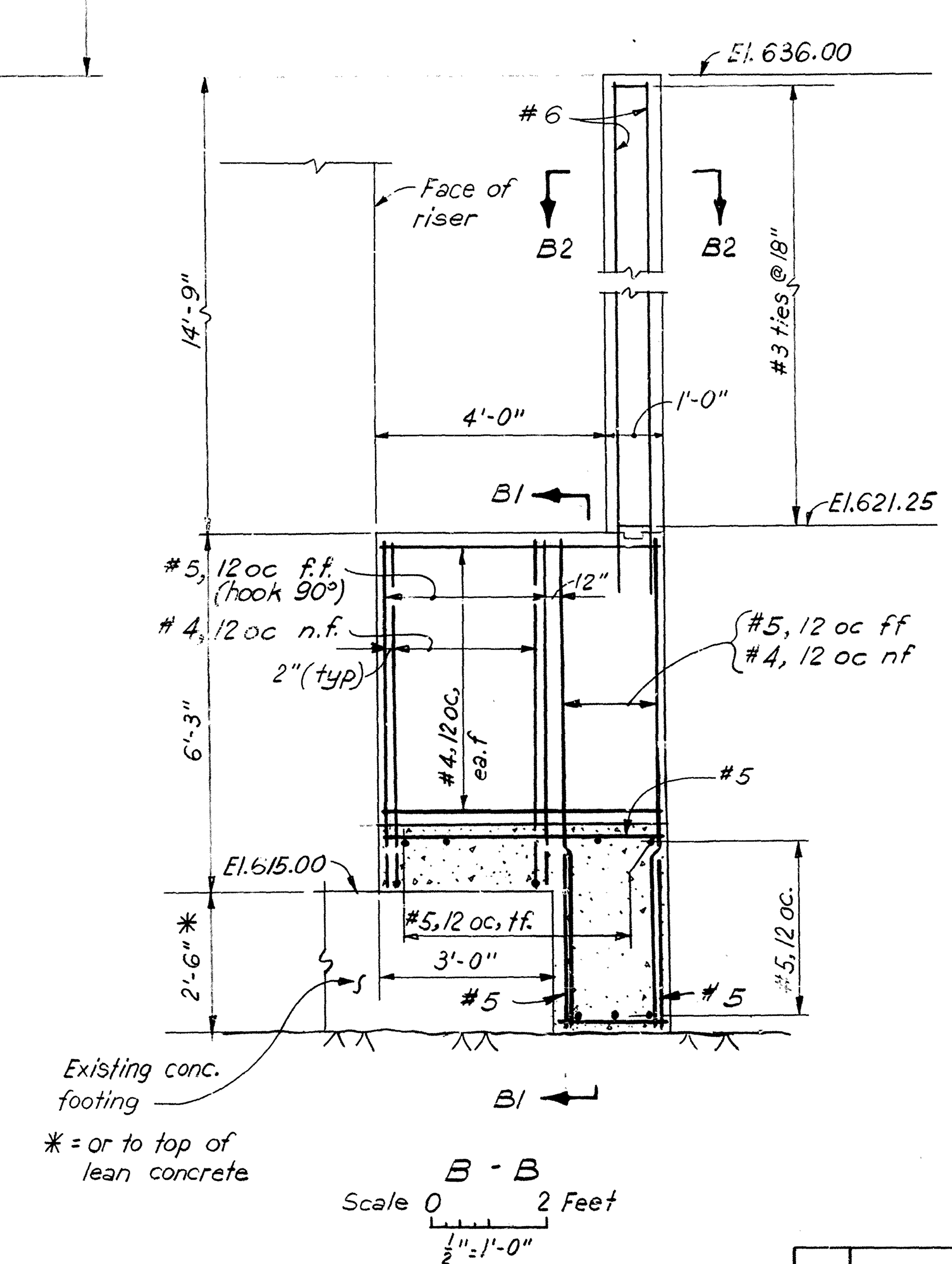
KENTUCKY POWER COMPANY BIG SANDY PLANT BIG SANDY, KENTUCKY	
BIG SANDY FLYASH DAM	SERVICE SPILLWAY
TEMPORARY WOOD SKIMMER	
CONSULTING ENGINEERS HARZA ENGINEERING COMPANY APPROVED	
CHICAGO, ILLINOIS	DATE 5-11-78 DWG. NO. 984 C III

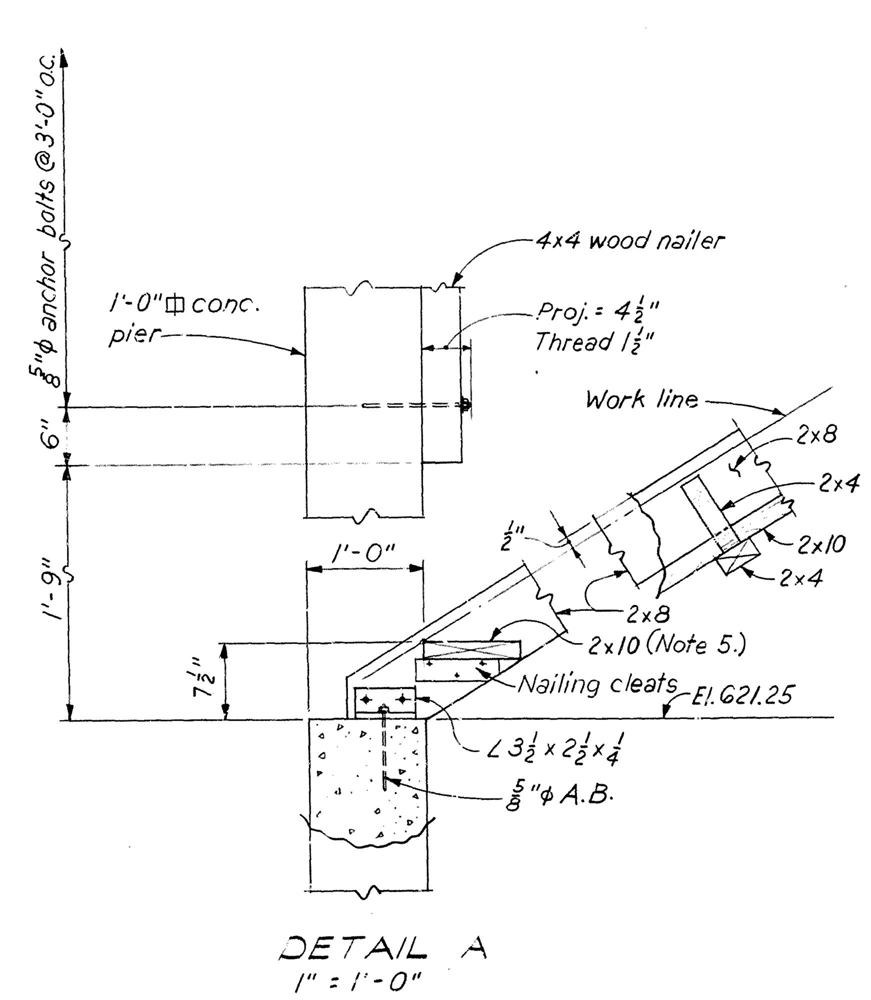
REV. NO.	DWG. TRANSMITTAL LETTER NO.	DATE	NATURE OF REVISION	BY	CHKD.	APPD.

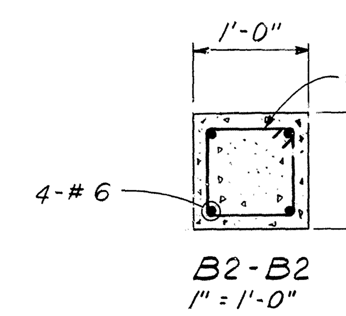
		REVIEWED	
DSGN.			
CHKD.	M.S.	CIVIL	
DWN.		MECH.	
CHKD.			
SUBM.		ELECT.	

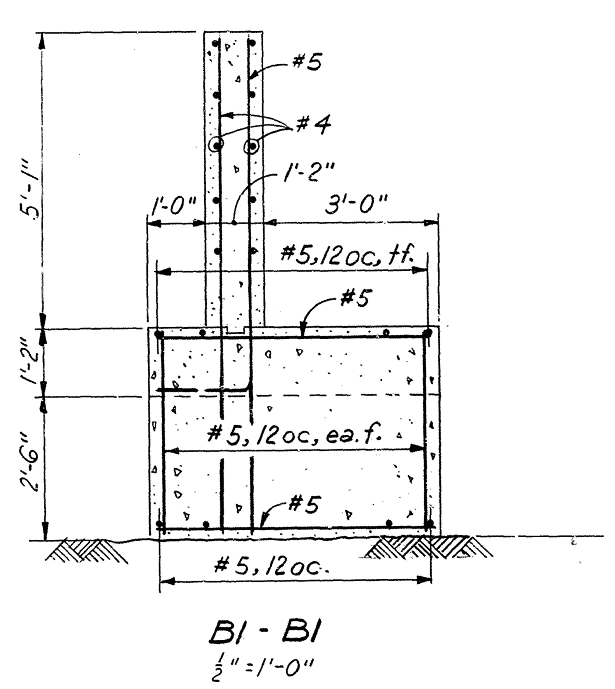


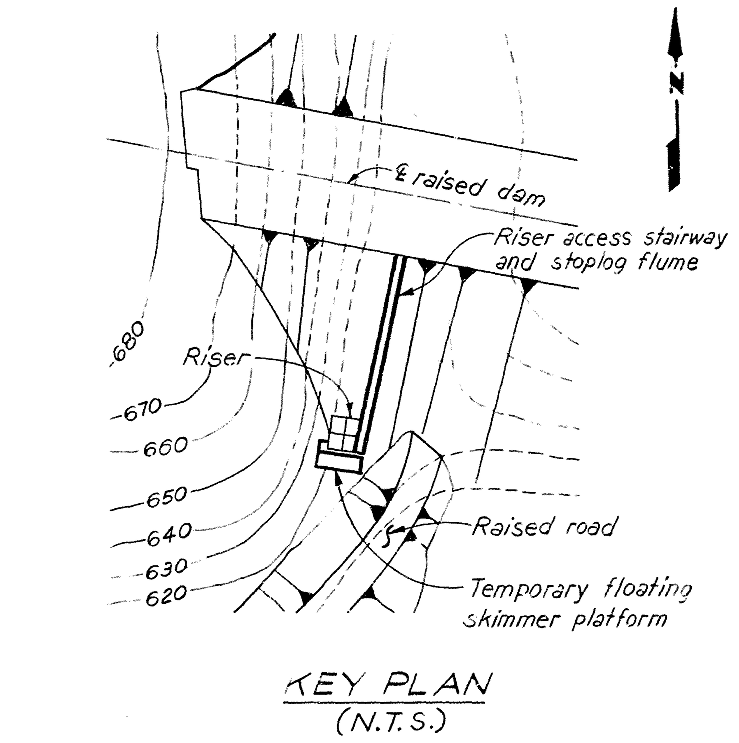


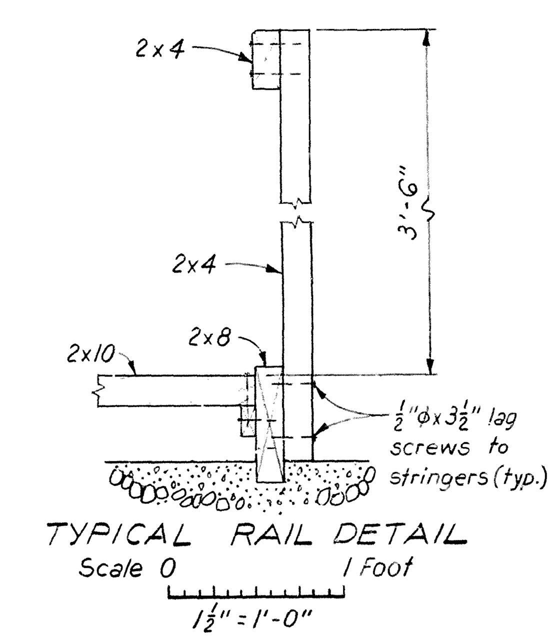












GENERAL NOTES:

1. Concrete: 4000 psi in 28 days
2. Reinforcing steel: ASTM-A615-76a, grade 60
3. Skimmer skirt shall be made of plywood exterior type DFPA, ⅝" min. thickness, or galvanized corr. sht. metal # 20 ga.
4. Location of riser access stairs and stoplog flume to be determined in field.
5. Provide skid resistant sand paint or equivalent on stair threads.
6. All wood for stairs shall be of Douglas fir construction grade, creosote treated or equal.
7. Lag screws and nails shall be galvanized.
8. Rubber sealing flaps shall be Neoprene Rubber Commercial grade Mc Master-Carr Cat. No. 8568K22 or equal

Scale 0 5 10 Feet
⅛" = 1'-0"
Unless otherwise noted

DSGN.	HS	REVIEWED	
CHKD.	HS	CIVIL	
DWN.		MECH.	
CHKD.		ELECT.	
SUBM.			

REV. NO.	DWG. TRANSMITTAL LETTER NO.	DATE	NATURE OF REVISION	BY	CHKD.	APPD.

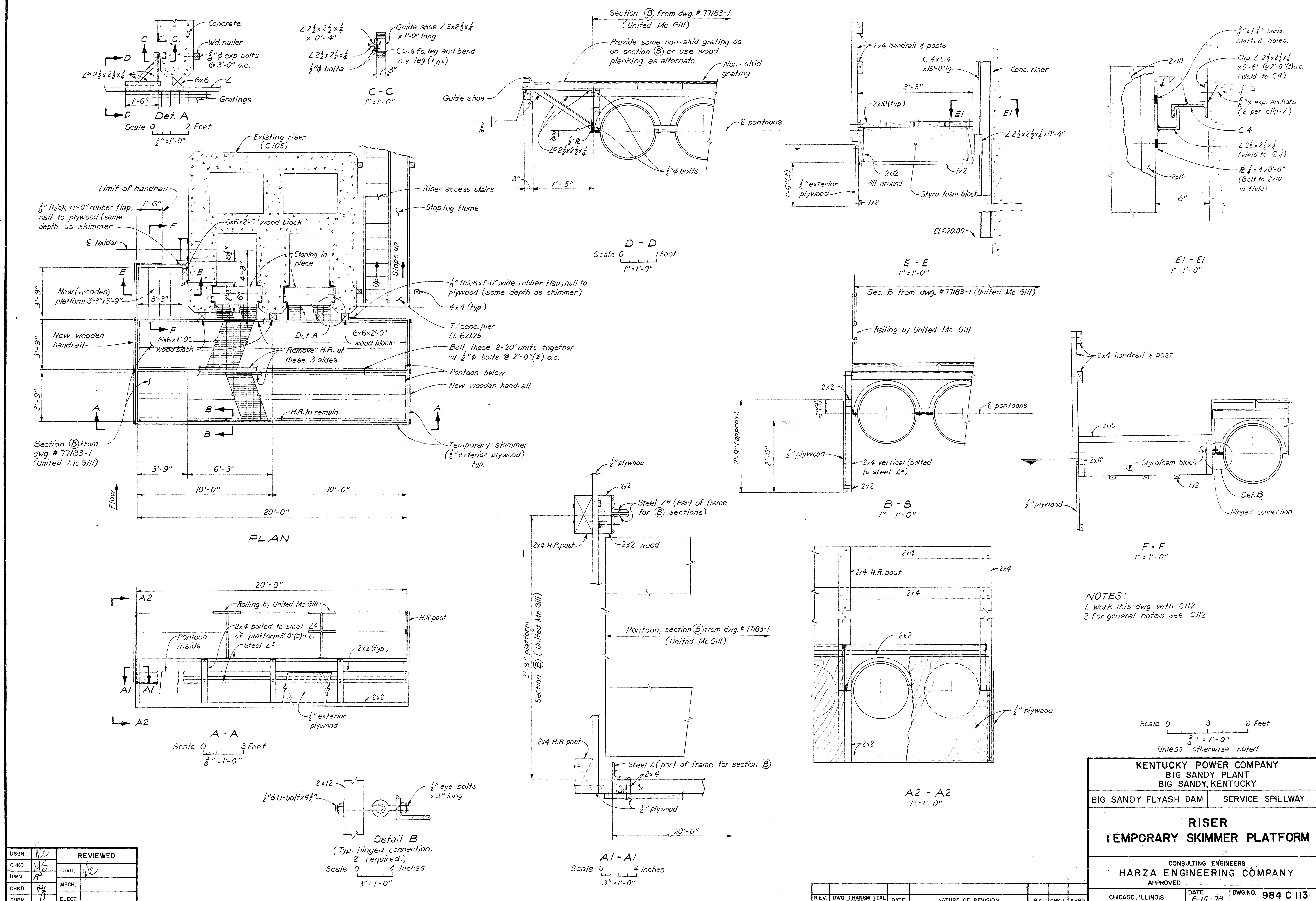
KENTUCKY POWER COMPANY
BIG SANDY PLANT
BIG SANDY, KENTUCKY

BIG SANDY FLYASH DAM | SERVICE SPILLWAY

RISER ACCESS STAIRS AND STOPLOG FLUME

CONSULTING ENGINEERS
HARZA ENGINEERING COMPANY
APPROVED

CHICAGO, ILLINOIS | DATE 6-15-78 | DWG. NO. 984 C 112



NOTES:
1. Work this dwg. with C112
2. For general notes see C112

KENTUCKY POWER COMPANY
BIG SANDY PLANT
BIG SANDY, KENTUCKY

BIG SANDY FLYASH DAM | SERVICE SPILLWAY

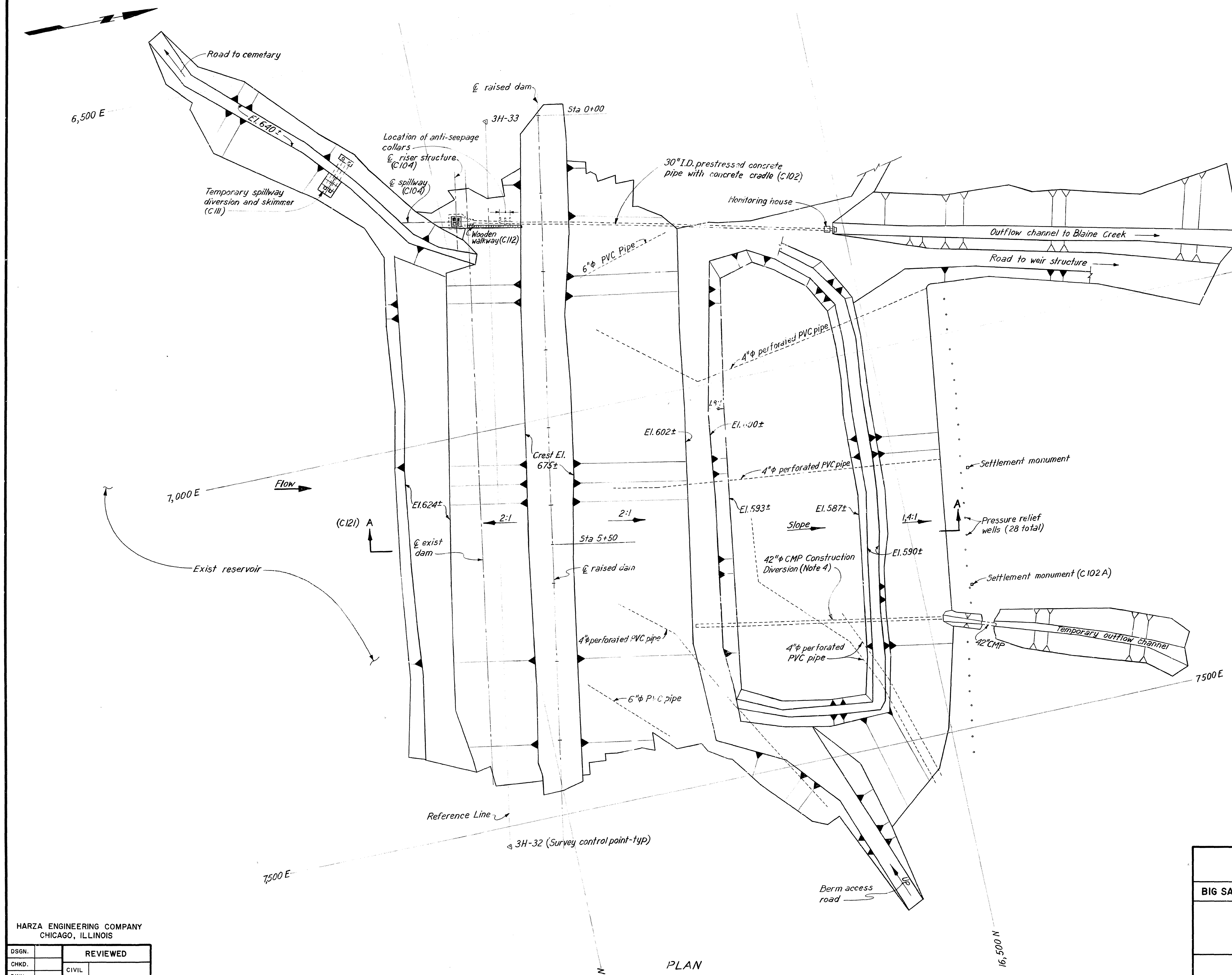
RISER
TEMPORARY SKIMMER PLATFORM

CONSULTING ENGINEERS
HARZA ENGINEERING COMPANY
APPROVED

CHICAGO, ILLINOIS — DATE 6-15-78 — DWG. NO. 984 C 113

REV. NO. | DWG. TRANSMITTAL LETTER NO. | DATE | NATURE OF REVISION | BY | CHKD. | APPD.

