HISTORY OF CONSTRUCTION

CFR 257.73(c)(1)

Fly Ash Reservoir II

Cardinal Plant
Brilliant, Ohio

August, 2016

Prepared for: Cardinal Operating Company - Cardinal Plant

Brilliant, Ohio

Prepared by: Geotechnical Engineering Services

American Electric Power Service Corporation

1 Riverside Plaza

Columbus, OH 43215

GERS-16-016

Table of CONTENTS

Attachments

Attachment A – Location Map
Attachment B – Construction Design Reports
Attachment C – Design Drawings
Attachment D – Instrumentation Location Map
Attachment E – Hydrology and Hydraulic Report
Attachment F – Maintenance Plan

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 Cardinal Power Plant in Wells Township, Jefferson County, near the town of Brilliant in eastern Ohio. It is owned by Buckeye Power and AEP Generation Resources (GENCO) and is operated by Cardinal Operating Company. The facility operates two surface impoundments for storing CCR; the Bottom Ash Complex and Cardinal Fly Ash Reservoir II (FAR II) Dam. The focus of this report is the FAR II Dam.

The FAR II Dam is a valley filled dam with a unique structure whose current configuration is the result of the original earth fill dam and two separate raisings (See figure 1). The original earth fill dam (Stage 1) consisted of a 180 feet high arched earth embankment incorporating a zoned cross section. At 925 feet NGVD, the dam featured a 70-foot wide by 1,055-feet long crest. The maximum operating pool that could be achieved with the original configuration was El. 913. In 1997, the original dam was raised, referred to as Stage 2. Following this raising, the dam was 237 feet high with a 30-foot wide crest. In 2013, the dam was raised 13 feet using back-to-back MSE walls, bringing the dam into its current, Stage 3 configuration. The principal features of the typical section are the MSE wall themselves and a vinyl sheet pile wall extending from the existing clay core to the top of the PMF flood level for seepage cutoff purposes.

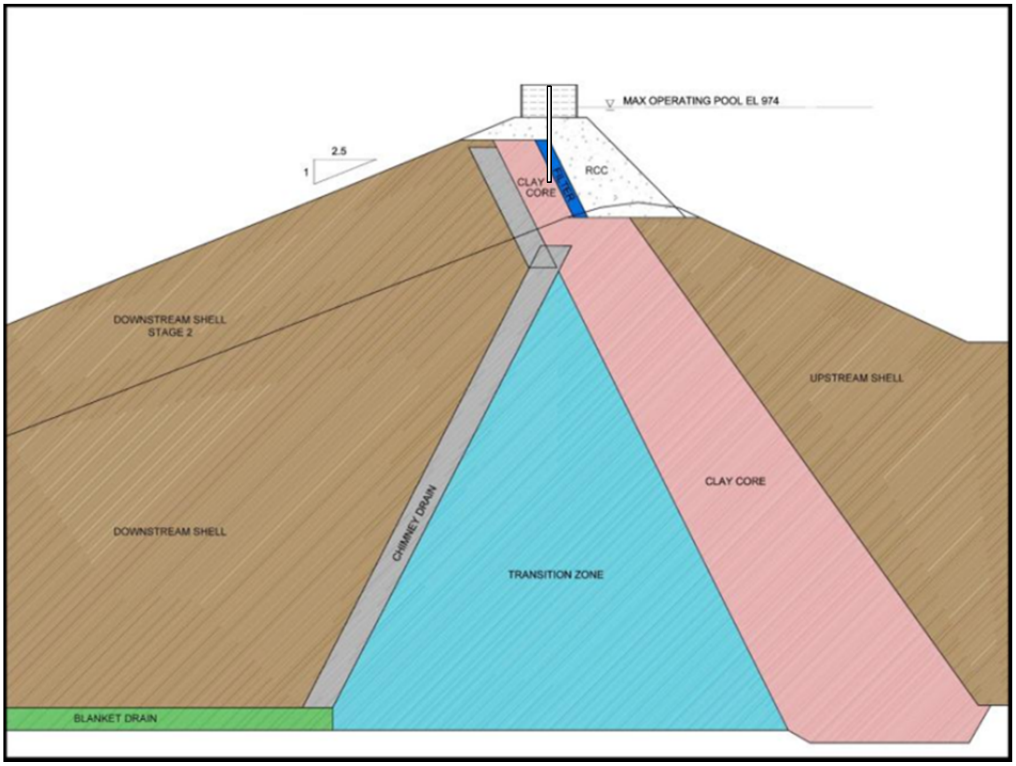


Figure 1. Cross-section of FAR II dam also depicting the original zoned earth dam and the two Raisings.

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 Cardinal Power Plant is located at 306 County Road 7 East, Brilliant, OH, 43913 County, near the town of Brilliant, Jefferson County, Ohio. It is owned by Buckeye Power and AEP Generation Resources (GENCO) and operated by Cardinal Operating Company. The facility operates the FAR II dam, ODNR# 0205-010.

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 purpose of Cardinal FAR II surface impoundment is to provide for the continued disposal of fly ash coal combustion byproduct produced by the Cardinal Generating Plant. Cardinal has three units rated at 600, 600 and 630 megawatts (MW) respectively which produce a total of approximately 560,000 cubic yards of fly ash per year. In addition, FAR II surface impoundment is the leachate treatment pond for the Cardinal Fly Ash Reservoir I (FAR I) Landfill.

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 Cardinal FAR II Ash Pond is located within the Upper Ohio-Wheeling Water Shed (HUC 05030106) which is approximately 1,517.0 square miles (970,876 acres) (USGS). The Cardinal FAR II is valley fill type of reservoir with a diked embankment at the lowest side of the valley. The Cardinal FAR II is located within the Blockhouse Run watershed, which drains directly into the Ohio River. Approximately one mile upstream of the Ohio River, Blockhouse Run splits into two branches, designated as the East Branch and the West Branch.

The Blockhouse Run watershed upstream of FAR II shown in the figure below is approximately 1352 acres (see figure. 2).

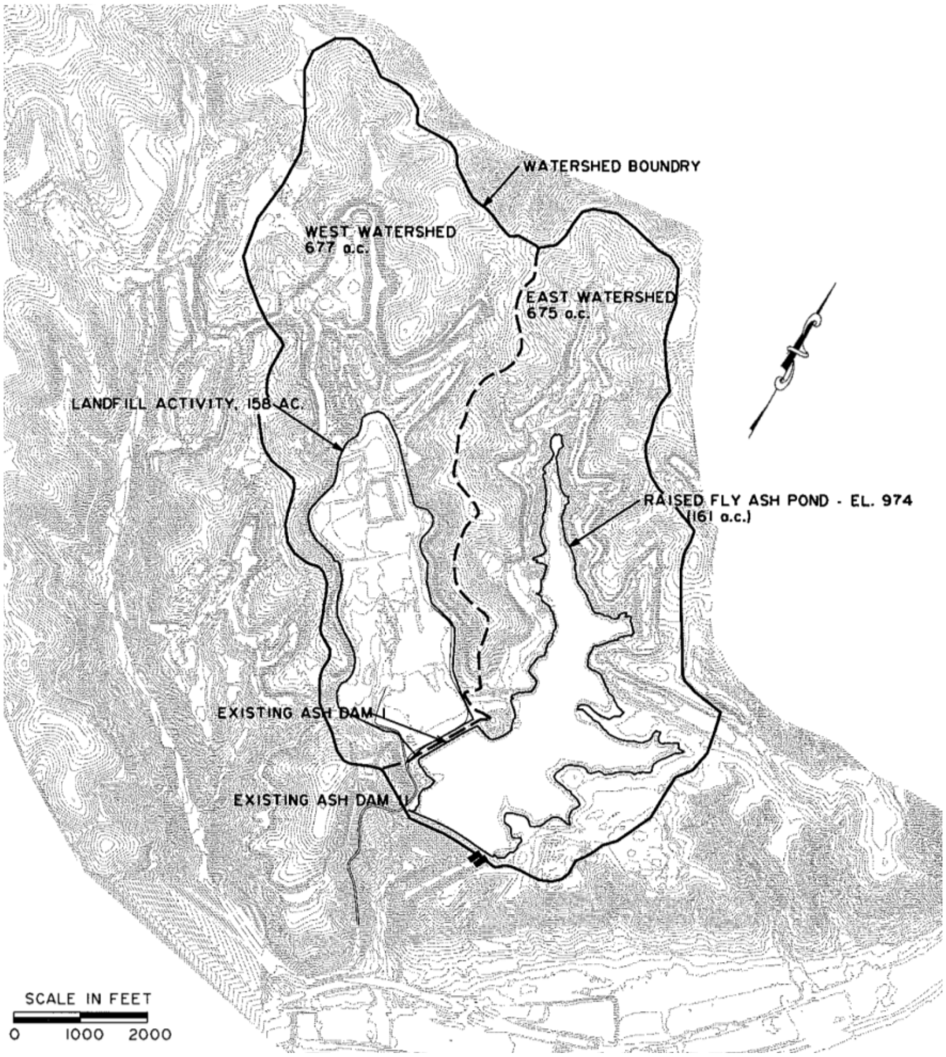


Figure 2. Blockhouse Run Water Shed Upstream of FAR II Dam.

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 geotechnical reports in Attachment B provide the specific properties of the foundation materials. The depth of overburden in the valley at the location of the proposed dam generally ranges between ten and thirty feet. It was found that some of the valley material was deposited by the erosion of the rock strata above the valley bottom.

Since the overburden is saturated and appeared to be heterogeneous, with some material having a softer consistency than that of the sample tested, it was determined to be unsuitable as a foundation material, and was removed in the area below the dam and in the valley slopes up to approximately elevation 800 feet NGVD.

7.1 FOUNDATION MATERIALS

Below the overburden, the top 100 feet of rock at the valley bottom downstream of the proposed cut-off trench consists of a green to gray calcareous claystone with limestone nodules, underlain by layers of sandstone, carbonaceous shales and coals.

Upstream of the core the calcareous claystone is missing and the top part of the rock consists of a fissile carbonaceous shale. The initial borings indicated the apparent existence of clay lenses in the foundation bedrock. In addition, slickenside surfaces, some of which were open and apparently filled with brecciated material, were observed in the borings.

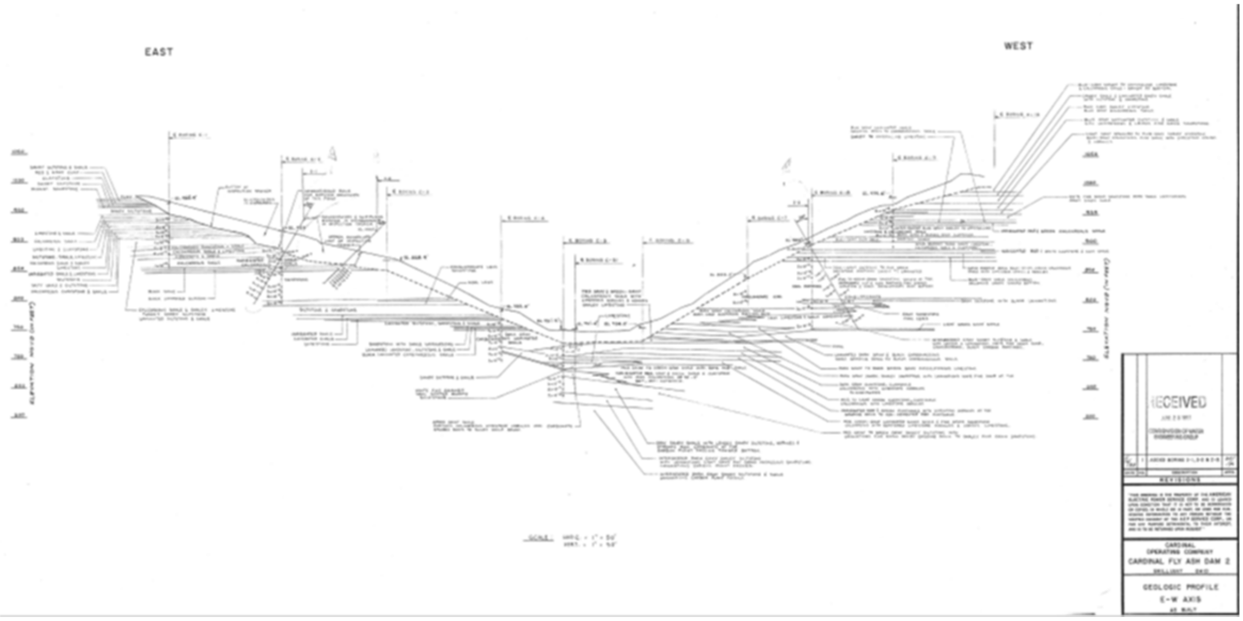


Figure 3. E-W Geologic Cross section Under the body of the Dam.

7.2 ABUTMENT MATERIALS

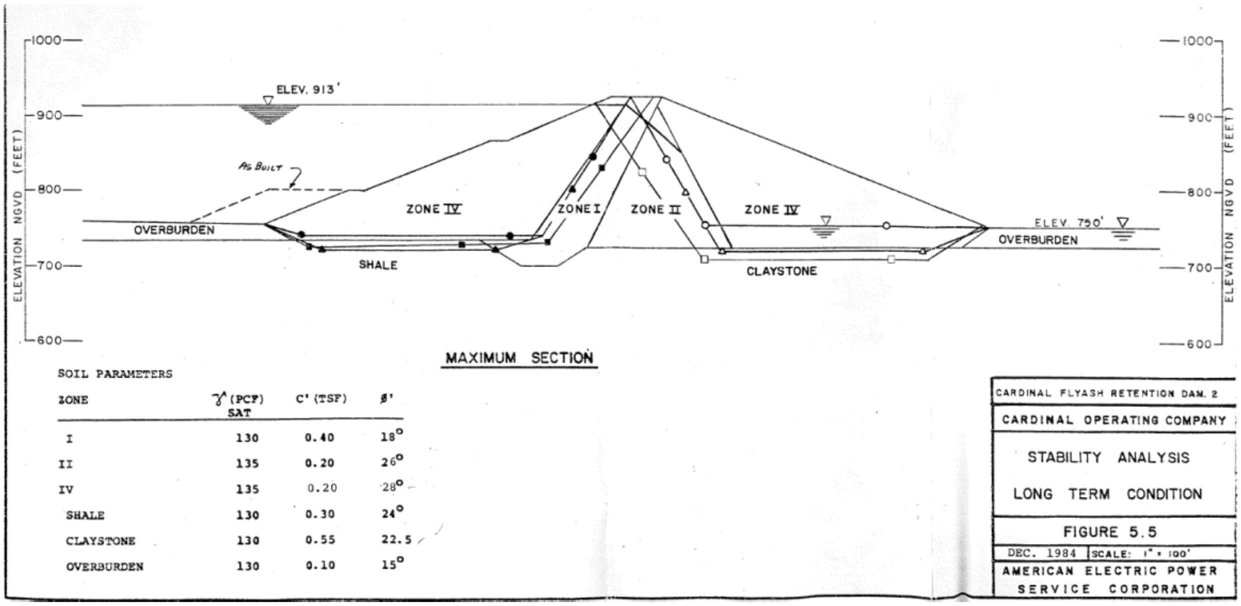
Figure 3 shows a geologic section at the location of the dam. At the abutments location, a cut to rock was made at the proposed abutment as shown in Drawing No. 13-3023. The orientation of the trimmed faces has been designed so that the upstream core of the dam intersects the abutments at right angles. This symmetrical configuration resulted in balanced seating of the clay core against the rock which reduces interface seepage and minimize the potential for cracking of the core.

A grout curtain was provided in the abutments of the dam. The dam was arched in the upstream direction and camber was provided to compensate for settlement. Slope protection consisted of RCC Facing for stage 2 in the upstream and grass and riprap on the downstream for stage 1 and 2 slopes with riprap in the groin of the dam. Stage 3 does not require slope protection.

8.0 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.]

In the following sections, the geotechnical aspects of the key components of the project are discussed. Complete details on the design are presented in the design reports and the design drawings (Attachments B and C) .



SOIL PARAMETERS

ZONE	γ (PCF) SAT	C' (TSF)	Ø'
I	130	0.40	18°
II	135	0.20	26°
IV	135	0.20	28°
SHALE	130	0.30	24°
CLAYSTONE	130	0.55	22.5
OVERBURDEN	130	0.10	15°

Figure 4. Cross section showing main constructed zones.

8.1 ORIGINAL EMBANKMENT

The dam has been designed as an earth embankment with five (5) different zones as shown on Drawing No. 13-3027 (figure 4 above). Zone I serve as an upstream impervious core to reduce seepage and dissipate total heads through the dam. Material for this zone came from Borrow Area I. The inorganic clays of low to medium plasticity found in this area are excellent materials for the core of a dam since they are relatively impervious, resistant to piping and have acceptable shear strength. Zone I soils were compacted with a sheepsfoot roller in lifts of 6 in. maximum loose thickness, at a water content ranging from optimum - 1% to optimum + 2%, to 100% of the maximum dry density as determined by the standard Proctor compaction test (ASTM D 698-78, Method A).

Mine spoil from Borrow Area II used in the transition zone (Zone II) between the clay core and the chimney drain. This zone serves two purposes. First, it complements the clay core in reducing seepage and dissipating total head. Secondly, it works as a filter to prevent piping of the clay core. This material is free of boulders or rock fragments larger than 6 inches. Zone II was compacted with a sheepsfoot vibratory roller in lifts of 9 in maximum loose thickness, at a water content ranging from optimum

8.2 1997 RAISING

The 1997 raising of the dam has been designed as an earth embankment with Roller Compacted concrete (RCC) zone as shown on Drawing No. 13-30041. The RCC zone having steeper slopes than the original dam minimized the amount of fill required for the construction of the downstream shell (Zone

IV). All other zones that compose the original dam were be extended in order to meet the geometry of the raised section for the dam. These zones include clay core (Zone I), chimney drain (Zone III A), Drainage blanket (Zone III B), outlet of the blanket drain (Zone III C), and downstream earth fill (Zone IV).

8.2 2013 RAISING

The back-to-back MSE wall solution was specifically developed to avoid the need for the placement of a large amount of downstream fill and the associated large stress increase and corresponding risk of slope failure. Additionally, MSE walls are flexible and can accommodate the anticipated differential settlement as the foundation for the walls transition from bedrock at the abutments to as much as 216 feet of cohesive embankment fill at the dam high point. With this solution, seepage is controlled by positively connecting the existing clay core with the top of the dam through the use of a sheet pile wall. To minimize seepage further, the joints between the sheet piles was treated with a sealant prior to driving and the lower portion of the wall was imbedded within the cement-bentonite slurry wall. The exposed portion of the driven sheet piles was further sealed post-installation with caulk on the upstream side.
Compaction of the MSE wall backfill was simplified through the use of free draining granular materials which are moisture insensitive.

9.0 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…]

9.1 SERVICE SPILLWAY

The existing service spillway is a vertical concrete shaft structure with side opening for effluent discharge connecting into a sloping concrete shaft structure with one side opening, four feet wide, connecting into a 54 inch diameter pre-stressed concrete cylinder pipe (PCCP), designed for 200 feet of internal hydraulic pressure and 200 feet of overburden pressure. During most operating conditions, discharge through the service spillway is controlled by the-weir flow over the side openings in the shaft. The bottom of the sloping concrete shaft and the entire 54-inch concrete pipe were constructed within bedrock as part of the 1997 raising. Stop logs are utilized to maintain settling action and control the operating pool level.

The energy dissipator at the outlet of the spillway conduit, an impact-type structure, was designed for the probable maximum discharge that would occur during a PMF, estimated to be 330 cubic feet per second. Dimensions of the dissipator, Drawing No. 13-3065, are based on the design criteria of the U.S. Bureau of Reclamation's Engineering Monograph No. 25, "Hydraulic Design of Stilling Basins and Energy Dissipators".
Results of the reservoir routings establish a maximum operating level of 974.0 feet, with the 50-year design flood reaching a level of 975.5 feet, 1.5 feet above the maximum operating pool.

9.2 EMERGENCY SPILLWAY

As of 2013 construction, the existing emergency was raised to El. 975.5 through the use of a mass concrete gravity section in conjunction with reinforced concrete training walls, in a manner similar to the existing configuration. The new walls direct the flow into the existing spillway outlet channel, as shown on Drawing Nos. 13-30083-A and 13-30089-A. A profile of the new emergency spillway is shown on Drawing No. 13-30090-A. In accordance with State of Ohio dam safety requirements for Class 1 dams, the new emergency spillway was designed to pass the design probable maximum flood (PMF) without overtopping the dam. The new spillway features a 108 foot long by 15 foot wide concrete control section positioned at El. 975.5, or 1.5 feet above the maximum operating pool. The training walls are located above elevation 975.5 and will consequently not be exposed to a continuous pool reducing corrosion concerns.

Based on the flood routing, the calculated peak discharge from the dam is 5,409 cfs at a maximum pool elevation of 981.9 feet NGVD. The PMF routing was also checked with the service spillway blocked, which resulted in a maximum pool elevation of 982.8 and 0.2 feet of freeboard.

The engineering drawings of the engineering structures and appurtenances are included in Attachment C.

10.0 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 table below describes the normal pool elevations and maximum pool elevations as well as maximum depth of CCR within the impoundment. The maximum pool elevation have been determined based on the 100% PMP storm analysis based on the Ohio State Requirements. Complete results of the hydrology and hydraulic analysis are included in a report prepared for the 2013 dam raising and included in Attachment E.

Maximum Normal Pool Elevation	974 ft above msl
Current Normal Pool Elevation	963 ft above msl
Maximum Pool Elevation following peak discharge from inflow design flood	982.8 ft above msl
Expected Maximum depth of CCR within impoundment	200 ft

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

[…and any identifiable 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 ponds operations could be adversely affected. These structures include service spillway, weir structures and influent sluicing piping and structures. See design drawings in Attachment C for location and details of all appurtenances.

During an extreme flood event, natural debris may tend to collect along the service spillway. However, the spillway is wide and complete blockage would not be an expected condition. In addition, at the current operating level, the pond capacity is sufficient to contain the design storm.

12.0 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 instrumentation program for the Cardinal FAR II dam consists of piezometers, settlement monuments and a weir. The location of the instruments is shown in plan in Drawing No. 13-3024 and in section in Drawing No. 13-3027, 13-3028, 13-30040, 13-30041, 13-30042, and 13-30098 (Attachment D).

Pneumatic piezometers were installed before construction and as the embankment is being placed, in the foundation, abutments and every zone of the dam.

The piezometers are read on 30 days basis. This information is used to monitor the buildup of pore pressure during and after construction and to evaluate the embankment stability in terms of effective stresses.

Settlement monuments are installed approximately every 50 ft along the downstream edge of the dam crest.

A 90° V notch weir was installed 50 ft downstream of the downstream toe of the dam. Seepage from the underdrain system and from seeps in the abutments, which are likely· to develop, were directed towards the weir. The weir is read on a minimum of every 30 days.

13.0 AREA – CAPACITY CURVES FOR THE CCR UNIT 257.73 (c)(1)(IX)

[Area-capacity curves for the CCR unit.]

Figure 5 shows the area capacity curves for the Cardinal FAR II and is included in the Hydrology and Hydraulic Analysis Report by SM&E, September 2012 in Attachment E.

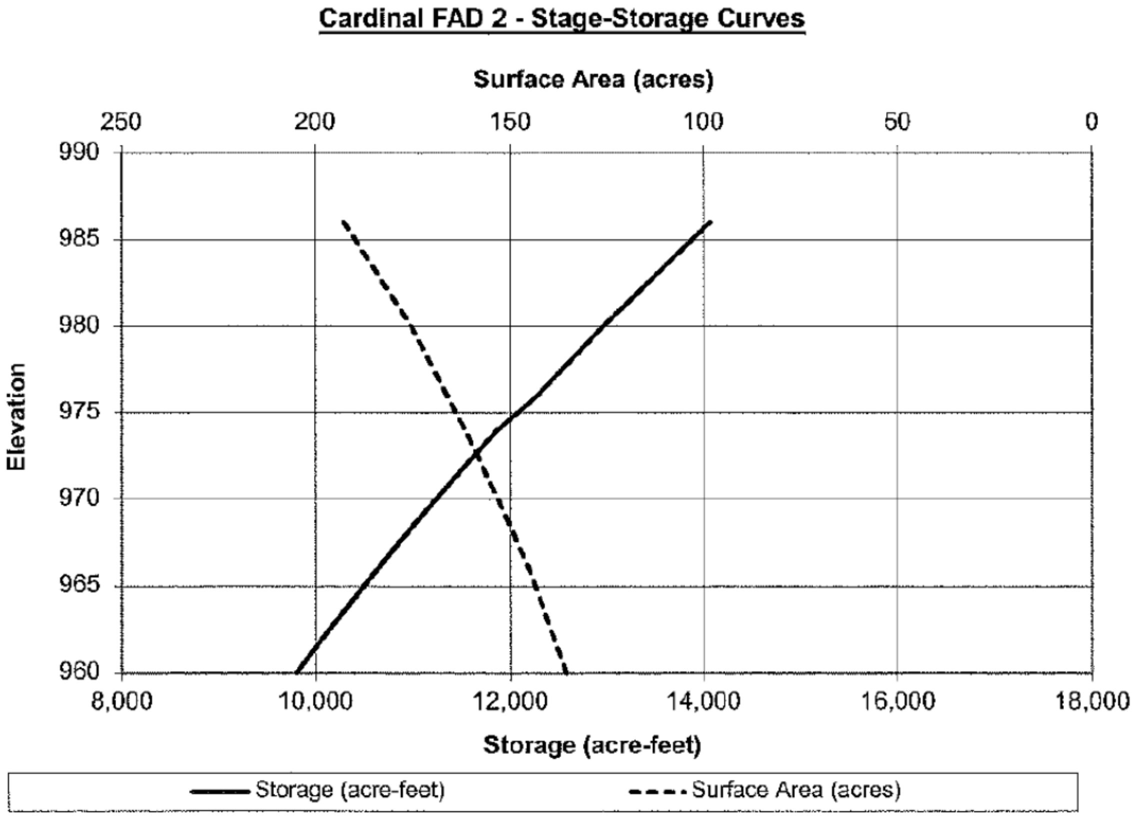


Figure 5. Capacity curves the Dam.

14.0 257.73 (c)(1)(x) DESCRIPTION OF EACH SPILLWAY AND DIVERSION

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

The CCR's are sluiced into the facility through a series of pipes designed to handle the various required capacities. The pipes discharge into the facility at locations shown on design maps. The CCR effluent is decanted through a reinforced concrete vertical drop inlet connected to an inclined shaft connected to a 54 inch diameter pre-stressed concrete cylinder (PCCP) outlet outflow pipe. The outflow pipe leads to a dissipation structure to an outfall at the Ohio River. Complete details of each spillway structure are included with the design drawings in Attachment C. Hydrology and Hydraulic Analysis which include calculations for each spillway structure are included in Attachment E.

The FAR II pond is valley fill pond with no diversions present for this facility.

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.]

Readily available portions of the original and the raising construction specifications are included in Appendix B. The full list of the specification is included in the approved permit application.

As required by the CCR rules the FAR II 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 Ohio the impoundment is inspected quarterly basis.

An impoundment maintenance plan is provided in Attachment F. 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.]

Overall, the existing dam has performed well since it was originally put into service in the mid- 1980s. This having been said, a few incidents have occurred which were investigated and resolved as discussed in the following sections.

16.1 DURING CONSTRUCTION SLIP (1997)

During the 1997 dam raising, an apparent undrained slope failure took place in late October within the downstream mine spoil zone near the toe of the slope as fill was being placed. At the time of the slope failure, the downstream mines spoil shell had been constructed up to El. 900. The slope failure exhibited a head scarp at roughly El. 827. Construction was halted and the failure was investigated. The investigation suggested that the failure resulted from pore pressure build up within the newly placed cohesive mine spoil soils due to an accelerated placement rate in conjunction with above optimum moisture contents. The failure was remediated by removing the majority of the slide mass in conjunction with the construction of a large rock fill toe drain/berm.

The toe berm is shown on the as-built drawings, dated March 31, 2000, from the 1997 dam raising which were submitted to ODNR.

16.2 CRACKING IN RCC (1997)

Subsequent to the completion of the 1997 dam raising, a number of cracks were observed within the RCC section. It was believed that these cracks were related to differential settlement along the crest as the amount of fill above bedrock varied in thickness as well as related to shrinkage of the RCC mass during curing. The RCC mix design and thermal gradients through the RCC zone were also considered attributing factors to the cracking. These cracks are described in a report entitled Cracks in RCC Zone and Post-Construction Performance of Dam , dated June 1, 1999.

Subsequent monitoring of the cracks suggested that no further significant movement had taken place and the cracks were sealed.

16.3 RIGHT ABUTMENHT SEEPAGE IN (2004)

In February of 2004, significant seepage emanating midway up the right downstream groin was observed. This seepage carried ash with it leading to a concern for a potential piping failure. The seepage was assessed and was believed to have occurred through the jointed right abutment bedrock. Laboratory analysis (grain size, mineralogical, and X-ray diffraction) of the seepage confirmed that it was carrying fly ash and not material from the dam itself. The seepage was initiated only after the water level in the reservoir reached the level of the more permeable sandstone bedrock layer. This layer was inadvertently exposed within the reservoir due to an surficial soil slumping which had occurred near upstream of the right abutment in 1984, prior to the construction of the Stage 1 dam. Over time this seepage has been reduced and no longer carries ash resulting from a self-healing process. This issue was described in detail in report prepared by AEP submitted to ODNR in 2004, as well as in the peer reviewed paper Amaya, Massey-Norton and Stark (2009). Subsequent monitoring of the right abutment downstream groin seepage has indicated that the seepage from the right abutment is staying relatively constant.