DSGN. / CHKD. / DWN. / CHKD. / SUBM.
REVIEWED: CIVIL / MECH. / ELECT.



NOTES:

1. Work this drawing with C100, 101, 102, 102A, 102 B, 104, 110 and 111, and C 121
2. Plan view reproduced from Kentucky Power Co. drawing by J.P. Diaz, dated 12/12/78
3. Plan outline and other information shown obtained from field records and survey information supplied Kentucky Power Co.
4. Construction diversion pipe filled with concrete and abandoned from service

Scale 0 50 100 Feet
1"=50'

HARZA ENGINEERING COMPANY
CHICAGO, ILLINOIS

		REVIEWED	
DSGN.			
CHKD.		CIVIL	
DWN.		MECH.	
CHKD.			
SUBM.		ELECT.	

REV. NO.	DWG. TRANSMITTAL LETTER NO.	DATE	NATURE OF REVISION	BY	CHKD.	APPD.

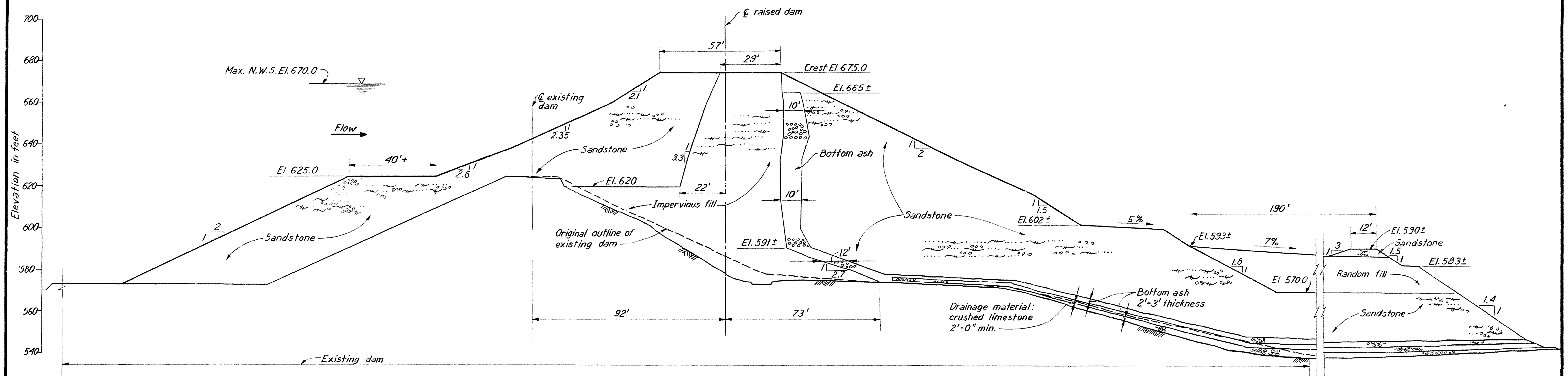
KENTUCKY POWER COMPANY
BIG SANDY PLANT
BIG SANDY, KENTUCKY

BIG SANDY FLYASH DAM | DAM RAISING

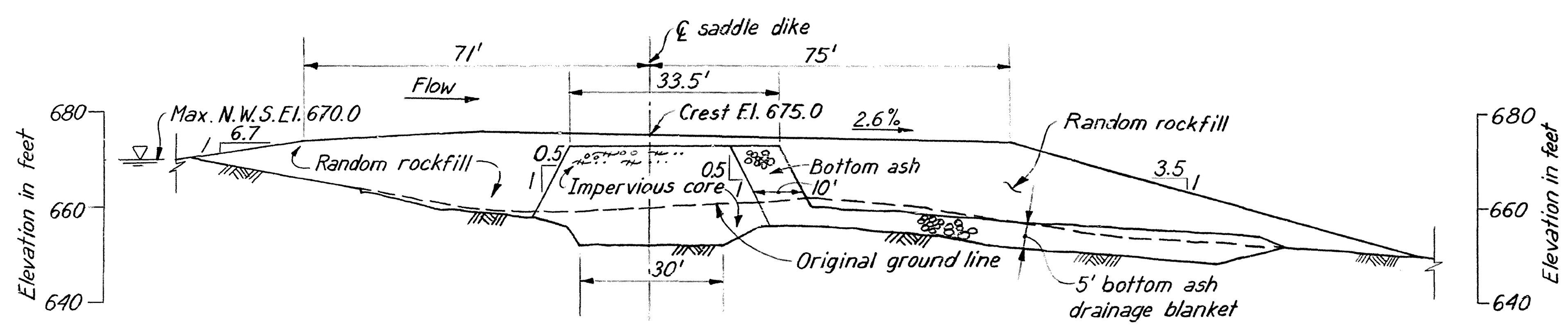
DRAWING OF RECORD:
DAM

AMERICAN ELECTRIC POWER
SERVICE CORP.
APPROVED ______________

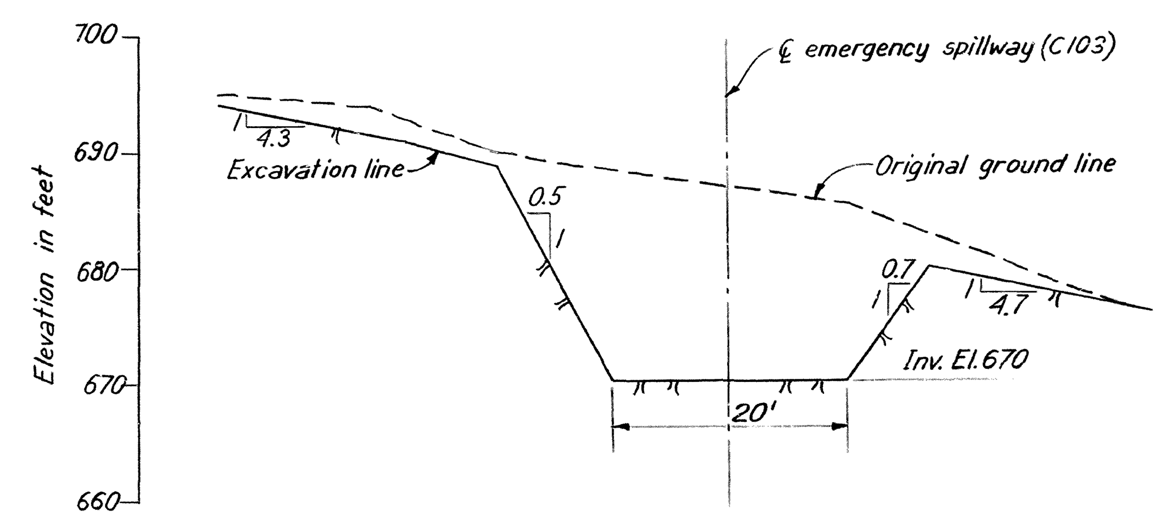
CHICAGO, ILLINOIS | DATE FEB. 1979 | DWG. NO. 984 C 120



DAM SECTION AT STA. 5+50



SADDLE DIKE SECTION
AT STA. 1+00



EMERGENCY SPILLWAY SECTION AT STA 1+00

Scale 0 10 Feet

NOTES:

1. Work this drawing with C101, C102, C102B, C103, C120.
2. Sections shown obtained from field records and survey information supplied by K.P.C.
3. Dimensions, elevations, slopes and grades, etc. shown obtained by scaling from field sections.

Scale 0 20 40 Feet
Unless noted

KENTUCKY POWER COMPANY
BIG SANDY PLANT
BIG SANDY, KENTUCKY

BIG SANDY FLYASH DAM	DAM RAISING

DRAWING OF RECORD: DAM AND SADDLE DIKE SECTIONS, EMERGENCY SPILLWAY

AMERICAN ELECTRIC POWER SERVICE CORP.
APPROVED ______________

CHICAGO, ILLINOIS	DATE FEB. 1979	DWG. NO. 984 C 121

REV. NO.	DWG. TRANSMITTAL LETTER NO.	DATE	NATURE OF REVISION	BY	CHKD.	APPD.

HARZA ENGINEERING COMPANY
CHICAGO, ILLINOIS

		REVIEWED	
DSGN.			
CHKD.		CIVIL	
DWN.			
CHKD.		MECH.	
SUBM.		ELECT.	

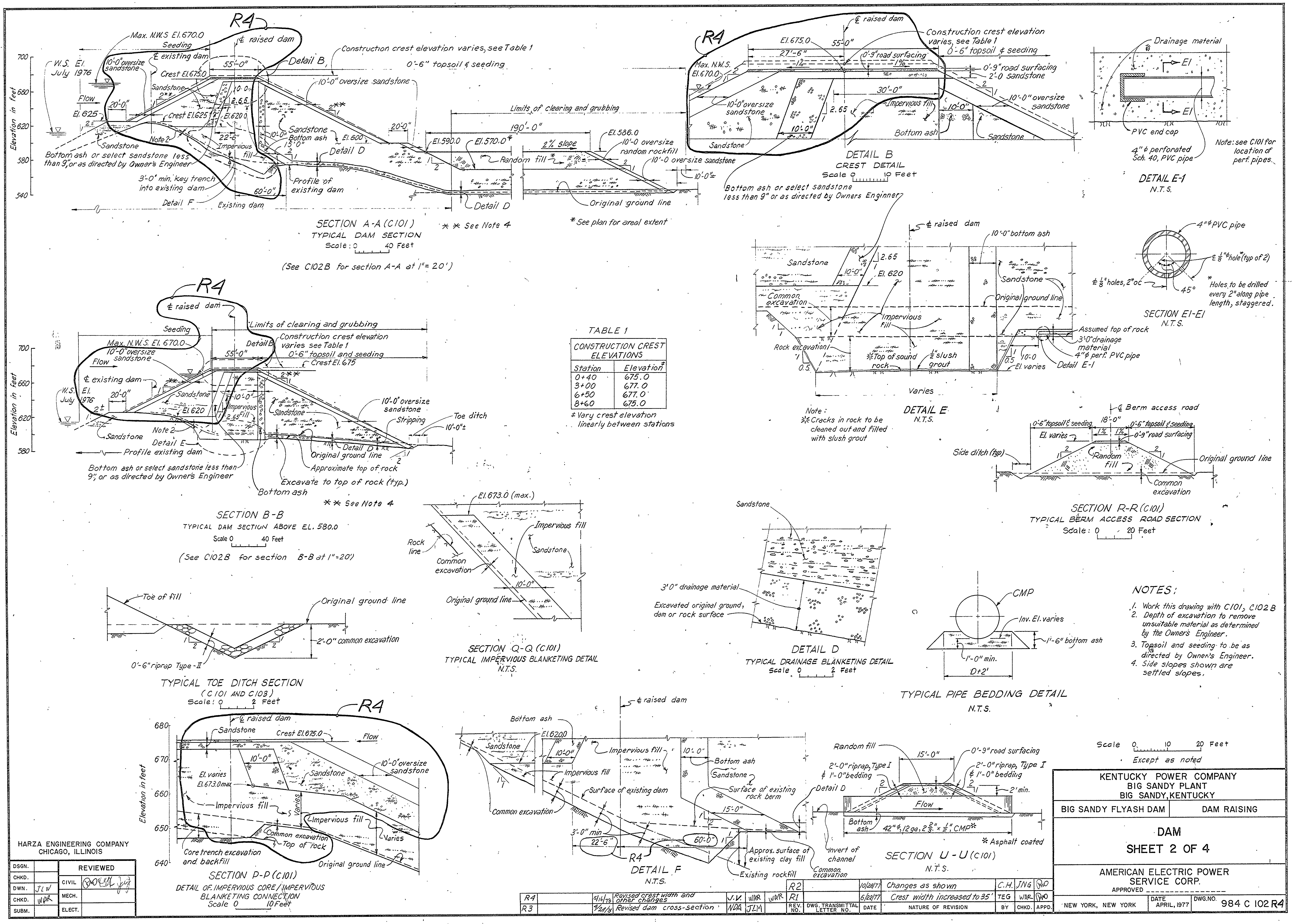


TABLE 1

CONSTRUCTION CREST ELEVATIONS	
Station	Elevation‡
0+40	675.0
3+00	677.0
6+50	677.0
8+60	675.0

‡ Vary crest elevation linearly between stations

NOTES:

1. Work this drawing with C101, C102B
2. Depth of excavation to remove unsuitable material as determined by the Owner's Engineer.
3. Topsoil and seeding to be as directed by Owner's Engineer.
4. Side slopes shown are settled slopes.

Scale 0 10 20 Feet
· Except as noted

REV. NO.	DATE	NATURE OF REVISION	BY	CHKD.	APPD.
R4	5/16/78	Revised crest width and other changes	J.V.	WAR	WAR
R3	9/21/78	Revised dam cross-section	NDA	JLM	
R2	10/28/77	Changes as shown	C.H.	JNG	
R1	6/28/77	Crest width increased to 35'	TEG	WAR	

HARZA ENGINEERING COMPANY
CHICAGO, ILLINOIS

		REVIEWED
DSGN.		
CHKD.		
DWN. JLW	CIVIL	
CHKD. WAR	MECH.	
SUBM.	ELECT.	

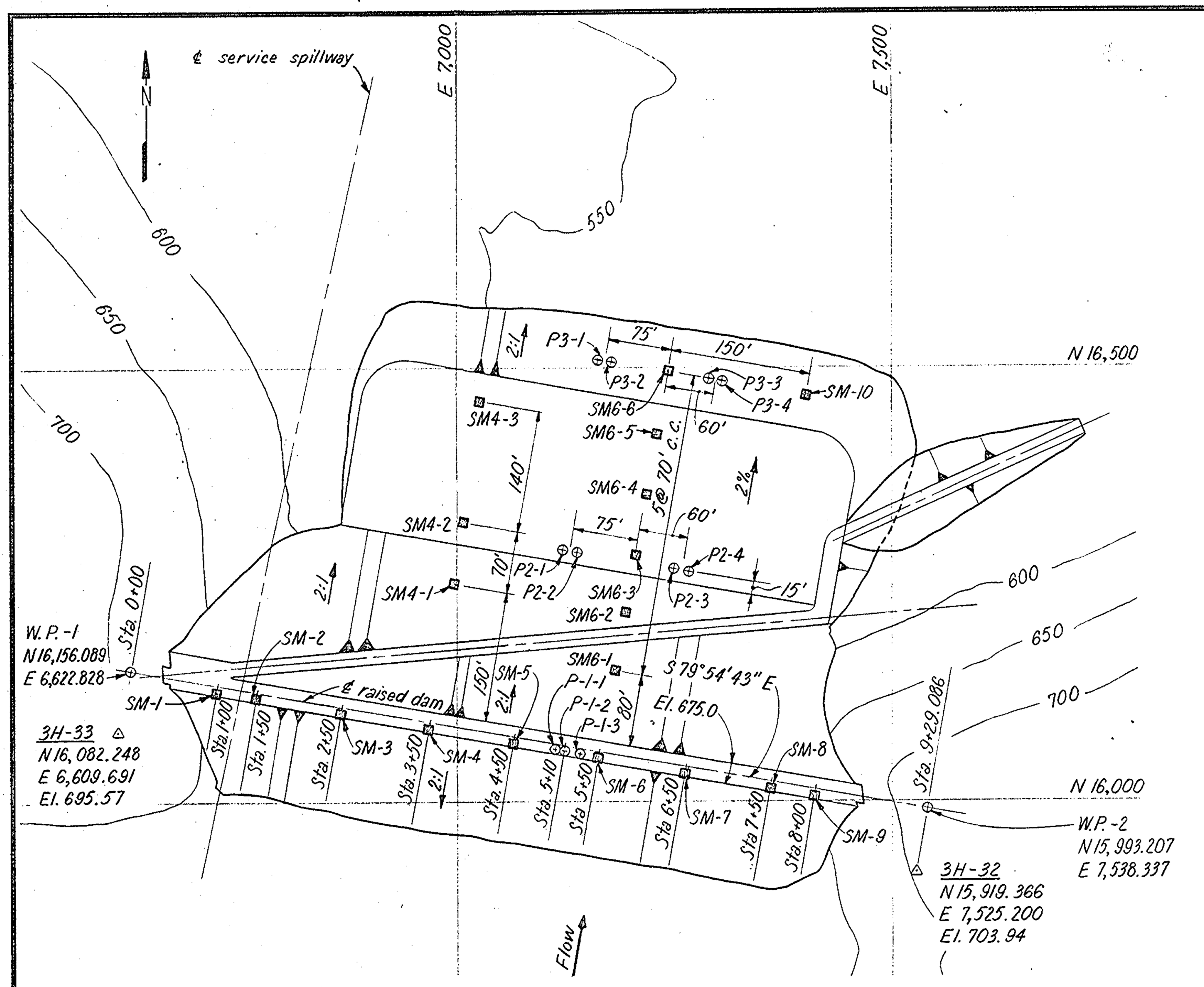
KENTUCKY POWER COMPANY
BIG SANDY PLANT
BIG SANDY, KENTUCKY

BIG SANDY FLYASH DAM — DAM RAISING

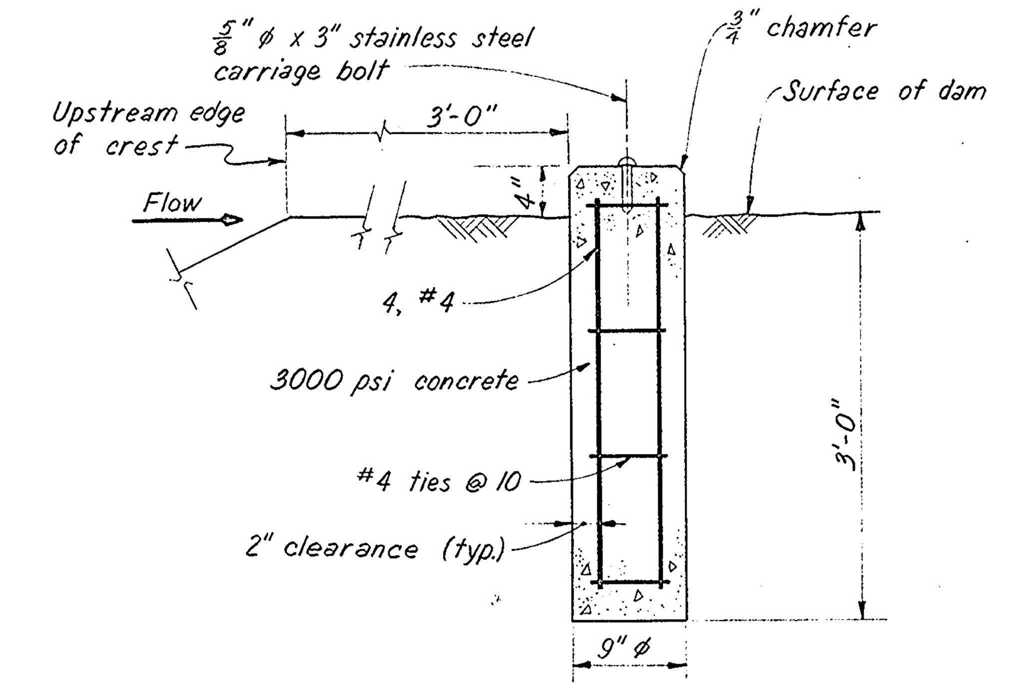
DAM
SHEET 2 OF 4

AMERICAN ELECTRIC POWER SERVICE CORP.
APPROVED

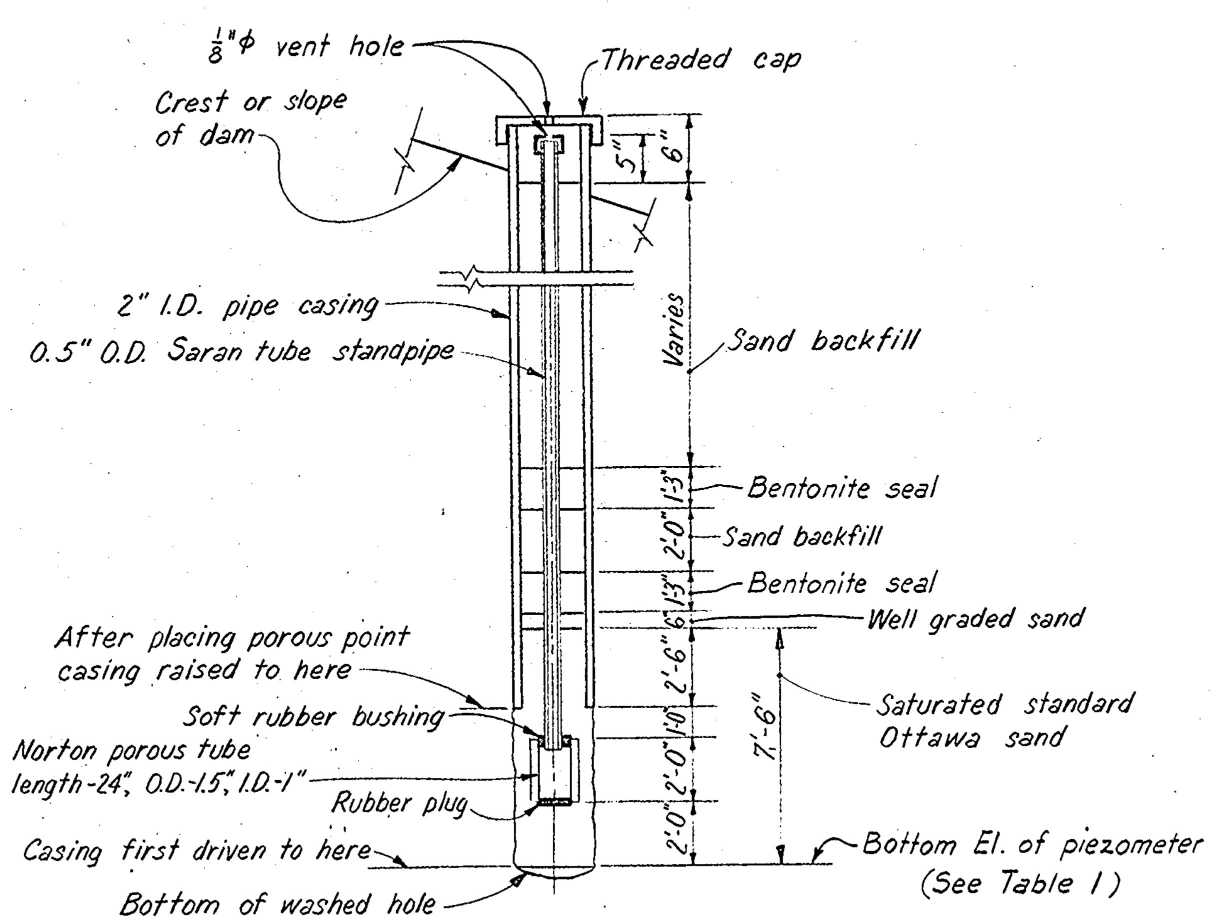
NEW YORK, NEW YORK — DATE APRIL, 1977 — DWG. NO. 984 C 102 R4



INSTRUMENT/ON LOCATION PLAN



SURFACE SETTLEMENT MONUMENT (19 REQ'D)
(SEE C 100)
N.T.S.



TYPICAL PIEZOMETER DETAIL
N.T.S.

TABLE 1
BOTTOM ELEVATION OF PIEZOMETER

Piezometer No.	Bottom Elevation
P1-1	*
P1-2	559
P1-3	625
P2-1	*
P2-2	530
P2-3	*
P2-4	530
P3-1	*
P3-2	530
P3-3	*
P3-4	530

* 15' below top of rock

LEGEND:
- ▫ SM-2 - Surface settlement movement.
- ⊕ P3-2 - Piezometer

NOTES:
1. Work this drawing with C 101

Scale 0 ___ 100 Feet
Except as noted

HARZA ENGINEERING COMPANY
CHICAGO, ILLINOIS

		REVIEWED	
DSGN.			
CHKD.		CIVIL	
DWN.			
CHKD.	WMP	MECH.	
SUBM.		ELECT.	

REV. NO.	DWG. TRANSMITTAL LETTER NO.	DATE	NATURE OF REVISION	BY	CHKD.	APPD.

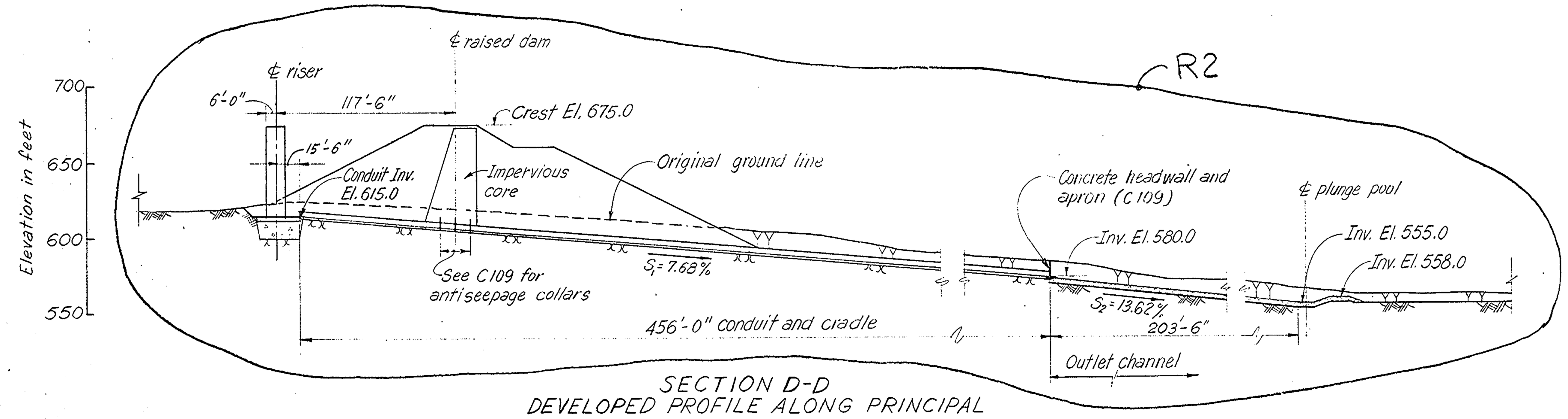
KENTUCKY POWER COMPANY
BIG SANDY PLANT
BIG SANDY, KENTUCKY

BIG SANDY FLYASH DAM | INSTRUMENTATION

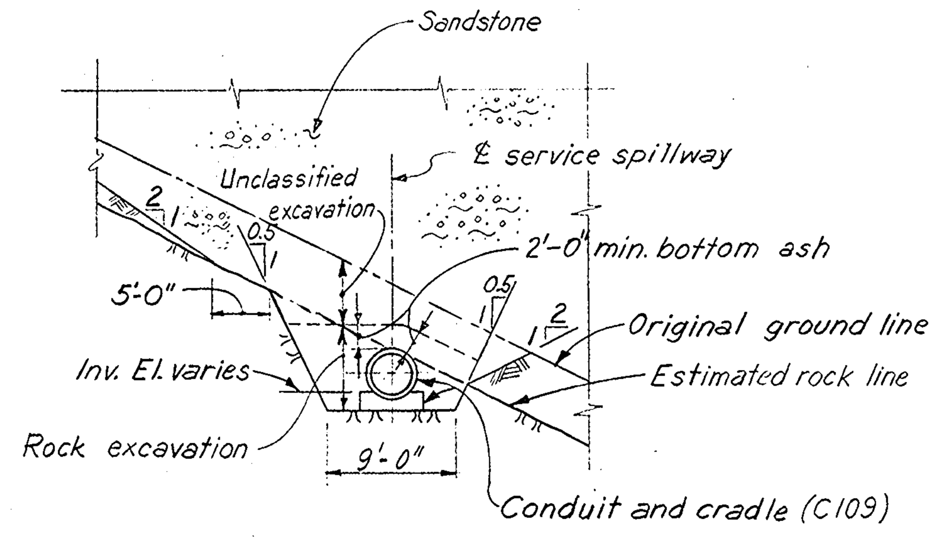
DAM
SHEET 3 OF 3

AMERICAN ELECTRIC POWER SERVICE CORP.
APPROVED

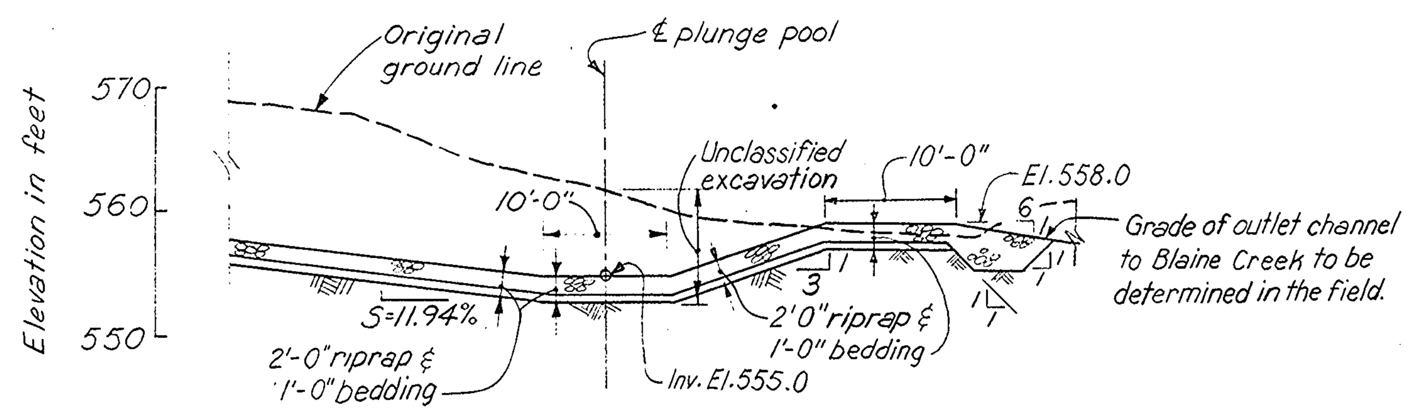
NEW YORK, NEW YORK | DATE APRIL, 1977 | DWG. NO. 984 C 102A



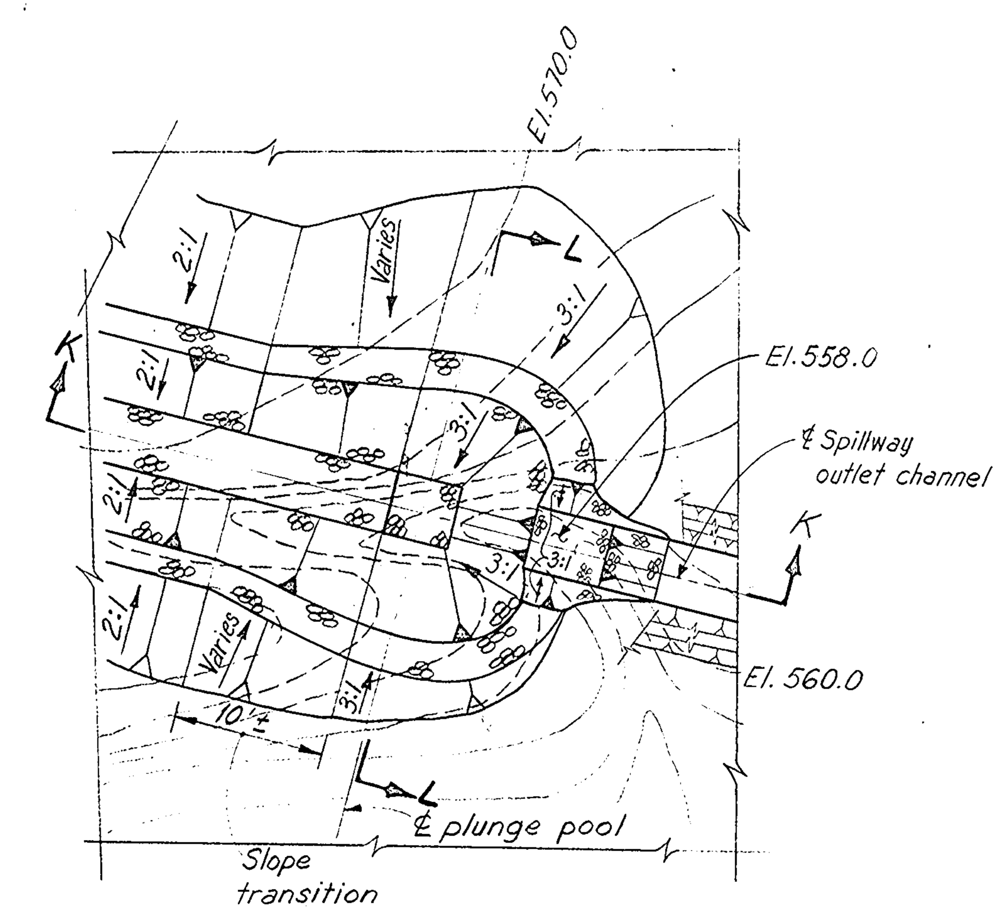
SECTION D-D
DEVELOPED PROFILE ALONG PRINCIPAL SPILLWAY CENTERLINE
Scale 0 50 Feet



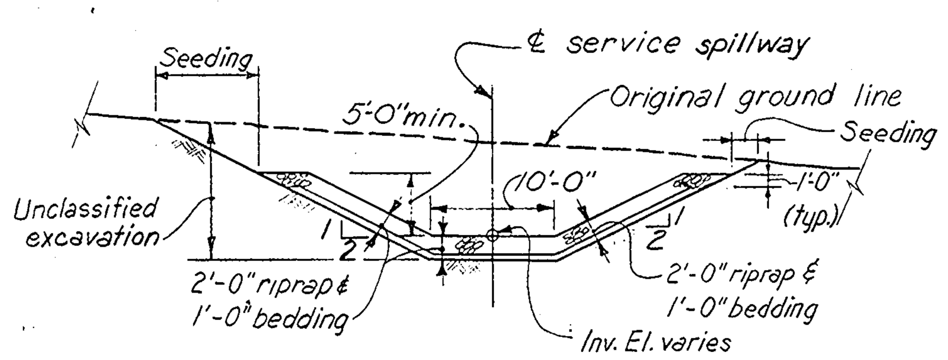
SECTION E-E
(984 C 101)
Scale 0 10 Feet



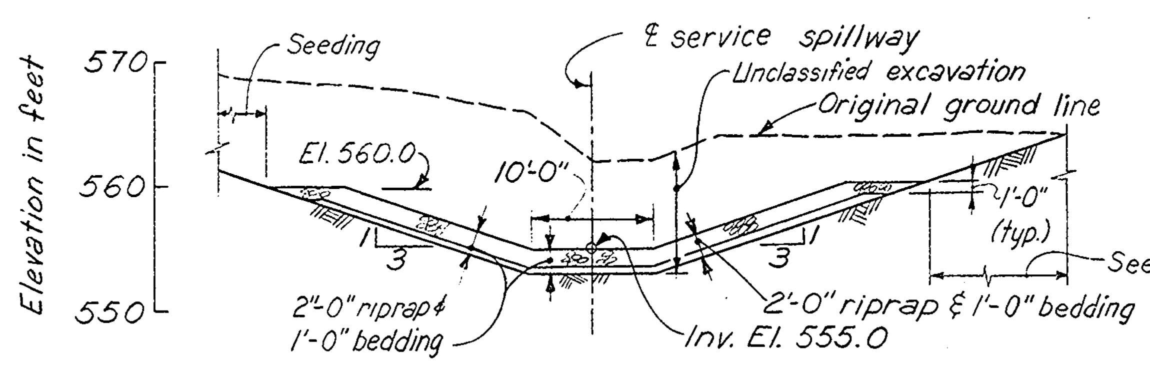
SECTION K-K
Scale 0 10 Feet



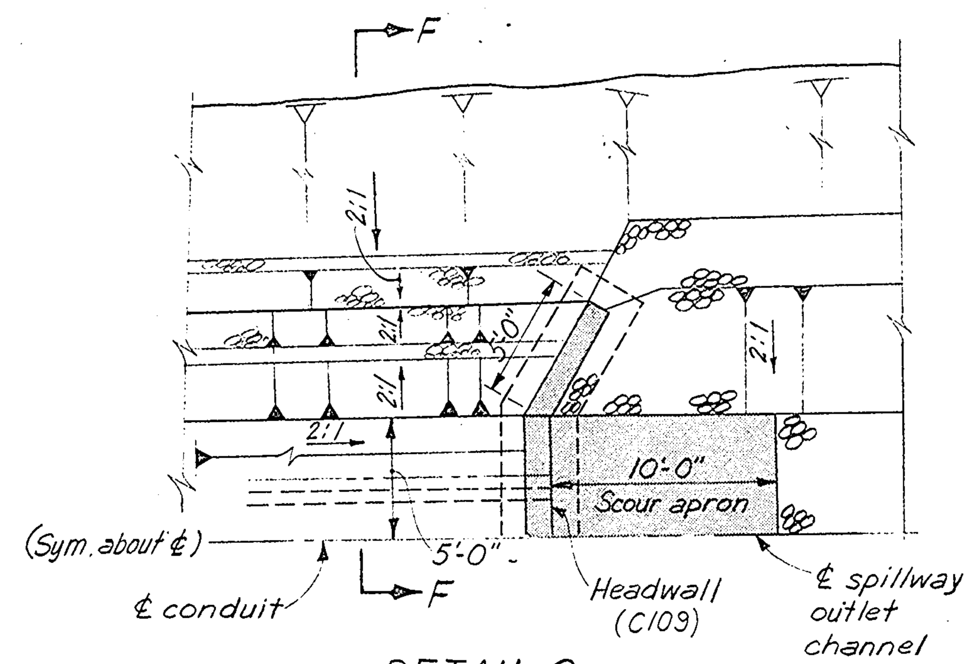
DETAIL A
(984 C 101)
PLAN - PLUNGE POOL
Scale 0 20 Feet



SECTION H-H
(984 C 101)
TYPICAL OUTLET CHANNEL SECTION
Scale 0 10 Feet



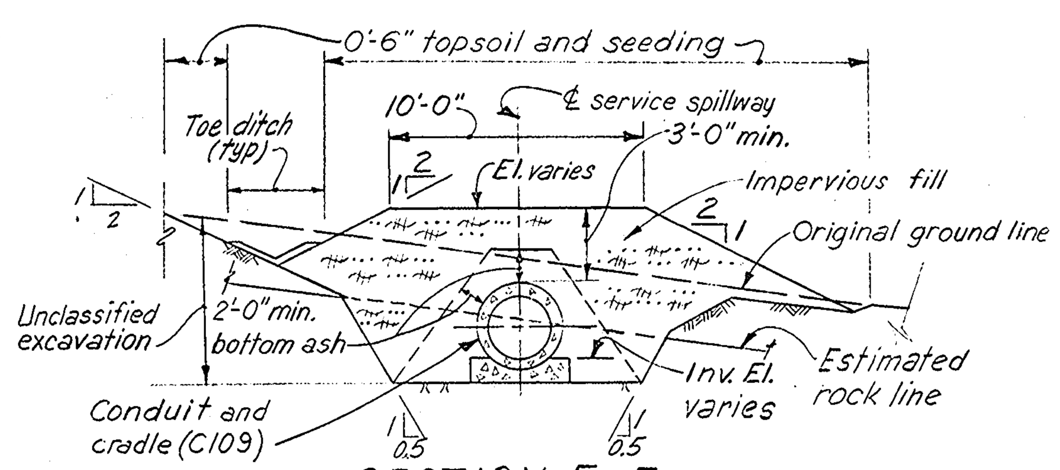
SECTION L-L
Scale 0 10 Feet



DETAIL C
(984 C 101)
CONDUIT OUTLET DETAIL
N.T.S.

REFERENCE DRAWINGS:
RISER C105 - C108
CONDUIT C109

NOTES:
1. Work this drawing with C101.
2. All riprap for the service spillway outlet channel and plunge pool is Riprap - Type I.
3. Topsoil and seeding to be at the direction of Owner's Engineer.



SECTION F-F
(984 C 101)
Scale 0 10 Feet

DSGN.	WDR	REVIEWED	
CHKD.		CIVIL	
DWN.	L.L.	MECH.	
CHKD.	WDR	ELECT.	
SUBM.			