17.0 REFERENCES

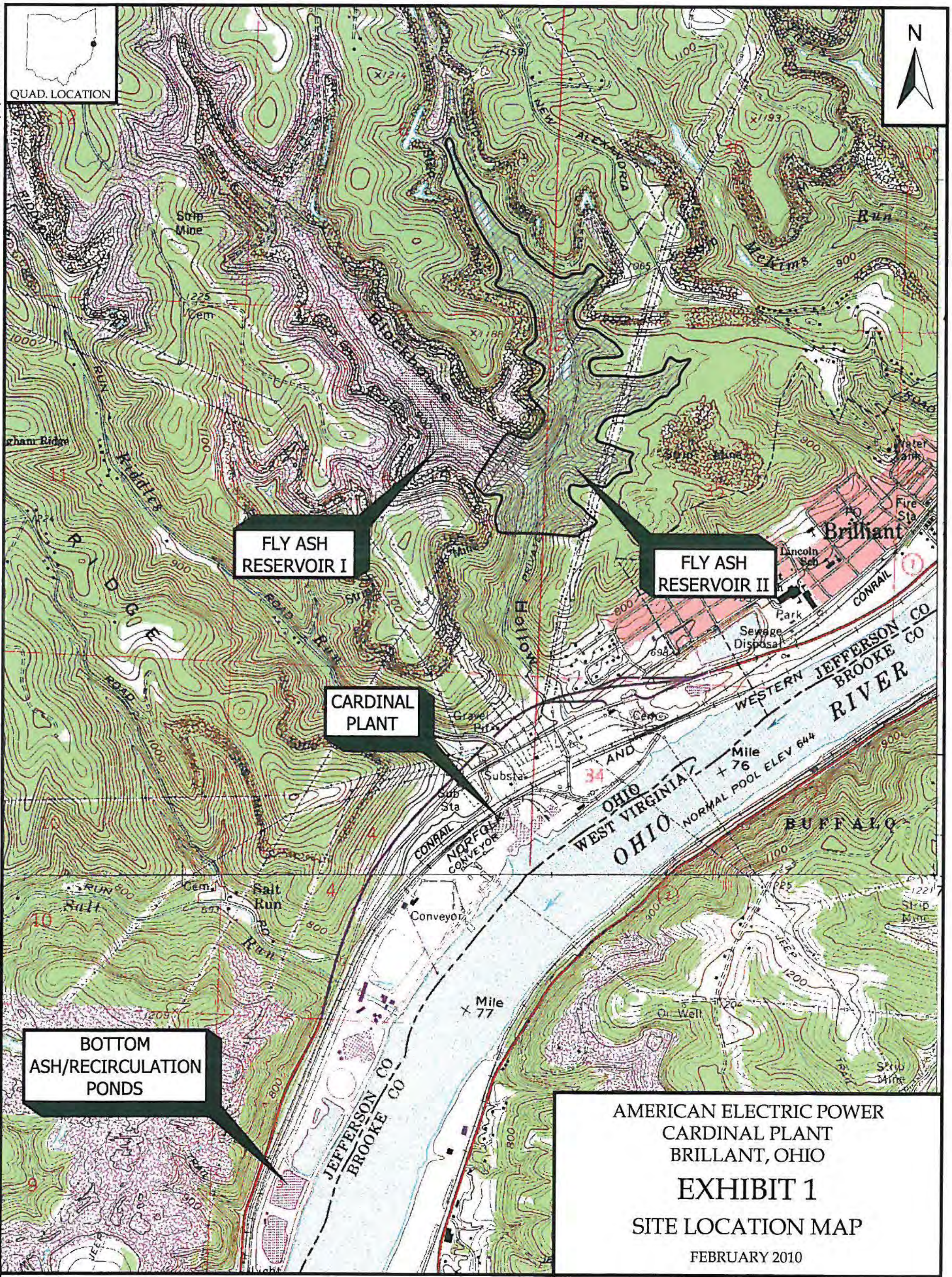
Amaya, P.J, Massey-Norton, J.T. and Stark, T.D., “Evaluation of Seepage from and Embankment Dam Retaining Fly Ash”, ASCE Journal of Performance of Constructed Facilities, Vol. 23, No. 6, December 1, 2009.

ATTACHMENT A

LOCATION MAP

PLOTTED: 2/5/2010 10:02:32 AM

P:\PR48249\cadd\Figure 1.dwg 12/10/2009 8:34:37 AM VanZandt, Ryan



AMERICAN ELECTRIC POWER
CARDINAL PLANT
BRILLANT, OHIO

EXHIBIT 1

SITE LOCATION MAP

FEBRUARY 2010

SOURCE:
7.5 MINUTE TILTONSVILLE, OHIO U.S.G.S. QUADRANGLE MAP

ATTACHMENT B

DESIGN REPORTS

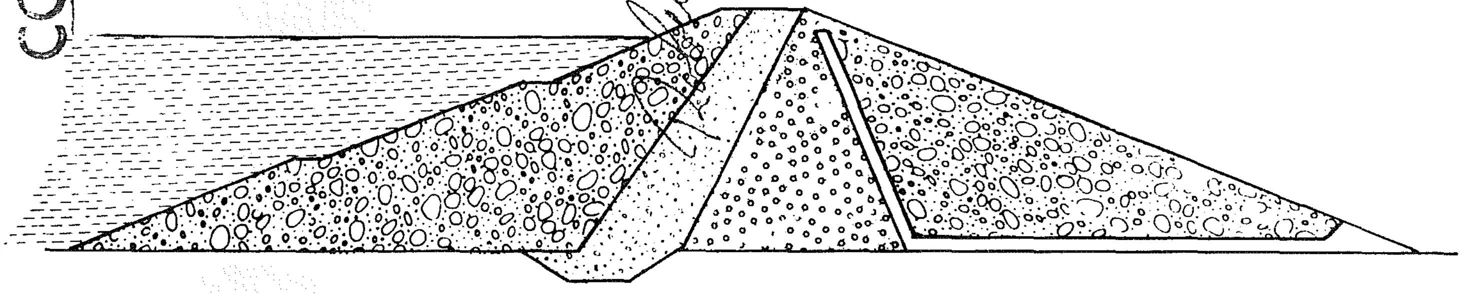
CARDINAL PLANT

FLYASH RETENTION POND II

Design Report For Proposed Dam

CONDITIONALLY

APPROVED
PERMIT NO. 85-1-47
MAY 1 1985



STATE OF OHIO
DEPARTMENT OF NATURAL RESOURCES
DIVISION OF WATER
DAM PERMITS
APPLICANT'S COPY

RECEIVED
JAN 24 1985
DIVISION OF WATER
SAFETY

FOR: CARDINAL OPERATING COMPANY

BY: AMERICAN ELECTRIC POWER SERVICE CORPORATION

AEP

DESIGN REPORT

PROPOSED DAM
FOR
FLYASH RETENTION POND II

CONDITIONALLY
APPROVED
PERMIT NO. 85-147
MAY 1 1985

CARDINAL PLANT
BRILLIANT, OHIO

PREPARED FOR
CARDINAL OPERATING COMPANY

Robert L. Gottemoeller

PREPARED BY
AMERICAN ELECTRIC POWER SERVICE CORPORATION
CIVIL ENGINEERING DIVISION
COLUMBUS, OHIO
DECEMBER, 1984

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

1.0 INTRODUCTION

Cardinal Operating Company, agent for Ohio Power Company and Buckeye Power Incorporated, proposes to build a second flyash retention dam at Cardinal Plant. The new facility will be constructed in two stages. The first stage dam will have a crest elevation at 925' NGVD (National Geodetic Vertical Datum) and will provide approximately 11 years of storage. For the second stage, the dam crest will be raised to an ultimate elevation of 970 ft. NGVD to create an additional 17 years of storage. This report presents the final design of the first stage dam and appurtenances in compliance with the Final Design Report Requirements Rule 1501:21-5-04 of the Administrative Code of the Ohio Department of Natural Resources, Division of Water.

1.1 General Description

The proposed dam site is located in Section 5 of Wells Township, Jefferson County near Brilliant in eastern Ohio, as shown in the Location Plan on the cover sheet of the Design Drawings. Drawing No. 13-3001 illustrates the project's general arrangement and construction sequence.

The design of the proposed dam (stage I only) consists of a 180 foot high arched earth embankment with a zoned cross section. The dam will have a 70 foot wide by 1055 foot long crest at an elevation of 925 feet NGVD. Drawing No. 13-3029 shows the proposed dam in plan. Longitudinal and transverse sections through the dam are presented in Drawing Nos. 13-3027, 3028 and 3030.

Approximately 80 acres will be inundated by damming Blockhouse Run downstream of the confluence of the east and west branches 1,000 feet downstream of the existing flyash dam. Prior to construction of the proposed dam, the pipeline which is presently located in the valley in the area of the proposed dam and which discharges fly ash into the existing retention pond will be rerouted along the west bank of Blockhouse Hollow. This work will prevent disruption of service during dam construction and is scheduled for the second part of 1984 and the first part of 1985. Dam construction will begin in spring 1985 and be completed by late 1987.

During construction of the main dam, a 40 foot high cofferdam with a crest elevation of 800 feet NGVD will serve to impound the discharge from the service spillway of the existing dam and the runoff of the east branch of Blockhouse Run. The cofferdam will eventually become part of the dam's upstream shell. The service spillway (also refered to as principal or primary) will run along the east abutment at an approximate elevation of 780 feet NGVD. It will consist of a 42"Ø precast concrete cylinder pipe placed in a rock trench and completely embedded in 4500 psi concrete. A temporary conduit which will connect to the principal spillway and be used as the spillway for the cofferdam. Drawing Nos. 13-3032 and 13-3024 show the service spillway and cofferdam in plan and Drawing No. 13-3033 presents sections and details. A 110 foot wide unlined rock cut along the east abutment, as shown in plan in Drawing Nos. 13-3023 and 13-3024 and in section in Drawing No. 13-3027 and 3028, will serve as the emergency spillway.

✓ Once built, the dam will provide 4,780 acre-feet of storage volume with the pond at its maximum pool elevation (913 feet NGVD) and a total storage volume of

5,800 acre-feet with the pool at the level of the dam crest.

1.2 Classification

The Ohio River, Cardinal Generating Plant, State Route 7 and the Tidd-dale subdivision of Brilliant, Ohio all lie directly downstream of the proposed dam. Therefore, a failure of the dam causing a sudden and uncontrolled discharge of water would likely result in loss of human life, and damage to homes, high value utility installations and both a railroad and a public road. Also, the dam height and storage volume exceed the corresponding thresholds for class I dams as established in Section 1501:21-13-01 of the ODNR Administrative Code. For these reasons, the proposed facility has been designated as a class I dam.

1.3 Scope of Work

This report presents the final design of the proposed project. Section 2 briefly describes the geology and findings of the site investigation of the soil and rock conditions at the proposed dam location and borrow areas. The parameters, assumptions and analysis pertaining to the hydrology of the site are described in Section 3. The design of the dam and geotechnical analyses are discussed in Sections 4 and 5, respectively. Finally, Section 6 summarizes the cost estimates and presents the construction schedule for the dam and appurtenances.

2.0 SITE INVESTIGATION AND POTENTIAL BORROW MATERIALS

A total of fifty two (52) borings and fifty eight (58) test trenches have been made at the site to investigate the foundation and abutments of the proposed dam and potential Borrow Areas in Blockhouse Hollow. SPT (Standard Penetration Test) blow counts and RQD (Rock Quality Designation) values were recorded for the overburden and bedrock samples respectively. Both disturbed and undisturbed samples of the overburden soils and underlying rocks were collected for identification and for laboratory determination of index and engineering properties. Tests performed in selected sample materials include natural water content, Atterberg limits, sieve analyses, hydrometer, consolidation, permeability, direct shear and triaxial tests. All tests, except when otherwise noted, were conducted at American Electric Power Service Corporation (AEPSC) laboratory. Appendix A contains copies of the drill logs and descriptions of the test trenches, and Drawings No. 13-3002 and 13-3003 show their locations. Appendixes B, B-1 and B-2 presents the results of the laboratory tests performed on representative soil and rock samples.

All the boreholes and trenches in the vicinity of the dam were either grouted or backfilled at the completion of the work.

2.1 Regional Geology

Jefferson County is located in the unglaciated Kanawha section of the Appalachian Plateau province. Both the Monongahela and Conemaugh Series of the Pennsylvanian System of sedimentary rocks comprise the rock formations in the region. These rock formations consist of arenaceous, carbonaceous and calcareous shales, sandstones, limestones and coal seams. The base of the Pittsburgh No. 8 Coal separates the base of the Monongahela Series and the top of the Conemaugh Series.

The Conemaugh Series has an average thickness of 518 feet and extends to the top of the Upper Freeport No. 7 Coal (Lamborn, 1930). A thin overburden mantle ranging from 5 to 25 feet in thickness caps the bedrock.

The relief of the region is characterized by a maturely dissected plateau with deep and precipitous valleys separated by gently undulating divides.

Bedrock in Jefferson County generally strikes northeast 58 degrees and dips to the southeast from 7 to 17 feet per mile (Lamborn, 1930).

2.2 Site Geology

Geologic cross sections constructed from the borings and test trenches are shown in Drawings Nos. 13-3004 and 13-3005 and in Figure 2.1. The lower part of the Monongahela Series and the top part of the Conemaugh Series outcrop at the surface. The rock formations of relevance at the proposed dam site vary from a gray arenaceous shale overlying the Summerfield Limestone to a green to gray calcareous shale underlying the Harlem Coal. The marker bed for this interval is the Ames Limestone which is located at an approximate elevation of 750 feet NGVD. The Morgantown Sandstone is the most conspicuous sandstone formation present.

The overburden mantle in Blockhouse Hollow consist of residual soils, mine spoil, landslide debris and alluvial deposits.

There are no known geologic faults at the proposed dam site.

2.3 Foundation

The results of the site investigation indicate that removal of the overburden soils is required to provide a suitable foundation for the proposed dam. Also, the calcareous claystone which constitutes the foundation rock downstream of the proposed cut off trench is slickensided and brecciated. Upstream of the cut off trench, the foundation rock is a fissile carbonaceous shale. In the next two sections the findings of the foundation investigation are discussed in detail.

2.3.1 Overburden

The depth of overburden in the valley at the location of the proposed dam generally ranges between ten and thirty feet. The standard split spoon sampler blow counts in this material vary from less than ten to about fifty blows per foot. Test trenches were excavated in the dam site overburden in order to obtain a better understanding for the large spread in blow count values. It was found that some of the valley material was deposited by the erosion of the rock strata above the valley bottom. The high blow count can be attributed to the sampler coming in contact with gravel and cobble size pieces of these shale and sandstone strata. The low blow-counts were obtained when the sampler was driven through zones of inorganic clay free of rock fragments. The clay samples that were recovered from the borings and test trenches had a soft to stiff consistency and low to medium plasticity. The results of the Atterberg Limits and gradation tests on samples of the overburden soils in the valley bottom are presented in Appendix B, Figures B.1 and B.2.

The level of the groundwater has been measured along the valley bottom in boreholes and also, at piezometers located in both the overburden and foundation bedrock. The depth to the water table in the boreholes is recorded in the drill logs and is shown on Drawing No. 13-3004. In Appendix C, a summary sheet is presented for each piezometer describing its installation and portraying the measured total head. These measurements show that the groundwater table in the valley bottom occurs approximately parallel to the ground surface about 5 feet below grade. Also, there is no appreciable difference between the measured total head in bedrock and overburden.

A block sample was taken from test trench No. 2 which is located in the valley bottom as shown in Drawing No. 13-3002. Consolidation tests and unconsolidated undrained triaxial tests were performed on soil specimens trimmed from this block sample. The test results are presented in Appendix B, Figures B.3 through B.6. Based on this data, the undrained shear strength, Su, of the clay matrix was found to equal 1.2 TSF. Since the overburden is saturated and appeared to be heterogeneous, with some material having a softer consistency than that of the sample tested, it was determined to be unsuitable as a foundation material, and will be removed in the area below the dam and in the valley slopes up to approximately elevation 800 feet NGVD.

2.3.2 <u>Foundation Rock</u>

The top 100 feet of rock at the valley bottom downstream of the proposed cut-off trench consists of a green to gray calcareous claystone with limestone nodules, underlain by layers of sandstone, carbonaceous shales and coals. Upstream of the core the calcareous claystone is missing and the top part of the rock consists of a fissile carboneceous shale. The initial borings indicated the apparent existence of clay lenses in the foundation bedrock. In addition, slickenside surfaces, some of which were open and apparently filled with brecciated material, were observed in the borings.

Clay Seams

There is some question as to whether or not the clay found in the borings could have resulted from grinding and exposure to water while drilling. To check this hypothesis, a core section from boring D-37 that had air slaked into a cohesionless mass of sand and gravel size angular particles, was worked with a mortar and pestle through several cycles of wetting. A silty clay of low plasticity similar to that found in the borings was formed suggesting that a cohesive material could be produced from the rock during drilling. The Atterberg limits on this material are presented in Appendix B, Table B.1.

In order to determine the existence and continuity of "clay seams" in the foundation of the proposed dam, 15 additional borings were drilled along the valley bottom. Borings D45 through U52 were drilled using a standard size NX core barrel with an inside discharge bit, same as with the initial borings. Borehole D45 was cored in 5 foot runs, with the intent that if any plugging of the bit occurred, the run would be terminated and the core retrieved. Core run lengths were restricted to 2.5 feet in holes D46 through U57 to increase recovery. Fairly high rotation and feed pressure were used to maintain as rapid core advance as possible using minimum amounts of water in order to reduce and limit the exposure of clay layers to washing by the drill. It was thought that if any clay layers were present the bit would be plugged by being forced into the clay with low water volume and pressure. On borings U53 through U57 a bottom discharge bit was used. This bit would have had a greater tendency to plug in soft clay layers but would subject the core to less washing. Borings U-62 and D-63 were cored using a triple tube core barrel with a size NQ bottom discharge bit which produced the least disturbed cores of all the borings.

Core recovery was generally good, as shown on the logs, with only occasional loss, but considerable grinding of the core did occur. In the soft calcareous shales with limestone nodules it appeared that the larger nodules were broken away from the soft shale matrix, spun by the core bit, and ground into the softer shale below. The grinding was also evident in the sandstone units which contained soft shale lenses and the laminated shales above the lower coal seam which split very easily along the laminations. The grinding of the core was probably due to the fairly high rotation speed and feed pressure.

In borings D45 through U52, only in rare cases was the ground up material preserved and retrieved in the core, showing up as traces of clay on the ground joints. Very little clay was recovered from these holes, but that which was recovered could be attributed in each case to grinding of the rock core as evidenced by circular spin patterns on the ends of the core pieces. The clay recovered was actually a silt of low to medium plasticity, with finely ground shale fragments and a color generally matching the surrounding rocks.

In Borings U-53 through U-57, U-62 and D-63 in which the bottom discharge bit was used, the driller reported no indication of plugging of the bit. However, as shown on the logs, several silty clay lenses 1/8 to 1/2 inch thick and soft, broken and brecciated shale intervals were recovered. There is evidence indicating that the clay lenses were created by the drilling process, as in borings D45 through U52 described above, but that superior sample recovery was produced using the bottom discharge bit. In each occurrence, the apparent clay lenses were located where the core could be seen to have been ground by the drill. The clay had a color matching that of the surrounding rock, usually contained finely ground shale fragments, and in many instances occurred only on the exterior portion of a joint, grading to soft shale toward the center of the core.

Along with the clay lenses, there were frequent occurrences where the core could be clearly observed to have been ground, but where no water had been in contact with the core during the grinding. The result was a white rock flour. This rock flour could be observed as lenses between pieces of ground core and as a coating on the outside of the softer rocks. On the outside of the core the rock flour was covered by clay. When wetted the rock flour assumed the texture and color of the silty clay lenses and coating.

The silty clay recovered during the core boring could be duplicated by hand. In the soft calcareous shales, merely kneading a wetted portion of the shale with a spatula produced a plastic silty clay similar to that retrieved in the cores.

Two inspection trenches were excavated at the proposed dam site to investigate the existance of clay seams in the foundation rocks. At TP-42 (located in the vicinity of the downstream toe) the overburden consisted of 1 to 2 feet of boiler slag fill and 6 to 7 feet of weathered shale and sandstone with a thin layer of alluvial sand and gravel deposits just above bedrock. Approximately 8 to 9 feet of rock consisting of highly fractured and slickensided green to gray calcareous claystone was exposed. The top 4 to 5 feet of shale showed open joints filled with brecciated material. In the remaining exposed rock, the slickensides were tight. Several sets of joints were encountered and their orientation measured (see the trench description in Appendix A for strikes and dips). Block samples of the rock containing slickensides with and without brecciated material together with samples of the intact rock were collected for testing. No clay or silt lenses were observed in the block samples or the trench walls.

In TP-58, located in the area of the cut-off trench, the overburden soil consisted of 4 to 6 feet of angular cobbles 1/2 to 1 1/2 in. in size bound together by a very wet and soft clay matrix. Approximately 8 ft. of rock consisting of a fissile shale with carbonaceous partings and horizontal bedding planes were exposed. No evidence of clay seams was found. Samples of the overburden soil and clay shale were collected for laboratory determinations of engineering properties.

Drawing No. 13-3005A presents in profile the incidence of clay material and slickensides as observed in the foundation borings. Based on this drawing, the observations made during the latter set of borings, and the findings of the inspection trenches, it was concluded that continuous clay seams are not likely to exist in the area of the foundation and that what was identified as clay seams on the initial set of boring was generated by grinding of the rock and exposure to water during drilling.

Strength of Foundation Rocks

Unconfined compression tests were performed on core samples of the valley bottom rock by AEPSC personnel at the Polytechnic

Institute of New York (PINY) soils laboratory. The test results together with sample location and description are presented in Appendix B, Table B.2. Only two samples had unconfined compressive strength of less than 5 TSF; most of the test results were considerably higher. The lower strength values recorded are believed to correspond to core samples containing tightly closed slickensides which remained closed during drilling and handling, but failed in testing. Core samples that failed through the intact rock account for the higher unconfined compressive strength values.

Direct shear tests have been performed on block samples of the green to gray calcareous claystone obtained from TP-42 and the fissile shale from TP-58. These tests were conducted at Ohio State University (OSU), Columbus, Ohio, under the supervision of Dr. W. Wolfe.

Both intact and brecciated/slickensided speciments of the calcareous claystone from TP-42 were sheared at the as received water content and after saturation, and at strain rates varying from 0.016 to 0.000288 in./min. The results are presented in Appendix B, Figures B.7 through B.16). These test results indicate the following:

1. Excess pore pressure did not appear to have developed during shearing, even for saturated speciments sheared at a strain rate of 0.016 in/min, the fastest rate permitted by the equipment.

2. The calcareous claystone exhibits strain softening with the post-peak strength of the intact specimens approaching the strength of the brecciated/slickensided samples.

3. The strength of the intact claystone is dependent on the rate of strain, with lower strength at slower strain rates.

Based on these findings, it was decided that the envelope defined by the strength of the brecciated/slickensided samples and the post peak strength of the intact specimens sheared at slowest strain rate possible best represents the strength of the calcareous claystone and thus, will be used for analysis of both the end of construction and long term stability of the downstream foundation. This envelope can be approximated as follows:

$C' = 0.55$ TSF and $\phi' = 22.5^{\circ}$

The direct shear test results on saturated samples of the carbonaceous shale from TP-58 sheared at strain rates of 0.016 and 0.000288 in./min. are presented in Appendix B, Figures B.17 through B.24. These test results indicate that:

1. The shale exhibits strain softening.

2. The strain rate controls the development of excess pore pressures during shear.

Based on these findings, the following strength envelopes of the fissile clay shale were selected for the stability analysis of the upstream foundation:

1. Undrained loading or end of construction.

   Cu = 0.58 TSF and Øu = 15°

2. Steady state flow or long term condition

   C' = 0.30 TSF and Ø' = 24°

Acres American Incorporated conducted direct shear tests on the clay shale as part of their foundation investigation for the final raising of the existing dam. A copy of their 1972 report, "Raising of the Existing Dam, Geotechnical Data for Bidders, Volume 2" is included as Appendix F. Drawing No. SK 3134-A-280 of the Acres report summarizes their test results on the foundation clay shale.

2.3$\overset{3}{.2}$ Abutments

According to available geologic literature, the principal regional joint system in the dam site area is generally orientated in alignment with the axis of the Appalachian folding. The principal joint system is perpendicular to the direction of the main regional pressure. The minor set of joints is nearly parallel to the direction of the main regional pressure. The strike of the Appalachian folding closest to the dam site, as shown on the Mid-Atlantic Region Geological Highway map published by The American Association of Petroleum Geologists, gives the strike of the principal joint system as N27E, and the strike of the secondary joint system as N63W.

In underground coal mining, the system of main entries and sub-mains is usually orientated parallel to the cleat (principal joint system) or butt cleat (minor joint system). The orientation of entries in three underground mines in the Pittsburgh Coal in Blockhouse Hollow are as follows:

Goucher No. 2, Mine File Map No. 57: N25E, N65W
Goucher Mine, Mine File Map No. 16: N23E, N67W
Goucher Mine, Mine File Map No. 23: N24E, N64W

Two exposures of the rock strata which underlie the left abutment in the site area were inspected. One is located in a ravine on the east abutment just upstream of the proposed dam, and the second on Route 7 on the south side of Riddles Run. The strikes of the more prominent joints in the Morgantown Sandstone as measured in the two exposures are listed below:

Blockhouse Hollow	Riddles Run
N40E	N30W
N35-40E	N32E
N15E	N30W
N20E	N40E
N30E	N60W
N65W	N60W

While the exposure along Riddles Run was formed by blasting and that in Blockhouse Hollow is subject to stress relief and downslope movement, the joints measured roughly coincide with the orientation determined from the regional geology, and the orientation of the entries in the underground coal mines.

Using an average strike for the primary joint system of N25E and a strike for the secondary joint system of N65W, the primary and secondary joint systems are nearly parallel and perpendicular to the abutments, respectively. The primary joint system is probably being reinforced by stress relief due to erosional unloading which tends to open these joints further. It is probable, therefore, that seepage from the reservoir, as controlled by the joint system, would be greater parallel to the abutments than perpendicular to the abutments.

In order to assess the potential for seepage through the abutments and foundation of the proposed dam, field permeability tests were conducted in both vertical and angled boreholes along the longitudinal axis of the dam. The standard double packer test was performed on each rock strata encountered. Test results are tabulated in Appendix C and summarized in the geologic cross sections (Drawings Nos. 13-3004 and 13-3005). No apparent correlation between lithology and permeability at the proposed site can be detected from these test results. Bedrock permeabilities did tend, however, to decrease with depth, ranging from 10^{-3} to 10^{-6} cm/sec. This trend is believed to result from a decrease in the width and number of open joints and fractures with increasing depth.

The field permeability measured in the vertical boreholes indicates that the Morgantown Sandstone, which was expected to be highly jointed and fractured, appears to be relatively impervious except where the formation is exposed or near the land surface. This finding contradicts the observed performance of the existing dam. Seepage around the abutments of the existing dam, although not in alarming quantity and free of soil fines, is believed to be occurring primarily through the Morgantown Sandstone. This apparent contradiction can be explained by the fact that the primary and secondary joint systems are likely to have dips perpendicular to those of the nearly horizontal bedding planes. If as suspected, the joints have nearly vertical dips, it is likely that vertical boreholes in which the packer tests were run intersected only a small percentage of joints and thus, gave uncharacteristically low values of permeability. The results of packer tests conducted on boreholes inclined 30 to 45 degrees from vertical support this explanation as the measured permeability in the angled boreholes were consistantly higher than the values obtained from the vertical holes.

2.4 Borrow Areas

Based on the findings of the site investigation, two Borrow Areas for soils and two quarries for rip-rap have been selected. Drawing No. 13-3001 shows the location of these areas. In addition, bottom ash from Cardinal Plant has been investigated and found to be acceptable as granular material for the drain.

2.4.1 Borrow Area I

Overburden along the valley slope and top of the ridge on the east side of the proposed retention pond consists of 12 to 15 feet of weathered to partially weathered soft calcareous shales, siltstones and claystones. These soils are very similar in color, gradation and plasticity, and can be classified as a mixture of gravel and cobble size pieces of rock in a matrix of inorganic clay of low to medium plasticity. Results of index tests on these soils are presented in Appendix B, Table B.3 and Figures B.29 through B.190.

Tests to determine engineering properties have been performed on samples which appeared typical of the overburden soils in this area. Unconsolidated undrained (UU) triaxial tests were conducted at various water content on samples TP-9, TP-32B, TP-34A and a composite sample obtained by combining soils from TP-45 through TP-51. Soil specimens were prepared in 1.4 and 2.8 inch diameter molds with heights between 3 to 6.5 inches. Results of UU tests on TP-9 are shown in Appendix B, Figures B.193 through B.200. Results on UU tests on TP-32B and TP-34A are presented in an August 21, 1984 report by the Ohio State University which is included as Appendix B-1. Results of UU tests on the composite sample are presented in an August 6, 1984 report by Benedict, Bowman, Craig and Moos (BBCM) Soils Engineering Consultants which is included as Appendix B-2.

Figure B.201A, Appendix B, shows for Borrow Area 1 soils, the undrained shear strength at two normal stress levels, as a function of the water content at which the specimens were compacted. The standard Proctor compaction curves, Atterberg limits and natural water contents of the samples used in the UU tests are also presented. In addition, the statistical averages and range in values of the index properties for all samples from Borrow Area 1 are shown. Based on this figure, the following observations can be made:

1. The undrained shear strength consistently decreased as the water content at which the specimen was compacted increased, up to about optimum + 2%, after which the undrained shear strength leveled off.

2. The plasticity and compaction characteristics of TP-34A and the composite sample are reasonably close to the averages values of all the samples. Therefore, the results of the strength tests on these samples can be expected to approximate average conditions for Borrow Area 1 soils.

3. The similarity in the distributions of natural water and optimum water contents suggests that, on the average, the soils exist in the field at a water content very close to optimum. A compaction specification in terms of water content for Borrow Area 1 soils of OPT-1% to OPT + 2% appears attainable except in periods of excessive rain.

Figure B.201, Appendix B, presents for TP-34A and the composite sample, the strength envelopes for various compacted water content as determined from the UU triaxial tests. Based on these results, the undrained shear strength of Borrow

Area 1 soils compacted at optimum - 1% to optimum + 2% to 95% of the maximum dry density, as determined by the standard Proctor test, can be reasonably approximated as follows:

$$C_u = 1.1 \text{ TSF and } \phi_u = 6^{\circ}$$

Consolidated drained (CD) triaxial tests were performed by Casagrande Consultants, Arlington, Massachusetts on 1.4 inch diameter specimens from test trench No. 9. The specimens were compacted to an average density equal to 97% of the standard Proctor maximum dry density and a water content essentially equal to optimum. Test results are presneted in Appendix B, Figure B. 202 and B. 203.

Consolidated undrained ($\overline{CU}$) triaxial tests with pore pressure measurements were performed at OSU on specimens prepared by mixing TP-35C and TP-38C samples and compacting them to various water contents. Similar tests were conducted on specimens of the composite sample at BBCM soils laboratory. The results from these tests are presented in Appendices B-1 and B-2 respectively. For comparison, the strength envelopes from the CD and $\overline{CU}$ triaxial tests on Borrow Area 1 soils are summarized in Appendix B, Figure B.204. It appears from these results that, in the range of water contents of interest, the drained strength of Borrow Area 1 soils is more dependent on plasticity than on compacted water content. Based on these results, the shear strength of Borrow Area 1 soils for steady state flow condition can be conservatively approximated as follows:

$$C' = 0.4 \text{ TSF and } \phi' = 18^{\circ}$$

Figure B.205 presents the results of permeability tests performed by Casagrande Consultants on a sample from test trench No. 9. A 1.4 inch diameter, 3 inch high sample was prepared from material passing the No. 10 sieve, and tested in a triaxial cell at different confining pressures. The soil was compacted to approximately 96% of the Standard Proctor maximum dry density and at a water content essentially equal to optimum. Permeabilities in the order of magnitude of 10^{-8} cm/sec. were obtained. Similar results were obtained from triaxial cell permeability tests conducted at OSU soils laboratory on specimens from a mixture of TP-35C and TP-38B compacted at various water contents. The results of these tests are presented in Appendix B-1, page 45. Appendix B-2, Plate 67 presents the results of permeability tests on a consolidometer conducted at BBCM soils laboratory on specimens of the composite sample compacted at various water contents. Again permeabilities in the range of 10^{-8} cm/sec. were measured.