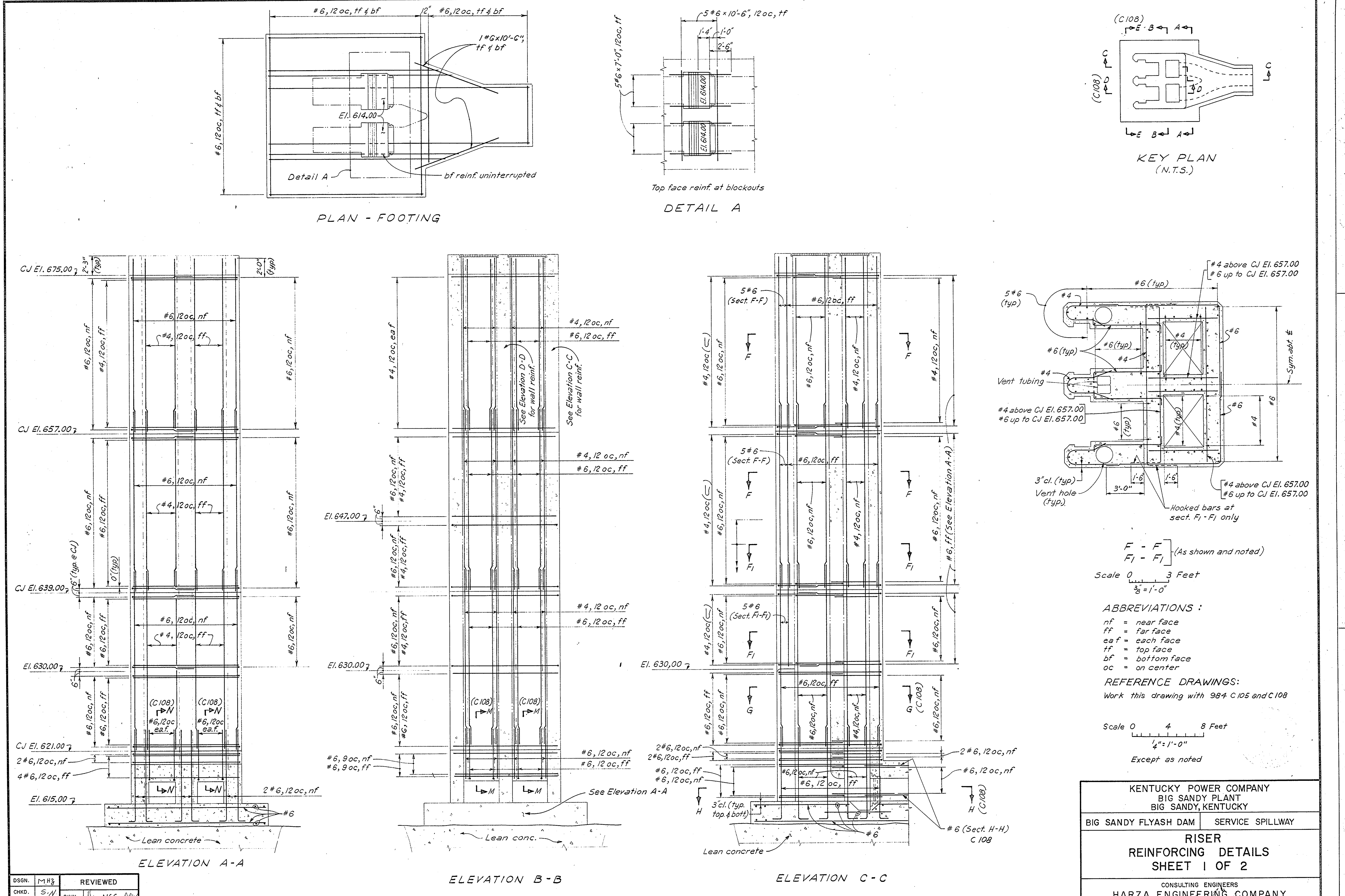
REV. NO.	DWG. TRANSMITTAL LETTER NO.	DATE	NATURE OF REVISION	BY	CHKD.	APPD.
R2		6/23/77	Revised inv. El. and length of conduit		WMR	
R1		5/6/77	Service Spillway Conduit Extension			

KENTUCKY POWER COMPANY
BIG SANDY PLANT
BIG SANDY, KENTUCKY

BIG SANDY FLYASH DAM | DAM RAISING

SERVICE SPILLWAY
EARTHWORK DETAILS

CONSULTING ENGINEERS
HARZA ENGINEERING COMPANY
APPROVED
CHICAGO, ILLINOIS | DATE April, 1977 | DWG. NO. 984 C 104 R2



ABBREVIATIONS:

nf = near face
ff = far face
ea f = each face
tf = top face
bf = bottom face
oc = on center

REFERENCE DRAWINGS:

Work this drawing with 984 C105 and C108

Scale 0 4 8 Feet
1/4" = 1'-0"
Except as noted

KENTUCKY POWER COMPANY
BIG SANDY PLANT
BIG SANDY, KENTUCKY

BIG SANDY FLYASH DAM — SERVICE SPILLWAY

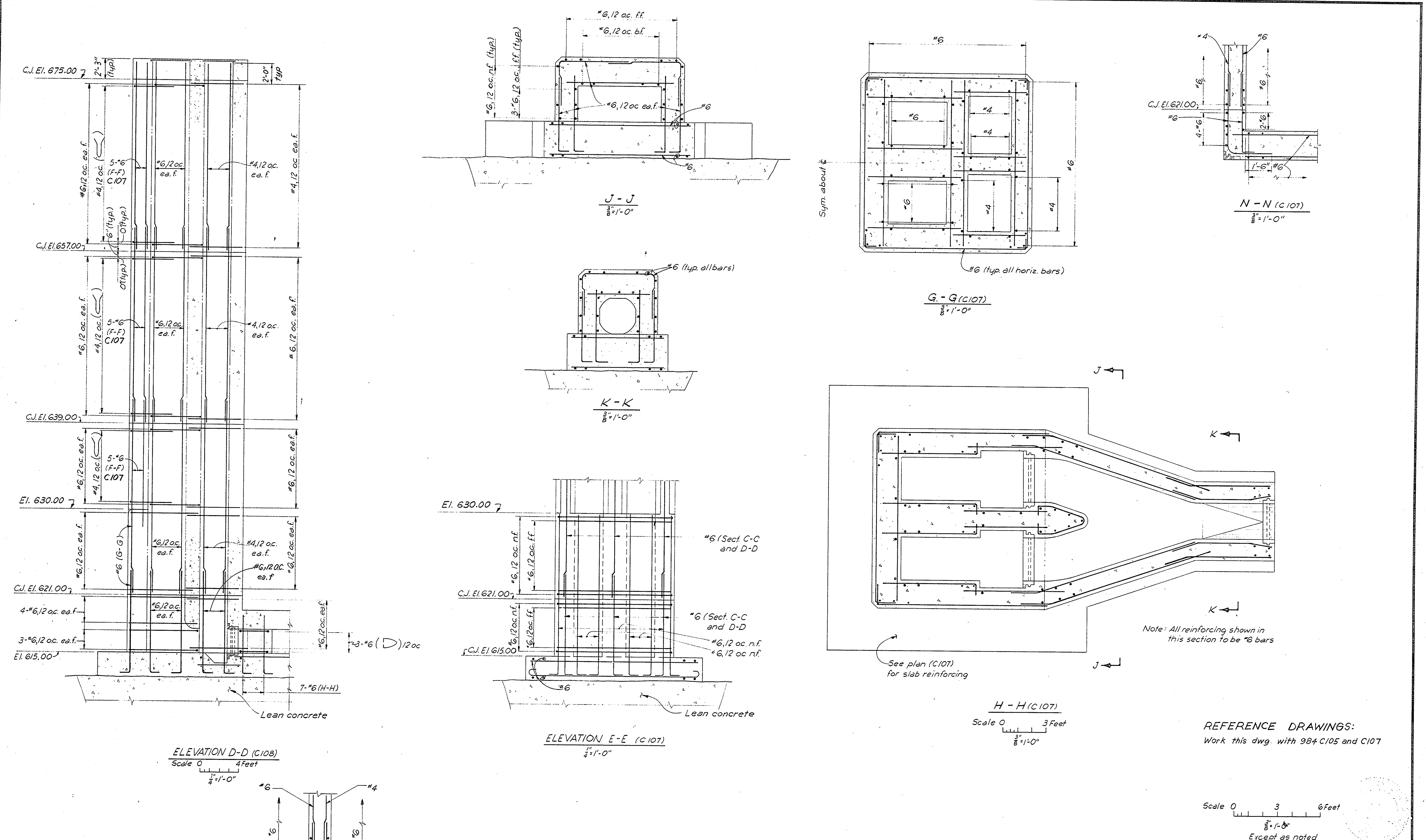
RISER REINFORCING DETAILS SHEET 1 OF 2

CONSULTING ENGINEERS
HARZA ENGINEERING COMPANY
APPROVED

CHICAGO, ILLINOIS — DATE 4-25-77 — DWG. NO. 984 C 107

REV. NO.	DWG. TRANSMITTAL LETTER NO.	DATE	NATURE OF REVISION	BY	CHKD.	APPD.

		REVIEWED	
DSGN.	MHZ		
CHKD.	S.N	CIVIL	MSG
DWN.	B.K.	MECH.	
CHKD.	MHZ		
SUBM.		ELECT.	



REFERENCE DRAWINGS:
Work this dwg. with 984 C105 and C107

Scale 0 3 6 Feet
3/8" = 1'-0"
Except as noted

DSGN.	MHZ	REVIEWED	
CHKD.	S.N.	CIVIL	HSG
DWN.	T.W. B.K.	MECH.	
CHKD.	MHZ	ELECT.	
SUBM.			

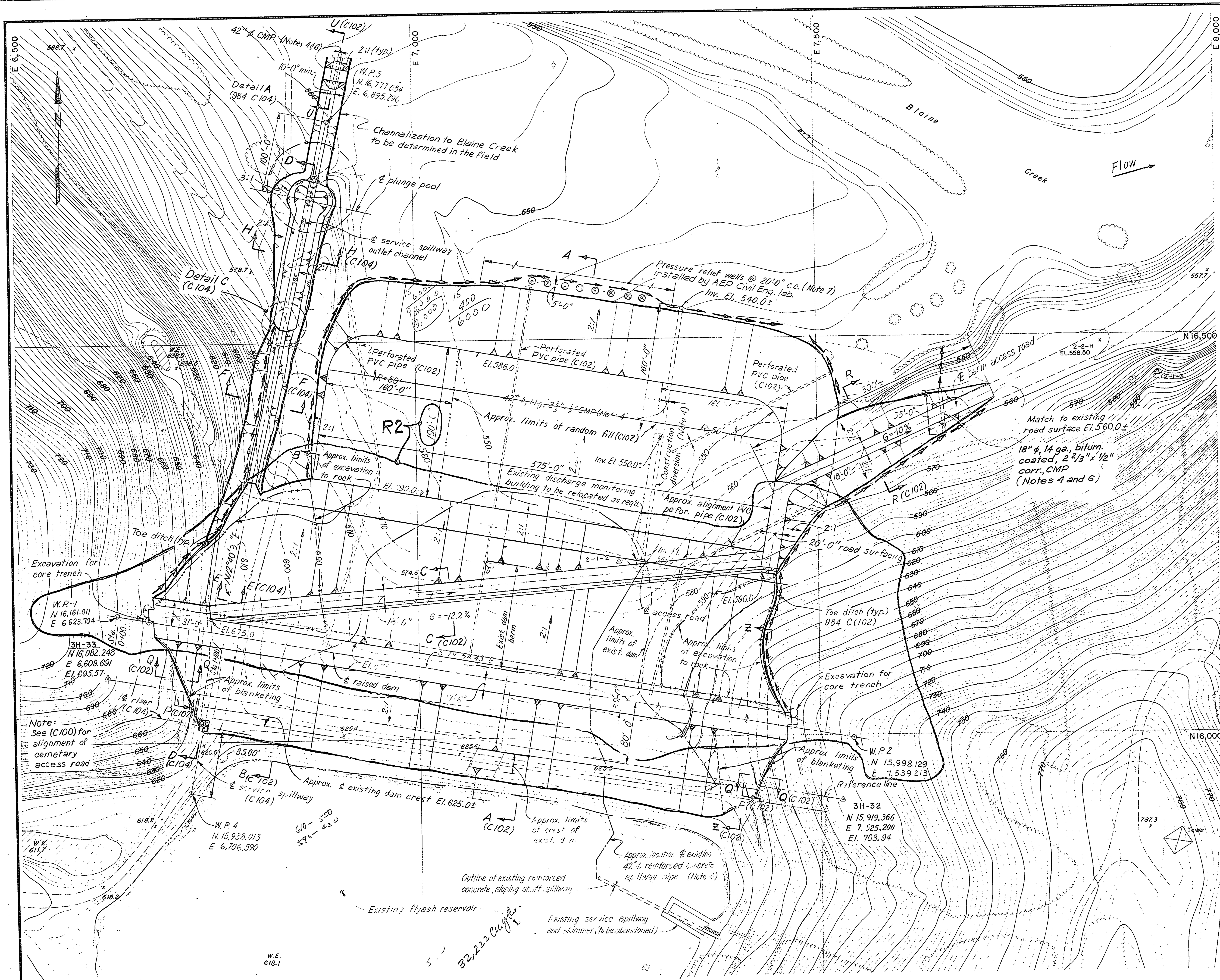
REV. NO.	DWG. TRANSMITTAL LETTER NO.	DATE	NATURE OF REVISION	BY	CHKD.	APPD.

KENTUCKY POWER COMPANY
BIG SANDY PLANT
BIG SANDY, KENTUCKY

BIG SANDY FLYASH DAM | SERVICE SPILLWAY

RISER
REINFORCING DETAILS
SHEET 2 OF 2

CONSULTING ENGINEERS
HARZA ENGINEERING COMPANY
APPROVED
CHICAGO, ILLINOIS | DATE 4-25-77 | DWG. NO. 984 C 108



HARZA ENGINEERING COMPANY
CHICAGO, ILLINOIS

DSGN.		REVIEWED	
CHKD.			
DWN.		CIVIL	
CHKD.	WAR	MECH.	
SUBM.		ELECT.	

PLAN

LEGEND:

- △ Kentucky Power Co. benchmark
- Fill slope
- Cut slope
- Limits of max. normal water surface El. 670.0
- Toe ditch (C102)
- ◎ Pressure relief well

NOTES:

1. Work this drawing with C102, C102A, and C104.
2. Topography supplied by Kentucky Power Co. Date of mapping 1976.
3. All exploration performed by Kentucky Power Co.
4. Corrugated metal pipe as manufactured by Armco or equal.
5. Both existing concrete spillway pipe and construction diversion to be concreted upon completion of construction as directed.
6. A minimum 2'-0" cover shall be placed over top of pipe. Location of CMP shall be determined in the field.
7. Pressure relief wells to extend 25'-0" into rock. Number of wells to be determined in the field.

Scale 0 50 100 Feet

REV. NO.	DWG. TRANSMITTAL LETTER NO.	DATE	NATURE OF REVISION	BY	CHKD.	APPD.
R2		6/22/77	Crest width changed to 35'	TEG	WAR	RMO
R1		5/16/77	Service Spillway Conduit Extension		WAR	RMO

KENTUCKY POWER COMPANY
BIG SANDY PLANT
BIG SANDY, KENTUCKY

BIG SANDY FLYASH DAM — DAM RAISING

DAM
SHEET 1 OF 3

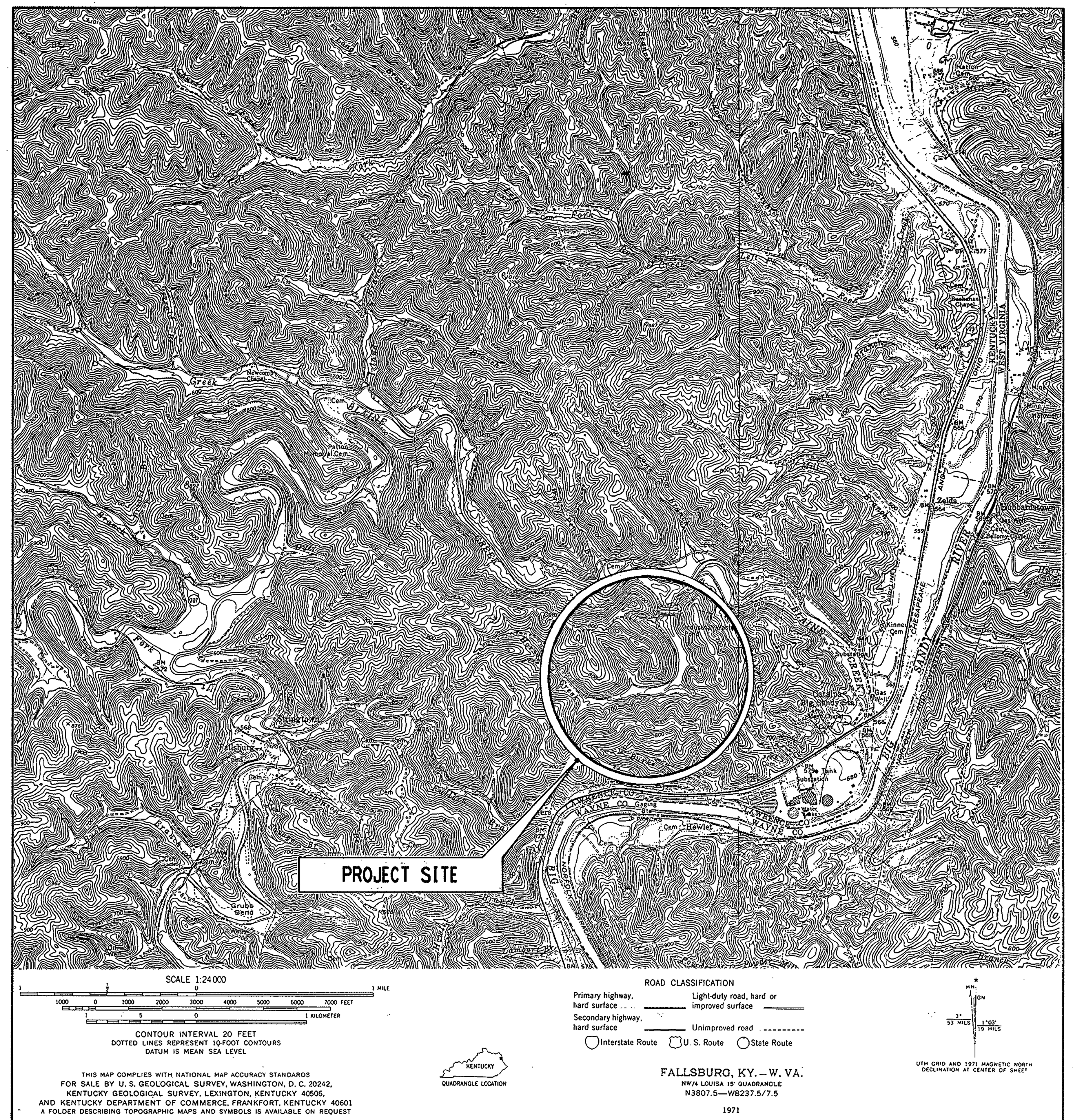
AMERICAN ELECTRIC POWER
SERVICE CORP.
APPROVED

NEW YORK, NEW YORK — DATE APRIL, 1977 — DWG. NO. 984 C 101R2

ATTACHMENT C-3

STAGE III DRAWINGS

DWG. NO. 12-30029



LOCATION MAP

DESIGN DRAWINGS

FOR

HORSEFORD CREEK FLYASH DAM

BIG SANDY PLANT

LOUISA, KENTUCKY

PREPARED FOR:

KENTUCKY POWER COMPANY

BY:

AMERICAN ELECTRIC POWER SERVICE CORP.

1 RIVERSIDE PLAZA, COLUMBUS, OHIO

MARCH 1993

DRAWING INDEX

DRAWING NUMBER	TITLE
12-30030	GENERAL ARRANGEMENT
12-30031	MAIN FLY ASH DAM, PLAN OF BORINGS
12-30032	SADDLE DAM AND EMERGENCY SPILLWAY - PLAN OF BORINGS
12-30033	MAIN FLY ASH DAM SECTIONS
12-30034	SADDLE DAM AND EMERGENCY SPILLWAY, GRADING PLAN
12-30035	SADDLE DAM AND EMERGENCY SPILLWAY, SECTIONS
12-30036	MAIN FLY ASH DAM, GRADING PLAN
12-30037	SADDLE DAM ROADWAY, PLAN AND PROFILE
12-30038	ACCESS STAIRWAY AND CULVERT DETAILS
12-30039	RISER - PLAN AND SECTIONS
12-30040	RISER - MISCELLANEOUS DETAILS
12-30041	TYPICAL-SUBSURFACE SECTIONS, MAIN DAM
12-30900	MAIN FLY ASH DAM AS-BUILT

DATE	NO.	DESCRIPTION	APPD.
3/19/93	0	ISSUED FOR CONSTRUCTION.	

REVISIONS

KENTUCKY POWER COMPANY
BIG SANDY PLANT
BIG SANDY, KENTUCKY

HORSEFORD CREEK FLY ASH DAM
COVER SHEET AND
INDEX DRAWING

DWG. NO. 12-30029-0

ARCH. — ELEC. — MECH. — STR. 95

SCALE: ——
DR: BBM
CH: P.J.A./RNS
SQ. LDR: RNS
DATE: 3-19-93

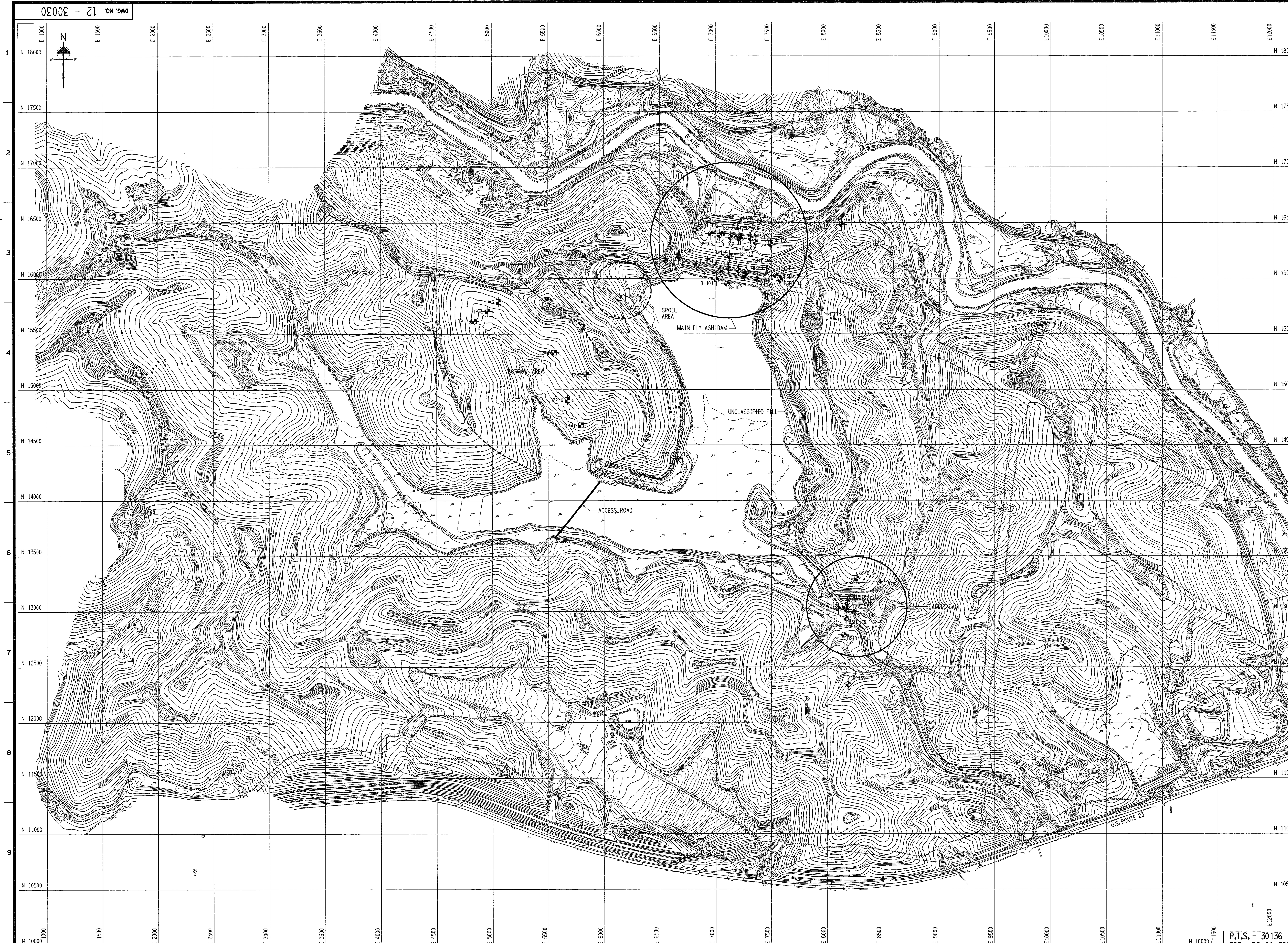
APPROVED BY
ENGINEERING DIVISION
DESIGN DEPARTMENT

EDR No. BS-00-5204

AMERICAN ELECTRIC POWER SERVICE CORP.