2.4.2 Borrow Area II

Overburden in the valley slope on the west side of the proposed retention pond consists of 30 to 50 feet of mine spoil, a highly variable mixture of inorganic clays of low plasticity with sand, gravel and large size rock pieces of unweathered shale, siltstone, limestone and sandstone.

Atterberg limits, gradation and compaction and specific gravity tests have been performed on disturbed samples obtained from test trenches in this area. The results are presented in Appendix B, Table B.4 and Figures B.206 through B.218

UU, $\overline{\text{CU}}$ and permeability triaxial tests on mine spoil from Borrow Area II (TP-40A and TP-41A) have been performed at OSU. The tests were conducted on 1.5 in. diameter by 3 in. high specimens prepared from the material passing the No. 4 sieve. Test results are presented in Appendix B-1.

Figure B.2194, Appendix B, presents, for Borrow Area 2 soils and the mine spoil used for construction of dam 1, the undrained shear strength at two normal stress levels as a function of the water content at which the samples were compacted. Also shown in this figure are the natural water content, compaction data and Atterberg limits of all the mine spoil samples tested. It can be observed that:

1. Similar to Borrow Area 1 soils, the undrained shear strength of the compacted mine spoil decreases as the compacted water content increases up to about optimum +2 at which point the strength starts to level off.

2. The compaction and plasticity characteristics of the mine spoil samples used in the UU tests fall reasonably close to the average values. The strength tests on these samples can be expected to yield reasonable design strength values.

3. That mine spoil exists in the field at a moisture content ranging from very dry of to slightly wet of optimum. A compaction

specification for mine spoil in terms of water content from optimum - 2% to optimum + 2% seems attainable.

Figure B.219, Appendix B, shows the strength envelopes for mine spoil compacted at various water contents as determined from the UU triaxial tests. Based on these results, the strength envelope for undrained loading of the mine spoil compacted on the dry side of optimum (from opt - 2% to opt + 1%) to 95% of the maximum dry density as determined by the standard Proctor compaction test, can be approximated as follows:

$$Cu = 1.15 \text{ TSF and } \phi u = 15.5^{\circ}$$

Similarly, the strength envelope for undrained loading of the mine spoil compacted wet of optimum (from opt - 1% to opt + 2%) can be approximated as follows:

$$Cu = 1.0 \text{ TSF and } \phi u = 11^{\circ}$$

Figure B.220, Appendix B, presents the strength envelopes as determined from CU tests on mine spoil compacted at various water contents. Based on this figure, the following design parameters were selected for the stability analysis under steady state flow:

1. Mine spoil compacted dry of optimum (-2% to + 1%)

   $$C' = 0.2 \text{ TSF} \quad \phi' = 28^{\circ}$$

2. Mine spoil compacted wet of optimum (-1% to +2%)

$$C' = 0.2 \text{ TSF} \qquad \emptyset' = 26^0$$

The results of the permeability triaxial tests on mine spoil from TP-55, 56 and 57 are presented in Appendix B-1, page 45. These tests were conducted at several confining pressures on 1.4 in. diameter by 3 in. high specimens compacted at various water contents. The laboratory measured permeability of the mine spoil falls in the 10^{-8} cm/sec. range.

Mine spoil similar to that found in Borrow Area II was used to construct the impervious central core and downstream shell of the last raising of the existing dam. The location of the borrow area used for the existing dam and the results of tests conducted on this material are presented in the 1972 report by Acres American included as Appendix F. Based on their test results, Acres American used the following strength parameters to analyze the stability of the existing dam:

i. Undrained shear strength of 2 TSF for end of construction condition.

ii. Effective cohesion and friction angle of 0.2 TSF and 26.5^0, respectively, for the steady state seepage condition.

2.4.3 Bottom Ash

A sampling and testing program spanning several months has been conducted to investigate the bottom ash produced by Cardinal Plant units 1, 2 and 3 and to determine its suitability as drain material.

Drawing No. 13-3070 shows a location plan of the Cardinal Plant bottom ash pond. Bottom ash was sampled at the discharge pipes and from stockpiles of material recovered for sale purposes. Unit 3 ash was sampled separately from Units 1 and 2 material. Results of specific gravity and gradation tests on units 1 and 2 ash are presented in Figures B.221 through B.261. Similar data and curves for Unit 3 bottom ash are shown in Figures B.262 through B.299. The following observations have been made:

1. Unit 1 and 2 bottom ash is well graded and the variation in particle size distribution and specific gravity is relatively small.

2. Unit 3 material ranges from poorly to well graded with a large variation in particle size distribution and specific gravity. Unit 3 ash can be as coarse as units 1 and 2 material but generally tends to have a finer gradation. Also, unit 3 ash has a lower average specific gravity than units 1 and 2 ash.

3. The $^{D}15$ size of the samples of units 1, 2 and 3 bottom ash with the coarser particle size distribution is about 0.5 mm, which is suitable as a filter for most ordinary, non dispersive clays.

4. Dry and wet sieving of the bottom ash gave essentially identical test results. Therefore, dry sieving can reliably be used in

the field for quality control during construction.

The results of constant head permeability tests performed at varied relative densities on units 1, 2 and 3 bottom ash are shown in Figure B.300 and Table B. 7. These tests and some of the specific gravity and gradation tests reported were conducted at Purdue University under the supervision of Dr. G. A. Leonards. The corresponding gradation curves of the bottom ash used in the permeability tests are presented in Figure B.301. As can be seen from this figure the units 1 and 2 ash with the coarsest particle size distribution of all the tested samples and unit 3 ash with about 9% of fines were used. Therefore, a wide range of bottom ash particle size distribution was covered. These permeability test results indicate the following:

1. The coefficients of permeability are about as expected for relatively clean granular materials. The Hazen-Willam formula:

   $$K, \text{cm/sec} = 100\ (D_{10}, \text{cm})^2$$

   gives a reasonable approximation of the permeability of the bottom ash in a loose state.

2. The permeability appears to vary linearly with relative density. Thus, if the in situ particle size distribution and relative density of the bottom ash are known, the combined use of the Hazen-Willam equation and the data shown in Figure B.300 permits

reasonable estimation of the field permeability.

3. It will be impractical to compact bottom ash in the field to relative densities much higher than 80%. Accordingly, the expected in situ permeability will range from 1 to 5 x 10^{-2} cm/sec, which is ideal for drain applications.

In order to determine the effect of fines on permeability, constant head permeability tests were conducted on bottom ash varying the percent of fines passing the No. 200 sieve. The results are presented on Figure B.302. This figure indicates that bottom ash compacted to relative density of about 70 to 80% will have a permeability larger than 10^{-2}cm/sec as long as the material passing the No. 200 sieve does not exceed 10 to 12%.

In order to investigate whether or not the bottom ash particles would break down during compaction and handling resulting in detrimental changes in gradation or permeability, tests were conducted in the before and after compaction condition. The test results are shown in Figures B.303 through B.307 and Table B. 8. The gradation characteristics and permeability of units 1, 2 and 3 bottom ash did not change significantly as a result of conducting relative density and Harvard miniature compaction tests. Thus, bottom ash can be expected to withstand compaction by light vibratory rollers during construction without significant particle breakdown or reduction in permeability.

Unconsolidated undrained triaxial tests were performed on samples of bottom ash that were prepared in a 2.8 inch diameter mold approximately 6 to 6.5 inches in height. These tests were not conducted in a saturated condition, because the gradation and permeability of this ash make it unlikely that excess pore pressures will develop in the field. Based on the test results (as shown in Figures B.308 and B.309), a shear strength envelope given by a straight line relationship with a friction angle of 38°. is reasonable and conservative.

3.0 HYDROLOGY

3.1 Introduction

where?

The existing hydrologic conditions at the proposed dam site are depicted in Figure D.1. Blockhouse Run, the major drainage feature in the project area, drains directly into the Ohio River. Approximately one mile upstream of the Ohio River, Blockhouse Run splits into two branches, designated as the East Branch and the West Branch.

The East Branch drains the eastern watershed as delineated in Figure D.1. Presently, the eastern watershed is relatively undisturbed (i.e. no man-made impoundments or developments) and the stream flow is not controlled.

The West Branch has been dammed to form the existing fly ash retention pond. Runoff from the western watershed is presently regulated by the existing principal spillway which discharges into the natural drainage channel just upstream of the confluence of the two branches.

The location of the proposed dam is shown on Drawing No. 13-3001. The dam will inundate approximately 80 acres or one-tenth of the area in the eastern watershed. Since the location of the proposed dam is situated downstream of the discharge points of the existing dam, runoff from the western watershed will also drain into the proposed reservoir. Therefore, the spillway system of the proposed dam has been designed to meet the ODNR class I design criteria based on the runoff from both watersheds .

The emergency spillway of the existing dam is presently located on the right abutment (looking downstream) as shown in Drawing No. 13-3001. This layout poses a threat to the safety of the proposed dam. In the event of a major flood, runoff from the western watershed would be routed through the emergency spillway of the existing dam and would be discharged towards the upstream shell of the proposed dam and could cause detrimental erosion. In order to ensure the safety of the proposed dam, the present emergency spillway of the existing dam will be permanently plugged and a new spillway constructed on the other abutment as shown in Drawing No. 13-3001. This new configuration will not affect the hydrologic characteristics of either watershed. Further details on the operational impacts of this change are discussed in section 3.7.

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

3.2 Basin Characteristics

Figure D.1 shows the limits of the watershed boundary for the proposed fly ash retention pond. The total drainage area above the proposed dam has been divided into two watersheds, East and West, for analysis of the storm runoff entering the new pond. A review of available topographic maps and aerial photos was made to determine essential basin characteristics for each watershed. Such characteristics include the drainage boundaries, areas, slopes, soil types, ground cover, land use and the 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.

The existing fly ash dam is located in the western watershed. Present land use within the drainage area is limited to reclaimed strip mine areas, some woodlands, and the fly ash reservoir. Reclamation of the reservoir area is the only projected change in land use in the West watershed.

Woodlands and scattered reclaimed strip mines constitute the existing land use in the East watershed. Construction of the proposed fly ash dam will inundate approximately 80 acres at an elevation of 913.0 feet NGVD, the maximum pool elevation.

Soil types in the areas have been identified by the Soil Conservation Service (SCS) of the U.S. Department of Agriculture and classified into hydrologic soil groups. Within the study area, all soils fall under the hydrologic soil group B. Table 3.1 lists the basin characteristics for the Western and Eastern watersheds.

3.3 Characteristics of Proposed Retention Pond

As previously referenced, Drawing No. 13-3001 shows the location of the proposed dam. Based on this layout, the pond will have the following surface areas and storage capacities.

ELEVATION,	(Ft. NGVD)	AREA (AC)	STORAGE (AC-FT)
Maximum Pool	913.0	80	4780
Top of Dam	925.0	95	5800

Figure D.2 presents the complete area-capacity-elevation curve developed for this study.

3.4 Design Requirements and Assumptions

No rainfall - runoff data were available for the site because the streams flow intermittently. Therefore, runoff hydrographs were generated using the U.S. Army Corps of Engineers (1981) HEC-1 computer program. The SCS dimensionless unit hydrograph method was employed in the calculation of the hydrographs. For each watershed, separate runoff hydrographs were computed and then later combined to form a single inflow hydrograph for the proposed reservoir.

Runoff from the West watershed has been examined from two perspectives. First, the area was analyzed for the present conditions of runoff draining into the existing pond and secondly, for projected conditions after the existing fly ash pond is reclaimed. The results of the analyses indicated that the storage capacity of the existing pond attenuates and reduces the inflow flood as it is routed through the reservoir. The discharges from the West watershed, as it exists, therefore, will be lower than those values estimated for the projected conditions since it is assumed that the reclaimed watershed will have an insignificant amount of valley storage. Accordingly, for the design of the emergency and service spillways of the proposed dam, projected land uses in both watersheds will be the controlling conditions.

3.4.1 Service Spillway

According to ODNR regulation 1501:21-13-04, design of the (principal) service spillway for class I dams must be such that the average frequency use of the emergency spillway is predicted to be less than once in fifty years. Since no stream-flow records are available to establish a flood-frequency curve, it was assumed that a 50-year rainfall event would produce the 50-year flood. The estimated precipitation, 3.5 inches, was obtained from the National Weather Bureau (1960) report TP-40. A 6 hour storm duration and normal soil moisture conditions were assumed for developing the inflow hydrograph.

3.4.2 Emergency Spillway

The ODNR regulation 1501:21-13-02 specifies that for class I dams, the spillway system shall safely pass the design flood equal to the probable maximum flood (PMF) without any overtopping of the dam. The PMF is the result of the probable maximum precipitation (PMP), 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. Generalized estimates of the PMP have been published by the Hydrometeorological Branch of the National Weather Service (1978). For the study area, a 6 hour PMP of 26.5 inches was used as the design rainfall event. The antecedent moisture conditions of the soil cover were assumed to be normal.

3.5 Analysis

All flood routings were conducted using the HEC-1 computer program. The program routes floods through the reservoir by the modified Puls method. In general, reservoir storage data and either spillway dimensions or discharge-rating curves are supplied by the user.

3.5.1 Service Spillway

Analysis of the service spillway system consisted of routing the 50-year flood to establish the maximum operating level. A preliminary design for the service spillway was determined and a stage - discharge curve was computed. Reservoir routings were conducted assuming various initial water levels in the proposed fly ash pond. The results were analyzed with respect to limiting the rise of the pool level to elevation 915.0 feet NGVD, the proposed crest elevation of the emergency spillway.

3.5.2 Emergency Spillway

Hydrologic reservoir routings 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 elevation in the proposed reservoir was assumed to be at the maximum operating level at the beginning of the design storm. Discharges from theservice spillway were neglected during the reservoir flood routing. Initially, a rectangular section was used and the program internally computed the discharge given a weir length. Having obtained a rough estimate for the opening, a trapezoidal section was designed and rated based on calculations of critical depth. The known stage - discharge curve was then used in the routing process to determine the maximum discharge and pool elevation. To this point, the analysis neglected any approach channels that would be required.

The topographic conditions at the proposed dam location dictate that an approach channel of significant length be constructed. Therefore, an initial design of the approach channel was made and a backwater analysis was conducted to examine the head losses between the control section and the pond. This work was completed using the U.S. Army Corps of Engineers (1976) HEC-2 computer program, Water Surface Profiles. Results of the backwater curves were used to revise the stage - discharge curve for the flood routing.

A trial-and-error procedure followed whereby the approach channel geometry or the control section was altered, a new backwater curve developed, and a revised reservoir routing completed. Final configurations and dimensions were established after successful routings were obtained.

Discharges from the emergency spillway are routed away from the dam through an outlet channel. Design of the channel size was based on the peak discharge from the reservoir. A water surface profile along the length of the channel was developed to examine the depth of flow and average velocity.

3.6 Results

3.6.1 Service Spillway

The proposed service spillway is a concrete shaft structure with side openings for effluent discharge connecting into a 42 inch diameter conduit buried underneath the earthen dam (see Drawing Nos. 13-3024, 13-3032, 13-3033 & 13-3060). The conduit is a prestressed concrete cylinder pipe (PCCP) designed for 200 feet of internal hydraulic pressure and 200 feet of overburden pressure. During most operating conditions, discharge through the service spillway will be controlled by the weir flow over the side openings in the shaft.

At the outlet of the spillway conduit an energy dissipator will be provided. The dissipator, an impact-type structure, was designed for the probable maximum discharge that would occur during a PMF, estimated to be 330 cubic feet per second. Dimensions of the dissipator, Drawing No. 13-3065, are based on the design criteria of the U.S. Bureau of Reclamation's Engineering Monograph No. 25, "Hydraulic Design of Stilling Basins and Energy Dissipators".

Results of the reservoir routings establish a maximum operating level of 913.0 feet, with the 50-year flood reaching a level of 914.5 feet, 0.5 feet below the crest of the emergency spillway.

Figure D4 presents the inflow and outflow hydrographs for the 50-year flood. Appendix D contains the computer listing of the reservoir routings.

3.6.2 Emergency Spillway

The development of the PMF hydrograph indicates a peak inflow to the proposed reservoir equal to 16,803 cubic feet per second. This value represents the combined hydrographs from the West and East watersheds. Values of the runoff from each watershed and the combined runoff are shown in Appendix D. The PMF inflow hydrograph is plotted in Figure D5.

The layout of the approach channel, control section, and outlet channel for the emergency spillway is shown in plan on Drawing Nos. 13-3024 & 3026. The spillway will be an unlined open channel excavated in rock of trapezoidal shape having an approach channel approximately 375 feet in total length. Entrance to the approach channel will approximate a trapezoid with a bottom width of 260 feet and invert elevation of 913.0 feet NGVD. The approach channel width is reduced as it follows an uphill slope to a section that will align with the upstream edge of the dam crest. From this point to the control section, a distance of a 190 feet, the channel will have a constant width of 110 feet and a horizontal grade of 915.0 feet NGVD with side slopes of 1.5 H:1V.

Based on the reservoir routing, the calculated peak discharge from the dam is 8789 cubic feet per second at a maximum pool elevation of 924.79 feet NGVD. Velocity through the control section is expected to be less than 14 feet per second at peak outflow.

3.7 Spillway System of Existing Fly Ash Dam

The present emergency spillway of the existing fly ash pond, located on the right abutment, will be permanently plugged sometime in the summer of 1984. The replacement spillway will be completed in late spring of 1985. During these construction activities the existing fly ash pond will operate without an emergency spillway. In the event of a major storm, runoff from the West watershed will be stored in the existing pond and released only through the service spillway.

The following table lists the amount of available storage to the top of the existing dam (elev. 1,000.0 feet NGVD) as a function of an arbitrary initial pool level. At present, the existing pond level is at approximately elevation 980.0 feet NGVD. The level of the existing pond is expected to rise one foot to 981.0 foot NGVD by the summer of 1984 when plugging of the emergency spillway is scheduled to take place, and to approximately 983.0 feet NGVD by the time the replacement emergency spillway is completed in 1985.

INITIAL POOL LEVEL (Feet, NGVD)	AVAILABLE STORAGE (Acre-Feet)
981.0	1531
982.0	1451
983.0	1370

Based on the present land use, which will not be changed during construction, the total runoff volume from the West watershed generated during the probable maximum flood equals 1250 acre-feet. As can be seen from the table above, the available storage during this construction period exceeds the PMF volume. Assuming an inital pool level of 983.0 feet NGVD, the PMF would result in a maximum pool elevation of 997.2 feet NGVD, leaving adequate freeboard.

Reservoir routings of the PMF were conducted to determine the time required to drain the flood waters through the service spillway to an acceptable pool level. Drawdown of the reservoir from elevation 997.2 to 984.0 feet NGVD could be achieved in a period of just over four days.

Summarizing, the existing fly ash pond will operate without an emergency spillway for a construction period of no more than a year. During this condition, the PMF can be stored in the existing reservoir without overtopping the dam. Also, the existing service spillway has the capacity to remove the flood waters from the reservoir within the allowable ten days as set forth in section 1501: 21-13-04 of the ODNR regulations.

3.8 Summary and Conclusions

The hydrologic/hydraulic studies for the proposed fly ash retention pond included estimating the PMF and 50-year flood hydrographs and designing the emergency and service spillways. The U.S. Army Corps of Engineers computer programs HEC-1 and HEC-2 were used in the analyses. Table 3.2 gives a complete summary of the study.

The proposed spillway system has enough capacity to pass the probable maximum flood without overtopping the dam. The water discharged through the emergency spillway is directed away from the dam such that it causes no threat to the stability of the structure.

TABLE 3.1 - BASIN CHARACTERISTICS

BASIN CHARACTERISTICS	WATERSHED EAST	WEST
Drainage area (acres)	730	677
Average land slope (%)	25	30
Hydrologic soil group	B	B
SCS curve number		
present	-	72
projected	70	70
Time of concentration (hours)		
present	-	0.4
projected	0.57	0.87

TABLE 3.2 HYDROLOGIC/HYDRAULIC SUMMARY FOR PROPOSED RETENTION POND

Drainage Area	2.2 Sq. Mi.
Design Floods (Inflow)	
PMF Peak	16803 cfs.
50-Yr Peak	766 cfs.
Peak Discharge	
PMF	8789 cfs.
50-Yr	40 cfs.
Maximum Pool Elevations, NGVD	
PMF	924.79 ft.
50-Yr	914.50 ft.
Emergency Spillway - Excavated Open Channel	
Crest Elevation, NGVD	915.0 ft.
Bottom Width	110.0 ft.
Side Slopes	1.5 H:1V
Service Spillway - Size	
Concrete Shaft and Conduit	
Shaft Opening 2 @ 3.5' x 6.0'	
Conduit - 42 inch prestressed concrete cylinder pipe	
Maximum Pool Level, NGVD	913.0 ft.

4.0 GEOTECHNICAL DESIGN

In the following sections, the geotechnical aspects of the key components of the project are discussed. Complete details on the proposed design are presented in the accompanying specification documents (Appendix G) and set of design drawings.

4.1 Dam Embankment

The proposed dam has been designed as an earth embankment with five (5) different zones as shown on Drawing No. 13-3027. Zone I will serve as an upstream impervious core to reduce seepage and dissipate total heads through the dam. Material for this zone will come from Borrow Area I. The inorganic clays of low to medium plasticity found in this area are excellent materials for the core of a dam since they are relatively impervious, resistant to piping and have acceptable shear strength. Zone I soils will be compacted with a sheepsfoot roller in lifts of 6 in. maximum loose thickness, at a water content ranging from optimum - 1% to optimum + 2%, to 100% of the maximum dry density as determined by the standard Proctor compaction test (ASTM D 698-78, Method A).

Mine spoil from Borrow Area II will be used in the transition zone (Zone II) between the clay core and the chimney drain. This zone will serve two purposes. First, it will complement the clay core in reducing seepage and dissipating total head. Secondly, it will work as a filter to prevent piping of the clay core. This material will be free of boulders or rock fragments larger than 6 inches. Zone II will be compacted with a sheepsfoot vibratory roller in lifts of 9 in. maximum loose thickness, at a water content ranging from optimum

- 1% to optimum + 2%, to 100% of the maximum dry density as determined by the standard Proctor compaction test (ASTM D 698-78, Method C).

Drawing No. 13-3025 shows, in plan, the under drain system for the dam. Bottom ash from Cardinal Plant units 1, 2 and 3 will be used in the chimney drain and lateral drainage blankets along the valley slopes in the downstream shell of the dam (Zone III A). Bottom ash from units 1 and 2 will be used in the outlet drainage blanket in the foundation of the downstream shell (Zone III B). A sand and gravel toe drain (Zone III C) will be placed at the end of the outlet drainage blanket. Bottom ash is expected to perform satisfactorily as a drain material as laboratory tests performed indicated that it has excellent mechanical properties such as gradation, permeability and shear strength. Bottom ash and the sand and gravel toe drain will be compacted in lifts of 9 in. maximum loose thickness in a saturated condition and with a vibratory roller to a relative density of at least 70% as determined by ASTM D 2049-69. The gradation requirements for the drainage zones and for Zones I and II soils are shown in Figure 4.1.

Mine spoil from Borrow Area II and material from the emergency spillway and abutments excavation will be used in the cofferdam and upstream and downstream shells of the proposed dam (Zone IV). Material for this zone will be free of boulders or rock fragments larger than 12 in. in size. Zone IV will be compacted with a sheepsfoot vibratory roller in lifts of 12 in. maximum loose thickness, to 100% of the maximum dry density, at a water content ranging from opt. - 2% to opt + 1%, as determined from the standard Proctor compaction test(ASTM D 698-78, Method C). Oversize rock will be used as rip rap (Zone V). Additional rip rap, if necessary, will come from the proposed quarries. Rip rap will be compacted by at least 4 passes with a D8 bulldozer or equivalent equipment.

4.2 Excavation

The location of the proposed dam was initially selected based on storage volume considerations. Subsequent investigation of the site revealed certain potential deficiencies which have been minimized by tailoring the design to meet the site conditions. On the east side, the dam location abuts against an existing knoll, which could increase potential seepage. In addition, the valley slopes on both abutments have an adverse orientation which would promote instability and make it difficult to achieve a tight connection between the clay core and abutments.

In order to correct these deficiencies, a cut to rock will be made at the proposed abutment as shown in Drawing No. 13-3023. The orientation of the trimmed faces has been designed so that the upstream core of the dam will intersect the abutments at right angles. This symmetrical configuration will result in balanced seating of the clay core against the rock which will reduce interface seepage and minimize the potential for cracking of the core. Furthermore, arching the dam in the upstream direction, as shown in Drawing No. 13-3026, will increase stability and will promote closure of any transverse cracks that may tend to develop.

The upstream core will extend to competent rock in the foundation as shown in Drawing No. 13-3023 to cut off potential seepage beneath the dam. The proposed excavation is based on the findings of the test pits, borings and the results of a geophysical survey. A copy of the report, "Geophysical Survey, Cardinal Flyash Dam II", is included as Appendix E. The actual depth of the cut-off trench and extent of the overburden removal for the foundation and abutments will be adjusted during construction to suit field conditions. The exposed rock surface in the excavation for the cut off trench will be slushed grouted and repaired with dental concrete.

4.3 Water Diversion

The cofferdam and the excavation for the cut off trench and service spillway are shown in plan in Drawing No. 13-3032 and in profile in Drawings No. 13-3028 and 13-3033. Because Blockhouse Run presently flows through the proposed dam site, excavation of the cofferdam foundation and the cut off trench and placement of fill in these areas will require a water diversion scheme. The proposed plan calls for temporarily relocating Blockhouse Run excavating and backfilling in sections and installing a temporary water diversion pipe (36" Ø CMP). The construction sequence is as follows:

1. Construct a temporary diversion channel as shown in the drawings and divert Blockhouse Run through it.

2. Starting 5 ft. from the east edge of the flowing creek and making a 1.5 (H) to 1 (V) cut, excavate the east portion of the cofferdam foundation and cut off trench down to the required grade (phase I excavation).

3. Partially backfill the excavation by placing the appropriate soils on a 1.5 (H) to 1 (V) slope to approximately elevation 755 NGVD (phase I backfill).

4. Install the diversion pipe, construct the end ditches and divert the creek to flow through the diversion pipe.

5. Excavate and backfill the west side of the cofferdam foundation and cut off trench (phase II excavation and backfill).

6. Construct the cofferdam and install a temporary 42" CMP at approximately elevation 780 ft. NGVD to connect to the service spillway.

The excavation described above will require dewatering of the site. The Contractor will be responsible for submitting a dewatering scheme for approval of the Owner's Engineer. Also, the Contractor will be permitted to submit an alternate water diversion scheme for the Owner's Engineer's consideration.

Once the cofferdam and service spillway are completed and the 42"Ø CMP is installed, the section of the diversion pipe which extend beyond the downstream toe of the cofferdam will be removed. The portion of the diversion pipe buried under the cofferdam will then be plugged with lean concrete. At this time, water will start to impound behind the cofferdam and the 42" Ø CMP will serve as the spillway. The cofferdam's operating pool will be at elevation 780.5 ft. NGVD.

4.4 Grout Curtain

A grout curtain along the longitudinal section of the dam will be provided to reduce potential seepage through the dam foundation and abutments. The primary and secondary lines of grout holes are shown in plan in Drawing No. 13-3029 and in profile in Drawing Nos.

13-3027 and 13-3028. The grout mixture will consist of a Portland cement, bentonite and water, with admixtures added as may be required. The mixture will be designed to suit the particular conditions encountered in the work. In general, grouting will be done in 10 to 20 ft. sections from the bottom of the hole up, at pressures not to exceed the overburden pressure. Split spacing will be carried out whenever the refusal criterion cannot be met. Additional lines of grout holes will be installed if required.

4.5 Pressure Relief Drains

A line of pressure relief drains will be installed in the abutments in areas were the overburden soils will not be removed and the lateral drainage blankets will not be directly in contact with exposed rock. The drains will consist of 6 in. diameter boreholes drilled to rock and backfilled with pea gravel as tentatively shown in plan in Drawing No. 13-3029 and in section in Drawing No. 13-3028. This system of drains is intended to intersect and drain seepage through the grout curtain and reduce the uplift pressures in the abutments. The exact location, number and spacing of pressure relief drains will be adjusted in the field to suit actual field conditions.

4.6 Geotechnical Instrumentation

The instrumentation program for the proposed dam will consist of piezometers, settlement monuments and a weir. The proposed location of the instruments is shown in plan in Drawing No. 13-3024 and in section in Drawing No. 13-3027 and 13-3028.

Pneumatic piezometers will be installed before construction and as the embankment is being placed, in the foundation, abutments and every zone of the dam. The piezometers will be read once a week during construction. This information will be used to monitor the build up of pore pressure during construction and to evaluate the embankment stability in terms of effective stresses.

Settlement monuments will be installed every 50 ft. along the downstream edge of the dam crest.

A 90° V notch weir will be installed 200 ft. downstream of the downstream toe of the dam. Seepage from the underdrain system and from seeps in the abutments, which are likely to develop, will be directed towards the weir.

An operation manual for the proposed dam, which will explain the use of these instruments and present the recommended schedule for reading the instruments during normal operation, will be prepared at a later date.

5.0 GEOTECHNICAL ANALYSES

Seepage, stability and settlement analyses have been conducted to evaluate geotechnical aspects of the proposed design. The under drain system was found to have ample discharge capacity to handle the anticipated flow through the dam embankment and foundation. The minimum safety factors, for stability at end of construction, long term condition and seismic loading were determined to be acceptable. A camber has been provided to accommodate the anticipated long term settlement. In the following sections, the assumptions made, methods used, and results of the geotechnical analyses are discussed in detail.

5.1 Seepage

Figure 5.1 presents the maximum section through the dam and shows the range in permeability that is anticipated for each of the embankment zones and top 75 feet of bedrock. The permeability values shown are based on the results of the packer tests on the foundation bedrock and the laboratory permeability tests on the embankment materials as discussed in Section 2 of this report.

Flow nets for two idealized conditions of flow through the embankment and foundation were constructed. For the first flow net, a perfect seepage cut off through the foundation bedrock was assumed. Also, it was assumed that no dissipation of total head would occur through the embankment other than at Zones I and II and that the permeability of both these zones will be homogeneous, isotropic and equal to 10^{-7} cm/sec. Figure 5.2 presents the resulting flow net. It is believed to approximate the most critical conditions of flow through the embankment that can be anticipated. The maximum expected flow and hydraulic gradient through the embankment equal 3.3×10^{-2} cf/day per linear ft. of dam and 1.25, respectively. The discharge capacity of the chimney drain (Zone III a) was checked and found to be adequate. The top phreatic surface through the embankment for use in the stability analysis under steady state flow was determined.

For the second flow net, the effect of the grout curtain was neglected and the permeability of the top 75 ft. of bedrock was assumed homogeneous, isotropic and equal to 5×10^{-5} cm/sec. Zones I and II and bedrock below the top 75 ft. were considered impervious. Figure

5.3 presents the flow net for seepage through the foundation. From this figure, a flow of 4.1 cf/day per linear ft. of dam, a maximum gradient across the core of 0.5 and the foundation uplift pressure shown were determined. Also, the outlet drainage blanket (Zone III b) was found to have sufficient discharge capacity to handle the anticipated flow through the embankment and foundation, and still provide an ample factor of safety of 10.

5.2 Stability

Stability analyses for end of construction, steady state seepage and seismic loading were performed on the maximum section of the dam using a computer program developed at Purdue University called STABL. The embankment stability was checked using circular slip surfaces and the Modified Bishop Method of Slices. Wedge type foundation failures were investigated using a simplified version of Janbu's Method of Slices. A pseudo static analysis with a horizontal seismic coefficient of 0.1 was conducted to simulate seismic loading. The stability of both the upstream and downstream shells of the dam were investigated.

Figures 5.4 and 5.5 summarize the results of the stability analyses for end of construction and steady state seepage respectively. The soil parameters and total heads used in the analyses are also depicted. The minimum factor of safety found for gravity loads was 1.4 and correspond to an upstream foundation slide at the end of construction. For the long term condition, the minimum factor of safety found was 1.5 and corresponds to a downstream foundation slide. For seismic loading, deep slip surfaces in the downstream shell were found to have a factor of safety of about 1.2.

Stability under rapid drawdown conditions was not investigated because this type of loading is not anticipated to occur.

The computer printouts for the stability analysis are presented in Appendix H.

5.3 Settlement

No long-term foundation settlement is expected to occur except in the areas of the abutment slopes where the overburden soil will not be removed. The total estimated settlement in these areas is less than 1 ft. (see Appendix H for Settlement Computations). Most of this settlement, as well as settlement of the embankment, is expected to occur during construction. Typically, the post construction settlement of dams of this type and size is about 1% of the dam height. In order to accommodate the post construction settlement the crest of the dam will be raised to elevation 926 ft. at the abutments and gradually to elevation 927 ft. at the mid point between abutments.

6.0 COST ESTIMATE AND CONSTRUCTION SCHEDULE

The cost of constructing the dam and appurtenances is estimated at $12,785,000. Table 6.1 lists the estimated cost of labor and materials for each construction item.

The construction schedule for the project is shown in Figure 6.1. Construction of the dam and appurtenances is expected to take three construction seasons with mobilization scheduled for May of 1984 and completion in October of 1987.

TABLE 6.1

CARDINAL PLANT - FLY ASH DAM II
COST ESTIMATE

Item	Cost Estimate	
	Material	Labor
I. Dam		
a. Clearing and Grubbing	-	$ 300,000
b. Excavation	-	$ 970,000
c. Foundation Preparation	$ 29,000	$ 42,000
d. Drilling and Grouting	$ 25,000	$ 407,000
e. Relief Wells	$ 3,000	$ 13,000
f. Stream Diversion	-	$ 46,000
g. Fill		
Zone I - Core	-	$1,202,000
Zone II- Transition	-	$1,088,000
Zone III- Drain	$297,000	$ 545,000
Zone IV - Shell	-	$2,928,000
h. Cofferdam	$ 50,000	$ 39,000
i. Rip-Rap	-	$ 400,000
j. Instrumentation	$ 30,000	$ 30,000
k. Collector Drains	$ 26,000	$ 26,000
Dam Sub Total	$460,000	$8,036,000
II. Service Spillway		
a. Riser	$135,000	$ 268,000
b. Conduit	$645,000	$ 424,000
c. Surge Shaft & Energy Dissipator	$ 71,000	$ 170,000
d. Outlet Ditch	-	$ 10,000
Service Spillway Sub Total	$851,000	$ 872,000
III. Emergency Spillway	-	$1,900,000
IV. Overflow Skimmer	$140,000	$ 15,000
V. Clean Up/Seeding	-	$ 60,000
VI. Mobilization	-	$ 74,000
VII. Access Roads		
a. Crest of Dam	$ 12,000	$ 2,000
b. Energy Dissipator	$ 8,000	$ 6,000
Access Roads Sub Total	$ 20,000	$ 8,000

TABLE 6.1 (Continued)

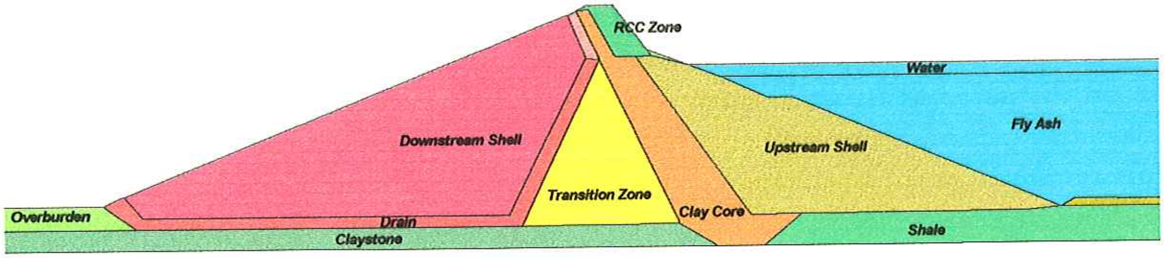
CARDINAL PLANT - FLY ASH DAM II
COST ESTIMATE

	Item	Cost Estimate Material	 Labor
VIII.	Borrow Area Reclamation	$ 41,000	$ 250,000
IX.	Sediment Control	-	$ 30,000
X.	Reservoir Clearing	-	$ 15,000
XI.	Access Stairs	$ 8,000	$ 5,000
	Sub Total Labor and Material	$1,520,000	$11,265,000
	Total Cost	$ 12,785,000	

CARDINAL PLANT

FLY ASH RETENTION POND II

FINAL DESIGN REPORT FOR PROPOSED EARTH FILL-ROLLER COMPACTED CONCRETE RAISING OF DAM



FOR: CARDINAL OPERATING COMPANY
BY: AMERICAN ELECTRIC POWER SERVICE CORPORATION

FINAL DESIGN REPORT

PROPOSED EARTH FILL-ROLLER COMPACTED CONCRETE RAISING OF DAM

FOR

FLY ASH RETENTION POND II

CARDINAL PLANT
BRILLIANT, OH

PREPARED FOR

CARDINAL OPERATING COMPANY

PREPARED BY
AMERICAN ELECTRIC POWER SERVICE CORPORATION
CIVIL AND MINING ENGINEERING DIVISION
COLUMBUS, OH

March 1997

DRAWING INDEX

DAM RAISING DRAWINGS:

DRAWING NUMBER	TITLE
13-30038-	Cover Sheet
13-30039-	Existing Dams and Reservoirs - General Arrangement
13-30040-	Grading and Drainage Plan
13-30041-	Profile and Sections
13-30042-	Sections and Details Sheet 1
13-30043-	Sections and Details Sheet 2
13-30044-	Fly Ash Drainage Shaft Plan, Sections and Details
13-30045-	Drainage Shaft Access Stairs Plan, Sections and Details
13-30046-	Site construction Plot Plan
13-32000-	Drainage Shaft Masonry and Reinforcing Plant, Sections and Details Sheet 1
13-32001-	Access Stairs and Skimmer Plan, Sections and Details.
13-32002-	Training Walls Plan Sections & Details
13-32203-	Access Stairs & Skimmer Plan, Sections & Details
13-32004-	Drainage Shaft Bulkhead, Plan, Sections & Details

REFERENCE DRAWINGS:

13-3002-1-	Boring and Test Pit Location Plan Sheet 1
13-3003-	Boring and Test Pit Location Plan Sheet 2
13-3004-	Geologic Profile N-S Axis
13-3005-1-	Geologic Profile E-W Axis
13-3032-1-	Existing Service Spillway & Cofferdam-Layout& Grading Plan

APPENDICES

Item No.	Description
A	Boring Logs Drilled in 1996
B	Laboratory Testing Data - 1996
C	Post Construction Field Instrumentation Data
D	Hydrology Computations
E	Seepage and Stability Analyses
F	Bid Documents (Under Separate Cover)

1501: 21-5-04. FINAL DESIGN REPORT

1.0 INTRODUCTION

Cardinal Operating Company, agent for Ohio Power Company and Buckeye Power Company Incorporated, proposes to increase storage capacity of the Fly Ash Retention Pond II (FAR II) at the Cardinal Plant by raising the existing dam crest from 925 feet NGVD (National Geodetic Vertical Datum) to 970 feet NGVD. Roller Compacted Concrete (RCC) and earth fill will be utilized in the dam crest raising. The storage capacity provided by the existing earth fill dam crest at 925 feet NGVD is projected to be exhausted in the year 1999. Increasing the dam crest to 970 feet, NGVD will provide capacity that is projected to be exhausted in 2014 (15 years). The existing dam was approved by the Ohio Department of Natural Resources, Division of Water on April 30, 1985, (Permit No. 85-147). This report presents the final design for the dam raising and appurtenances in compliance with the Final Design Report requirements rule 1501:21-5-04. of the Ohio Laws and Administrative Rules of the Ohio Department of Natural Resources, Division of Water. Also provided, where appropriate, is information related to the existing, permitted dam.

1.1 Location and General Description of the Dam

The proposed dam raising site is located in Section 5 of Wells Township, Jefferson County, near the town of Brilliant in eastern Ohio, as shown on the Cover Sheet, Location Plan Existing Dams and Reservoirs Drawing No. 13-30038. Drawing No. 13-30039 illustrates the General Arrangement.

The existing earth fill dam consists of a 180-feet high arched earth embankment with a zoned cross section. At 925 feet NGVD, the dam has a 70-foot wide by 1,055 feet long crest. Drawing No. 13-30040, Fly Ash Dam II Raising, Grading and Drainage Plan shows the proposed and existing conditions. It is proposed to raise the dam crest from 925 feet NGVD to 970 feet NGVD utilizing Roller Compacted Concrete (RCC) and earth fill. The RCC design will consist of a 45-foot high RCC structure with an earth fill downstream shell. The proposed RCC crest will measure 30 feet wide by 1,400 feet long. Drawing Nos. 13-30040 and 13-30041 show the proposed Earth Fill-RCC dam in plan view and cross sections.

Drawing No. 13-30040 depicts the location of the proposed Earth Fill-RCC extension. In addition, Drawing No. 13-30039 depicts the outline of the reservoir, the locations of state, county and township roads; utilities; topography and other pertinent information.

1.2 Classification

The Ohio River, Cardinal Generating Plant, State Route 7 and the Tidd-dale subdivision of Brilliant, OH all lie directly downstream of the proposed dam. Therefore, a sudden failure of the existing dam at 925 feet NGVD or the proposed Earth Fill-RCC dam at 970 feet NGVD will likely result in loss of human life, and 1 damage to homes, high value utility installations and both a railroad and a public road. Also, the existing and proposed dam heights and storage volumes exceed the thresholds for class I dams as established in Section 1502:21-13-01 of the ODNR Administrative Rules. For these reasons, the proposed Earth Fill-RCC dam is classified as a class I dam.

1.3 Purpose of Dam

The purpose of the proposed Earth Fill-RCC extension from elevation 925 feet to 970 feet NGVD is to provide for the continued disposal of fly ash coal combustion by-product produced by the Cardinal Generating Plant. Cardinal has three units rated at 600, 600 and 630 megawatts (MW) respectively which produce a total of 560,000 cubic yards of fly ash per year. The proposed Earth Fill-RCC extension will provide for fly ash disposal into the year 2014.

1.4 Impact of Dam to Human Life, Health and Property

The Ohio River, Cardinal Generating Plant, State Route 7 and the Tidd-dale subdivision of Brilliant, Ohio all lie directly downstream of the proposed dam. Therefore, a sudden failure of the existing dam at 925 feet NGVD or the proposed Earth Fill-RCC dam at 970 feet NGVD will likely result in loss of human life, and damage to homes, high value utility installations and both a railroad and a public road.

2.0 SITE INVESTIGATION

2.1 Regional Geology

The regional geology of Jefferson County is discussed in the design report for Stage I construction of the Cardinal Dam II (1). Proposed Stage II construction involves raising of the existing dam to elevation 970 feet. The geological investigation and evaluation conducted for the purposes of constructing the existing dam are still valid. These data will be used in the design and construction of the Stage II of the Cardinal Dam II.

2.2 Site Geology

Geologic cross sections for the stage I dam were constructed from the borings and test trenches at the time the original dam was designed. These sections are shown in Figure 2.1, Reference 1 and in Drawing Nos. 13-3004 and 13-3005 which are included as reference drawings to this report. This information will be utilized for designing of the raised dam.

2.3 Foundation

The results of the site investigation conducted during the design and construction phases for the original dam will be utilized in the raising of the original dam to elevation 970 feet. The findings obtained during the construction of the raised dam will be compared with the results from the original investigation. Any deviation from these results will be addressed at the time and necessary modifications will be made to accommodate the specific field conditions for the raised dam. In the following sections, the foundation requirements related to the raising of the dam are discussed in more detail.

2.3.1 Overburden

Unsuitable overburden material will be removed during construction of the raised dam in the areas below the toe extensions of the dam and along the valley slopes up to elevation of 980 NGVD. As indicated by the original investigation the thickness of the overburdens generally ranges between ten and thirty feet.

2.3.2 Foundation Rock

The lateral and downstream extension to the existing dam will be founded on the rock. The surface of the foundation rock will be cleaned and loose pieces of the rock will be removed. Particular attention will be given to the rock seepage zones. Grouting will be performed as required and a suitable drainage system will also be provided in these zones. In addition, the water collected in these zones will be conveyed to the drainage blanket and/or chimney drains as directed by field conditions.

2.3.3 Borrow Areas

The locations of the borrow areas are shown on the original Drawing No. 12-3001, which is attached to this report as a reference drawing. The soils from these areas will be utilized for construction of impervious clay core (Zone I) and downstream shell (Zone IV). The bottom ash from Cardinal Plant bottom ash pond will be used as a granular material for the drains. Clean granular aggregate will be utilized for extension of the drainage blanket at the toe of the dam.

2.3.4 Existing Dam

The crest of the existing dam will support the raised portion of the dam. A berm will be provided at the heel of the RCC section to improve the stability of this section. Field investigations conducted in the Spring of 1996 indicated the soil condition to be satisfactory. The soil in the upstream Zone IV was not saturated in its upper part which is critical to the raising. It presumably is saturated below the water level as indicated by readings of piezometers.

The impervious clay core Zone I was performing as designed, preventing seepage of the water through the dam. The soil in this zone was dry as well as the soil downstream from the clay core.

2.4 Proposed Design of Dam

The proposed raising of the dam has been designed as an earth embankment with the RCC facing as shown on Drawing No. 13-30041. The RCC zone having steeper slopes than the existing dam will minimize the amount of fill required for the construction of the downstream shell (Zone IV). All other zones that compose the original dam will be extended in order to meet the geometry of the proposed section for the raised dam. These zones include clay core (Zone I), chimney drain (Zone III A), drainage blanket (Zone III B), outlet of the blanket drain (Zone III C), and downstream earth fill (Zone IV). The use of a reinforced fly ash fill and stabilized fly ash fill as a substitute for the earth fill in the Zone IV was also evaluated. Cement and lime was used as a stabilizing material in order to improve the strength of the fly ash. Based on results of the analyses the cost necessary for mixing, placement and in-place testing of the stabilized fly ash fill was too high compared to earth fill, and therefore was eliminated.

2.4.1 Emergency Spillway

The existing emergency spillway will be modified as shown in Section 2-2 of Drawing No. 13-30041. The modified spillway system will safely pass the design PMF without any overtopping of the dam. As shown on Section 2-2, the upstream zone, top and downstream slope of the emergency spillway will be constructed of the RCC. In addition, the downstream slope will have a stepped surface in order to dissipate energy of the flowing water.

2.4.2 Service Spillway

The service spillway will change from the vertical concrete shaft structure connected to a 42-inch diameter conduit to a new inclined rectangular shaft structure along the face of the RCC zone. The inclined shaft will connect to a 54-inch diameter prestressed concrete cylinder pipe (PCCP).

The 54-inch PCCP connects into a transition manhole at the toe of the downstream slope. From this transition manhole, the outlet pipe will be a 42-inch diameter welded steel pipe laid above grade down the abutment slope. The steel pipe discharges into a drop manhole which will be connected to the existing 42-inch diameter PCCP outlet pipe.

The section of the existing outlet pipe upstream of the new drop manhole will be grouted and abandoned in place.

The total length of pipe to be grouted is about 1,000 ft. The existing access bridge and skimmer will be utilized to support the grout supply line. Three hundred (300) psi strength flowable grout will be used in this grouting operation. The grout will be pumped and discharged initially at distance 500 ft from the bottom of the overflow tower or inlet of the pipe. This discharge point will be moved if necessary, in order to improve the grouting operations. As the pipe is filled, the supply line will gradually be retrieved until the entire pipe and the bottom 5 ft of the overflow tower is filled with the grout. It is our intention not to fill the overflow tower with grout, but rather allow it to be filled with fly ash and water. Filling the tower with heavier grout would jeopardize its stability. Prior to the grouting operation, a concrete bulkhead will be installed at the outlet of the pipe in the proximity of the drop manhole, where the pipe will be cut. The ungrouted portion of the pipe will be connected to the new manhole and incorporated into the new overflow discharge system. AEP plans to solicit input from the Contractors regarding grouting of the pipe and if the proposed grouting method is modified or changed, the ODNR, Division of Water will be notified.

2.5 Field Investigation at Dam and Borrow Areas

2.5.1 Existing Dam and Borrow Areas

The results of the site investigation related to overburden foundation and borrow areas for the existing dam are discussed in detail in the original Design Report for Dam. Boring logs and description of test trenches are included in Appendix A of the original report. The results of the 1996 laboratory testing on samples of these soils are included in Appendix B.

Field permeability tests were conducted in order to assess the potential for seepage through the abutment and foundation rock. Test results are tabulated in Appendix C of original Design Report for Dam and summarized in Drawing Nos. 13-3004 and 13-3005.

No specific field seismic investigations were performed for the dam. A pseudo static analysis with a horizontal seismic coefficient of 0.10 g was conducted to obtain stability of the dam during an earthquake loading. The seismic coefficient was selected based on seismic zone map. Zone 2 with a coefficient of 0.10 g was assumed for the site of the dam.

2.5.2 Recent Field Investigation and Laboratory Testing

The field investigation conducted in the Spring of 1996 included seven borings. Three of these borings were inclined borings in the upstream Zone

IV and four borings were vertical borings taken within clay core Zone I. The purpose of these borings was to evaluate current soil conditions within the original dam and to provide soil samples for laboratory testing to obtain current soil strength parameters to be used in the stability evaluation of the raised dam. Location of these borings is shown in Drawing No. 13-30040. The log of borings is included in Appendix A.

Tests to determine strength parameters of the dam soils were conducted in the AEP Civil Engineering Laboratory. The results of these tests are included in this final design report.

Tests to obtain composition and strength properties for roller-compacted concrete (RCC) mix were also performed in the AEP lab. The test's results are included in Appendix B. Based on these results the proposed mix No. 1 consisting of 60% fly ash, 40% bottom ash and 8% cement is recommended to be used on the upstream facing of the Cardinal Dam. For interior zone of the upstream RCC shell, it is recommended that a lower strength No. 1 mix with 4% cement was used. The decision to use these two mixes was based on the strength and freeze-thaw resistance. Moisture-density relationship curves for the mixes tested, and gradation curves for fly ash and bottom ash material together with five soil saturation tests and triaxial tests are included in the Appendix B. The saturation tests were performed to obtain degree of saturation for soils of the dam in order to select the type of tests required to determine soil strength parameters above and below ground water table. Based on these results unconsolidated undrained (UU) strength was used above ground water table, and anisotropically consolidated undrained (ACU) strength was used below ground water table.

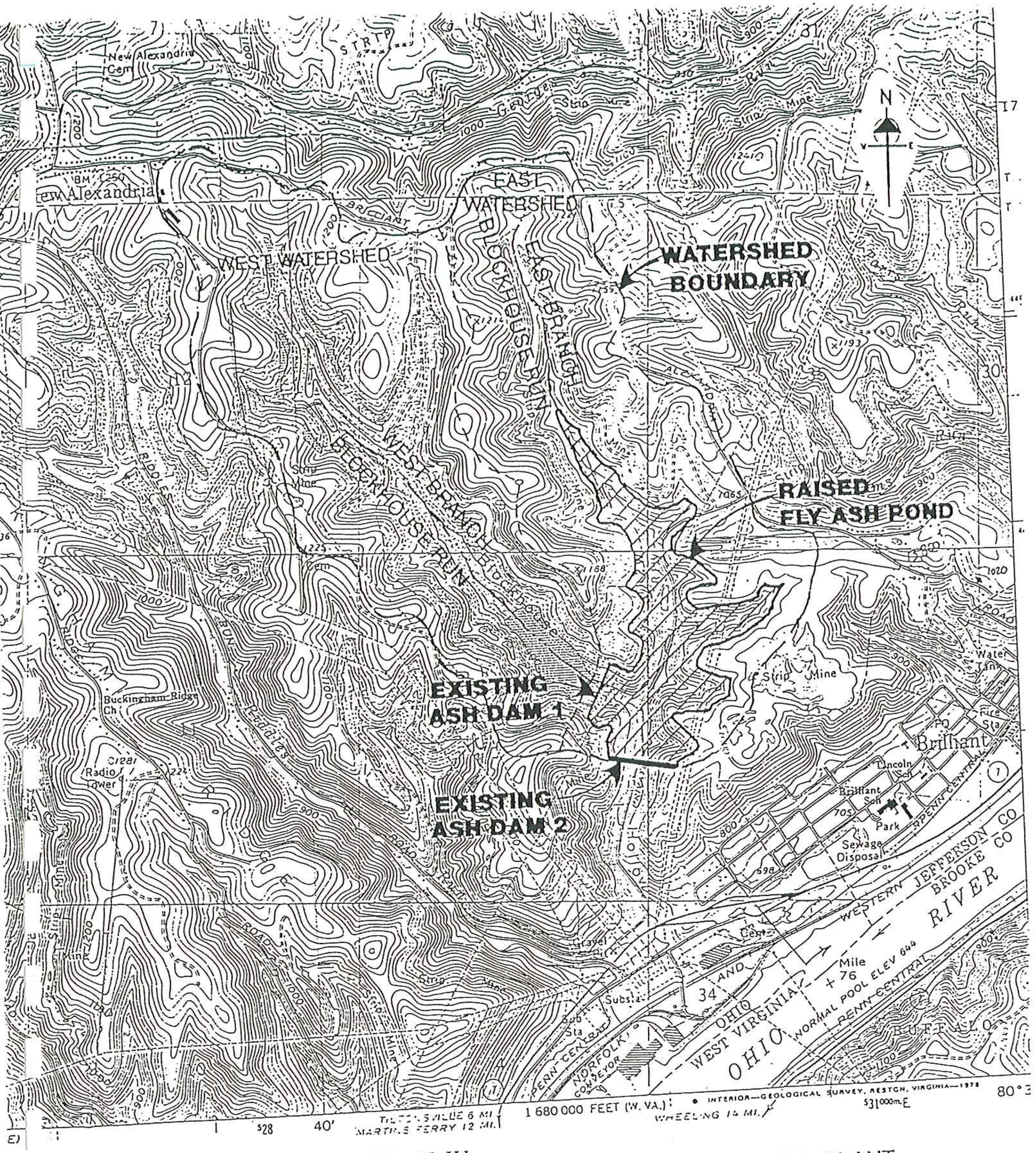
For construction of the downstream shell, in addition to an earth fill option, two other possibilities were evaluated. As a second option, we considered reinforced fly ash fill with erosion protection matting embedded into a topsoil facing layer placed over the fly ash fill. The third option included the same low strength RCC mix proposed for interior zone of the upstream RCC shell. The earth fill option was the most economical and provided required stability, therefore, the other two options were eliminated.

3.0 HYDROLOGY

3.1 Introduction

The existing hydrologic conditions at the proposed dam site are depicted in Figure 3.1. Blockhouse Run, the major drainage feature in the project area, drains directly into the Ohio River. Approximately one mile upstream of the Ohio River, Blockhouse Run splits into two branches, designated as the East Branch and the West Branch.

The East Branch drains the eastern watershed as delineated in Figure 3.1. The active fly ash dam II inundates the East Branch. The West Branch has been dammed to form the old fly ash retention pond I.



STEUBENVILLE WEST, OHIO – W. VA.
SW/4 STEUBENVILLE 15' QUADRANGLE
40080-C6-TF-024

1968
PHOTOREVISED 1985
DMA 4864 I SW–SERIES V852

SCALE
1000 0 1000 2000 FEET

CARDINAL PLANT
FLY ASH POND 2

FIGURE 3.1
WATERSHED MAP

The location of the dam is shown on Drawing No. 13-30039. Extension of the dam, and raising the fly ash pond level to a spillway elevation 961 ft. will inundate approximately 138 acres or 18 percent of the area in the eastern watershed. Since the location of the dam is situated downstream of the discharge points of the old dam, runoff from the western watershed drains into the existing reservoir. Therefore, the spillway system of the proposed dam raising has been designed to meet ODNR Class I design criteria based on the runoff from both watersheds. The following sections present the hydrologic considerations and analyses performed during the design phase of this project.

3.2 Basin Characteristics

Figure 3.1 shows the limits of the watershed boundary for the raised fly ash retention pond. The total drainage area above the dam has been divided into two watersheds, East and West, for analysis of the storm runoff entering the reservoir.

A review of available topographic maps and aerial photos was made to determine essential basin characteristics for each watershed. Such characteristics include the drainage boundaries, areas, slopes, soil types, ground cover, land use and the 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.

The old fly ash dam is located in the western watershed. Present land use within the drainage area is limited to reclaimed strip mine areas, some woodlands, and the inactive fly ash reservoir I. Reclamation of the reservoir area is actively in progress in the form of an ash landfill above the level of the settled fly ash.

Woodlands and scattered reclaimed strip mines constitute the existing land use in the East watershed. Construction of the proposed fly ash dam will inundate approximately 138 acres at elevation 960.0 feet NGVD, the maximum operating pool elevation.

Soil types in the areas have been identified by the Soil Conservation Service (SCS) of the U.S. Department of Agriculture and classified into hydrologic soil groups. Within the study area, all soils fall under the hydrologic soil group B. Table 3.2.1, below, lists the basin characteristics for the Western and Eastern watersheds.

TABLE 3.2.1 - BASIN CHARACTERISTICS

	WATERSHED	
BASIN CHARACTERISTICS	**EAST**	**WEST**
Drainage area (acres)	730	677
Average land slope %	25	30
Hydrologic soil group	B	B
SCS curve number	70	70
Time of concentration (hours)	0.57	0.87

3.3 Characteristics of Proposed Retention Pond

A previously referenced, Drawing No. 13-30039 shows the location of the existing dam. Based on this layout, the reservoir will have the following surface areas and storage capacities - as shown below on Table 3.3.1.

TABLE 3.3.1 - SURFACE AREAS AND STORAGE CAPACITIES

ELEVATION	(Ft. NGVD)	AREA (AC)	STORAGE (AC-FT)
Maximum Pool	960	138	9800
Top of Dam	970	157	11350

Figure 3.2 presents the complete area-capacity-elevation curve developed for this dam.

3.4 Design Requirements and Assumptions

Rainfall - runoff data was not available for the site because the streams flow intermittently. Therefore, runoff hydrographs were generated using the U.S. Army Corps. of Engineers' HEC-1 computer program. The SCS dimensionless unit hydrograph method was employed in the calculation of the hydrographs. For each watershed, separate runoff hydrographs were computed and then later combined to form a single inflow hydrograph for the proposed reservoir.

Runoff from the West watershed was analyzed based on fully reclaimed condition of the old fly ash reservoir I. The reclaimed condition will result in a gently sloped grassed field.

In the East watershed, the reservoir surface is modeled as a subbasin to convert direct rainfall into a runoff hydrograph. The ash sluice water of 13.3 mgd (20.6 cfs) is represented as a base flow in the East watershed.

Once computed the runoff hydrographs from the two watersheds are combined and routed through the reservoir.

3.4.1 Principal Spillway

According to ODNR regulation 1501:21-13-04, design of the principal (service) spillway for class I dams must be such that the average frequency use of the emergency spillway is predicted to be less than once in fifty years. Since stream-flow records are not available to establish a flood-frequency curve, it was assumed that a 50-year rainfall event would produce the 50-year flood. The estimated precipitation, 3.5 inches, was obtained from the National Weather Bureau (1960) report TP-40.

AREA CAPACITY CURVES

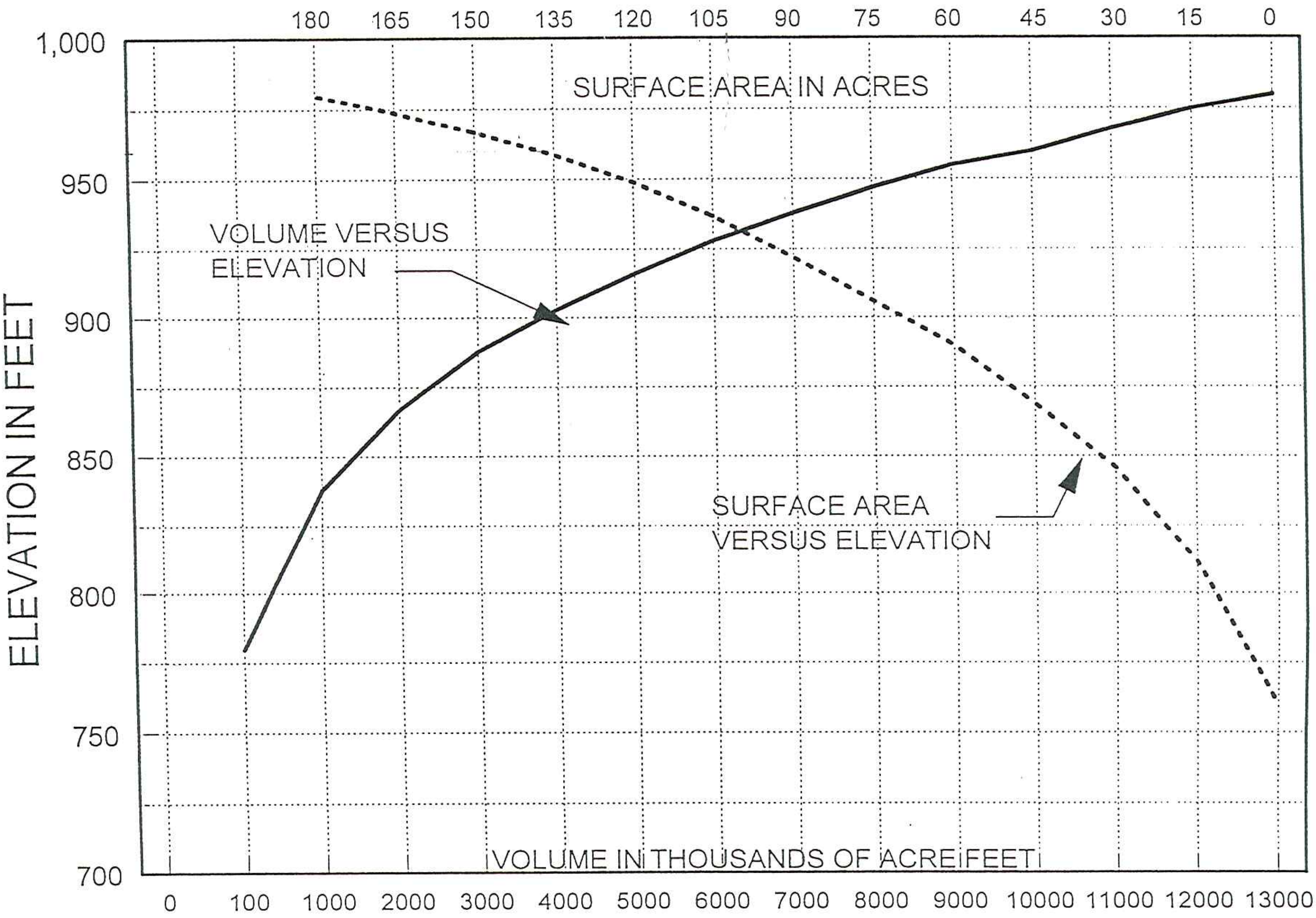


FIGURE 3.2

A 6-hour storm duration and average soil moisture conditions were assumed for developing the inflow hydrograph.