3303
10-MAR-1993

IGS0189



LEGEND - EXISTING

- x671.4 SPOT ELEVATION
- INTERMEDIATE CONTOUR
- 680 INDEX CONTOUR
- DEPRESSION CONTOUR
- TREES AND TREELINE
- STRUCTURE AND BUILDING
- FENCE
- POLE
- ROADS
- EDGE OF WATER
- MANHOLES / CATCH BASIN
- POWER POLE
- B-101 BORING
- TP=1 TEST PIT

NOTE:

1. THIS MAP PREPARED BY:----- FROM PHOTOGRAPHY THIS MAP HAS BEEN COMPLIED AT 1"= 00' WITH 2 FT. CONTROUR INTERVALS AND MEETS OR EXCEEDS ALL NATIONAL MAP ACCURACY STANDARDS.

REFERENCE DRAWINGS

- 12-30031 MAIN FLY ASH DAM, PLAN OF BORINGS
- 12-30032 SADDLE DAM AND EMERGENCY SPILLWAY PLAN OF BORINGS
- 12-30033 MAIN FLY ASH DAM SECTIONS
- 12-30034 SADDLE DAM AND EMERGENCY SPILLWAY GRADING PLAN
- 12-30035 SADDLE DAM AND EMERGENCY SPILLWAY SECTIONS
- 12-30036 MAIN FLY ASH DAM, GRADING PLAN
- 12-30037 SADDLE DAM ROADWAY, PLAN AND PROFILE
- 12-30038 ACCESS STAIRWAY AND CULVERT DETAILS
- 12-30039 RISER - PLAN AND SECTIONS
- 12-30040 RISER - MISCELLANEOUS DETAILS
- 12-30041 TYPICAL-SUBSURFACE SECTIONS, MAIN DAM

DATE	NO.	DESCRIPTION	APPD.
5/15/98	0	ISSUED FOR CONSTRUCTION	RSA

REVISIONS

S:/BS/12/GHS/30030.DGN

KENTUCKY POWER COMPANY

BIG SANDY PLANT
HORESFORD CREEK

LAWRENCE, COUNTY KENTUCKY

GENERAL ARRANGEMENT

DWG. NO. 12 - 30030 - 0

SCALE: 1"=300'

CIVIL ENGINEERING DIVISION

DR:
CH:
ENGR.
PROJ. ENGR.
DATE:

APPROVED BY Carl S. Togni

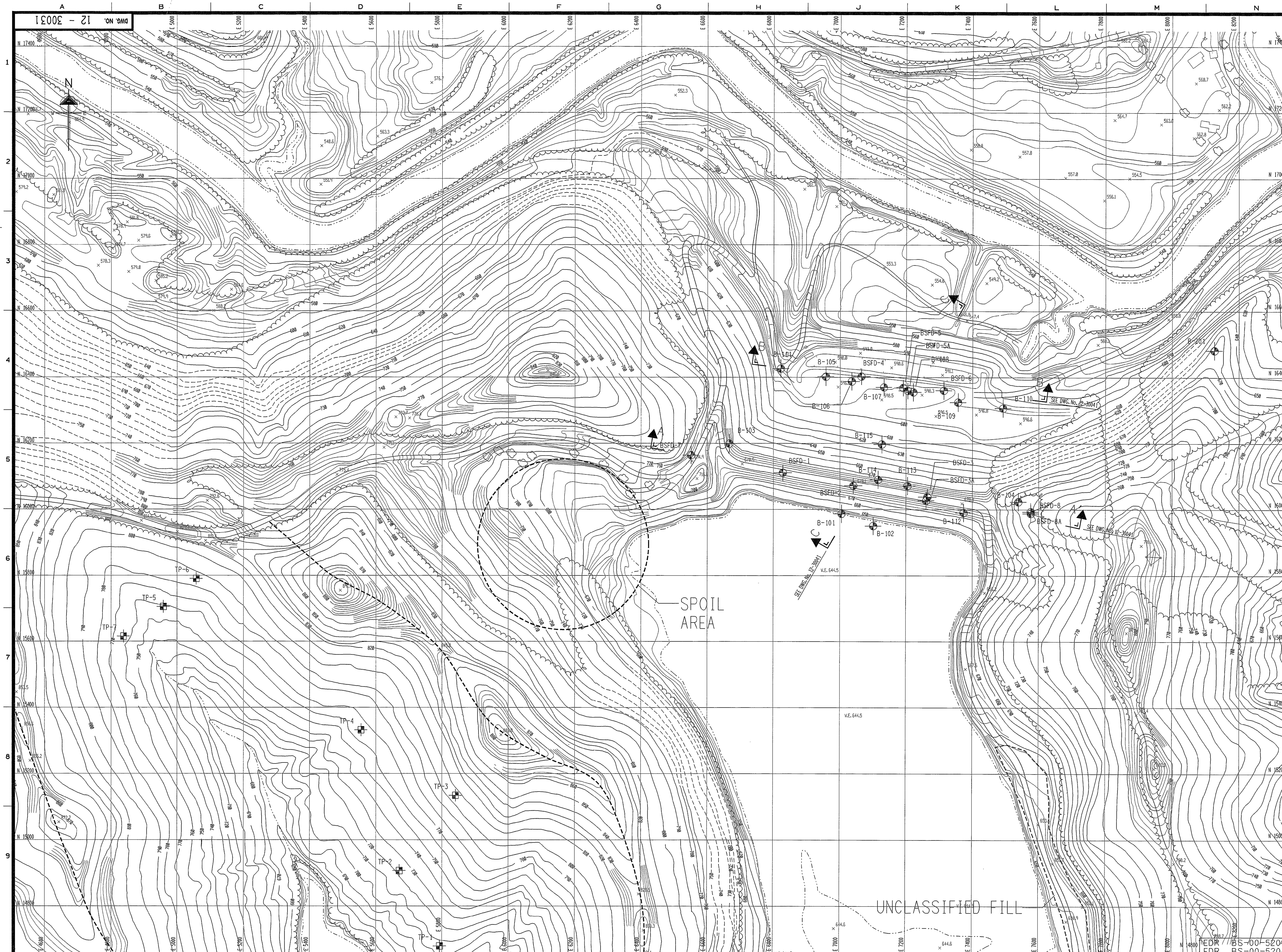
AEP AMERICAN ELECTRIC POWER

1 RIVERSIDE PLAZA COLUMBUS, OH 43215

IGS1092

3303 15-MAY-1998

SYSTEM DATE-15-MAY-1998
SYSTEM TIME-13:32
15th FLR. CED



LEGEND - EXISTING

x671.4	SPOT ELEVATION
	INTERMEDIATE CONTOUR
680	INDEX CONTOUR
	DEPRESSION CONTOUR
	TREES AND TREELINE
	STRUCTURE AND BUILDING
	FENCE
	POLE
	ROADS
	EDGE OF WATER
	MANHOLES / CATCH BASIN
	POWER POLE
B-201	BORING
TP-7	TEST PIT

REFERENCE DRAWINGS

12-30030 GENERAL ARRANGEMENT
12-30032 SADDLE DAM AND EMERGENCY SPILLWAY PLAN OF BORINGS
12-30033 MAIN FLY ASH DAM SECTIONS
12-30034 SADDLE DAM AND EMERGENCY SPILLWAY GRADING PLAN
12-30035 SADDLE DAM AND EMERGENCY SPILLWAY SECTIONS
12-30036 MAIN FLY ASH DAM, GRADING PLAN
12-30037 SADDLE DAM ROADWAY, PLAN & PROFILE
12-30038 ACCESS STAIRWAY AND CULVERT DETAILS
12-30039 RISER - PLAN AND SECTIONS
12-30040 RISER - MISCELLANEOUS DETAILS
12-30041 TYPICAL-SUBSURFACE SECTIONS, MAIN DAM

DATE	NO.	DESCRIPTION	APPD.
5/15/98	0	ISSUED FOR CONSTRUCTION	RJA

REVISIONS

S:/BS/12/GHS/30031.DGN

KENTUCKY POWER COMPANY

BIG SANDY PLANT HORSEFORD CREEK

LAWRENCE, COUNTY KENTUCKY

MAIN FLY ASH DAM BORING LOCATION PLAN

DWG. NO. 12 - 30031 - 0

SCALE: 1" = 100'

CIVIL ENGINEERING DIVISION

DR:
CH:
ENGR.
PROJ. ENGR.
DATE:

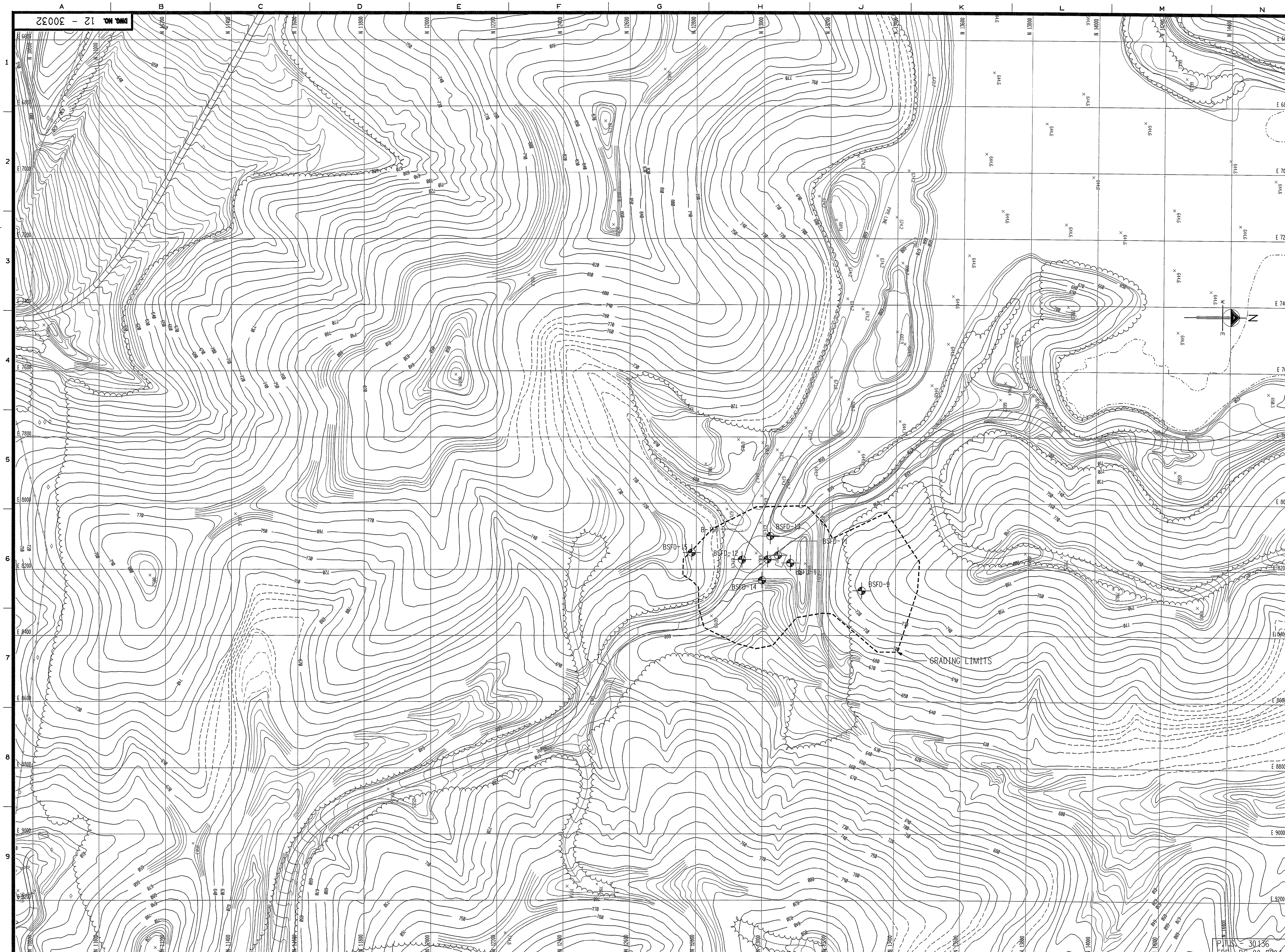
APPROVED BY Carl S. Togni

AMERICAN ELECTRIC POWER

1 RIVERSIDE PLAZA COLUMBUS, OH 43215

SYSTEM DATE-15-MAY-1998
SYSTEM TIME-13:35
15th FLR. CED

IGS1092



LEGEND - EXISTING

x671.4	SPOT ELEVATION
	INTERMEDIATE CONTOUR
680	INDEX CONTOUR
	DEPRESSION CONTOUR
	TREES AND TREELINE
	STRUCTURE AND BUILDING
—x—x—	FENCE
	POLE
	ROADS
	EDGE OF WATER
	MANHOLES / CATCH BASIN
	POWER POLE
BSFD-9	BORING

REFERENCE DRAWINGS

12-30030 GENERAL ARRANGEMENT
12-30031 MAIN FLY ASH DAM, PLAN OF BORINGS
12-30033 MAIN FLY ASH DAM SECTIONS
12-30034 SADDLE DAM AND EMERGENCY SPILLWAY GRADING PLAN
12-30035 SADDLE DAM AND EMERGENCY SPILLWAY SECTIONS
12-30036 MAIN FLY ASH DAM, GRADING PLAN
12-30037 SADDLE DAM ROADWAY, PLAN & PROFILE
12-30038 ACCESS STAIRWAY AND CULVERT DETAILS
12-30039 RISER - PLAN AND SECTIONS
12-30040 RISER - MISCELLANEOUS DETAILS
12-30041 TYPICAL-SUBSURFACE SECTIONS, MAIN DAM

DATE	NO.	DESCRIPTION	APPD.
5/15/98	0	ISSUED FOR CONSTRUCTION	RSA

REVISIONS

S:/BS/12/GHS/30032.DGN

KENTUCKY POWER COMPANY

BIG SANDY PLANT HORSEFORD CREEK

LAWRENCE, COUNTY KENTUCKY

SADDLE DAM AND EMERGENCY SPILLWAY BORING LOCATION PLAN

DWG. NO. 12 - 30032 - 0

SCALE:
DR: 1" = 100'
CH:
ENGR.
PROJ. ENGR.
DATE:

CIVIL ENGINEERING DIVISION

APPROVED BY Carl S. Togni

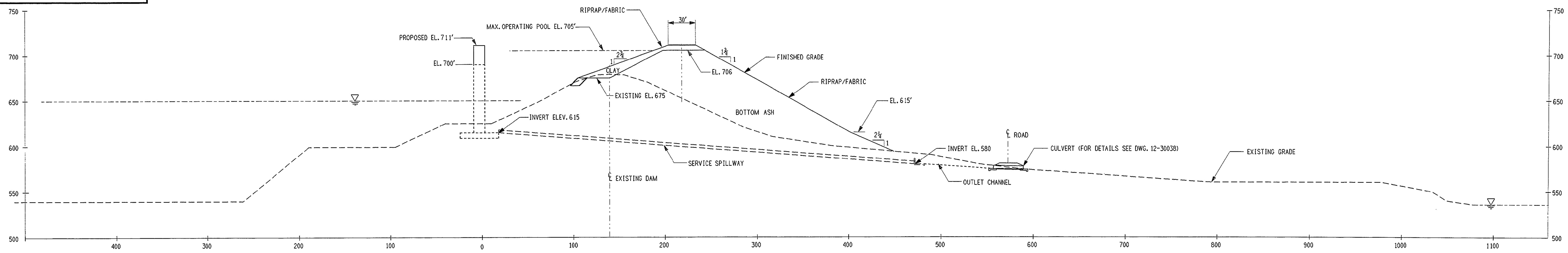
AEP AMERICAN ELECTRIC POWER

1 RIVERSIDE PLAZA COLUMBUS, OH 43215

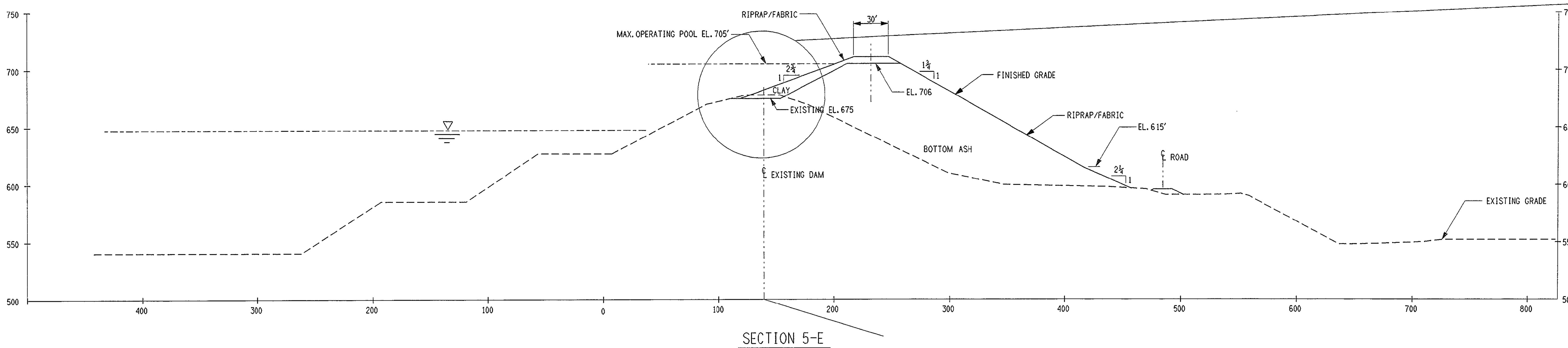
3303

SYSTEM DATE-15-MAY-1998
SYSTEM TIME-13:38
15th FLR.CED

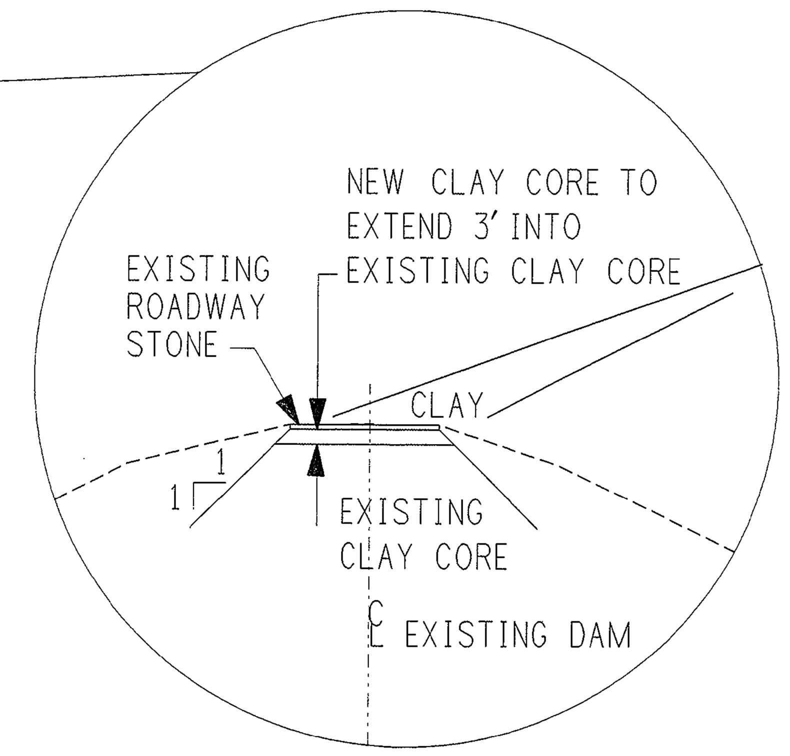
DWG. NO. 12-30033



SECTION 3-E
SCALE: 1" = 50' HORZ & VERT
(FOR LOCATION SEE DWG No. 12-30036)