3.4.2 Emergency Spillway

The ODNR regulation 1501:21-13-02 specifies that for class I dams, the spillway system shall safely pass the design flood equal to the probable maximum flood (PMF) without any overtopping of the dam. The PMF is the result of the probable maximum precipitation (PMP), 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. Generalized estimates of the PMP have been published by the Hydrometerological branch of the National Weather Service (1978). For the study area, a 6-hour PMP of 26.5 inches was used as the design rainfall event. The antecedent moisture conditions of the soil cover were assumed to be average.

3.5 Analysis

All reservoir flood routings were conducted using the HEC-1 computer program. The program routes floods through the reservoir by the modified Puls method. In general, reservoir storage data and either spillway dimensions or discharge-rating curves are supplied by the user.

3.5.1 Principal Spillway

Analysis of the principal spillway system consisted of routing the 50-year flood to establish the invert of the emergency spillway. A preliminary design for the principal spillway was determined and a stage-discharge curve was computed. A maximum operating level of elevation 960 was predetermined based on the projected life of the dam raising. Reservoir routings of the 50-year storm were then performed.

3.5.2 Emergency Spillway

Hydrologic reservoir routings 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 elevation in the proposed reservoir was assumed to be at the maximum operating level at the beginning of the design storm. Initially, a rectangular section was used and the program internally computed the discharge given a weir length. Having obtained a rough estimate for the opening, a trapezoidal section was designed and rated based on calculations of critical depth using the Corps of Engineers HEC-2 computer program, Water Surface Profiles.

The calculated stage - discharge curve was then used in the routing process to determine the maximum discharge and pool elevation.

Discharges from the emergency spillway are routed away from the dam through an outlet channel that was designed for the first phase construction. Design of the channel size was based on the peak discharge from the reservoir.

3.6 Results

3.6.1 Principal Spillway

The proposed new principal spillway is a sloping concrete shaft structure with one side opening, four-feet wide, connecting into a 54- inch diameter prestressed concrete cylinder pipe (PCCP). During most operating conditions, discharge through the principal spillway will be controlled by weir flow over the opening in the shaft. The existing energy dissipator will be utilized.

The maximum operating level is set at elevation 960 feet. This corresponds to a maximum stop log elevation of 958.6 based on the base inflow of 20.6 cfs.

The peak inflow during the 50-year, 6-hour storm is 698 cfs, which results from 3.5 inches of rainfall. The reservoir level will rise to elevation 960.9 feet based on an initial pool level of elevation 960. The peak outflow from the dam will be 44 cfs. Appendix D contains the computer listing of the reservoir routing.

3.6.2 Emergency Spillway

The development of the PMF hydrograph indicates a peak inflow to the reservoir equal to 16,730 cubic feet per second. This value represents the combined hydrographs from the West and East watersheds. Values of the runoff from each watershed and the combined runoff are shown in Appendix D.

The layout of the control section and outlet channel for the emergency spillway is shown on Drawing Nos. 13-30040 & 13-30041.

The emergency spillway will be a section of the Earth Fill-RCC that has been lowered to elevation 961. It will have a bottom width of 110 feet and vertical side slopes. The downstream slope of the dam will be formed with steps to provide some energy dissipation. Discharges over the emergency spillway are directed into the excavated channel constructed during the Phase I construction.

Based on the flood routing, the calculated peak discharge from the dam is 6388 cfs at a maximum pool elevation of 968.1 feet NGVD. This peak discharge is less than the design value of 8789 cfs that was used to size the emergency spillway outlet channel for the existing dam.

3.7 Spillway System of Existing Dams

During the initial construction of the proposed dam raising, flow will continue to pass through the existing spillway tower and 42-inch diameter outlet pipe. As the pond level nears elevation 913, construction of the new principal spillway will be completed. The existing principal spillway will then be abandoned in place by grouting the outlet pipe between the tower and the new manhole (No. 1). A flow path to the new spillway structure will be constructed and all future discharges will be passed through this structure.

3.8 Summary and Conclusions

The hydrologic/hydraulic studies for the proposed dam raising included estimating the PMF and 50-year flood hydrographs and designing the emergency and service spillways. The U.S. Army Corps of Engineers computer programs HEC-1 and HEC-2 were used in the analyses. Table 3.8.1, below gives a complete summary of the study.

The proposed spillway system has enough capacity to pass the probable maximum flood without overtopping the dam. The water discharged through the emergency spillway is directed away from the dam such that it causes no threat to the stability of the structure.

TABLE 3.8.1 HYDROLOGIC/HYDRAULIC SUMMARY
FOR PROPOSED RAISING OF RETENTION POND

Drainage Area		**2.2 Sq. Mi.**
Designs Floods (Inflow)		
	PMF Peak	16,730 cfs.
	50-Yr Peak	698 cfs.
Peak Discharge		
	PMF	6388 cfs.
	50-Yr	44 cfs.
Maximum Pool Elevations, NGVD		
	PMF	968.1 ft.
	50-Yr	960.9 ft.
Emergency Spillway - Overflow Section of Earth Fill - RCC		
	Crest Elevation, NGVD	961.0 ft.
	Bottom Width	110.0 ft.
	Side Slopes	0 H:1V
Principal Spillway - Size		
	Sloping Concrete Shaft and Conduit	
	Shaft Opening @ 4.0' x 3.5'	
	Conduit Size	54" & 42"
	Maximum Operating Pool Level, NGVD	960.0 ft.

4.0 GEOTECHNICAL DESIGN.

Performance of the existing dam, geotechnical aspects incorporated in its design, and applicability of these aspects in the design of a portion of the dam proposed for raising are discussed in the following sections.

4.1 Performance of the Existing Dam Embankment

Quarterly inspection of the existing dam and appurtenant structures is conducted by plant personnel. The personnel of the American Electric Power Service Corporation (AEPSC) perform yearly inspections to review overall performance of the dam and to ensure that the remedial work has been completed in accordance with recommendations resulting from previous inspections. An inspection on a biennial basis is conducted by an outside independent consultant. The purpose of this inspection is to reinspect the dam, to review the results of periodic inspections conducted by the plant and AEPSC personnel, and to provide an independent assessment of the current status of the dam. The results of a dam safety and performance inspection are summarized below:

- Geotechnical design criteria used in the design of the existing dam, based on its performance since construction to present, is satisfactory.
- The long term movement of the dam as established by deformation survey is within expected range for the earth fill dam.
- Some new seepage in the left abutment downstream of the dam has been developed subsequent to construction of the dam, as the reservoir water level has been raised. No evidence of fines in the discharge was detected. All seepage areas are being monitored on a regular basis.
- The slide on the valley slopes near the right abutment, we believe, was the result of the service road drainage problem. This problem was corrected in 1993. It appears that the slide has stabilized. Nevertheless, the area of the slide is inspected on a regular basis.
- Piezometers were installed in various zones of the dam during its construction. Readings for the piezometers show a trend which indicates a normal condition reflective of post construction period and an increase in the reservoir level.

In conclusion, the dam appears to be performing satisfactorily. Inspection of the dam indicates there was no visual evidence of excessive deformation, movement and cracking which would be an indication of deterioration in the long-term stability of the dam.

4.2 Geologic Assumptions for Raised Dam

Geologic conditions considered in the design of the raised portion of the dam include the site geologic features and rock types that may adversely effect the stability and performance of the raised dam. Major geologic assumptions addressed here have been evaluated and incorporated in the design of the existing dam. These assumptions included evaluations of geologic structures at the site such as rock, faults, joints, shear zones and bedding plans. Presence of clay seams in the foundation rock was also investigated as a potential instability condition. It was concluded that continuous clay seams are not likely to exist in the foundation rock at the site.

Seepage through the foundation and abutment rock was evaluated. Zones of rock with higher in-situ permeability were identified and grouted during construction of the existing dam. The same approach will be used to correct any seepage through the abutments of the raised portion of the dam. These seepage zones, if discovered during construction, should be grouted and drained as necessary. Although no sinkholes or downstream boils have developed at the dam since its construction, seepage has occurred at the downstream toe on both the right and left abutments. After stripping of the foundation for the downstream overlay, a drainage blanket will be provided over the entire area stripped.

4.3 Proposed Raising of the Dam

The raising of the dam to a crest elevation of 970 feet is required in order to increase the storage capacity of the fly ash reservoir for future disposal of fly ash produced by the Cardinal units. Start of construction operations is scheduled for the Spring of 1997.

The proposed raising of the dam was designed as an embankment with an upstream RCC zone. The face of this zone will consist of a higher strength mix. In the interior portion of this zone we are considering using the lower strength RCC which is more in line with the strength of the soil. Mixing, placement and compaction of the RCC material will be performed in accordance with acceptable standards for this material during placement of a test fill section in the summer of 1997.

An RCC test section will be constructed in the area of emergency spillway channel immediately downstream of the toe of the raised dam. The purpose to construct the test fill will be to confirm mix design mixing process, placement, composition of the RCC fill and to identify any potential problem areas. Also, the layout of the testfill should, in so far as feasible, contribute to improvement of the get-away channel.

Results of this fill testing will be included in the construction progress reports which will be submitted to the ODNR on a regular basis during construction.

The earth fill portion of the dam will be constructed in accordance with the requirements established in the design of the original dam. The requirements pertinent to the different zones shown on Drawing No. 13-30041 are discussed in Section 4.0 of Design Report for Proposed Dam (1). The main points addressed in this report are summarized in the following paragraphs.

The underdrain system for the dam will be extended and daylighted at the toe of expanded dam.

Mine spoil fill from the borrow areas will be utilized in the construction of downstream shell. Bottom ash and fly ash from the plant will be incorporated in the RCC material.

The upstream clay core will be extended to competent rock in the abutments at right angles to the rock surface. Since the dam is arched in the upstream direction it should reduce seepage at the interface and minimize the cracking of the core.

The RCC upstream zone will follow the flat arch of the dam. The rock will be excavated to achieve a right angle contact with the RCC.

Water diversion during proposed construction of the dam is described in Section 3.7.

Grouting of the abutments' rock will be performed on an as-needed basis confirmed in the field during foundation preparation stage.

Geotechnical instrumentation already in place will be maintained. Any destroyed points will be replaced whenever possible or new ones will be installed in the proximity of the destroyed one.

5.0 GEOTECHNICAL ANALYSIS.

Seepage, stability and settlement analyses of the raised dam have been conducted to evaluate geotechnical aspects of the proposed earth fill - RCC design. The underdrain system was designed to have ample discharge capacity to handle the anticipated flow of the raised dam through the dam embankments and foundation. The minimum safety factors for stability at end of construction, steady seepage and seismic cases were determined to be acceptable. The assumptions made, methods used, and results of the geotechnical analyses are discussed in the following sections.

5.1 Seepage Analyses

5.1.1 General

Seepage analyses were coordinated by AEP. Actual work was performed by Geo/Environmental Associates, Inc., Knoxville, TN. The analysis of the existing Cardinal Fly Ash Retention Pond II, was conducted first with coefficients of permeability (k) for the various embankment and foundation materials that were used in the design of the original dam. For the stability analyses, it was assumed that the k value of the transition zone was the same as the clay core. A seepage analysis was then performed for the proposed raising of the existing facility to a crest elevation of 970 feet NGVD to assess the top flow line and the maximum seepage through the embankment and the abutments to various pool levels. The seepage analyses were performed with the aid of a finite element seepage computer program entitled SEEP/W. The program utilizes a finite element method to solve steady-state and transient free water surface problems for seepage through porous media. SEEP/W was developed by GEO-SLOPE Programming Limited of Calgary, Alberta, Canada.

In general, given a total known head at an inlet condition and an unknown head at an outlet condition (i.e., review node boundary condition), the program adjusts the total head and the amount of flow at the nodal points based upon input coefficients of permeability and geometry until equilibrium is reached. Equilibrium conditions are then shown as a final free water surface.

5.1.2 Assumptions and Selection of Parameters

Actual flow rates from the main drain and the abutments are summarized in Table 5.1.2.1.

TABLE 5.1.2.1 SUMMARY OF MEASURED FLOW RATES

Pool Level (ft. NGVD)	Seepage Flow Rate (gpm)		
	Outlet Drain	Right Abutment	Left Abutment
894	23	265	25

The coefficients of permeability (k) used in the seepage simulations are concluded in Table 5.1.2.2.

TABLE 5.1.2.2 SUMMARY OF COEFFICIENTS OF PERMEABILITY USED IN SIMULATIONS

Material	Coefficient of Permeability (ft/min.)	
	Simulation No.1	
	Vertical	Horizontal
Downstream Shell	2 x 10^{-4}	1.8 x 10^{-3}
Blanket Drain	1 x 10^{-1}	9 x 10^{-1}
Fly Ash	2 x 10^{-5}	1.8 x 10^{-4}
Clay Core	4 x 10^{-8}	3.6 x 10^{-7}
Roller-compacted Concrete	1 x 10^{-8}	9 x 10^{-8}
Upstream Shell	4x10^{-7}	3.6 x 10^{-6}
Transition Zone	4 x 10^{-8}	3.6 x 10^{-7}
Overburden	2 x 10^{-6}	1.8 x 10^{-5}

Simulation No. 1 was performed for the existing dam with a crest elevation of 925 feet NGVD and a normal pool of 894 feet NGVD. As shown by the results included in Section I, Simulation No.1 provides a reasonable estimation of the top flow line and equipotential levels that more or less agree with readings of piezometers. As a result, the k values shown in Table 5.1.2.2 are used in the simulation of future stages of construction.

5.1.3 Predictions For Future Stages

Simulation No.2 was performed to assess conditions prior to the start of the new stage of construction. This simulation was performed for the current embankment configuration and a pool level at elevation 903 feet NGVD, assumed to be an anticipated start for RCC placement. Simulation No. 3 was performed for the future dam with a normal pool level of 960 feet NGVD. Finally, a model of the proposed construction was developed with a crest elevation of 970 feet NGVD and a peak pool level of 968 feet NGVD performed for the future dam with a normal pool level of 960 feet NGVD. Finally, a model of the proposed construction was developed with a crest elevation of 970 feet NGVD and a peak pool level of 968 feet NGV (Simulation No. 5) to simulate conditions that could develop on the roller-compacted concrete during the design storm event. Output data from Simulation Nos. 2, 3, 4 and 5 are used in the stability analyses summarized in 5.2.

The flow rate through the right abutment (looking downstream) was calculated to increase from the maximum current rate of 265 gpm (observed only on two occasions, 11/30/95 and 3/14/96) to a rate of 493 gpm after the normal pool achieves an elevation of 960 feet NGVD. This assumption is conservative since the average rate of the current flows through the right abutment is about 172 gpm.

An increase in the seepage flow rate was also based on a relationship of current-past seepage rates versus and pool water elevations projected to a final pool elevation of 960 feet. The projected flow was determined to be around 400 gpm which is somewhat less than calculated flow.

The rate of seepage at the left abutment, for the normal pool at the elevation of 960 feet was predicted to increase from the current 25 gpm to a seepage rate of 45 gpm based on the assumption that some new springs may develop as the normal pool level in the pond is increased to elevation of 960 feet. Two new springs have developed as water levels in the pond was raised. The relationship diagram of the current flows and water elevations for the left abutment appears inconclusive thus, it has not been used to project the future seepage flow through this abutment. A ratio of the final and current water elevations multiplied by the current flow was used to estimate the future flow. The plots for seepage flow through both abutments are included in Appendix C. Due to geological conditions, seepage rate through the right abutment was always considerably higher than the seepage through the left abutment. Therefore, it is reasonable to assume that the same condition will prevail in the future. In support of this conclusion, three springs developed in the right abutments during initial impoundment of the Fly Ash Retention Dam II reservoir.

All existing springs are located some distance downstream from the toe of the existing and raised dam, and thus shall not endanger the stability of the dam. A drainage blanket and collection ditches have been installed around the springs in the right abutment in order to stabilize surrounding areas and to control surface drainage and soil erosion.

Based on the above evaluation, the total calculated/estimated rate of seepage through the right abutment dam and left abutment is approximately 598 (493+45+60) gpm. Although the total flow rate through the raised dam is estimated to be only 60 gpm, flow through the right abutment, more than the flow from the left abutment has the potential to partially discharge into the extended downstream toe drainage blanket. Therefore, the existing blanket drain, extended beneath the raised portion of the embankment was evaluated for this condition and was found to have the capacity to accommodate an assumed flow of approximately 434 gpm from the abutments, which is considerably higher than any expected seepage from the abutments based on up-to-date measured data.

Moreover, a lateral drainage blanket along the contract surface of the rock and fill will be placed in both abutments of the downstream extended shell.

In addition, to further reduce discharge of the water from the right abutments into the toe drainage blanket, excavated areas will be cleaned and inspected for seepage. The zones of rock mass with higher in-situ permeability at the contact of the clay core and the rock will be grouted. All other seepage zones, if discovered, will be drained as necessary by installing a drain pipe between seepage zones and groin ditch.

The rock surface in the extended zone of the left abutment is already exposed and there are no visible areas with high in-situ permeability.

5.2 Slope Stability Analyses

5.2.1 General

Stability analyses were also performed by Geo/Environmental Associates, Inc. for the proposed raising of Cardinal Dam II using the SLOPE/W software package developed by GEO-SLOPE International. SLOPE/W utilizes the limit equilibrium theory to solve for the factor of safety of earth and rock slopes. Specifically, Bishop simplified method was employed to estimate safety factors for several simulations of construction for the facility. Phreatic surfaces and pressure heads for the analyses were estimated with the SEEP/W software package, and imported into SLOPE/W. The SEEP/W output is included in Appendix E of this submittal. Material parameters used in the analyses have been based on recommendations by a consultant, John Lowe, III. A summary of the conditions analyzed, along with tabulated stability analysis results are presented herein. Additionally, SLOPE/W graphical output for each of the stability analyses is included in Appendix E, Section 2. Minimum factors of safety used in the stability analyses of the Cardinal dam are shown in table 5.2.1. These factors are based on the minimum factors of safety recommended by Federal Energy Regulatory Commission (FERC) Engineering Guidelines for Embankment Dams(2).

TABLE 5.2.1 MINIMUM REQUIRED FACTORS OF SAFETY

Loading Conditions	Minimum Factor of Safety	Slope to be Analyzed
End of Construction	1.3	Upstream & Downstream
Steady Seepage with Max Pool	1.5	Upstream & Downstream
Steady Seepage with Surcharge Pool	1.4	Downstream
Steady Seepage with Maximum Pool plus Earthquake	>1.0	Upstream & Downstream

5.2.2 End of Construction Condition

Static and dynamic stability analyses have been performed on both the upstream and downstream slopes for the proposed end-of-construction condition of the facility. The results of these analyses are included in Appendix E, Section II, Simulation 1.

The end-of-construction condition was modeled with the proposed RCC (roller-compacted concrete) zone constructed to its maximum elevation of 970 feet, NGVD. The pool and fly ash elevations were modeled at 903 feet, NGVD, and 895 feet, NGVD, respectively. Furthermore, the slope of the downstream face of the dam was modeled as 2.5H:1.0V (2.5 horizontal to 1.0 vertical).

The parameters used for the end-of-construction stability analyses are shown in Table 5.2.2.1. ACU (anisotropically consolidated undrained) parameters were used for saturated embankment materials (See Appendix E, Section II, Attachment II-1), and modeled for the end-of-construction condition. Three trial failure surfaces were estimated at the time of consolidation (existing stage), and ACU parameters were derived based on methods published by Mr. Lowe (Lowe, 1967). Included in Attachment II-1 are the SLOPE/W models used for the ACU calculations, detailed slice information for each of the slices in the critical trial failure surface No. 1 and Lotus 5.0 spreadsheets calculating ACU parameters along all three trial failure surfaces. After the ACU parameters were estimated for the applicable materials, the two graphs in Attachment II-1, provided by Mr. Lowe, were used to estimate anisotropic material strength parameters to be used in the end-of-construction stability analyses. Parameters for the unsaturated materials were selected from the material profile constructed by Mr. Lowe (Attachment II-1).

Table 5.2.2.1 END-OF-CONSTRUCTION SIMULATION MATERIAL PARAMETERS

Material	Unit Weight, γ (pcf)	Angle of Internal Friction ϕ, (degrees)	Cohesion, c, (psf)
RCC Zone	95	0	14400
Fly Ash	90	26	0
Saturated Upstream Shell	128	Variable*	Variable*
Clay Core	125	6.8	2200
Saturated Clay Core	128	Variable*	Variable*
Transition Zone	125	11	2000
Saturated Transition Zone	128	Variable*	Variable*
Downstream Shell	125	11	2000

Material	Unit Weight, γ (pcf)	Angle of Internal Friction ϕ, (degrees)	Cohesion, c, (psf)
Chimney and Foundation Drain	100	38	0
Overburden	123	0	1700
Claystone	140	22.5	1100
Shale	140	15	1100
* ACU parameters vary based on the major to minor principal stress ratio, K_c (See Appendix E - Attachment II-1 for actual parameters used).			

The ACU conditions which were calculated at the time of consolidation (existing stage), were derived based on methods published by Mr. Lowe (Lowe, 1967). Included in Attachment II-1 are the SLOPE/W models used for the ACU calculations, a sample calculation for determining the ACU parameters, and Lotus 5.0 spread sheets calculating ACU parameters for each of the failure slices.

Safety factors have been calculated for static and dynamic (pseudo-static) conditions in both the upstream and downstream directions for the end-of-construction simulation. Graphical output for the end-of-construction simulation is included in Appendix E Section II - Attachment II-2. A summary of the results is presented in table 5.2.2.2.

TABLE 5.2.2.2 END-OF-CONSTRUCTION SIMULATION STABILITY ANALYSIS RESULTS

Analysis Condition*	Calculated Minimum Safety Factor
CRITICAL DOWNSTREAM STATIC FAILURE SURFACE	1.43
UPSTREAM STATIC TRIAL FAILURE SURFACE 1	1.30
UPSTREAM STATIC TRIAL FAILURE SURFACE 2	1.37
UPSTREAM STATIC TRIAL FAILURE SURFACE 3	1.47
* End-of-construction simulations were subjected to ACU conditions, and pore water pressures applied at the resultant centroid on the sides of the failure surface slices.	

5.2.3 Steady-State-Condition With Maximum Storage

Static and dynamic stability analyses were simulated on both the upstream and downstream slopes for the proposed steady-state condition with maximum storage (Simulation No. 2). The dam was modeled with the RCC zone constructed to its maximum elevation of 970 feet, NGVD. The pool and fly ash were both modeled at 960 feet NGVD. Again, the slope of the downstream face of the dam was modeled as 2.5 horizontal to 1.0 vertical (2.5H:1.0V). Table 5.2.3.1 summarizes the strength parameters used in this analysis.

TABLE 5.2.3.1 STEADY-STATE CONDITION MATERIAL PARAMETERS

Material	Unit Weight, γ (pcf)	Angle of Integral Friction, ϕ , (degrees)	Cohesion, c, (psf)
RCC Zone	95	0	14,400
Fly Ash	90	26	0
Upstream Shell	128	30	0
Clay Core	125	28	0
Saturated Clay Core	128	28	0
Transition Zone	125	30	0
Saturated Transition Zone	128	30	0
Downstream Shell	125	30	0
Chimney Drain and Foundation Drain	100	38	0
Overburden	123	0	1,700
Claystone	140	22.5	1,100
Shale	140	15	1,100

SLOPE/W was employed to estimate safety factors for static and dynamic conditions in both the upstream and downstream directions for the steady-state condition with maximum storage. Graphical output for the simulation is included in Appendix E, Section II - Attachment II-3. A summary of the results is presented in Table 5.2.3.2.

TABLE 5.2.3.2
STEADY-STATE CONDITION WITH MAXIMUM POOL STABILITY ANALYSIS RESULTS

Analysis Condition	Calculated Minimum Safety Factor
MINIMUM DOWNSTREAM STATIC FAILURE SURFACE (ALONG FACE)	1.45
MINIMUM DOWNSTREAM DYNAMIC FAILURE SURFACE (ALONG FACE)*	0.99
DOWNSTREAM STATIC FAILURE SURFACE	1.62
DOWNSTREAM DYNAMIC FAILURE SURFACE*	1.12
DOWNSTREAM STATIC FAILURE SURFACE (DAYLIGHTING UPSTREAM RCC ZONE)	1.90
DOWNSTREAM DYNAMIC FAILURE SURFACE (DAYLIGHTING UPSTREAM RCC ZONE)*	1.26
*Based on a conservative earthquake coefficient = 0.15g.	

5.2.4 Steady-State Condition With Surcharge Pool

The final simulation entailed examining a static loading along the downstream slope for a steady-state condition with a surcharge pool. As before, the dam was modeled with the proposed RCC zone constructed to its maximum elevation of 970 feet, NGVD. The surcharge pool condition was modeled with the pool at elevation 968 feet, NGVD, and the fly ash at elevation 960 feet, NGVD. Again, the slope of the downstream face of the dam was modeled as 2.5 horizontal to 1.0 vertical (2.5H:1.OV). The steady-state condition material parameters provided in Table 5.2.3.1 were applied to the conditions for Simulation No. 3.

Safety factors for the steady-state condition with a surcharge pool are provided in Table 5.2.4.1. Graphical output for the simulation is included in Appendix E, See Attachment II-4.

TABLE 5.2.4.1 STEADY-STATE CONDITION WITH SURCHARGE POOL STABILITY ANALYSIS RESULTS

Analysis Condition	Calculated Minimum Safety Factor
MINIMUM DOWNSTREAM STATIC FAILURE SURFACE (ALONG FACE)	1.45
DOWNSTREAM STATIC FAILURE SURFACE	1.60
DOWNSTREAM STATIC FAILURE SURFACE(DAYLIGHTING UPSTREAM RCC ZONE)	1.86

Because the surcharge loading condition could only develop during the relatively short duration probably maximum flood, earthquake loading conditions are not applicable for the surcharge loading condition.

5.3 Settlement

No long term foundation settlement is expected to occur due to raising of the dam. Most of foundation and embankment settlement occurred during construction of the original dam and operational stage preceding the raising.

6.0 COST ESTIMATE AND CONSTRUCTION SCHEDULE

The cost of constructing the dam to the elevation 970 ft., and appurtenances is estimated to be 7,380,000. Table 6.1 lists the estimated cost for each construction item.

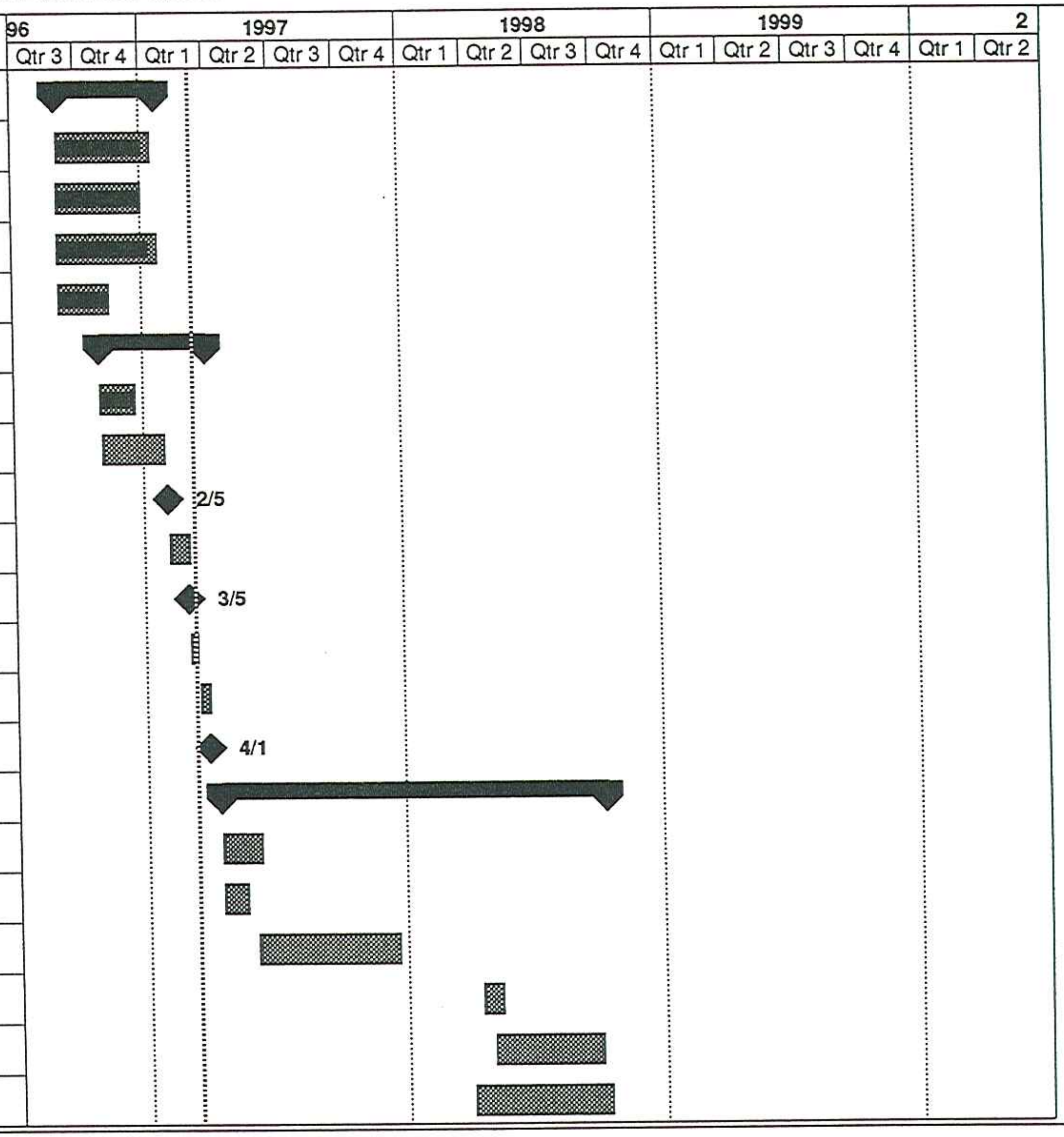
The construction schedule for the project is shown in Figure 6.1. Construction of the dam and appurtenances is expected to take two constructions seasons with mobilization scheduled for April 1, 1997 and completion in October of 1998.

TABLE 6.2 - COST ESTIMATE

DESCRIPTION	UNITS	QUANTITY	UNIT PRICE	COST$
1. MOBILIZATION	L.S.	1	425,000	425,000
2. CLEARING AND GRUBBING	ACRES	16	3,500	56,000
3. STRIPPING	S.Y.	110,000	1.0	110,000
4. COMMON EXCAVATION	C.Y.	54,000	2	108,000
5. DRAINAGE AGGREGATE	C.Y.	34,000	6	204,000
6. FOUNDATION PREPARATION	S.Y.	17,000	3	51,000
7. CLAY CORE-FILL ZONE I	C.Y.	24,000	6	144,000
8. PLACE AGGREGATE DRAIN FILL ZONE II, III	C.Y.	34,000	2.5	85,000
9. FILL ZONE IV	C.Y.	550,000	4	2,200,000
10. SEDIMENT CONTROL	L.S.	1	25,000	25,000
11. BORROW AREA RECLAMATION	ACRE	32	2,500	80,000
12. OVERFLOW STRUCTURE	L.S.	1	40,000	40,000
13. EMERGENCY SPILLWAY	L.S.	1	150,000	150,000
14. SEEDING	S.Y.	41,000	0.5	22,000
15. DRAINAGE PIPE	L.F.	500	5	25,000
16. ENERGY DISSIPATOR	L.S.	1	15,000	15,000
17. OVERFLOW SKIMMER	L.S.	1	20,000	20,000
18. ACCESS STAIRS	L.S.	1	22,000	22,000
19. INSTRUMENTATION	L.S.	1	75,000	75,000
20. GROUNDWATER WELLS	L.S.	1	65,000	65,000
21. DRILLING & GROUTING	L.S.	1	40,000	40,000
22. OVERFLOWING PIPING (2 MH, AND 54" RCPP, 42" STEEL & 42" RCPP)	L.S.	1	170,000	170,000
23. ROLLER COMPACTED CONCRETE	C.Y.	88,000	36	3,168,000
24. GROUTING OF EXISTING 42" RCPP	C.Y.	400	150	60,000
25. RIPRAP	L.S.	1	20,000	20,000
TOTAL				7,380,000

CARDINAL FLY ASH RESERVOIR
DAM CREST RAISING
AEPSC LABOR CONTRACTING

ID	Task Name	Start	Finish
1	**DEVELOP CONTRACT DOCUMENTS**	**9/5/96**	**1/24/97**
2	PREPARE & ISSUE BID DRAWINGS	9/5/96	1/16/97
3	PREPARE & ISSUE SPECIFICATIONS	9/5/96	1/1/97
4	DEVELOP SCOPE OF WORK	9/5/96	1/24/97
5	DEVELOP BID QUANTITIES	9/5/96	11/15/96
6	**BID / AWARD CONTRACT**	**11/2/96**	**4/1/97**
7	PREPARE BIDDERS LIST	11/2/96	12/20/96
8	PREPARE / ROUTE BIDDERS LIST	11/4/96	1/31/97
9	CONDUCT PRE-BID MEETING	2/5/97	2/5/97
10	BID PROJECT	2/5/97	3/5/97
11	RECEIVE BIDS	3/5/97	3/5/97
12	PREPARE RECOMMENDATIONS	3/5/97	3/14/97
13	ROUTE RECOMMENDATION FOR AWARD	3/17/97	3/31/97
14	AWARD CONTRACT	4/1/97	4/1/97
15	**CONTRACT WORK**	**4/15/97**	**10/13/98**
16	CONTRACTOR MOBILIZE AT SITE	4/15/97	6/9/97
17	EROSION AND SEDIMENT CONTROL 1997	4/15/97	5/19/97
18	DOWNSTREAM SHELL WORK 1997	6/2/97	12/19/97
19	EROSION AND SEDIMENT CONTROL 1998	4/15/98	5/12/98
20	CONSTRUCT RCC FILL	5/1/98	10/1/98
21	CONSTRUCT SPILLWAY	4/1/98	10/13/98



Project: CARDINAL FLY ASH RESER'
Date: 3/11/97

Task	▨	Summary	▬	Rolled Up Progress	▬
Progress	▬	Rolled Up Task	▨		
Milestone	◆	Rolled Up Milestone	◇		

REFERENCES

(1) American Electric Power Service Corporation, "Cardinal Plant Fly Ash Retention Pond II, Design Report for Proposed Dam," Civil Engineering Division, Columbus, Ohio, December 1984.

(2) Federal Energy Regulatory Commission "Engineering Guidelines For The Evaluation of Hydropower Projects," April 1991 pages 4-28.

Dam Raising Design Report
Cardinal Fly Ash Reservoir No. 2

Submitted to

Ohio Department of Natural Resources
Division of Soil and Water Resources

Submitted and Owned by

Cardinal Operating Company
Brilliant, Ohio

Prepared by

American Electric Power Service Corporation
1 Riverside Plaza, Columbus Ohio 43215

and

S&ME, Inc.
6190 Enterprise Ct.
Dublin, Ohio 43016

January 2013

TABLE OF CONTENTS

LIST OF TABLES

LIST OF APPENDICES

DRAWING INDEX

DRAWING NUMBER	TITLE
13-30080-A	Cover Sheet (ODNR)
13-30081-A	Reservoir General Arrangement
13-30082-A	Dam Control Plan and Profile
13-30083-A	Site Plan
13-30084-A	Trench Excavation, Slurry Wall, and Sheet Pile Wall
13-30085-A	MSE Wall Elevation
13-30086-A	MSE Wall Details
13-30087-A	Dam Raising Typical Sections
13-30088-A	Right Abutment Plan and Sections
13-30089-A	Emergency Spillway and Downstream Slope Grading Plan
13-30090-A	Emergency Spillway Profile and Sections
13-30091-A	Service Spillway General Plan, Profile and Elevation
13-30092-A	S1.0 Service Spillway Inlet Structure Details
13-30093-A	S1.1 Service Spillway Inlet Structure Details
13-30094-A	S1.2Service Spillway Inlet Structure Details
13-30095-A	S2.0 Emergency Spillway Retaining Wall Plan
13-30096-A	S2.1 Emergency Spillway Retaining Wall Sections
13-30097-A	Instrumentation Plan
13-30098-A	Erosion and Sedimentation Plan

REFERENCE DRAWINGS	TITLE
13-3004-0	Geologic Profile N-S Axis
13-3005-1	Geologic Profile E-W Axis
13-30040-5	Grading and Drainage Plan
13-30041-6	Profile and Sections
13-30042-3	Sections and Details Sheet 1
13-30043-5	Sections and Details Sheet 2
13-30053-3	Grout Holes Plan and Section
13-32004	Drainage Shaft Masonry & Reinforcement Plans, Sections, & Details
13-30099-A	Boring Plan

1. INTRODUCTION

Cardinal Operating Company, agent for Ohio Power Company and Buckeye Power Company Incorporated, proposes to increase storage capacity of the Fly Ash Retention Pond II (FAR II) at the Cardinal Plant by raising the existing dam crest from 970 feet NGVD (National Geodetic Vertical Datum) to 983 feet NGVD. The storage capacity provided by the existing dam with a crest at 970 feet NGVD is projected to be exhausted in the year 2013. Increasing the dam crest to 983 feet NGVD will provide storage capacity through 2019 (6 more years). The original dam (crest El. 925) was approved by the Ohio Department of Natural Resources, Division of Water on April 30, 1985 (Permit No. 85-147). The dam was subsequently raised beginning in 1997, bringing the crest to El. 970 (present crest elevation), as approved by the Ohio Department of Natural Resources, Division of Water on April 30, 1996, (Permit No. 97-264). The purpose of this report is to summarize the technical design of the dam raising in fulfillment of the requirements of Section 1501:21-5-04 of the Ohio Administrative Code. Cardinal Operating Company previously submitted a Preliminary Design Report for the project in October of 2011, and an earlier version of this Final Design Report in May, 2012, both of which are superseded by this submittal. In addition to this report, a plan package and construction specifications have been prepared which are submitted under separate cover. Please note that, separately, Cardinal Operating Company applied for a wastewater permit from the Ohio Environmental Protection Agency for the modified facility. This permit was issued on August 20, 2012 (Application No. 877122).

1.1 Location and General Description of the Dam

The proposed dam raising site is located in Section 5 of Wells Township, Jefferson County, near the town of Brilliant in eastern Ohio, as shown on the Cover Sheet included as Drawing No. 13-30080-A of the Dam Raising Drawings. The original earth fill dam consisted of a 180 feet high arched earth embankment incorporating a zoned cross section. At 925 feet NGVD, the dam featured a 70-foot wide by 1,055-feet long crest. The maximum operating pool that could be achieved with the original configuration was El. 913. Throughout this document the original dam is referred to as either the 1986 dam or the Stage 1 dam. The present dam, which reflects the modifications associated with the 1997 raising, and as shown on the Record Drawings dated March 31, 2000, is 225 feet high with a 30-foot wide crest. The current dam is referred to as either the present dam or the Stage 2 dam throughout this document.

A cross-section through the highest section of the existing dam is included on reference drawing No. 13-30042-3. As shown on this drawing, the existing dam consists of several zones. These zones, from upstream to downstream, consist of 1)upstream mine spoil structural fill shell, 2)upstream fine bottom ash filter zone, 3)cohesive clay core, 4)mine spoil transition zone, 5)bottom ash chimney drain/filter which transitions into a blanket drain which exits at the toe, and 6)downstream mine spoil structural fill shell. In addition, although not formally a part of the dam, the impounded ash rests on top of the upstream mine spoil shell which influences the stability of the upstream slope and seepage through the dam and abutments. The level of impounded ash is constantly changing; the approximate topography of the impounded ash as determined by bathymetric survey on September 7, 2011 is shown on drawing No. 13-30081. Also shown on the dam cross-section are the features associated with the 1997 raising which incorporated an upstream RCC (cement stabilized bottom ash) block along with extensions of the upstream bottom ash filter, clay core, chimney drain and downstream mine spoil shell. At the completion of the 1997 raising, the upper portion of the entire dam crest (approximately 30 feet wide) consists of a minimum of 9 feet of RCC to both protect the dam from erosion and serve as a roadway. The RCC zone will serve as the foundation for the proposed dam raising.

The current maximum design operating pool is El. 960, although the pool is maintained several feet below this level though the use of a stop log configuration within the service spillway structure. Drawing No. 13-30083-A of the plan package entitled Site Plan shows the as-built topographic data for the current dam configuration. It is proposed to raise the dam crest from 970 feet NGVD to 983 feet NGVD utilizing back-to-back mechanically stabilized earth (MSE) walls constructed on top of the existing crest, as depicted on Drawing No. 13-30087-A. Seepage will be controlled through the use of a vinyl sheet pile wall which will extend from the top of the MSE walls through the existing RCC and into the clay core. The existing service spillway will be raised to bring the maximum design operating pool up to El. 974. Plan section, and details of the service spillway raising are shown on Drawing No. 13-30091-A. The existing emergency spillway will be raised using mass concrete in conjunction with new training walls to safely pass flood events exceeding the 50-year flow. The proposed emergency spillway plan is depicted on Drawing No. 13-30089-A.

Table 1.1.1 Existing And Proposed Characteristics Of Reservoir

Location	Statistic	Existing	Proposed
DAM	Classification	Class I	Class I
	Crest Elevation	970.0	983.0
	Maximum Height	237 feet	250 feet
	Crest Width	30 feet	22 feet
	Emergency Spillway El.	961.0	975.5
	Emergency Spillway Width	110 feet	108 feet
RESERVOIR	Max. Operating Pool El.	960.0	974.0
	Max. Operating Pool Area	138 ac.	161 ac.
	Max. Operating Pool Volume	9,800 ac-ft	11,868 ac-ft
	Emergency Spillway Area	137 ac.	184 ac.
	Emergency Spillway Volume	9,900 ac-ft	12,200 ac-ft
	Top of Dam Area	153 ac.	184 ac.
	Top of Dam Volume	11,350 ac-ft	13,500 ac-ft

In addition to the dam information, Drawing No. 13-30081-A depicts the outline of the reservoir at the proposed maximum operating pool level and at the probable maximum flood (PMF) level, as well as the locations of state, county and township roads; utilities; topography and other pertinent information.

1.2 Classification

The Ohio River, Cardinal Generating Plant, State Route 7 and the Tidd-dale subdivision of Brilliant, OH all lie directly downstream of the proposed dam. A sudden failure of the existing dam at 970 feet NGVD or the proposed dam at 983 feet NGVD will likely result in loss of human life, and damage to homes, high value utility installations and both a railroad and a public road. Also, the existing and proposed dam heights and storage volumes exceed the thresholds for class I dams as established in Section 1502:21-13-01 of the ODNR Administrative Rules. For these reasons, the proposed Earth Fill-RCC dam is classified as a class I dam.

1.3 Purpose of Dam

The purpose of the proposed raising from elevation 970 feet to 983 feet NVGD is to provide for the continued disposal of fly ash coal combustion byproduct produced by the Cardinal Generating Plant. Cardinal Generating Plant has three units rated at 600, 600 and 630 megawatts (MW), respectively, which produce a total of 560,000 cubic yards of fly ash per year. The proposed dam raising will provide for fly ash disposal through the year 2019.

2. PROPOSED DESIGN OF DAM

The proposed dam raising project will involve raising the main dam 13 feet using back-to-back MSE walls. The principal features of the typical section, as shown on Drawing No. 13-30087-A, are the MSE wall themselves and a vinyl sheet pile wall extending from the existing clay core to the top of the PMF flood level for seepage cutoff purposes. Other important features of the dam raising include a raised service (principal) spillway incorporating stoplogs connected to the existing service spillway, and a higher emergency spillway constructed at the location of the existing emergency spillway. These items are discussed further in the following sections.

2.1 Main Dam

The raising of the dam will be accomplished using back-to-back MSE walls to achieve the proposed crest elevation of 983. With this approach, the need for a large amount of fill on the downstream side of the dam is obviated. A new vinyl sheet pile wall will be installed within a slurry stabilized trench to create a continuous seepage barrier extending from the existing clay core to the top of the proposed dam. It should be understood that the sheet pile wall is principally a seepage control feature as opposed to a structural feature. The sheet pile wall will be supported on both sides by the new MSE walls. The MSE walls will generally feature full height precast concrete panels, high density polyethylene (HDPE) geogrid reinforcement, free-draining granular backfill, and concrete coping traffic barriers. The geogrid reinforcement will be imbedded into the precast panels eliminating connection strength issues. The use of geogrid and PVC sheet piles will eliminate corrosion concerns, while burial within the stone will remove UV light degradation concerns. The precast concrete panels will be reinforced with epoxied-coated rebar and will be designed to resist corrosion. As the panels will be manufactured off-site in a controlled setting, the quality should be higher (in terms of proper placement of reinforcement, consolidation of concrete and proper curing) than for equivalent cast-in-place concrete.

At the existing emergency spillway, the top of the sheet pile wall will be terminated at El. 963 within the mass concrete section. To the left of the emergency spillway, the top of the sheet pile wall will terminate within the training wall footing, providing a continuous barrier to seepage. At the right abutment, the back-to-back MSE wall configuration will be turned up the present access road until El. 983 is reached to avoid impacting the existing fly ash service lines. The sheet pile/slurry trench cutoff wall will be extended into intact rock at both abutments to minimize near surface seepage through the overburden.

2.2 Emergency Spillway

As part of this project, the existing emergency spillway will be raised to El. 975.5 through the use of a mass concrete gravity section in conjunction with reinforced concrete training walls, in a manner similar to the existing configuration. The new walls will direct the flow into the existing spillway outlet channel, as shown on Drawing Nos. 13-30083-A and 13-30089-A. A profile of the new emergency spillway is shown on Drawing No. 13-30090-A. In accordance with State of Ohio dam safety requirements for Class 1 dams, the new emergency spillway has been designed to pass the design probable maximum flood (PMF) without overtopping the dam. The new spillway will feature a 108 foot long by 15 foot wide concrete control section positioned at El. 975.5, or 1.5 feet above the maximum operating pool. The training walls will be located above elevation 975.5 and will consequently not be exposed to a continuous pool reducing corrosion concerns. Modified Service Spillway

The existing service spillway consists of a sloping concrete shaft structure with one side opening, four feet wide, connecting into a 54 inch diameter pre-stressed concrete cylinder pipe (PCCP). The bottom of the sloping concrete shaft and the entire 54-inch concrete pipe were constructed within bedrock as part of the 1997 raising. Stop logs are utilized to maintain settling action and control the operating pool level. The existing service spillway will be extended with a new vertical concrete shaft structure. Other than the extension, all other aspects of the spillway will remain the same, with the flow discharging at the base of the dam into the same receiving stream. Stop logs will be incorporated into the new vertical section to continue to allow for the incremental raising of the operating pool.

3. SITE INVESTIGATION

3.1 Regional Geology

The regional geology of Jefferson County is discussed in the design report for Stage 1 construction of the Cardinal Dam II (1). The geological investigation and evaluation conducted for the purposes of constructing the existing dam are still valid. Additionally, an extensive review of the regional geology with special focus on permeable bedrock units is presented in the recently completed Hydrogeologic Study for the project.

3.2 Site Geology and Subsurface Stratigraphy of Natural Ground

Geologic cross sections for the stage 1 dam were constructed from the borings and test trenches at the time the original dam was designed. These sections are shown in Drawing Nos. 13-3004 and 13-3005 which are included as reference drawings to this report.

3.3 Subsurface Investigation

A large amount of historic subsurface data is available at the location of the dam as presented in the 1986 and 1996 design reports. In recognition of this, as part of the current dam raising effort, a subsurface exploration program which focused on filling the gaps in the data with respect to the proposed construction. The Plan of Borings, included as Drawing No. 13-30098-A of the reference drawings, shows the locations of all known explorations performed at the project site, including recently completed work.

3.3.1 Field Work

During June of 2011, 16 borings and 10 test pits were performed in support of the present dam raising focusing on the left and right abutments and potential low spots around the rim of the reservoir. Please note that at the time of 2011 investigation, a higher raising was envisioned necessitating exploration at locations which will not be impacted by the reservoir as part of the current plan. Test pits were extended to depths between 9 and 14 feet, or until the top of the bedrock surface, if encountered first. The borings were extended to depths between 10.9 and 117 feet below existing grades. The boring locations were selected and field located by S&ME by referencing existing site features. Subsequently, AEP surveyed in the exact exploration locations and surface elevations at the boring locations.

In February of 2012, 5 additional borings, 3 of which included cores of the RCC, were performed through the crest of the existing dam. These borings were extended to between 34.5 and 35.5 feet to develop information to assess the strength and permeability of the RCC and the strength of the upper portion of the underlying clay core which will be subject to increased stress under the weight of the MSE walls. At Borings B-1202, B-1203 and B-1205, the RCC was first cored using a diamond-tipped core barrel and conventional concrete coring equipment. At Borings B-1202 and B-1205, a 3-inch core was obtained, while at Boring B-1203, an 8-inch core was obtained.

Finally, on April 3, 2012, , seven additional test pits were completed to confirm the top of rock elevation along the left abutment. The test pit locations were surveyed using GPS equipment.

3.3.2 Exploration Methods

The borings were performed with a track or truck-mounted drill rig and were advanced between sampling attempts using a 3¼-inch I.D. hollow-stem auger. Disturbed, but representative, samples were obtained by lowering a 2-inch O.D. split-barrel sampler to the bottom of the hole and driving it into the soil by blows from a 140-pound hammer freely falling 30 inches (Standard Penetration Test, ASTM D1586). The samples were obtained at continuous intervals until auger refusal was encountered. Split barrel samples were examined immediately after recovery and representative portions of each sample were placed in air tight jars and retained for subsequent laboratory testing.

Upon encountering auger refusal, a changeover was made to rock coring techniques to verify the presence and condition of the bedrock. Bedrock cores were then obtained by using a NQ rock-core barrel with a diamond bit with water as the circulating fluid. Recovered cores were catalogued in the field and preserved in compartmented boxes and delivered to our laboratory for inspection, classification, and testing. The rock coring was performed in accordance with ASTM D 2113.

All borings except B-1101D were backfilled with cement-bentonite grout either at completion or within the same week that drilling was completed. A standpipe was used to ensure the overburden soils did not cave before the grouting occurred in B-1106 and B-1108. B-1101D was drilled without sampling and a standpipe was left in the boring to allow for the use of a downhole camera to view the presence of possible voids encountered in B-1101A, 1101B and 1101C.

Test pits were completed using a backhoe with a 44-inch bucket to expose the near surface stratigraphy. The stratigraphy was identified and logged. Samples of the exposed material were obtained from the pit wall at select locations, excavator bucket or cuttings pile, depending on depth. The samples were preserved in air-tight containers.

3.3.3 Water Pressure (Packer) Testing

Water pressure (packer) tests were completed in Borings B-1108, B-1109 and B-1115 following the completion of rock coring. The packer tests were performed to assess the permeability of the bedrock. The packer tests were completed using a pneumatic double packer setup to test approximate 10-foot intervals. The individual packers were inflated using nitrogen. The tests were run by lowering the packer setup to the desired test interval, inflating the packers, and pumping water into the zone between the packers. After a steady water pressure reading was obtained, the time and water meter readings were noted. A constant pressure was maintained during the test by opening or closing a by-pass valve located between the water pump and the pressure gauge. Tests were generally run for a time period of 3 to 5 minutes at which time a final water meter reading was recorded. Multiple tests were typically run at each depth interval for varying water pressures to allow for the calculation of the permeability values using multiple data sets. The results of these tests were incorporated into the Hydrogeology Report which is presented under separate cover.

3.3.4 Recording of Field Data

In the field, the following procedures and specific duties were performed by personnel from our office:

- examined all samples recovered from the borings;
- cleaned soil samples of cuttings and preserved representative portions in airtight glass jars;
- made seepage observations and measured the water levels in the borings;
- prepared a log of each boring or test pit;
- made hand-penetrometer measurements in soil samples exhibiting cohesion;
- directed the packer testing program after reviewing the rock core samples; and,
- provided liaison between the field personnel and the Project Manager so that the field investigation could be modified in the event that unexpected subsurface conditions were encountered.

At the completion of drilling, all samples were transported to the BBCM laboratory for further examination and testing.

3.3.5 Laboratory Testing

In the laboratory, soil samples were visually identified, with natural moisture content (ASTM D 2216), liquid/plastic limit determinations (S&ME adjustment to ASTM D 4318) and grain-size analyses (ASTM D 422) performed on selected representative specimens of anticipated subgrade soil. Results of these tests permit an evaluation of strength and compressibility characteristics of the soil by comparison with similar soils for which these characteristics have been previously determined. A summary of the laboratory results is presented on Plates 1 through 6 of Appendix B, Atterberg Limits results are presented on Plates 7 through 11, and grain size curves are presented on Plates 12 through 69 of Appendix B. Unconfined compression test results of the RCC core samples are presented as Plates 70 through 77 of Appendix B and permeability test results of the RCC are presented as Plates 78 through 81 of Appendix B. Two consolidated-undrained (CU) triaxial test series, and four unconsolidated-undrained (UU) triaxial tests were completed on samples of the upper portion of the existing clay core. The results of these tests are presented graphically in Appendix B on Plates 73 through 84.

Based upon the results of the laboratory testing program, the field logs were modified, if necessary, and copies of the laboratory-corrected boring logs and test pit logs are submitted as Plates 4 through 34 and 53 through 62 in Appendix A, respectively for the 2011 work and on Plates 71 through 80, and 85 through 91 for the 2012 work. In addition to the logs, rock core photographs and field test pit photographs are included as Plates 35 through 52 and Plates 62 through 70 of Appendix A, respectively. Photographs of the RCC cores are included as Plates 81 through 84 of Appendix A, and a table summarizing the unconfined compression tests is included below. Note unconfined compression tests were only performed on the vertical cores.

Table 3.3.5 Summary of Unconfined Compressive Strength Tests Performed on RCC Cores

Boring	Sample No.	Approximate Specimen Depth (feet)	Unconfined Compressive Strength (psi)
B-1202	C-2	0.5	1516
B-1202	C-4	1.3	705
B-1202	C-9	3.4	1211
B-1202	C-14	5.9	1264
B-1205	C-1B	0.5	595
B-1205	C-4	2.4	1411
B-1205	C-8	5.6	1602
B-1205	C-12	7.7	1127
			Avg. 1179

Shown on the boring logs are: descriptions of the soil stratigraphy encountered; depths from which samples were preserved; sampling effort (blow-counts) required to obtain the specimens in the borings; N_{60} values; seepage and groundwater observations; and, values of hand-penetrometer measurements made in soil samples exhibiting cohesion. For your reference, hand-penetrometer values are roughly equivalent to the unconfined compressive strength of the cohesive fraction of the soil sample. An explanation of the symbols and terms used on the boring logs, and definitions of the special adjectives used to denote the minor soil and rock components are presented as Plates 1 and 2 of Appendix A of this submission. Additionally, test pit exploration procedures and symbols and terms used on the test pit logs are presented on Plate 3 of Appendix A.

4. HYDROLOGIC AND HYDRAULIC ANALYSIS

4.1 Introduction

The existing hydrologic conditions at the proposed dam site are described herein. Blockhouse Run, the major drainage feature in the project area, drains directly into the Ohio River. Approximately one mile upstream of the Ohio River, Blockhouse Run splits into two branches, designated as the East Branch and the West Branch.

The East Branch drains the eastern watershed as delineated in the Watershed Map on Plate 2 of Appendix C. The active fly ash dam II inundates the East Branch. The West Branch has been dammed to form the old Fly Ash Reservoir I (FAR I).

The location of the dam is shown on the drawings. Extension of the dam will inundate approximately 161 acres, or 24 percent of the area in the eastern watershed. Since the location of the dam is situated downstream of the discharge points of the old dam, runoff from the western watershed drains into the existing reservoir. Therefore, the spillway system of the proposed dam raising has been designed to meet ODNR Class I design criteria based on the runoff from both watersheds. The following sections present the hydrologic considerations and analyses performed during the design phase of this project.

4.2 Basin Characteristics

Figure 3.1 shows the limits of the watershed boundary for the existing Fly Ash Reservoir II(FAR II). The total drainage area above the dam has been divided into two watersheds, East and West, for analysis of the storm runoff entering the reservoir, as shown on Plate 2 of Appendix C.

A review of available topographic maps and aerial photos was made to determine essential basin characteristics for each watershed. Such characteristics include the drainage boundaries, areas, slopes, soil types, ground cover, land use and the 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.

The old fly ash dam is located in the western watershed. Present land use within the drainage area is limited to reclaimed strip mine areas, some woodlands, and the inactive FAR I. Reclamation of the reservoir area is actively in progress in the form of a residual waste landfill above the level of the ponded fly ash. A built-out landfill condition was also analyzed for the western watershed, using the 2005 FAR I PTI. The PTI listed a Curve Number (CN) of 74, therefore the composite CN of the current FAR I condition of 75 was used. See Plates 4 through 6 of Appendix C.

Woodlands and scattered reclaimed strip mines constitute the existing land use in the East watershed. Construction of the proposed fly ash dam raising will inundate approximately 161 acres at Elevation 974.0 feet NGVD, the maximum operating pool elevation.

Soil types in the areas have been identified by the Soil Conservation Service (SCS) of the U.S. Department of Agriculture and classified into hydrologic soil groups. Within the study area, all soils fall under the hydrologic soil group B. Table 4.2.1, below, lists the basin characteristics for the Western and Eastern watersheds.

Table 4.2.1 Basin Characteristics

BASIN CHARACTERISTICS	WATERSHED			
	WEST		EAST	
	Woods	**Landfill**	**Woods**	**Reservoir**
Drainage area (acres)	519	158	514	161
Average land slope %	30	n/a	25	n/a
Hydrologic soil group	C	C	C	n/a
SCS curve number (CN)	70	91	70	100
Composite CN	75		n/a	
Time of concentration (hours)	0.87		0.57	0.1
TOTAL AREA (acres)	677		675	

4.3 Characteristics of Proposed Reservoir

A previously referenced, Drawing No. 2 shows the location of the existing dam. Based on this layout, the reservoir will have the following surface areas and storage capacities - as shown below in Table 4.3.1.

Table 4.3.1 Surface Areas and Storage Capacities

ELEVATION	(Ft. NGVD)	AREA (AC)	STORAGE (AC-FT)
Maximum Pool	974.0	161	11,868
Emerg. Spillway	975.5	165	12,200
Top of Dam	983.0	184	13,500

The area-capacity-elevation curve developed for this dam is shown on Plate 3 of Appendix C.

4.4 Design and Assumptions

Rainfall - runoff data was not available for the site because the streams flow intermittently. Therefore, runoff hydrographs were generated using the U.S. Army Corps of Engineers HEC-1 computer program. The SCS dimensionless unit hydrograph method was employed in the calculation of the hydrographs. For each watershed, separate runoff hydrographs were computed and then later combined to form a single inflow hydrograph for the proposed reservoir.

Runoff from the West watershed was analyzed based on current landfill construction activity. The landfill area was assumed to be in a disturbed (unvegetated) condition. A composite curve number was used to represent the unvegetated landfill and surrounding wooded areas. This is shown on Plate 4 of Appendix C.

In the East watershed, the reservoir surface is modeled as a subbasin to convert direct rainfall into a runoff hydrograph. The ash sluice water of 13.3 mgd (20.6 cfs) is represented as a base flow in the East watershed.

Once computed, the runoff hydrographs from the three subbasin watersheds are combined and routed through the reservoir.

4.4.1 Service Spillway

According to OAC 1501:21-13-04, design of the principal (service) spillway for class I dams must be such that the average frequency use of the emergency spillway is predicted to be less than once in fifty years. The estimated precipitation for a 50-year storm was obtained from the NOAA Atlas 14. For a 6-hour storm, the precipitation is 3.43 inches, whereas the 24-hour storm amount is 4.51 inches, as shown on Plate 9 of Appendix C.

Both 6-hour and 24-hour storm durations with average soil moisture conditions were checked. The 24-hour storm resulted in a higher maximum water surface, therefore this storm duration was used for developing the 50-year storm inflow hydrograph.

4.4.2 Emergency Spillway

OAC 1501:21-13-02 specifies that for class I dams, the spillway system shall safely pass the design flood equal to the probable maximum flood (PMF) without any overtopping of the dam. The PMF is the result of the probable maximum precipitation (PMP), 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. Generalized estimates of the PMP have been published by the

Hydrometerological branch of the National Weather Service, as shown on Plates 11 and 12 of Appendix C. For the study area, a 6-hour PMP of 26.5 inches was used as the design rainfall event. The antecedent moisture conditions of the soil cover were assumed to be average.

The layout of the control section and outlet channel for the emergency spillway is shown on the Emergency Spillway Plan.

The emergency spillway control section will be a section of mass concrete at Elevation 975.5. It will have a bottom length of 108 feet and side slopes consisting of access ramps at 2 to 15% grades. Downstream of the access ramps and control section, vertical concrete retaining walls wrap into the spillway and guide flow down the channel. The width of the control section along the flow direction will be 15 feet. The downstream channel of the spillway will be stepped. Steps will be formed of the mass concrete beginning at the downstream end of the control section and tying-in to the existing RCC steps. The calculations show that flow downstream of the control section becomes supercritical. The spillway channel transitions from an approximate 3.5H:2V slope along the proposed concrete steps to a 5H:2V slope along the existing RCC steps..