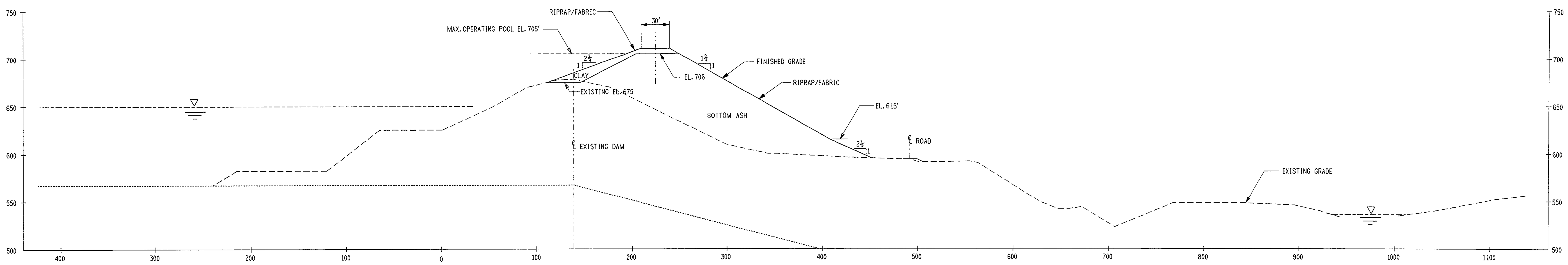


SECTION 5-E
SCALE: 1" = 50' HORZ & VERT
(FOR LOCATION SEE DWG No. 12-30036)

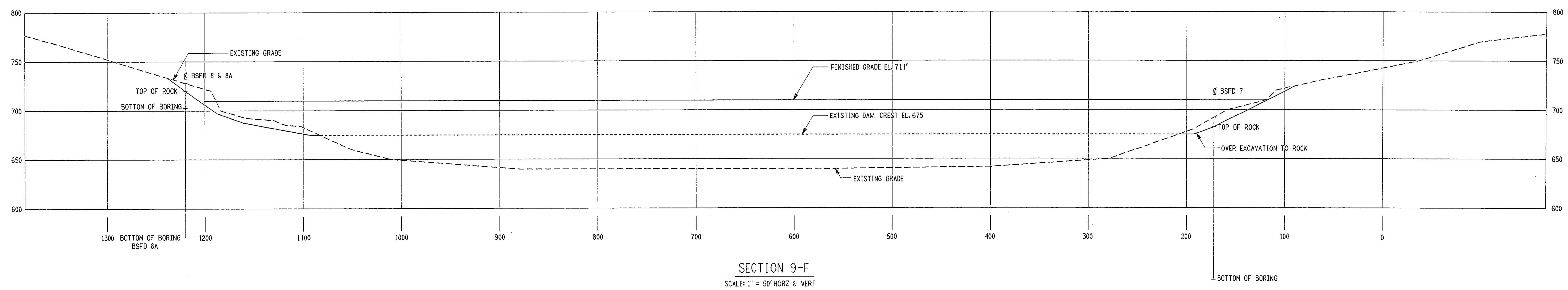


REFERENCE DRAWINGS

12-30030 GENERAL ARRANGEMENT
12-30031 MAIN FLY ASH DAM, PLAN OF BORINGS
12-30032 SADDLE DAM AND EMERGENCY SPILLWAY PLAN OF BORINGS
12-30034 SADDLE DAM AND EMERGENCY SPILLWAY GRADING PLAN
12-30035 SADDLE DAM AND EMERGENCY SPILLWAY SECTIONS
12-30036 MAIN FLY ASH DAM, GRADING PLAN
12-30037 SADDLE DAM ROADWAY, PLAN & PROFILE
12-30038 ACCESS STAIRWAY AND CULVERT DETAILS
12-30039 RISER - PLAN AND SECTIONS
12-30040 RISER - MISCELLANEOUS DETAILS
12-30041 TYPICAL-SUBSURFACE SECTIONS, MAIN DAM



SECTION 7-E
SCALE: 1" = 50' HORZ & VERT
(FOR LOCATION SEE DWG No. 12-30036)



SECTION 9-F
SCALE: 1" = 50' HORZ & VERT
(FOR LOCATION SEE DWG No. 12-30036)

DATE	NO.	DESCRIPTION	APPD.
5/15/98	0	ISSUED FOR CONSTRUCTION	DA

REVISIONS

S:/BS/12/GHS/30033.DGN

KENTUCKY POWER COMPANY
BIG-SANDY
HORSEFORD CREEK
LAWRENCE KENTUCKY

MAIN FLY ASH DAM
SECTIONS

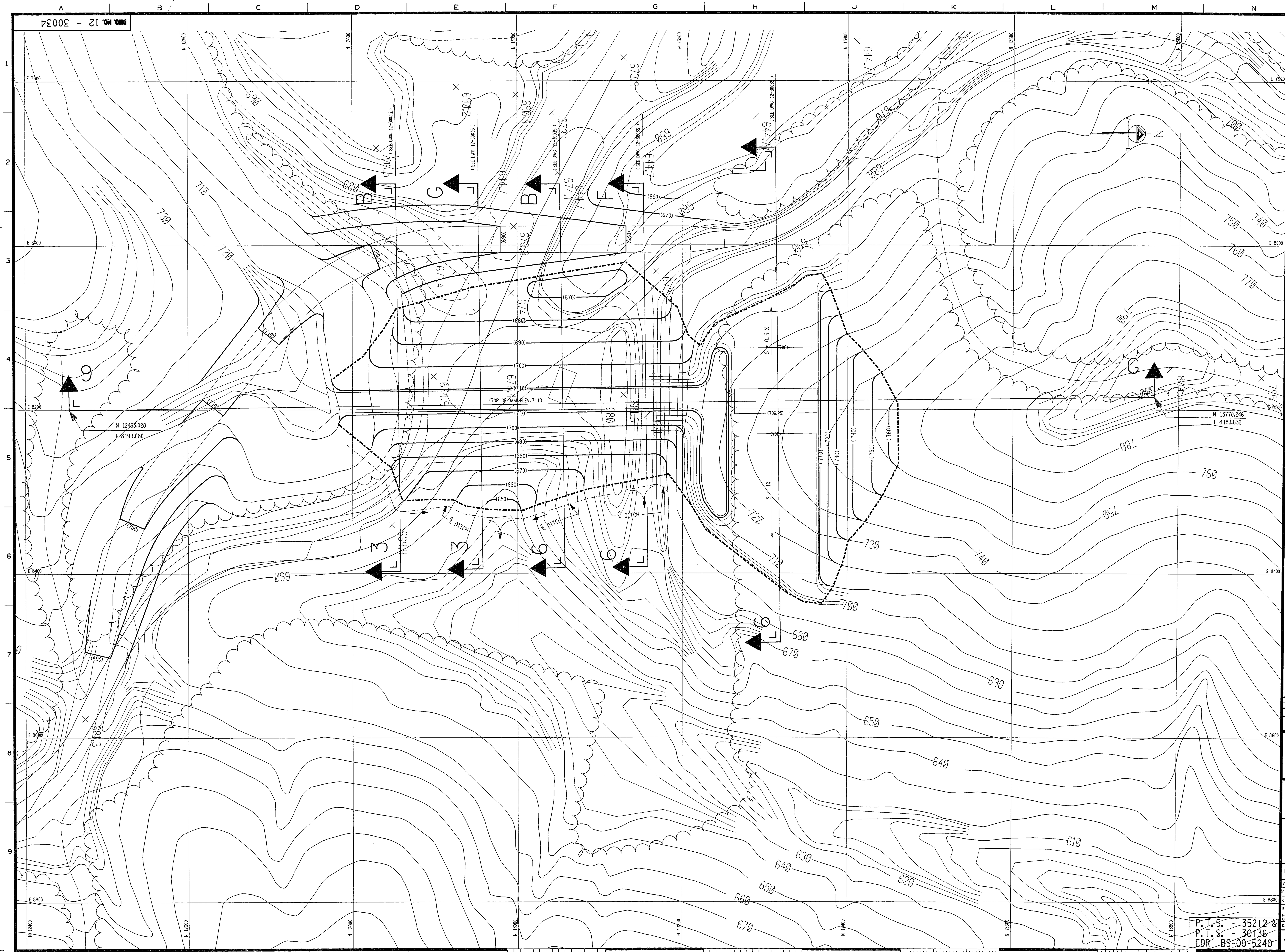
DWG. NO. 12-30033 - 0

SCALE: 1"=50'
CIVIL ENGINEERING DIVISION
DR:
CH:
ENGR.
PROJ. ENGR.
DATE:
APPROVED BY Carl A. Togni

P.T.S. - 30136
EDR BS-00-5204

AEP AMERICAN ELECTRIC POWER
1 RIVERSIDE PLAZA
COLUMBUS, OH 43215

IGS1092
3303
SYSTEM DATE-15-MAY-1998
SYSTEM TIME-13:41
15th FLR.CED



LEGEND - EXISTING

x671.4	SPOT ELEVATION
	INTERMEDIATE CONTOUR
680	INDEX CONTOUR
	DEPRESSION CONTOUR
	TREES AND TREELINE
	STRUCTURE AND BUILDING
-x-x-	FENCE
	POLE
	ROADS
	EDGE OF WATER
	MANHOLES / CATCH BASIN
	POWER POLE

REFERENCE DRAWINGS

12-30030 GENERAL ARRANGEMENT
12-30031 MAIN FLY ASH DAM, PLAN OF BORINGS
12-30032 SADDLE DAM AND EMERGENCY SPILLWAY PLAN OF BORINGS
12-30033 MAIN FLY ASH DAM SECTIONS
12-30035 SADDLE DAM SECTIONS
12-30036 MAIN FLY ASH DAM, GRADING PLAN
12-30037 SADDLE DAM ROADWAY, PLAN & PROFILE
12-30038 ACCESS STAIRWAY AND CULVERT DETAILS
12-30039 RISER - PLAN AND SECTIONS
12-30040 RISER - MISCELLANEOUS DETAILS
12-30041 TYPICAL-SUBSURFACE SECTIONS, MAIN DAM

DATE	NO.	DESCRIPTION	APPD.
5/15/98	0	ISSUED FOR CONSTRUCTION	[signature]

REVISIONS

S:/BS/12/GHS/30034.DGN

KENTUCKY POWER COMPANY

BIG SANDY PLANT
HORSEFORD CREEK

LAWRENCE, COUNTY KENTUCKY

SADDLE DAM AND EMERGENCY SPILLWAY GRADING PLAN

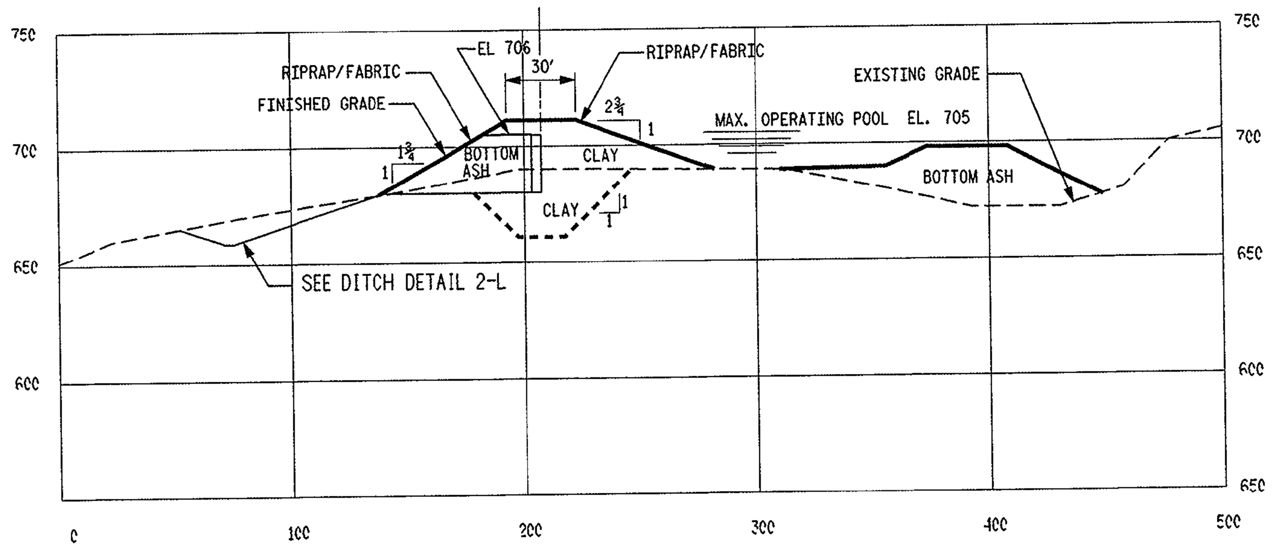
DWG. NO. 12 - 30034 - 0

SCALE: 1" = 40'
CIVIL ENGINEERING DIVISION
DR:
CH:
ENGR.
PROJ. ENGR.
DATE:
APPROVED BY [signature]

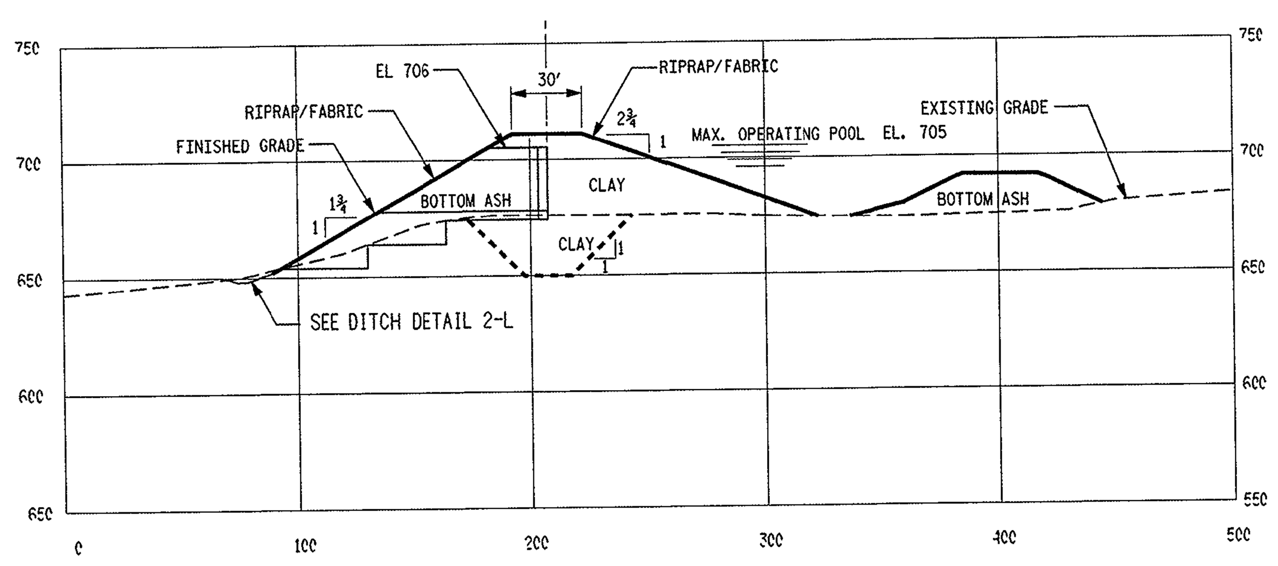
AEP AMERICAN ELECTRIC POWER
1 RIVERSIDE PLAZA COLUMBUS, OH 43215

3303
SYSTEM DATE-15-MAY-1998
SYSTEM TIME-13:42
15th FLR. CED

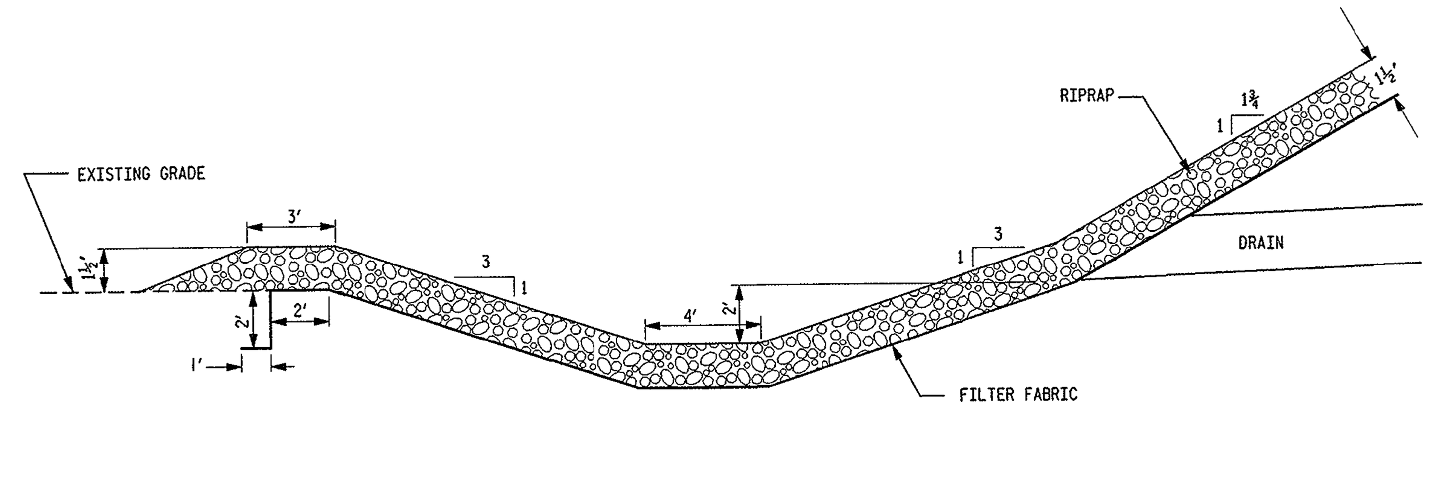
DWG. NO. 12-30035



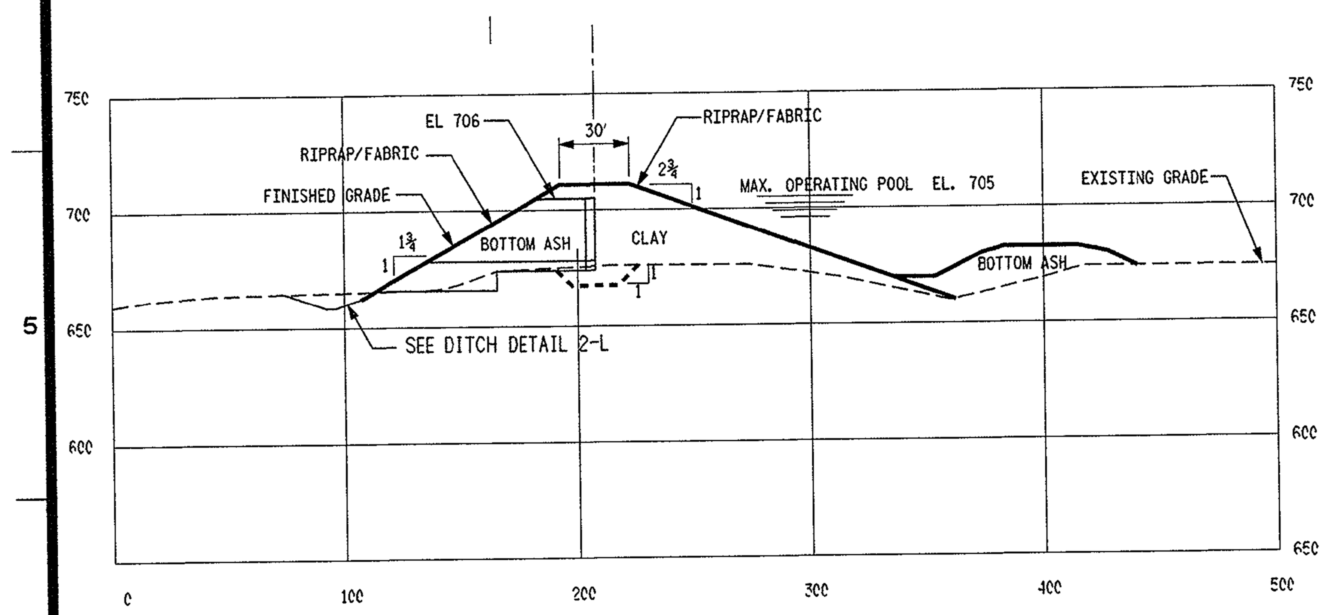
SECTION 3-B
SCALE: 1" = 50' HORZ & VERT
(FOR LOCATION SEE DWG No. 12-30034)



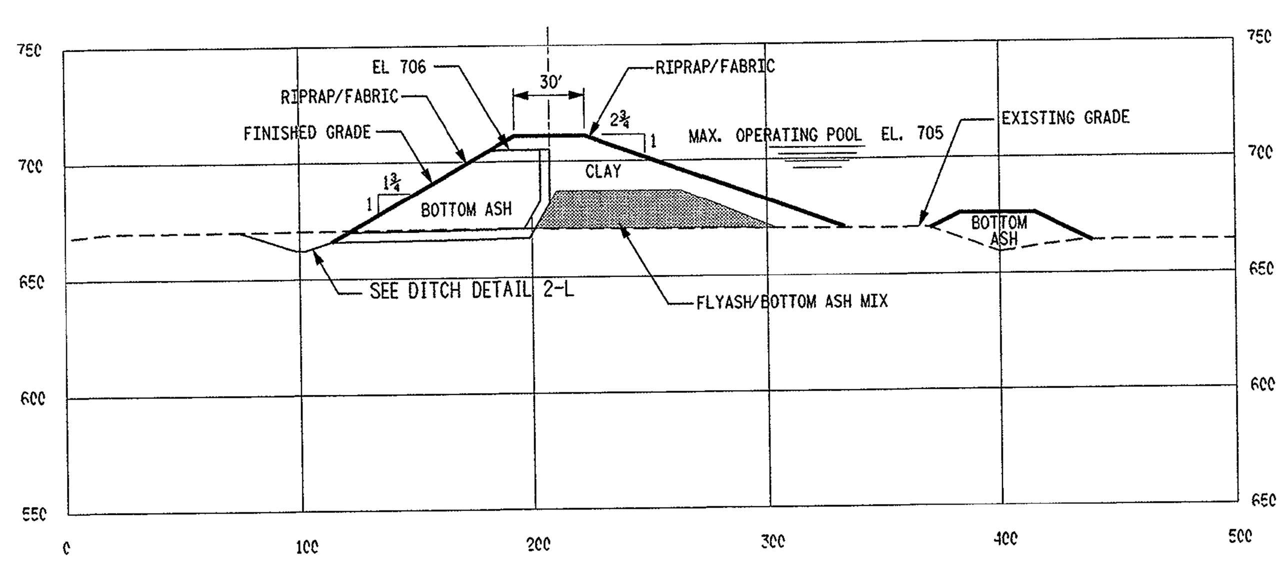
SECTION 3-G
SCALE: 1" = 50' HORZ & VERT
(FOR LOCATION SEE DWG No. 12-30034)