4.5 Analysis

All reservoir flood routings were conducted using the HEC-1 computer program. The program routes floods through the reservoir by the modified Puls method. In general, reservoir storage data and either spillway dimensions or discharge-rating curves are supplied by the user.

4.5.1 Service Spillway

Analysis of the service spillway system consisted of routing the 50-year storm to establish the invert of the emergency spillway. A design for the service spillway was determined and a stage-discharge curve was computed. A maximum operating level of elevation 974 was predetermined based on the projected life of the dam raising. Reservoir routings of the 50-year storm were performed using the maximum operating level of the reservoir.

Inflow was calculated as weir flow over the 4-foot stop log. Above Elevation 976, flow will enter through the top of the vertical service spillway structure. This flow was analyzed as both weir and orifice flow. Rating calculations for the service spillway are included on Plates 13 through 19 of Appendix C.

4.5.2 Emergency Spillway

Hydrologic reservoir routings were conducted to analyze the emergency spillway and its ability to pass the probable maximum flood without overtopping the dam. A flat rectangular control section was designed with a width of 15 feet and length of 108 feet. Discharge over the spillway was rated based on calculations of critical depth using the Corps of Engineers HEC-RAS computer program. Cross sections were taken at changes in geometry, slope or surface roughness. Manning's n roughness coefficients were input based on the expected channel surface conditions. Based on literature (see Plates 40 through 42 of Appendix C), a relatively high Manning’s roughness coefficient of n=0.07 was used to model the stepped spillway surface. As shown on the drawings, proposed reinforced concrete training walls extend from the crest of the dam to a point approximately 3 feet beyond the proposed stepped channel transitions into the existing steps. Downstream from the training walls section, the spillway width becomes 110 feet, consistent with the current configuration.

The calculated relationship between stage and discharge was then used in the routing process to determine the maximum discharge and pool elevation. This information was used as the emergency spillway rating and input into HEC-1.

Discharges from the emergency spillway are routed away from the dam through an existing outlet channel.

4.6 Results

4.6.1 Service Spillway-Hydraulic Capacity

The proposed new principal spillway is a vertical concrete shaft structure with a 4-foot wide opening on one side. The spillway shaft will tie into the existing inclined spillway structure. The existing structure drains into a 54-inch diameter Prestressed Concrete Cylinder Pipe (P.C.C.P.), which then ties into a 42-inch steel pipe extending down the dam. The existing energy dissipator at the outlet of the steel pipe will be utilized. During most of the operating conditions, discharge through the service spillway will be controlled by weir flow over the stop logs in the opening of the shaft. The maximum operating level is set at elevation 974.0 feet. This corresponds to a maximum stop log elevation of 972.5 based on the base inflow of 20.6 cfs.

The peak inflow during the 50-year, 24-hour storm is 486 cfs, which results from 4.51 inches of rainfall according to NOAA Atlas 14. The reservoir level will rise to elevation 975.2 feet based on an initial pool level of elevation 974. The peak outflow from the dam will be 58 cfs. The HEC-1 output for the reservoir routings are contained on Plates 44 through 75 of Appendix C.

4.6.2 Service Spillway-Structural Capacity

The 54-inch P.C.C.P. portion of the service spillway was also analyzed for additional internal and external pressures due to the 13-foot dam raising. The pipe is installed under the dam embankment and was trenched into bedrock. Pipe crushing calculations were performed to analyze the additional loading on the pipe from the raised dam. Previous calculations (see 2000 As-Built Drawing No. 13-30043-5) indicate that the pipe was designed to handle 80 feet of overburden material at 125 pcf. The proposed top of dam will be 74.6 feet above the pipe, therefore the existing concrete pipe will be suitable to handle the additional load. Additional information on as-built drawing 13-30043-5 also indicates that the pipe is capable of handling internal pressure up to 35 psi. It is possible that at high headwater elevations, the spillway pipe could become pressurized. Under the maximum pool elevation of 983.0, the maximum static head on the downstream portion of the pipe would be 80.5 feet, or 34.9 psi. As the water will be flowing through the pipe, the actual pressure on the pipe will be less than this value; therefore the pressure should not exceed the pipe rating of 36 psi. See Plates 20 and 21 of Appendix C.

4.6.3 Emergency Spillway

The development of the PMF hydrograph indicates a peak inflow to the reservoir equal to 16,329 cfs. This value represents the combined hydrographs from the West and East watersheds. Values of the runoff from each watershed and the combined runoff are shown in Appendix C.

Based on the flood routing, the calculated peak discharge from the dam is 5,409 cfs at a maximum pool elevation of 981.9 feet NGVD. The PMF routing was also checked with the service spillway blocked, which resulted in a maximum pool elevation of 982.8 and 0.2 feet of freeboard.

Both 6-hour and 24-hour storm durations were checked. The 6-hour storm resulted in a higher maximum water surface, therefore this storm duration was used for developing the PMF inflow hydrograph.

Depth of flow in the spillway was determined based on the HEC-RAS analysis. In the proposed spillway section, the training walls were kept a minimum of 1 foot above the critical water surface depth of 4.5 feet, as shown on Plates 23 and 32 of Appendix C. The training wall height downstream of the steps transition was kept to a minimum of 1 foot above the resultant water surface depth during the PMF event (2 to 2.5 feet). The existing wall height of 4 feet meets this requirement. The HEC-RAS output is presented as Plates 25 through 36 of Appendix C. The structural analysis of the raised emergency spillway is presented elsewhere in this report.

4.7 Summary and Conclusions

The hydrologic/hydraulic studies for the proposed dam raising included estimating the PMF and 50-year flood hydrographs and designing the emergency and service spillways. The U.S. Army Corps of Engineers computer programs HEC-1 and HEC-RAS were used in the analyses. The Hydrograph presented on Plate 43 of Appendix C displays the resultant inflow and outflow hydrographs from HEC-1 based on the PMF event. Table 4.7.1, gives a complete summary of the study.

The proposed spillway system has enough capacity to pass the probable maximum flood without overtopping the dam. The water discharged through the emergency spillway is directed away from the dam such that it causes no threat to the stability of the structure.

Table 4.7.1 Hydrologic/Hydraulic Summary for Proposed Raising Of Dam

HYDROLOGIC AND HYDRAULIC SUMMARY		
Drainage Area	AREA (AC)	2.2 Sq. Mi.
Design Floods (Inflow)		
	PMF Peak	16,329 cfs
	50-Yr Peak	547 cfs
Peak Discharge		
	PMF	5,409 cfs
	50-Yr	58 cfs
Maximum Pool Elevations, NVGD		
	PMF	981.9 ft
	50-Yr	975.2 ft
Emergency Spillway - Overflow Control Section - Concrete		
	Crest Elevation, NGVD	975.5 ft
	Bottom Width	105.0 ft
	Side Slopes	Vertical
Service Spillway - Size		
	Top of Vertical Concrete Structure	976.0 ft
	Stop Log Width	4.0 ft
	Conduit Size	54" & 42"
	Maximum Operating Pool Level, NGVD	974.0 ft

5. GEOTECHNICAL DESIGN

Performance of the existing dam, geotechnical aspects incorporated in its design, and applicability of these aspects in the design of a portion of the dam proposed for raising are discussed in the following sections.

5.1 Performance of the Existing Dam Embankment

Overall, the existing dam has performed well since it was originally put into service in the mid-1980s. This having been said, a few incidents have occurred which were investigated and resolved as discussed in the following sections.

5.1.1 Specific Issues Which Have Been Evaluated and Corrected

During the 1997 dam raising, an apparent undrained slope failure took place in late October within the downstream mine spoil zone near the toe of the slope as fill was being placed. At the time of the slope failure, the downstream mines spoil shell had been constructed up to El. 900. The slope failure exhibited a head scarp at roughly El. 827. Construction was halted and the failure was investigated. The investigation suggested that the failure resulted from pore pressure build up within the newly placed cohesive mine spoil soils due to an accelerated placement rate in conjunction with above optimum moisture contents. The failure was remediated by removing the majority of the slide mass in conjunction with the construction of a large rock fill toe drain/berm. The toe berm is shown on the as-built drawings, dated March 31, 2000, from the 1997 dam raising which were submitted to ODNR.

Subsequent to the completion of the 1997 dam raising, a number of cracks were observed within the RCC section. It was believed that these cracks were related to differential settlement along the crest as the amount of fill above bedrock varied in thickness as well as related to shrinkage of the RCC mass during curing. The RCC mix design and thermal gradients through the RCC zone were also considered attributing factors to the cracking. These cracks are described in a report entitled *Cracks in RCC Zone and Post-Construction Performance of Dam* , dated June 1, 1999. Subsequent monitoring of the cracks suggested that no further significant movement had taken place and the cracks were sealed.

In February of 2004, significant seepage emanating midway up the right downstream groin was observed. This seepage carried ash with it leading to a concern for a potential piping failure. The seepage was assessed and was believed to have occurred through the jointed right abutment bedrock. Laboratory analysis (grain size, mineralogical, and X-ray diffraction) of the seepage confirmed that it was carrying fly ash and not material from the dam itself. The seepage was initiated only after the water level in the reservoir reached the level of the more permeable sandstone bedrock layer. This layer was exposed within the reservoir due to an overburden landslide which had occurred near upstream of the right upstream in 1984, prior to the construction of the Stage 1 dam. Over time this seepage has been reduced and no longer carries ash resulting from a hypothesized self-healing process. This issue was described in detail in report prepared by AEP submitted to ODNR in 2004, as well as in the peer reviewed paper Amaya, Massey-Norton and Stark (2009). Subsequent monitoring of the right abutment downstream groin seepage has indicated that the seepage from the right abutment is staying relatively constant.

5.1.2 Annual Inspection Program

Quarterly inspection of the existing dam and appurtenant structures is conducted by plant personnel. The personnel of the American Electric Power Service Corporation (AEPSC) perform yearly inspections to review overall performance of the dam and to ensure that the remedial work has been completed in accordance with recommendations resulting from previous inspections. An inspection on a less frequent basis is conducted by an outside independent consultant. The purpose of this inspection is to re-inspect the dam, to review the results of periodic inspections conducted by the plant and AEPSC personnel, and to provide an independent assessment of the current status of the dam. The inspection reports are submitted to ODNR, with the most recent dated November 8, 2011. The results of a dam safety and performance inspection are summarized below:

Geotechnical design criteria used in the design of the existing dam, based on its performance since construction to present, seem to be satisfactory. The long term movement of the dam as established by deformation survey appears to be within expected range for the earth fill dam and generally appear to be slowing, implying a secondary compression response.

The seepage emanating from the dam is continuously monitored both in magnitude and for the presence of eroded materials. Since the significant seepage outbreak which in 2004 (previously discussed), the quantity of seepage has reduced and has remained constant. As shown in the November, 2011 inspection report, despite a pool rise of roughly 25 feet since 2004, the discharge from the right abutment seepage has held steady. This data should be compared to the planned increase in the operating pool of 14 feet.

Piezometers were installed in various zones of the dam during its construction. Readings for the piezometers show a trend which appears to indicate a normal condition reflective of post construction period and an increase in the reservoir level, as detailed in the annual inspection reports.

In conclusion, the dam appears to be performing satisfactorily.

5.2 Geologic Assumptions for Raised Dam

Geologic conditions considered in the design of the raised portion of the dam include the site geologic features and rock types that may adversely affect the stability and performance of the raised dam. Major geologic assumptions addressed here were evaluated and incorporated in the design of the existing dam. These assumptions included evaluations of geologic structures at the site such as rock type, faults, joints, shear zones and bedding planes. Presence of clay seams in the foundation rock was also investigated as a potential instability condition. It was concluded that continuous clay seams are not likely to exist in the foundation rock at the site.

5.3 Borrow Areas

The proposed dam raising exclusively involves the use of imported and manufactured materials, and as such no on-site borrow will be used as part of the project. Backfill between the MSE walls will consist of crushed limestone imported from off-site. A minimal amount of compacted granular material will be placed behind the left abutment training wall, which will consist of crushed limestone obtained from the on-site stockpile.

5.4 Dam Raising Scheme with Respect to Geotechnical Issues

The back-to-back MSE wall solution was specifically developed to avoid the need for the placement of a large amount of downstream fill and the associated large stress increase and corresponding risk of slope failure. Additionally, MSE walls are flexible and can accommodate the anticipated differential settlement as the foundation for the walls transition from bedrock at the abutments to as much as 216 feet of cohesive embankment fill at the dam high point. With this solution, seepage is controlled by positively connecting the existing clay core with the top of the dam through the use of a sheet pile wall. To minimize seepage further, the joints between the sheet piles will be treated with a sealant prior to driving and the lower portion of the wall will be imbedded within the cement-bentonite slurry wall. The exposed portion of the driven sheet piles will be further sealed post-installation with caulk on the upstream side. Compaction of the MSE wall backfill will be simplified through the use of free draining granular materials which are moisture insensitive.

Geotechnical instrumentation already in place will be maintained as possible. Destroyed points will be replaced whenever possible or new ones will be installed in the proximity of a destroyed one. The existing and proposed deformation monuments are shown on Drawing No. xxxxx. The existing piezometers are shown in section view on Record Drawing No. 13-30042-4, Section 3-3. Proposed instrumentation will consist of sets of upstream and downstream tilt meters (single axis) affixed to the top of the MSE wall panels (not the coping) in conjunction with survey monuments. This instrumentation will provide an excellent indication of any movement in the dam, whether rotational or translational. This information, in conjunction with the close monitoring of the seepage from the downstream drains will provide the operator with sufficient information to understand the performance of the raised dam. Furthermore, as the pool level will be raised incrementally, the instrumentation will provide an early warning of any deviations from the expected performance, when the head is low relative to the raised portion of the dam, mitigating the potential for a catastrophic failure.

5.5 General Discussion of Foundation for Raised Dam

The foundation for the raised portion of the dam will generally consist of the existing RCC crest with the exception of either abutment where the raised section will be supported on natural ground. The foundation for the main dam was extensively investigated as part of the Stage 1 design report. During the construction of the original dam, beneath the clay core, all surficial soils were removed and the core was extended into intact bedrock. As the core transitioned up the abutments, vertical cuts were made into the hillside to reach intact rock to promote good contact. The bedrock beneath the abutments was grouted during the original dam construction and again during the 1997 dam raising. Where the dam raising will extend beyond the present crest at the abutments, it is planned to remove the majority of the overlying soil (generally less than 8 feet thick). The seepage cutoff trench will be extended into rock at both abutments. In the following sections, the foundation requirements related to the raising of the dam are discussed in more detail.

5.6 Abutment Seepage Under Raised Pool

As part of the design of the dam raising, the potential for increased seepage through the abutments was considered. As previously indicated, a significant seepage event through the right abutment bedrock occurred in 2004. However, since this event, the quantity of seepage has stabilized. This is shown in the November, 2011 inspection report, where despite a pool rise of roughly 25 feet since 2004, the discharge from the right abutment seepage has held steady. This data should be compared to the planned increase in pool associated with the dam raising of 14 feet. The reason for this apparent discrepancy is likely related to the fact that the ash is generally discharged at the rear of the reservoir, such that the only the finest particles make their way to the dam and abutments. For this reason, as the level of impounded ash increases, it tends to reduce the permeability of the abutment walls from what would be anticipated if the reservoir only contained water. In addition to this observation, it should be noted that the abutments which will be impacted by the proposed raising were grouted as part of the 1997 raising, as shown on Reference Dwg. No. 13-30053-3. For these reasons, it is not planned to perform additional abutment grouting during construction of the raising.

Although an abutment grouting program is not planned in conjunction with the construction of the dam raising, future abutment grouting has not been ruled out. In the case of the right abutment, the flow rate through the existing, monitored, seeps will continue to be measured and recorded. In contrast to the right abutment, no seeps have been identified at the left abutment, likely due to the greater width of the ridge at this location. It is also recognized that as the downstream side of the left abutment (beyond the emergency spillway channel) is covered with trees and brush, it is difficult to closely observe the slope at present. To this end, as part of the maintenance plan for the dam raising, the upper portion of the slope will be cleared of brush and small trees (although larger tress will not be remove), and the surface stabilized with grass. The slope will then be routinely walked looking for signs of seepage or distinct seeps as part of the regular dam inspection program. If a seep is found, an attempt will be made to monitor the flow rate and a summary of the issue will be presented to ODNR for discussion. Depending upon the severity and the change with time, an abutment grouting program may be developed. The specific abutment grouting, including number, configuration and depth of the grout holes would be developed after the seep has been identified.

5.7 Foundation Preparation Considerations

5.7.1 Existing Dam

The majority of the raised dam is supported on the top of the existing dam. With the exception of the sheet pile cutoff, the new MSE walls and backfill will be placed directly on the existing RCC surface. Surface preparation will consist of the removal of loose/weathered RCC material. As previously indicated, 5 borings were performed to investigate the condition of the existing RCC and the strength of the underlying cohesive clay core. These borings indicate that beneath the weathered surface layer the RCC is in good condition exhibiting minimal cracking, as evidenced by the recovered large diameter core sample.

5.7.2 Abutments

The widened abutments will be founded on bedrock. The surface of the foundation rock will be cleaned and loose pieces of the rock will be removed prior to the installation of the MSE and cast-in-place retaining walls (left abutment only). The sheet pile cutoff wall will be extended into intact bedrock cutting off near surface seepage. As previously indicated, additional bedrock grouting is planned as part of the dam raising contract, but may be considered in the future based on the observation of the abutment slopes.

6. GEOTECHNICAL ANALYSIS

In support of the design, calculations and analyses were performed to assess the suitability of the proposed configuration, including global stability, MSE wall internal and external stability, steady state seepage and settlement. The targeted factors of safety (where applicable) used for design for these various failure modes were based on US Army Corps of Engineers requirements. When evaluating these analyses and the computed factors of safety, it should also be recognized that the pool level will be raised incrementally over a period of 6 years through the use of stop logs. Because of this, the performance of the dam can be observed prior to the most severe loading and unanticipated responses can be addressed under lower heads than the full design value. The calculations and the results are summarized in the following sections of this report. The analyses themselves (computer output where applicable), including parameter selection rational, are presented within Appendix D.

6.1 Discussion of Anticipated Foundation Behavior

As previously indicated, the foundation for the MSE walls will consist of the existing RCC section. As shown on Reference Drawing No. 13-30042-3 which depicts the typical configuration for the main portion of the dam, the RCC zone is approximately 50 feet thick on the upstream side and 9 feet thick on the downstream side. For this reason, the upstream RCC zone effectively functions as a block while the downstream RCC zone functions more like a thick protective cover. The point between the two zones is the location of the cutoff wall which will be physically severed as part of the raising.

Based on the above understanding, the upstream and downstream MSE walls were evaluated separately in terms of global stability, sliding and bearing pressure. It is believed that the underlying 9-foot thick section will tend to function as a spread footing for the downstream wall with respect to sliding and bearing capacity. For this reason, external sliding stability was evaluated at both the MSE wall/RCC interface as well as at the underlying RCC/clay core interface. Likewise, the upstream wall was evaluated both at the MSE wall/RCC interface and considering the MSE wall and RCC block as a single unit.

A large number of potential global slope stability cases were evaluated in an attempt to assess all possible failure modes. Owing to the complex geometry of the MSE walls, in some cases the walls were modeled as a simple surcharge load to reduce the potential for computational difficulties (the software not analyzing what it appears to be). For this reason, there are some variations in the computed factors of safety. Finally, the RCC zones were modeled both as monolithic blocks as well as smaller blocks recognizing the existing cracks. This is discussed in more detail in Section 6.4.

6.2 Seepage Analysis of Main Dam

A seepage analysis was performed with the aid of the computer program Slide™ (Version 5.0) developed by Rocscience, Inc. The program utilizes the finite element method to perform steady-state unsaturated groundwater analysis. The cross section developed for the analysis was provided by AEP and modified to reflect the proposed raising. The cross section reflects the section through the highest point of the dam. The seepage analysis was performed in conformance with the US Army Corps of Engineers Manual 1110-2-1901 entitled *Seepage Analysis and Control for Dams*.

Coefficients of permeability (k) for the various embankment and foundation materials developed by AEP and their consultants for the Stage 1 and Stage 2 Dam design reports were used for the present analysis. New material zones include the reinforced zone (No. 57 stone), ODOT Item 304 surface course, the cement-bentonite slurry wall, and the vinyl sheet pile wall. A summary of the parameters are shown in the following table.

Table 6.2.1: Summary of Coefficients of Permeability Used in Seepage Analysis

Material	Vertical Coefficient of Permeability, k_v (ft/min)	K_h/k_v
Fly Ash	2×10^{-5}	9
RCC	2×10^{-5}	9
Mine Spoil - Upstream Shell (Zone IV)	4×10^{-7}	9
Clay Core (Zone I)	4×10^{-8}	9
Transition Zone (Zone II)	4×10^{-8}	9
Mine Spoil - Downstream Shell - Stage 1 & 2 (Zone IV)	2×10^{-4}	9
Bottom Ash Filter Zone	2×10^{-4}	1
Chimney Drain (Zone III A)	1×10^{-1}	9
Blanket Drain (Zone III B/C	1×10^{-1}	9
Brown Clay	4×10^{-8}	9
Rock Toe Buttress	1×10^{-2}	9
Overburden	2×10^{-6}	1
Claystone	1×10^{-10}	1
Shale	1×10^{-10}	1
Reinforced Zone - Coarse Aggregate	1×10^{-1}	1
Item 304 Aggregate	2×10^{-4}	1
Vinyl Sheet Pile Wall	2×10^{-10}	1
Cement-Bentonite Cutoff Wall	2×10^{-5}	1

The main purpose of this analysis was to look at the impact of the higher pool on the existing chimney and toe drain system. In addition to the computer analysis, the phreatic surface was considered as measured in the field in the many piezometers. The piezometer data suggests that the phreatic surface is well controlled with the existing chimney drain.

6.2.1 Hydraulic Boundary Conditions

The following boundary conditions were assigned to the finite element based models.

- A 'Constant Head' boundary set at El. 974 was used to represent the level of water in the fly ash reservoir at the proposed maximum operating pool. The upstream end of the model was truncated and also assigned a 'Constant Head' boundary at El. 974.
- The model was terminated approximately 20 feet below the top of bedrock. No boundary conditions were assigned to the bottom boundary indicating a 'No Flow' condition.
- The downstream MSE wall was removed and a A 'No-Flow' boundary was placed on the downstream side of the sheet pile cut-off wall as the permeability of the wall is sufficiently low to assume no flow across this boundary.
- 'Unknown' boundary conditions were set on the remainder of the model to allow the program freedom to calculate values at these locations. These locations include the downstream slope face and the downstream ground surface.

6.2.2 Finite Element Discretization and Mesh

The following steps were performed during the development of the seepage model:

- 6 Noded Triangles were used to generate the finite element mesh for the models.
- The density of nodes was manually increased to minimize the number of ‘Poor Quality Elements’ based on the Mesh Quality function available in Slide.
- Poor quality elements were defined as elements with one of the following characteristics:
    1. Maximum side length to minimum side length ratio greater than 10.
    2. Minimum interior angle less than 20 degrees.
    3. Maximum interior angle greater than 120 degrees.

6.2.3 Seepage Analysis Models and Results

Prior to conducting the seepage analysis for the dam raising, an analysis was performed for the existing geometry of the dam and the results from the most recent piezometer readings have been superimposed on the graphical output for comparison to the computer generated results. The results indicate the finite element based seepage analysis appears to correspond with the actual phreatic surface measured in the field for the existing conditions. Graphical output from the seepage analyses for the existing and proposed geometries are presented in Appendix D-1

6.3 Internal and External MSE Wall Analyses (Excluding Global Stability)

It is planned to use MSE walls constructed with Tensar's ARES panel wall system. This system, which is approved by ODOT for the support of highways, consists of full height precast concrete panels with the geogrid reinforcement directly cast into the panel eliminating connection strength issues. Additionally, this system avoids the use of metallic reinforcement, eliminating corrosion concerns. Consistent with ODOT practice, the final design of the geogrid strength and spacing, along with the detail design of the precast panels, will be performed by Tensar working for the contractor, as described in detail in the MSE wall specification. However, to verify the suitability of the system for this project, S&ME performed internal and external analyses which are presented in Appendix D-2 of this report. Internal analyses consisted of examining connection strength and the strength of the geogrid. External analyses consisted of sliding, eccentricity, bearing capacity, and deep seated stability, the latter of which was examined using 2-D slope stability analysis as discussed in Section 6.3. These analyses are based on the use of free-draining, angular granular backfill and the presence of a live surcharge loading due to truck traffic.

The internal stability of the MSE walls was assessed using the MSEW v3.0 design software manufactured by ADAMA Engineering, Inc., in general accordance with AASHTO criteria as presented in the publication FHWA-NHI-10-043 S&ME examined two different loading conditions. The first is the typical long term condition with the water level at the maximum pool elevation. The principal question for this case is the earth pressure acting on the back of the reinforced zone associated with the other wall. Note that no structural resistance is assumed to be provided by the sheet pile wall such that any pressure acting on the sheet pile will be transmitted into the opposite MSE wall. For the second case, the stability of the downstream wall was examined under the weight of the impounded water at the maximum flood pool. As the sheet pile wall will be nearly water tight, particularly for the short term flood condition, and that the free-draining granular backfill within the reinforced zone of the upstream wall will not appreciable cause any head loss, the full hydrostatic pressure was assumed to act on the back of the downstream wall.

As previously indicated, the external stability of the MSE walls (bearing capacity and sliding type failures) was assessed in different ways for the two walls in consideration of the differing thicknesses of the underlying RCC. For the bearing capacity analyses, the downstream RCC section was assumed to act as a spread footing reducing the loading intensity on the underlying clay core. Hand calculations were performed to assess the factor of safety for bearing capacity for the end of construction and long term loading conditions. Two-dimensional slope stability analyses were performed to examine the bearing capacity for the upstream wall/RCC section. The results of these analyses are included in Appendix D-2. Table 6.3.1 summarizes the lowest factors of safety obtained for each loading condition.

Table 6.3.1: Summary of Upstream MSE Wall Analyses (Other than Global Stability)

Analysis Condition	Minimum Requirement	End of Construction	Max Operating Pool	PMF Pool
Direct Sliding on Reinforcement	FS = 1.5	2.48†	2.11†	1.58†
Direct Sliding at MSE Wall/RCC Interface	FS = 1.5	3.51†	2.87†	2.17†
Eccentricity	E / L < $^1/_6$	0.115	0.139	0.088
Overturning	FS = 1.5	4.35	1.63	2.78
Geogrid Strength*	FS = 1.5	1.58	1.81	4.16

† Factor of safety does not reflect reduction in lateral earth pressure due to back-to-back wall geometry
***For design purposes only-final strength by contractor's engineer**

Table 6.3.1: Summary of Downstream MSE Wall Analyses (Other than Global Stability)

Analysis Condition	Minimum Requirement	End of Construction	Max Operating Pool	PMF Pool
Direct Sliding on Reinforcement	FS = 1.5	2.48†	2.48	1.4
Direct Sliding at MSE Wall/RCC Interface	FS = 1.5	3.51†	3.51	2.23
Eccentricity	E / L < $^1/_6$	0.088	0.115	-
Overturning	FS = 1.5	4.35	1.63	2.78
Geogrid Strength*	FS = 1.5	1.58	1.58	1.58
Bearing Capacity of MSE/RCC Mass	FS = 2.0	FS = 4.1 (Drained Analysis) FS = 5.4 (Undrained Analysis)		

†Factor of safety does not reflect reduction in lateral earth pressure due to back-to-back wall geometry
***For design purposes only-final strength by contractor's engineer**

An additional analysis was performed to determine the factor of safety for direct sliding with the downstream MSE wall and RCC foundation sliding over the clay core / mine spoil foundation. The factor of safety for this scenario during the PMF was 1.4, meeting the requirements for the Maximum Surcharge Loading condition.

6.4 Global Slope Stability Analysis

Embankment dams should exhibit adequate factors of safety against a slope stability failure for static, seismic and other loading conditions. As part of this project, S&ME focused on evaluating the cross-section through the high point of the dam. Additional slope stability runs were performed for the section through the existing emergency spillway. The following sections of this report discuss the analyses that were performed, explain the rational supporting parameter selection, and present the results.

6.4.1 Methodology

Two dimensional slope stability analyses were performed with the aid of the computer program SLIDE™ (Version 6.0) utilizing Spencer's limit equilibrium method (Spencer, 1973) with a deterministic approach. Graphical output was generated for each model examined displaying the critical slip surface corresponding to the lowest factor-of-safety with colored contours of the other factors of safety included. In some cases, an additional failure surface was shown where it was deemed the minimum factor of safety did not clearly depict the overall embankment stability. The analyses were performed under End of Construction, Long Term (Static), Rapid Drawdown, and seismic loading conditions in conformance with the US Army Corps of Engineers Manual 1110-2-1902 entitled *Slope Stability*. The required minimum factors of safety for these loading conditions are summarized in Table 6.5.1. The phreatic surface was modeled based on current piezometer data collected from at the site and the results of the finite element seepage analysis. However, the phreatic surface was entered manually to minimize the potential for computation uncertainty as compared to directly using the finite element analysis output pressures.

As the proposed MSE wall raising is limited in extent, it is recognized that a significant change in stress will only be realized in the upper portion of the embankment. It is believed that the embankment soils in this zone may initially exhibit an undrained response, while the embankment soils beyond the zone of significant stress increase will not see an increase in pore pressure and can be appropriately modeled with drained strengths. To this end, the zone of influence for the end of construction analysis was determined based on results of a finite element deformation analysis performed by AEP. Where the anticipated deformation of the dam from the new loading was less than 0.005 feet, long term steady state parameters were used as influence of the new loading on this zone is expected to be negligible. The zone modeled with undrained strength parameters is designated by hatched material colors in the slope stability graphical output in the End of Construction stability runs. Graphical output from the finite element analysis is included in Appendix D-4.

Rapid Drawdown analyses were completed on the inboard slopes to model drawdown of the reservoir from the PMF level to the maximum operating pool. Seismic slope stability analyses were performed based on a pseudo-static slope stability approach. The rapid drawdown analyses were performed using the Duncan, Wright, and Wong 3-stage analysis approach (Duncan et al. 1990) in addition to Spencer's method to calculate the minimum factor of safety.

Table 6.4.1: Minimum Factors of Safety for Slope Stability

Analysis Condition	Required Minimum Factor of Safety*	Slope to Be Analyzed
End of Construction	1.3	Upstream and Downstream
Long Term (Steady-State Seepage)	1.5	Upstream and Downstream
Max Surcharge Pool	1.4	Downstream
Rapid Drawdown	1.1	Upstream
Seismic (Pseudo-Static)	1.0	Upstream and Downstream

* Based on the US ACOE EM-110-2-1902 guidelines (Table 3-1) and ODNR historic requirements.

6.4.2 Shear Strength Parameters

Shear strength parameters representing the existing dam zones were developed by AEP and their consultants for the design of the Stage 1 and Stage 2 dams. These values were used as the starting point for the present global stability analyses but were modified in some cases to reflect the results of the current investigation or to investigate particular failure modes. Additionally, the proposed raising will include several new material zones: the MSE wall reinforced zone (No. 57 stone), ODOT Item 304 surface course, the cement-bentonite slurry wall, and the vinyl sheet pile wall. The shear strength parameters for these new materials were estimated based on past experience. It should also be noted that the strength of these materials does not appreciably impact the global stability analyses. A summary of the shear strength parameters are shown in Tables 6.5.1.1, 6.5.1.2, and 6.5.1.3. The following sections discuss the shear strength values in more detail for the most critical zones, the strength of which greatly impacts the computed factors of safety.