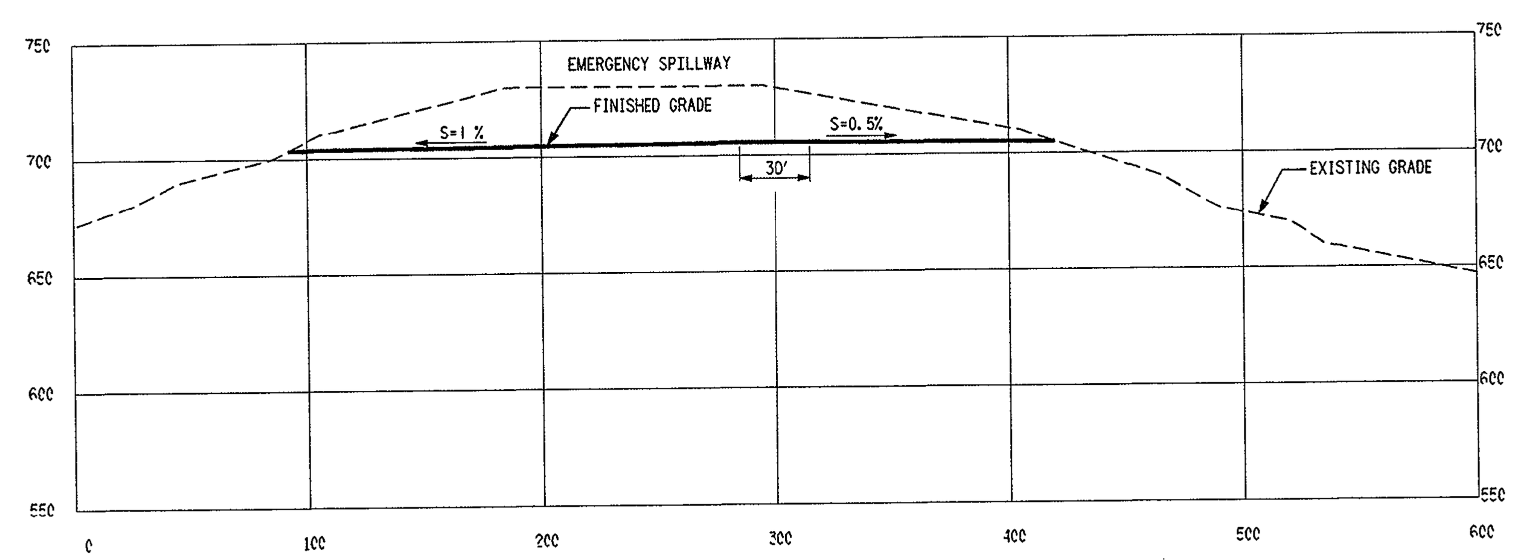
SECTION 2-L
DITCH DETAIL
SCALE: 1" = 4'



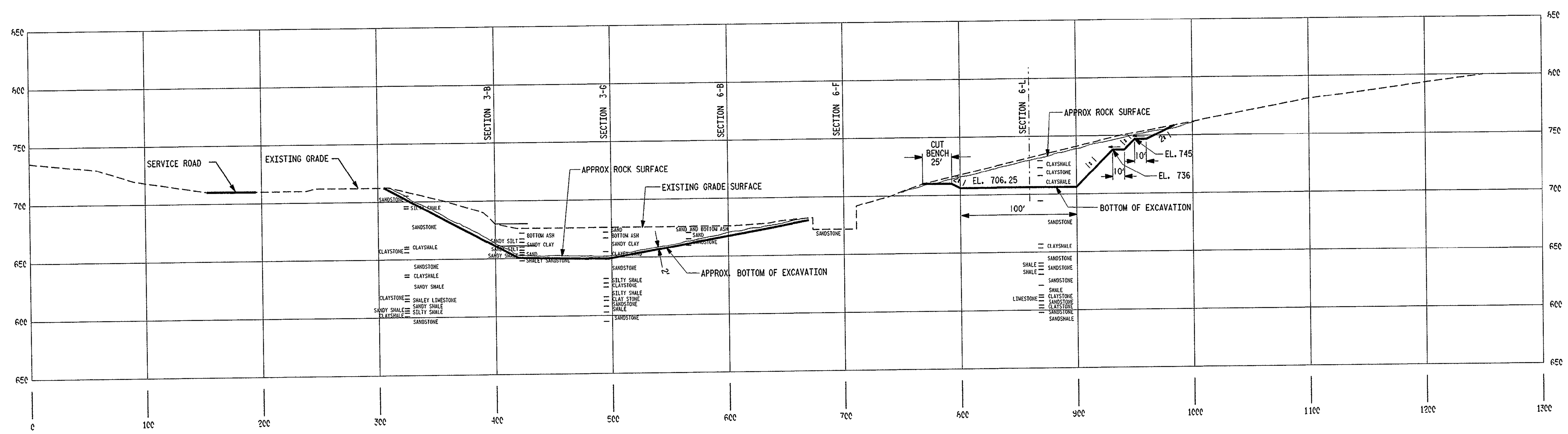
SECTION 6-B
SCALE: 1" = 50' HORZ & VERT
(FOR LOCATION SEE DWG No. 12-30034)



SECTION 6-F
SCALE: 1" = 50' HORZ & VERT
(FOR LOCATION SEE DWG No. 12-30034)



SECTION 6-L
SCALE: 1" = 50' HORZ & VERT
(FOR LOCATION SEE DWG No. 12-30034)



SADDLE DAM
SECTION 9-G
SCALE: 1" = 50' HORZ & VERT
(FOR LOCATION SEE DWG No. 12-30034)

REFERENCE DRAWINGS

- 12-30030 GENERAL ARRANGEMENT
- 12-30031 MAIN FLY ASH DAM, PLAN OF BORINGS
- 12-30032 SADDLE DAM AND EMERGENCY SPILLWAY PLAN OF BORINGS
- 12-30033 MAIN FLY ASH DAM SECTIONS
- 12-30034 SADDLE DAM AND EMERGENCY SPILLWAY GRADING PLAN
- 12-30036 MAIN FLY ASH DAM, GRADING PLAN
- 12-30037 SADDLE DAM ROADWAY, PLAN & PROFILE
- 12-30038 ACCESS STAIRWAY AND CULVERT DETAILS
- 12-30039 RISER - PLAN AND SECTIONS
- 12-30040 RISER - MISCELLANEOUS DETAILS
- 12-30041 TYPICAL-SUBSURFACE SECTIONS, MAIN DAM

DATE	NO.	DESCRIPTION	APPD.
5/15/98	0	ISSUED FOR CONSTRUCTION	PJA

REVISIONS

S:/BS/12/GHS/30035.DGN

KENTUCKY POWER COMPANY
BIG-SANDY PLANT
HORSEFORD CREEK
LAWRENCE KENTUCKY

SADDLE DAM AND
EMERGENCY SPILLWAY
SECTIONS

DWG. NO. 12-30035 - 0

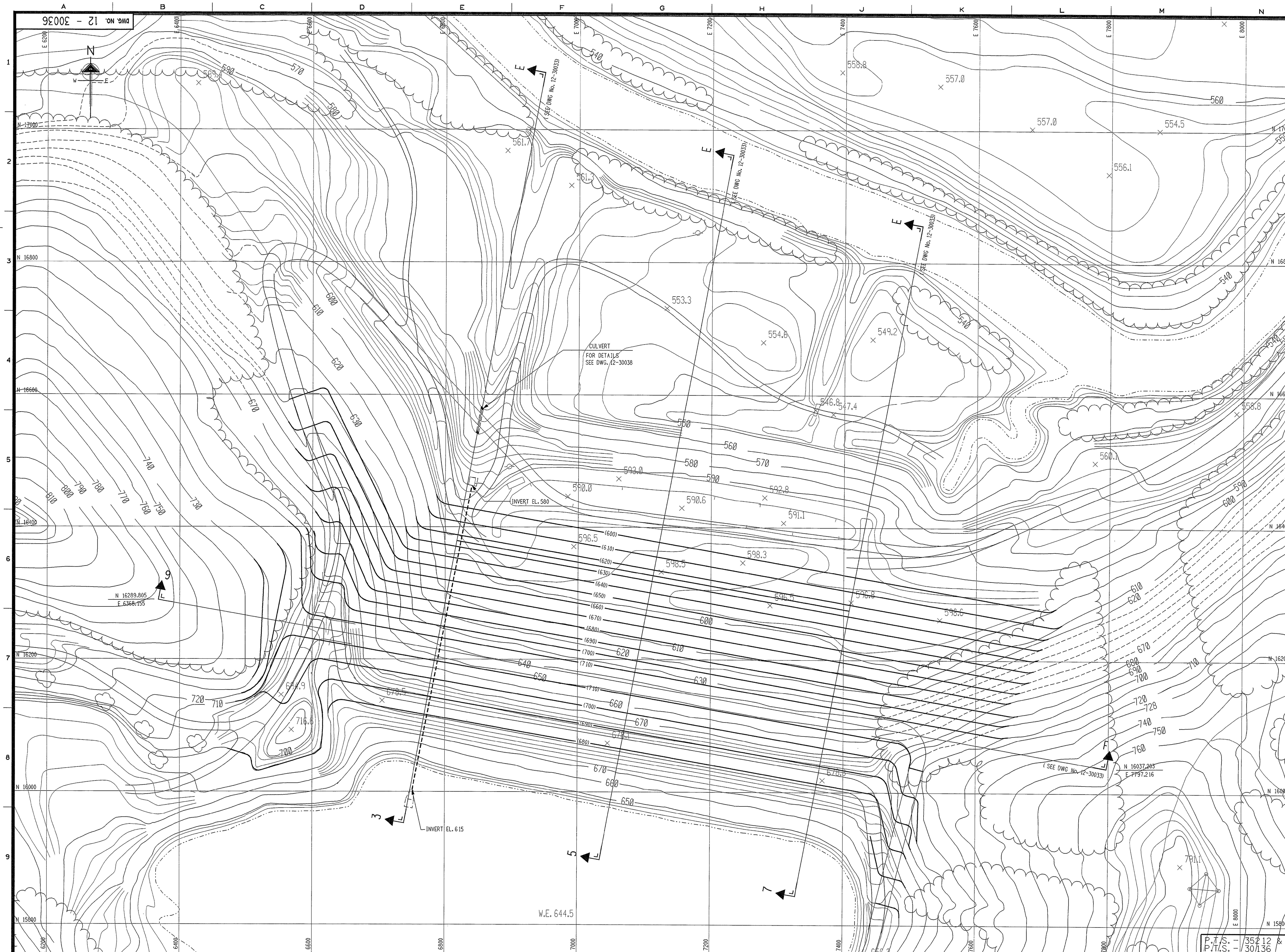
SCALE: 1"=50' CIVIL ENGINEERING DIVISION

Original Signed By:
CARL S. TOGNI

P.T.S. - 35212 &
P.T.S. - 30136
EDR BS-00-5204

AEP AMERICAN ELECTRIC POWER
1 RIVERSIDE PLAZA
COLUMBUS, OH 43215

IGS1092



LEGEND - EXISTING

SPOT ELEVATION
INTERMEDIATE CONTOUR
680 INDEX CONTOUR
DEPRESSION CONTOUR
TREES AND TREELINE
STRUCTURE AND BUILDING
FENCE
POLE
ROADS
EDGE OF WATER
MANHOLES / CATCH BASIN
POWER POLE

REFERENCE DRAWINGS

12-30030 GENERAL ARRANGEMENT
12-30031 MAIN FLY ASH DAM, PLAN OF BORING
12-30032 SADDLE DAM AND EMERGENCY SPILLWAY PLAN OF BORINGS
12-30033 MAIN FLY ASH DAM SECTIONS
12-30034 SADDLE DAM AND EMERGENCY SPILLWAY GRADING PLAN
12-30035 SADDLE DAM AND EMERGENCY SPILLWAY SECTIONS
12-30037 SADDLE DAM ROADWAY, PLAN & PROFILE
12-30038 ACCESS STAIRWAY AND CULVERT DETAILS
12-30039 RISER - PLAN AND SECTIONS
12-30040 RISER - MISCELLANEOUS DETAILS
12-30041 TYPICAL-SUBSURFACE SECTIONS, MAIN DAM

DATE	NO.	DESCRIPTION	APPD.
5/15/98	0	ISSUED FOR CONSTRUCTION	RH

REVISIONS

S:/BS/12/GHS/30036.DGN

KENTUCKY POWER COMPANY

BIG SANDY PLANT
HORSEFORD CREEK

LAWRENCE, COUNTY KENTUCKY

MAIN FLY ASH DAM
GRADING PLAN

DWG. NO. 12 - 30036 - 0

SCALE: 1" = 50'

CIVIL ENGINEERING DIVISION

DR:
CH:
ENGR.
PROJ. ENGR.
DATE:

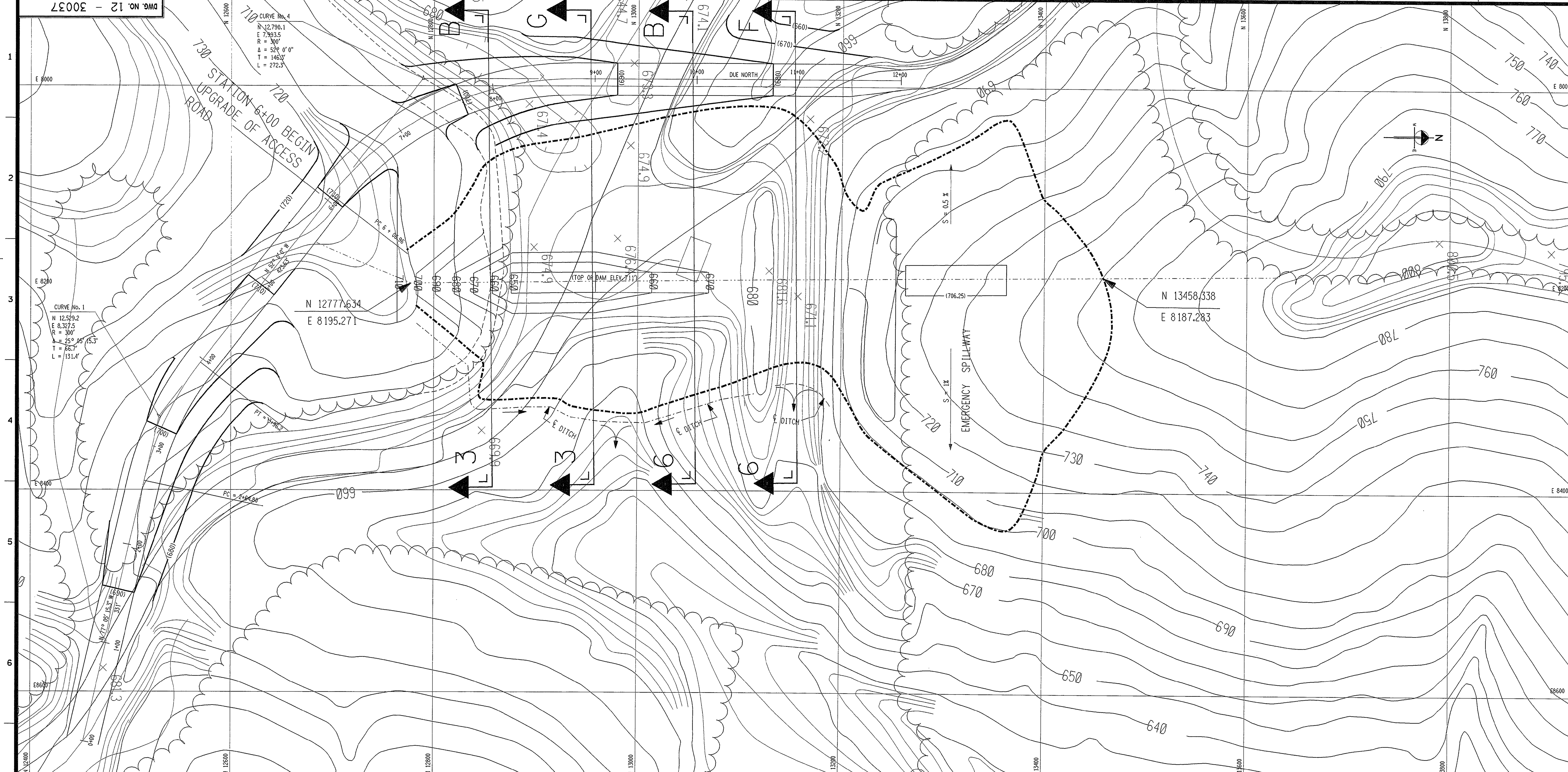
APPROVED BY Carl S. Togw

AMERICAN ELECTRIC POWER

1 RIVERSIDE PLAZA
COLUMBUS, OH 43215

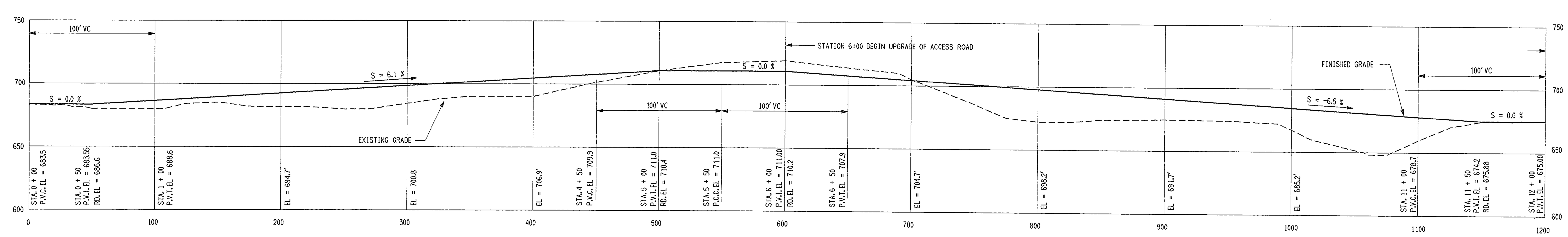
SYSTEM DATE-15-MAY-1998
SYSTEM TIME-13:46
15th FLR. CED

IGS1092



ROAD GRADING PLAN

SCALE : 1" = 40'



ROADWAY ℄ PROFILE

SCALE : 1" = 40' HORZ.& VERT.

LEGEND - EXISTING

- x671.4 SPOT ELEVATION
- INTERMEDIATE CONTOUR
- 680 INDEX CONTOUR
- DEPRESSION CONTOUR
- TREES AND TREELINE
- STRUCTURE AND BUILDING
- FENCE
- POLE
- ROADS
- EDGE OF WATER
- MANHOLES / CATCH BASIN
- POWER POLE

REFERENCE DRAWINGS

- 12-30030 GENERAL ARRANGEMENT
- 12-30031 MAIN FLY ASH DAM, PLAN OF BORINGS
- 12-30032 SADDLE DAM AND EMERGENCY SPILLWAY PLAN OF BORINGS
- 12-30033 MAIN FLY ASH DAM SECTIONS
- 12-30034 SADDLE DAM AND EMERGENCY SPILLWAY GRADING PLAN
- 12-30035 SADDLE DAM SECTIONS
- 12-30036 MAIN FLY ASH DAM, GRADING PLAN
- 12-30038 ACCESS STAIRWAY AND CULVERT DETAILS
- 12-30039 RISER - PLAN AND SECTIONS
- 12-30040 RISER - MISCELLANEOUS DETAILS
- 12-30041 TYPICAL-SUBSURFACE SECTION, MAIN DAM

DATE	NO.	DESCRIPTION	APPD.
5/15/98	0	ISSUED FOR CONSTRUCTION	RLA

REVISIONS

S:/BS/12/GHS/30037.DGN

KENTUCKY POWER COMPANY

BIG SANDY PLANT
HORSEFORD CREEK

LAWRENCE, COUNTY KENTUCKY

SADDLE DAM
CUT-OFF TRENCH EXCAVATION PLAN
ROADWAY PLAN AND PROFILE

DWG. NO. 12 - 30037 - 0

SCALE: 1" = 40'

CIVIL ENGINEERING DIVISION

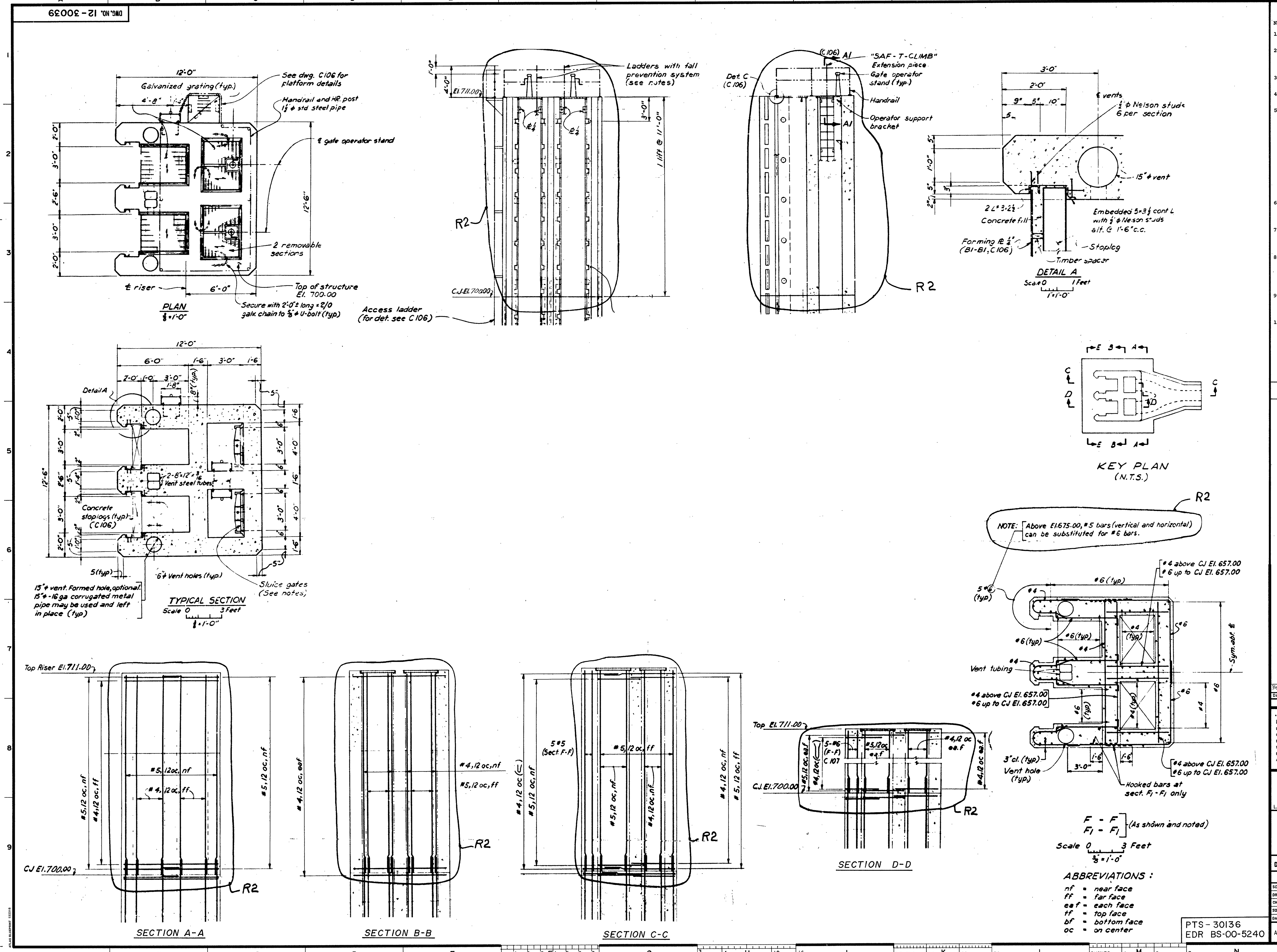
DR:
CH:
ENGR.
PROJ. ENGR.
DATE:

APPROVED BY Carl S. Togni

AMERICAN ELECTRIC POWER
1 RIVERSIDE PLAZA
COLUMBUS, OH 43215

SYSTEM DATE-15-MAY-1998
SYSTEM TIME-13:48
15th FLR. CED

IGS1092



NOTES:

1. ALL CONCRETE SHALL BE OF 4,000 PSI STRENGTH.
2. ALL REINFORCING STEEL SHALL BE DEFORMED BARS AND SHALL CONFORM TO ASTM-A613 GRADE 60 SPECIFICATIONS.
3. EXISTING CONCRETE SHALL BE REMOVED A MINIMUM OF 2'-6".
4. NEW REINFORCING SHALL BE LAP-SPLICE A MINIMUM OF 2'-0".
5. THE INDICATED CONCRETE AREA SHALL BE PAINTED WITH TWO COATS OF "SIKAGARD 664/9 HI-BILD' EPOXY RESIN OR EQUAL. THE MINIMUM THICKNESS OF EACH COAT SHALL BE 6 TO 9 MILS. SURFACE PREPARATION AND EPOXY APPLICATION SHALL BE IN ACCORDANCE WITH MANF. RECOMMENDATIONS. NO MEASUREMENT FOR PAYMENT WILL BE MADE FOR EPOXY PROTECTIVE COATING. THE ENTIRE COST FOR ALL WORK AND MATERIALS SHALL BE INCLUDED IN THE UNIT PRICE FOR CONCRETE.
6. ALL BOLTS, NUTS AND WASHERS SHALL CONFORM TO ASTM-A307 SPECIFICATIONS, AND SHALL BE GALVANIZED.
7. ALL EXPOSED CORNERS OF THE STRUCTURE SHALL HAVE A 3/4" CHAMFER (MIN) UNLESS OTHERWISE NOTED.
8. ALL EXPOSED MISC. METAL INCLUDING: GRATING, LADDERS, HANDRAILS AND ANGLES SHALL BE STRUCTURAL STEEL, CONFORMING TO ASTM-A36 SPECIFICATIONS AND SHALL BE GALVANIZED.
9. THE SLUICE GATE OPENINGS ARE 36" X 30". GATES SHALL BE OF "RODNEY HUNT HY-Q240H" SERIES, OR EQUAL.
10. PROVIDE "SAF-T-CLIMB" FALL PREVENTION SYSTEMS FOR 3 LADDERS WITH ONE REMOV. EXTEN. PC., 2 SAFETY BELTS AND 2 SLEEVES AS SUPPLIED BY POTTER-WEIL CO., CHICAGO, OR EQUAL.

REFERENCE DRAWINGS

12-30030 GENERAL ARRANGEMENT
12-30031 MAIN FLY ASH DAM, PLAN OF BORINGS
12-30032 SADDLE DAM AND EMERGENCY SPILLWAY PLAN OF BORINGS
12-30033 MAIN FLYASH DAM SECTIONS
12-30034 SADDLE DAM AND EMERGENCY SPILLWAY GRADING PLAN
12-30035 SADDLE DAM AND EMERGENCY SPILLWAY SECTIONS
12-30036 MAIN FLY ASH DAM, GRADING PLAN
12-30037 SADDLE DAM ROADWAY, PLAN & PROFILE
12-30038 ACCESS STAIRWAY AND CULVERT DETAILS
12-30040 RISER - MISCELLANEOUS DETAILS
12-30041 TYPICAL-SUBSURFACE SECTIONS, MAIN DAM

DATE	NO.	DESCRIPTION	APPD.
3/19/93	0	ISSUED FOR CONSTRUCTION	

REVISIONS

KENTUCKY POWER COMPANY
BIG SANDY PLANT
HORSEFORD CREEK
LAWRENCE COUNTY KENTUCKY

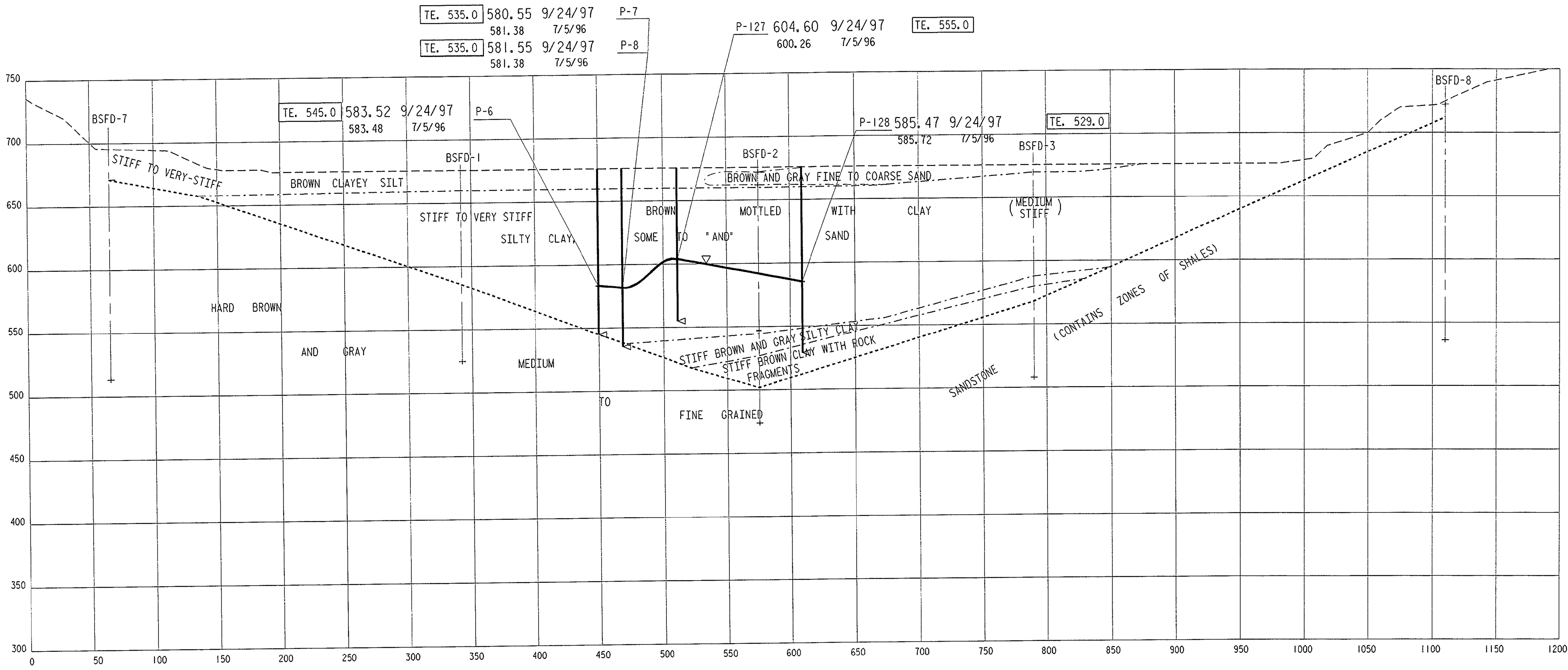
RISER
PLAN AND SECTIONS

DWG. NO. 12-30039-0

ARCH. - ELEC. - MECH. - STR. 93

SCALE: AS SHOWN
DR: RBM
CH: P.J.A. / RNS
SQ. LDR: RNS
DATE: 3-19-93
APPROVED BY: ENGINEERING DIVISION; DESIGN DEPARTMENT

AMERICAN ELECTRIC POWER SERVICE CORP.

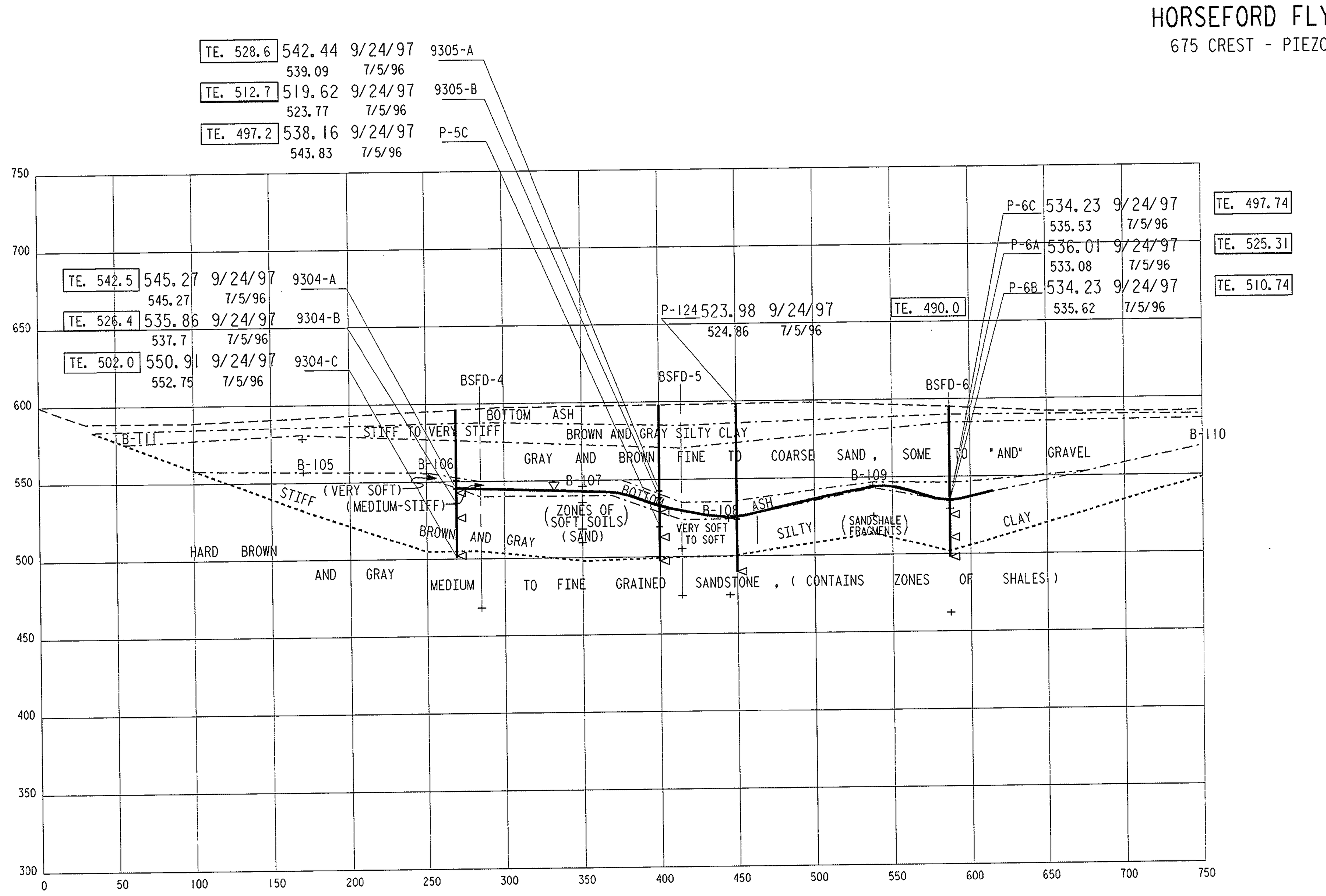


SECTION A-A

BIG-SANDY PLANT

HORSEFORD FLY ASH DAM

675 CREST - PIEZOMETER DATA

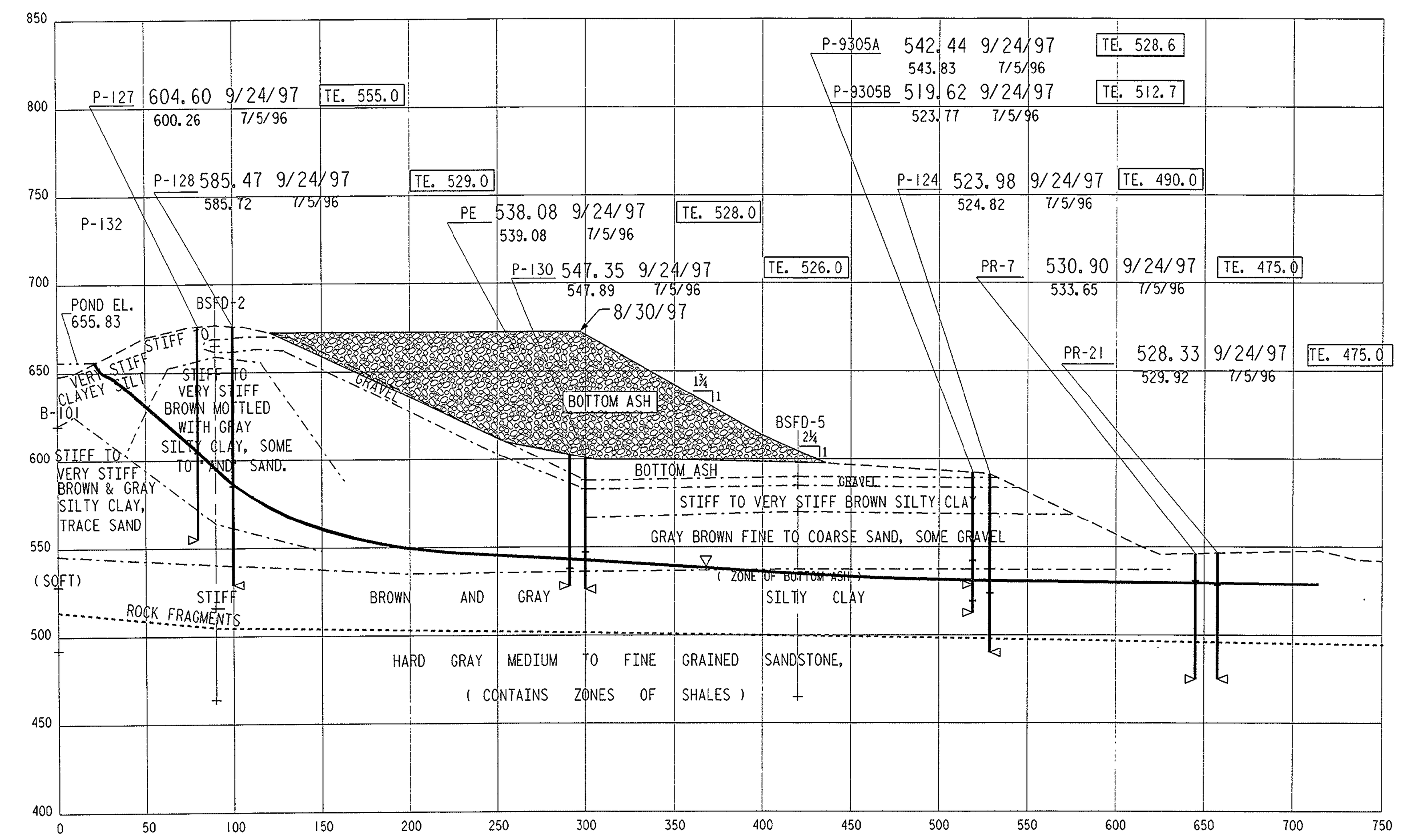


SECTION B-B

BIG-SANDY PLANT

HORSEFORD FLY ASH DAM

590 BERM - PIEZOMETER DATA



SECTION C-C

BIG-SANDY PLANT

HORSEFORD FLY ASH DAM

CROSS-SECTION - PIEZOMETER DATA

NOTES:

BSFD-1 = BORINGS DRILLED IN 1989 FOR THE DESIGN AND CONSTRUCTION OF STAGE II

B-100 = BORINGS DRILLED FOR DESIGN AND CONSTRUCTION OF STAGE II IN 1967 AND 1976 RESPECTIVELY

TE = TIP ELEVATION

REFERENCE DRAWINGS

12-30030 GENERAL ARRANGEMENT

12-30031 MAIN FLY ASH DAM, PLAN OF BORINGS

12-30032 SADDLE DAM AND EMERGENCY SPILLWAY PLAN OF BORINGS

12-30033 MAIN FLYASH DAM SECTIONS

12-30034 SADDLE DAM AND EMERGENCY SPILLWAY GRADING PLAN

12-30035 SADDLE DAM AND EMERGENCY SPILLWAY SECTIONS

12-30036 MAIN FLY ASH DAM, GRADING PLAN

12-30037 SADDLE DAM ROADWAY, PLAN & PROFILE

12-30038 ACCESS STAIRWAY AND CULVERT DETAILS

12-30039 RISER - PLAN AND SECTIONS

12-30040 RISER - MISCELLANEOUS DETAILS

DATE	NO.	DESCRIPTION	APPD.
10/19/96	2	REVISED AS PER READING 12/7/95	
1/2/96	1	REVISED AS PER READING 12/7/95	WPM
3/19/93	0	ISSUED FOR CONSTRUCTION	

REVISIONS

IgsD - /usr5/stru/big_sandy/u4/30041.400

KENTUCKY POWER COMPANY

BIG-SANDY HORSEFORD CREEK

LAWRENCE KENTUCKY

TYPICAL SUBSURFACE SECTIONS MAIN-DAM

DWG. NO. 12-30041 - 2

SCALE:	CIVIL-ENGINEERING-DIVISION	
DR: 1"=50'	APPROVED BY: BRUCE BENNETT	HJ
CH: RBM		
ENGR. PJA		
PROJ. ENGR. RNS	FRANK VAN PELT JR. V. A. LEPORE	WPM
DATE: 3/19/93		

AEP AMERICAN ELECTRIC POWER — 1 RIVERSIDE PLAZA COLUMBUS, OH 43215

P.T.S. - 30136
EDR BS-00-5204

3303

15-MAY-1998

SYSTEM DATE-15-MAY-1998
SYSTEM TIME-13:52
15th FLR. CED

IGS1092

<u>ATTACHMENT D</u>

INSTRUMENTATION LOCATION MAP

DRN BY:	BIG SANDY POWER STATION	DWG NO: FIGURE 2 rev 1
DATE:	SHEET 2 of 3	AEP AMERICAN ELECTRIC POWER — AEP SERVICE CORP. 1 RIVERSIDE PLAZA COLUMBUS, OH 43215
SCALE: 1"=200'	FLY ASH DAM	

BY: s145543
PLOT TIME: 10:23:29 AM
PLOT DATE: 6/15/2012