6.4.2.1 Existing RCC

The 2D global stability analyses performed in support of the 1997 raising modeled the RCC zone with a cohesion of 14,400 psf (100psi). As representative of the intact RCC, this value may be considered conservative when compared to the unconfined compression strengths performed during the present investigation which ranged from 595 to 1602 psi with an average of 1191 psi. These recent tests also match well with the results of 128 unconfined compression tests perform on specimens recovered from 13 core holes during March, 1999, as reported in the June 1, 1999 post-construction report. These tests recorded strength values which ranged from 288 psi to 2367 psi with an average of 1155 psi. Having said this, it is recognized that a number of cracks are present within the RCC zone, as discussed in detail in the June 1, 1999 report entitled "Cracks in RCC Zone and Post- Construction Performance of Dam". The cracks discussed in this report are largely transverse and were believed to be associated with the cooling of the RCC in conjunction with differential settlement between the abutments and the middle of the dam. Transverse cracks do not affect 2D limit-equilibrium global slope stability analyses which are performed in the transverse direction. Of more concern is the presence of horizontal cracks within the RCC presumably occurring between lifts with incomplete bonding. Photographs of the three RCC cores obtained during this investigation are shown in Appendix A. As can be seen, the two 3-inch diameter cores exhibit a number of horizontal joints, whereas the one 8-inch diameter core is fully intact. Based on this, it is clear that at least some horizontal joints exists within the RCC mass however the smaller diameter cores may tend to exaggerate the number.

To examine of the impact of horizontal joints within the RCC mass, three global slope stability analyses were performed placing the horizontal joint at difference elevations. As horizontal joints are under compression due to the weight of the overlying material, it is expected that any such joints should be tightly closed. The shear strength of the joint was model with a friction angle of 38 degrees (cemented bottom ash to cemented bottom ash interface) but with no cohesion. The full hydrostatic head (associated with the pool level of interest) was also assumed to act within the joint, as the models assume that all head loss occurs within the clay core and sheet pile wall located downstream from the RCC.

6.4.2.2 Compacted Mine Spoil and Clay Core

Both the upstream and downstream RCC zones are supported principally on the compacted mine spoil and cohesive clay core soils (designated Zone IV and Zone I respectively). The upper portion of these zones will be subject to stress increase under the weight of the new MSE walls, with the relative increase in stress decreasing with depth. A review of the construction documents for the Stage 1 and 2 dams indicates that the clay core was required to be compacted to 100% of standard proctor density at a moisture content of -1% to +2% of optimum and in 6-inch loose lifts. The mine spoil was to be compacted in 12-inch loose lifts to 100% of standard proctor density at a moisture content of -2% to +1% of optimum. The clay core was to consist of cohesive soil and recompacted shale fragments. In contrast, the mine spoil could consist of mainly granular material to cohesive soils. In consideration of this, it is believed that the clay core material represents the most critical material in terms of end of construction (undrained) bearing capacity and global stability. For this reason, additional borings were performed to obtained “undisturbed” samples of the clay core to confirm its current condition. In the laboratory, two CU triaxial test series and 4 UU triaxial tests were performed.

The soil shear strength parameter justification for the upper portion of the clay core and new material zones used for the dam raising is presented in Appendix D-3. The shear strength parameters for the upper clay core, including drained, undrained, and seismic conditions, were developed based on the results of specialty laboratory testing performed during this investigation, field test results (hand penetrometer readings), and correlations with index test results. As the other mine spoils zones (Zones II & IV) and the original clay core materials were constructed with similar materials and specifications, the undrained strength parameters for these materials were reduced from the values previously assumed during the design of the Stage 1 and Stage 2 dams to be consistent with the undrained parameters developed from the recent laboratory testing. A summary of the specialty testing is also presented in Appendix D-3.

Table 6.5.1.1: Summary of End of Construction Shear Strength Parameters

Material	Unit Weight (pcf)	Angle of Internal Friction (deg)	Cohesion (psf)
Fly Ash	95	30	0
RCC	95	0	14,400*
Mine Spoil - Upstream Shell (Zone IV)	125	0	2,000
Clay Core (Zone I)	128	0	2,000
Transition Zone (Zone II)	128	0	2,000
Mine Spoil - Downstream Shell (Zone IV)	125	0	2,000
Bottom Ash Filter Zone	100	38	0
Chimney Drain (Zone III A)	100	38	0
Blanket Drain (Zone III B/C	100	38	0
Brown Clay	125	26	0
Rock Toe Buttress	110	38	0
Overburden	123	15	1,000
Claystone	140	22.5	1100
Shale	140	15	1100
Reinforced Zone - Coarse Aggregate	105	38	0
Item 304 Aggregate	130	38	0
Cement-Bentonite Cutoff Wall	100	0	720*

Table 6.5.1.2: Summary of Long Term Steady State Shear Strength Parameters

Material	Unit Weight (pcf)	Angle of Internal Friction (deg)	Cohesion (psf)
Fly Ash	95	30	0
RCC	95	0	14,400*
Mine Spoil - Upstream Shell (Zone IV)	125	30	0
Clay Core (Zone I)	128	30	0
Transition Zone (Zone II)	128	30	0
Mine Spoil – Downstream Shell (Zone IV)	125	30	0
Bottom Ash Filter Zone	100	38	0
Chimney Drain (Zone III A)	100	38	0
Blanket Drain (Zone III B/C	100	38	0
Brown Clay	125	26	0
Rock Toe Buttress	110	38	0
Overburden	123	15	1,000
Claystone	140	22.5	1,100
Shale	140	15	1,100
Reinforced Zone – Coarse Aggregate	105	38	0
Item 304 Aggregate	130	38	0
Granular Fill	115	34	0
Cement-Bentonite Cutoff Wall	100	0	720*

*Strength modified to account for existing cracks-see discussion.

Table 6.5.1.3: Summary Seismic Stability Shear Strength Parameters

Material	Unit Weight (pcf)	Angle of Internal Friction (deg)	Cohesion (psf)
Mine Spoil - Upstream Shell (Zone IV)	125	6.8	2,000
Clay Core (Zone I)	128	6.8	2,000
Clay Core ('97 Raising)	128	14.4	520
Transition Zone (Zone II)	128	11	2000
Mine Spoil - Downstream Shell (Zone IV)	125	11	2,000

6.4.3 Pseudo-static Coefficient Determination

Seismic analyses were performed using a pseudo-static analysis with a horizontal seismic coefficient of 0.06g. Following recommendations by Youd (2008), the seismic factors were estimated using a probabilistic procedure based on deaggregation plots developed by the USGS Earthquakes Hazard Program, as appropriate for regions without a well-defined active fault. The plots were developed for 2% probability of exceedence in 50 years. Compatible modal pairs of earthquake magnitude, M and peak acceleration, a_{max} from both a near-source and distant-source earthquake event were considered. The seismic coefficient chosen is based on the near source event, determined to be the most critical. Development of the seismic parameters is presented in Appendix D-3. Liquefaction is not considered a concern for this dam, as the dam is founded directly on bedrock and consists of properly compacted soil and bottom ash.

6.4.4 Analyses and Results

The graphical computer outputs for the slope stability analysis has been included with this report in Appendix D-5. The minimum factors of safety computed for all loading conditions and scenarios examined for this report are in conformance with the USACE requirements. Tables 6.5.3.1 and 6.5.3.2 summarize the lowest factors of safety computed for the various loading conditions.

Table 6.5.3.1 : Slope Stability Analysis Results - End of Construction

Failure Mode	Failure Surface Type	Slope	Calculated Minimum Factor of Safety
Global – Failure Surface Below RCC	Circular	Upstream	3.79
		Downstream	1.58
Failure Surface Through MSE Wall	Circular	Downstream	1.71
	Non-circular		1.60
Shallow Failure Surface Through MSE Wall	Circular	Downstream	2.22
Failure Along Sheet Pile Interface	Composite	Downstream	1.81

Table 6.5.3.2 : Slope Stability Analysis Results – Steady State Seepage

Failure Mode	Required Minimum Factor of Safety	Slope	Calculated Minimum Factor of Safety
Global – Failure Surface Below RCC	Circular	Upstream	2.96
		Downstream	1.76
Failure Surface Through MSE Wall	Circular	Downstream	1.77
	Non-circular		1.79
Shallow Failure Surface Through MSE Wall			2.14
Failure Along Sheet Pile Interface	Composite	Downstream	1.75
Failure through Horizontal Crack in RCC	Non-Circular	Downstream	1.59
		Upstream	2.0
Surcharge Pool	Circular	Downstream	1.68

Table 6.5.3.3 : Slope Stability Analysis Results – Surcharge Pool, Rapid Drawdown, & Seismic

Loading Condition	Analysis	Slope	Calculated Minimum Factor of Safety
Surcharge Pool	Flood Pool EL 982	Downstream	1.68
Rapid Drawdown	Drawdown from Flood Pool to Max Pool	Downstream	2.74
Seismic	Global	Upstream	2.65
		Downstream	1.11

6.5 Analysis of Raised Emergency Spillway Section

The existing emergency spillway will be raised using mass concrete. The mass concrete will extend from the top of the existing spillway at El. 961 up to El. 975.5, a height of 14.5. Of this amount, the lower 13 feet will eventually be exposed to the permanent pool. For this reason, the mass concrete will function as both a structural and a seepage control element. Seepage through the mass concrete will be controlled by minimizing the number of cracks through the use of low heat of hydration concrete and thermal control during placement. Additionally, control and expansion joins will be installed. All joints will be protected with water stops and sealed with caulk.

The mass concrete was analyzed as a conventional concrete gravity dam for uplift (flotation), sliding and overturning along the RCC/concrete interface. Loading due to water pressure at El. 975.5 (just before spillover) and at max flood pool was examined, along with ice loading (maximum operating pool only). Additionally, seepage and global slope stability analyses were carried out in a manner similar to that used for the main dam section. The results of these analyses are shown in Table 6.5.3.2. Finally, the potential for settlement was considered. The existing emergency spillway section including the upstream RCC is supported directly on bedrock associated with the Stage 1 spillway cut. As the majority of the weight of the mass concrete will be applied to the upstream RCC, it is believed that minimal settlement will occur. There is however the potential for differential settlement to take place between the main mass concrete and the smaller amount of the concrete covering the top part of the existing spillway steps. To this end, an expansion joint with a water stop is included at this transition.

Table 6.5.3.2: Slope Stability Analysis Results – Raised Emergency Spillway Section

Analysis	Description	Calculated Minimum Factor of Safety
Long Term (Steady-State Seepage) Slope Stability	Composite Failure Surface	2.17
Seismic (Pseudo-Static) Slope Stability	Composite Failure Surface	2.22
Uplift	Pool EL 975.5	2.5
	Pool EL 982.0	2.0
Slidin	Pool EL 975.5	2.1
	Pool EL 982.0	2.2
Overturning	Pool EL 975.5	2.1

6.6 Settlement

The settlement of the existing dam under the weight of the MSE wall raising was evaluated. In recognition of the complex foundation conditions consisting of various types of compacted fill as well as RCC, it is believed that the best estimate of settlement may be determined by extrapolation from the settlement records of the existing dam. Specifically, the settlement of the Stage 1 dam under the weight of the Stage 2 raising was measured through a combination of monitoring points. This data has previously been presented to ODNR and is summarized in the Appendix D-6. However the proposed raising is roughly 30 percent of the load of the Stage 2 raising. For this reason, the observed settlement for Stage 2 was reduced by this amount. Based on this computation, it is anticipated that primary compression settlement under the weight of the planned raising will vary from less than one-half inch at the abutments, to as much as 1.5 to 2 inches near the high point of the dam. Additionally, the overall dam is experiencing secondary compression, as evidence by the creep type behavior of the existing monitoring points. This long term settlement will continue and may result in an additional ¼ to ½ inch of settlement over the next 10 years, by which time the storage will be exhausted. Such settlement is well within the tolerance of the ARES system, as each panel is essentially an independent system with differential settlement accommodated between the panels. New survey monuments will placed on the top of dam at an approximate spacing of 200 feet.

Long-term settlement of the existing service spillway pipe under the influence of the planned raising is estimated to be less than ½ inch as this pipe is fully supported on a cast-in-place concrete cradle on top of bedrock.

.

7. CONSTRUCTION APPROACH

Owing to the limited access to the dam crest, construction of the planned raising will be more difficult and will require a higher level of planning and coordination than typically required for dam construction or for similar MSE walls along highways. Recognizing this, AEP and S&ME have discussed several different construction approaches and believe that the planned raising is constructible. The general approach is discussed as follows. Please note that the following approach is presented to demonstrate the feasibility of the dam raising and in the end, the selected contractor will likely employ a somewhat different approach.

Construction Sequence:

1. The conceptual construction approach developed by S&ME and AEP involves constructing the RCC trench and slurry wall as a continuous operation at the beginning of the job while access to the dam is still maintained. It is planned to use a rock trencher to excavate through the existing RCC. Such equipment is readily available and is presently being used just across the border in Pennsylvania to construct natural gas pipelines. The use of a rock trencher will minimize disturbance of the remainder of the RCC. Once the RCC trench is complete, slurry wall methods will be used to excavate and construct a cement-bentonite slurry wall across the dam. Although the slurry has been specified with a maximum strength of 15 psi, equivalent to a medium-stiff to stiff clay, the project specifications require that the slurry wall contractor and the sheet pile installer coordinate their means and methods prior to the start of slurry wall construction. As with the RCC trenching, the slurry wall will be constructed in one continuous operation across the dam. At this time, the sheet pile will most likely not be installed, as once the sheet pile wall is in place, access across the dam is severely limited

2. To improve access to the dam and to minimize the need for large trucks to cross the dam in the future, this, the existing haul road on the east side of the reservoir will be improved prior to the start of the dam raising.

3. Following the completion of the trenching/slurry wall operations, construction of the MSE walls will commence working from the right to the left abutment. In general, it is planned to place an approximate 90 to 100 ton crane on the existing dam surface. Concrete panels and the sheet piles will be delivered to the crane from the left abutment. The crane will set the sheet piles into the hardened, albeit weak, cement-bentonite wall and will also set the MSE wall panels. To further minimize seepage, prior to insertion into the slurry wall, the female sheet pile interlocks will be sealed with a hydrophilic sealant. Post-insertion, the exposed portion of the interlocks will be

furthered sealed with a bead of caulk on the upstream side. Prior to installing the sheet pile wall, the Contractor is required to perform borings along the sheet pile wall alignment at 100 foot intervals to verify existing top of clay core elevations.

4. Consistent with highway construction, the panels will be temporarily braced until backfilled. Stone and geogrid will be delivered to the left abutment. The stone will be delivered to the point of placement with a telescoping, movable conveyor belt to eliminate the need for stone trucks to drive on top of the newly constructed walls. Within the MSE walls, only light equipment will be used including skid-steers to place the stone and a smaller compactor. As all fill will consist of open-grade No. 57 stone, only minimal compaction effort will be required.

5. Recognizing that the elevation of the raised emergency spillway is higher than the top of the existing dam, the project specifications require the contractor to minimize the time between the construction of the emergency spillway modification and the completion of the rest of the dam raising. The raised emergency spillway will be constructed using mass concrete. To minimize the potential for construction joint seepage, the specifications require the first 6 foot high lift to be poured monolithically, as this section encases the sheet pile and will be subject to the greatest head. Above this point (El. 967), the contractor may place the concrete in two foot lifts to reduce thermal issues.

6. While the existing emergency spillway is being constructed, a separate crew will focus on raising the service spillway. In this way the full width of the existing dam will be available to support the concrete trucks and form work laydown. The intent is to fully construct the spillway extension but not close it off until the remainder of the raising is complete. The raised structure will allow access to the top of the existing concrete channel to allow the last stoplogs to be inserted and seal off post-completion.

7. Once the sheet piles are fully covered with the MSE wall backfill, access across the entire dam will again be available. At this point, the precast concrete barriers will be set, incorporating gaps so that this cannot impound water and to facilitate instrumentation access, and the Item 304 wearing course will be installed.

8. COST ESTIMATE AND CONSTRUCTION SCHEDULE

It is proposed to construct the raised dam over a period of 8 months beginning in April of 2013. A complete construction schedule has been developed by AEP and is included following the cost estimate. Please note that AEP has retained a contractor to perform access road improvements which is reflected on the construction schedule. This work began in November of 2012 and is separate from the dam raising construction, which is scheduled to begin in April of 2013.

The cost associated with this work is estimated to be $ 7,453,000 as depicted in detail on the following page. This cost estimate includes costs associated with Construction Administration and Materials Testing, as well as updating the Emergency Action Plan and Operation, Maintenance, and Inspection Manual.

Table 8.1.1 ENGINEER'S ESTIMATE OF PROBABLE CONSTRUCTION COST

ITEM NO.	DESCRIPTION	UNIT	QUANTITY	TOTAL UNIT PRICE	TOTAL
1	Mobilization	L.S.	1	$ 1,000,000	$ 1,000,000
2	Topsoil Stripping and Stockpiling	C.Y.	50	$ 15	$ 750
3	RCC Excavation (Rock Trencher)	C.Y.	1,334	$ 115	$ 153,410
4	Cement Bentonite Slurry Wall	S.F.	31,375	$ 18	$ 564,750
5	RCC Foundation	S.Y.	4,850	$ 0.25	$ 1,213
6	Abutment Foundation	S.Y.	500	$ 2.25	$ 1,125
7	Granular Fill	C.Y.	400	$ 35	$ 14,000
8	Flowable Fill Backfill	C.Y.	365	$ 120	$ 43,800
9	Select MSE Wall Backfill - No. 57 Stone	C.Y.	11,324	$ 70	$ 792,680
10	Granular Surface Course - Item 304	C.Y.	1,075	$ 55	$ 59,125
11	Bentonite-Aggregate Composite	C.Y.	250	$ 450	$ 112,500
12	Lean Concrete Backfill	C.Y.	50	$ 250	$ 12,500
13	Erosion Control	L.S.	1	$ 35,000	$ 35,000
14	PVC Pipe-Perforated 6"	L.F.	1,330	$ 8	$ 10,640
15	PVC Pipe-Solid 6"	L.F.	150	$ 20	$ 3,000
16	Demolition	L.S.	1	$ 5,000	$ 5,000
17	Fencing & Gates	L.S.	1	$ 12,000	$ 12,000
18	Mechanically Stabilized Earth Wall	S.F.	31,375	$ 40	$ 1,255,000
19	Service Spillway Modifications	L.S.	1	$ 100,000	$ 100,000
20	West Abutment Access Road Improvements	L.S.	1	$ 20,000	$ 20,000
21	Portable Concrete Barrier	L.F.	2,440	$ 36	$ 87,840
22	8' Railing Section	EA	32	$ 1,000	$ 32,000
23	Modular Floating Dock	L.S.	1	$ 19,000	$ 19,000
24	Floating Debris Boom	LF	300	$ 15	$ 4,500
25	Biaxial Geogrid	S.Y.	1,965	$ 6	$ 11,790
26	Non-Reinforced Mass Concrete	C.Y.	3,000	$ 250	$ 750,000
27	Emergency Spillway Training Walls	C.Y.	225	$ 900	$ 202,500
28	Exploratory Drilling	L.F.	375	$ 50	$ 18,750
29	Vinyl Sheet Pile Wall	S.F.	52,075	$ 18	$ 937,350
30	Instrumentation	L.S.	1	$ 20,000	$ 20,000
31	Updates to EAP and OMI	L.S.	1	$ 15,000	$ 15,000
32	Construction Administration and Testing	L.S.	1	$ 480,000	$ 480,000

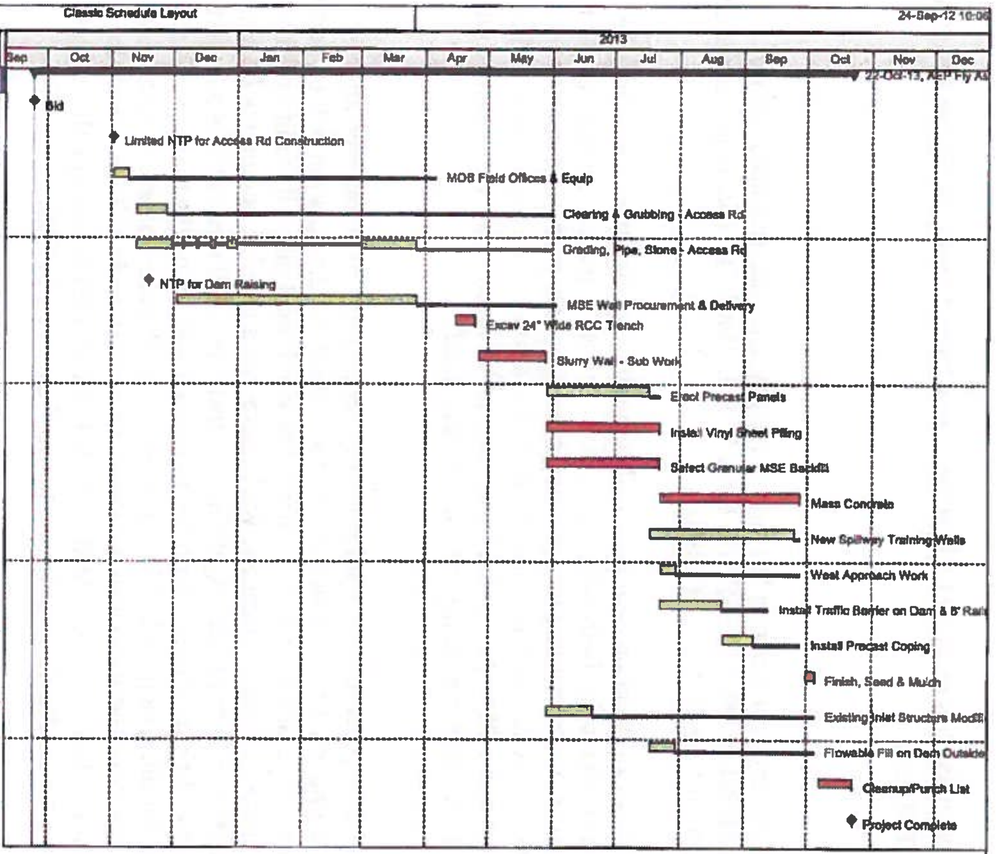
Subtotal	**$ 6,775,223**
Contingency, 10%	**$ 677,522**
Subtotal	**$ 7,452,945**

AEP Fly Ash Retention Dam II - Rev 1

Classic Schedule Layout

24-Sep-12 10:06

Activity ID	Activity Name	Original Duration	Total Float	Start	Finish	Predecess...	Successors
	AEP Fly Ash Retention Dam II - ...	[illegible]	0	24-Sep-12	22-Oct-13		
100	Bid	0	0	24-Sep-12*			110
110	Limited NTP for Access Rd Construction	0	0	02-Nov-12*		100	120
120	MOB Field Offices & Equip	5	40	02-Nov-12	09-Nov-12	110	130, 140
130	Clearing & Grubbing - Access Rd	10	64	13-Nov-12	27-Nov-12	120	160
140	Grading, Pipe, Stone - Access Rd	34	40	13-Nov-12	27-Mar-13	120	160
150	NTP for Dam Raising	0	0	19-Nov-12*			160, 200
160	MSE Wall Procurement & Delivery	115	67	02-Dec-12	27-Mar-13	150, 130, ...	220
200	Excav 24" Wide RCC Trench	7	0	15-Apr-13*	24-Apr-13	150	210
210	Slurry Wall - Sub Work	20	0	26-Apr-13	28-May-13	200	220, 230, 300
220	Erect Precast Panels	32	3	20-May-13	17-Jul-13	210, 160	240, 310, 250
230	Install Vinyl Sheet Piling	35	0	29-May-13	22-Jul-13	210	240
240	Select Granular MSE Backfill	35	0	29-May-13	22-Jul-13	220, 230	260, 270, 245
245	Mass Concrete	45	0	23-Jul-13	27-Sep-13	240	290
250	New Spillway Training Walls	45	3	18-Jul-13	24-Sep-13	220	290
260	West Approach Work	5	40	23-Jul-13	30-Jul-13	240	290
270	Install Traffic Barrier on Dam & 6' Railing Sections	20	15	23-Jul-13	21-Aug-13	240	280
280	Install Precast Coping	10	15	22-Aug-13	05-Sep-13	270	290
290	Finish, Seed & Mulch	5	0	30-Sep-13	04-Oct-13	250, 260, 280, 245	320
300	Existing Inlet Structure Modifications	15	70	29-May-13	20-Jun-13	210	320
310	Flowable Fill on Dam Outside MSE Walls	8	45	18-Jul-13	30-Jul-13	220	320
320	Cleanup/Punch List	10	0	07-Oct-13	22-Oct-13	300, 310, 290	330
330	Project Complete	0	0		22-Oct-13	320	



▬ Actual Work
▭ Remaining Work ◆
🟥 Critical Remaining Work
◆ Milestone
▼ Summary
— Float Bar

Page 1 of 1

TASK filter: All Activities

(c) Primavera Systems, Inc

REFERENCES

(1) American Electric Power Service Corporation, "Cardinal Plant Fly Ash Retention Pond II, Design Report for Proposed Dam", Civil Engineering Division, Columbus, Ohio, December 1984.

(2) American Electric Power Service Corporation, "Cardinal Plant Fly Ash Retention Pond II, Final Design Report for Proposed Dam Earth Fill-Roller Compacted Concrete Raising of Dam", American Electric Power Service Corporation, Columbus, Ohio, March 1997.

(3) Buhac, H.J. and Amaya, P.J., “Raising of Cardinal Fly Ash Retention Dam, *Dam Maintenance and Rehabilitation-Proceedings of the International Congress on Conservation and Rehabilitation of Dams*, Madrid, Spain, November 11-12, 2002, pp. 425-432.

(4) American Electric Power Service Corporation, "Cardinal Fly Ash Retention Dam II Raising, Cracks in RCC Zone and Post-Construction Performance of Dam”, American Electric Power Service Corporation, Columbus, Ohio, June 1, 1999.

(5) American Electric Power, "Seepage Analysis FAR Dam II-Cardinal Plant, Brilliant, Ohio”, American Electric Power, Columbus, Ohio, October 2004.

(6) American Electric Power Civil Laboratory, “Deformation Review, Report of Survey, Cardinal Plant Flay Ash Dam - Reservoir II”, American Electric Power Civil Laboratory, Groveport, Ohio, September 6, 2011.

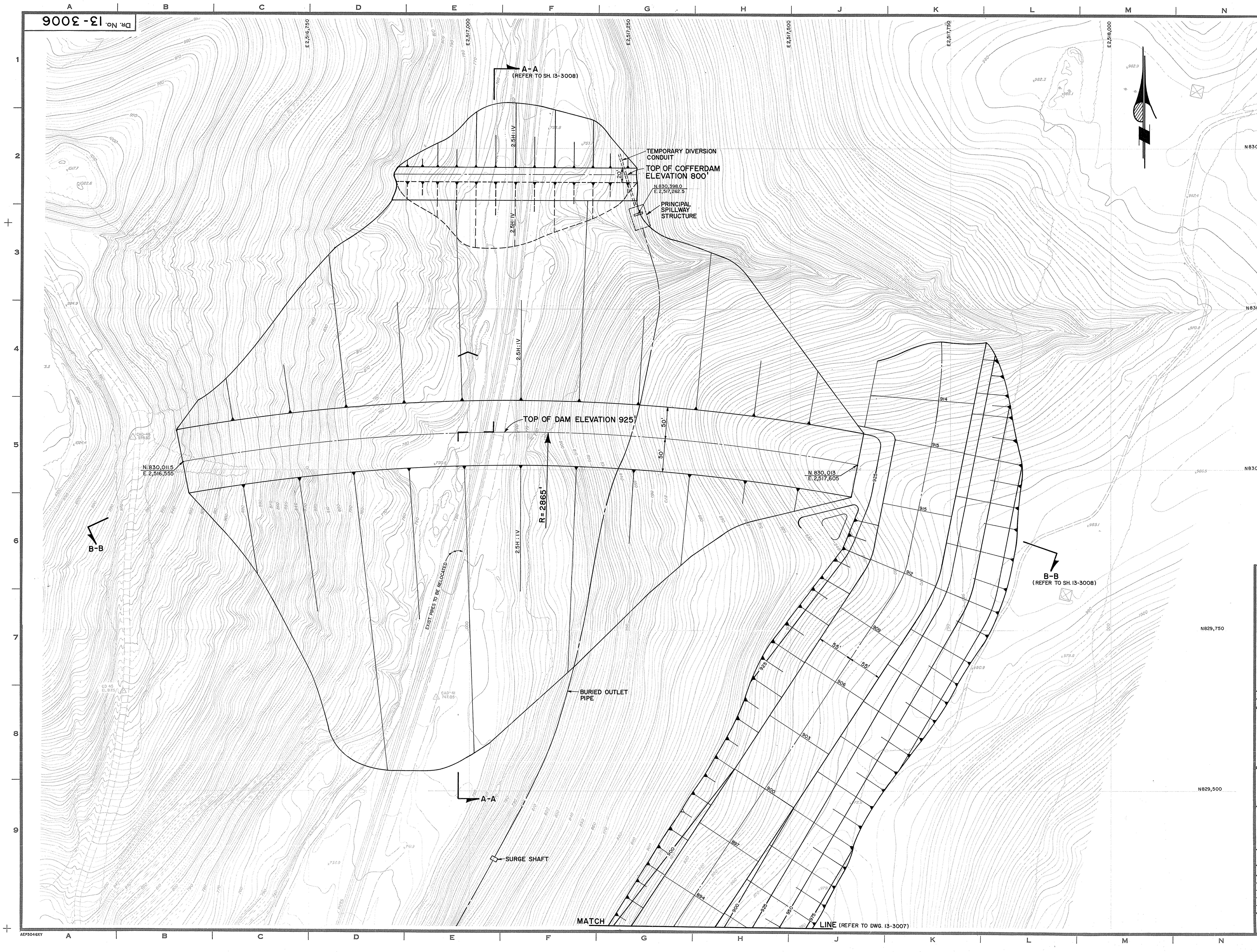
(7) Amaya, P.J, Massey-Norton, J.T. and Stark, T.D., “Evaluation of Seepage from and Embankment Dam Retaining Fly Ash”, *ASCE Journal of Performance of Constructed Facilities*, Vol. 23, No. 6, December 1, 2009.

(8) American Electric Power, “Cardinal Plant Dam and Dike Inspection 2011 (includes seepage and piezometer data)”, American Electric Power Civil Laboratory, Groveport, Ohio, November 8, 2011.

ATTACHMENT C

DESIGN DRAWINGS

DR. No. 13-3006



DATE	NO.	DESCRIPTION	APPD.

REVISIONS

BUCKEYE POWER CO.
OHIO POWER CO.
CARDINAL FLY ASH DAM 2
BRILLIANT OHIO

DAM &
EMERGENCY SPILLWAY

DR. NO. 13-3006

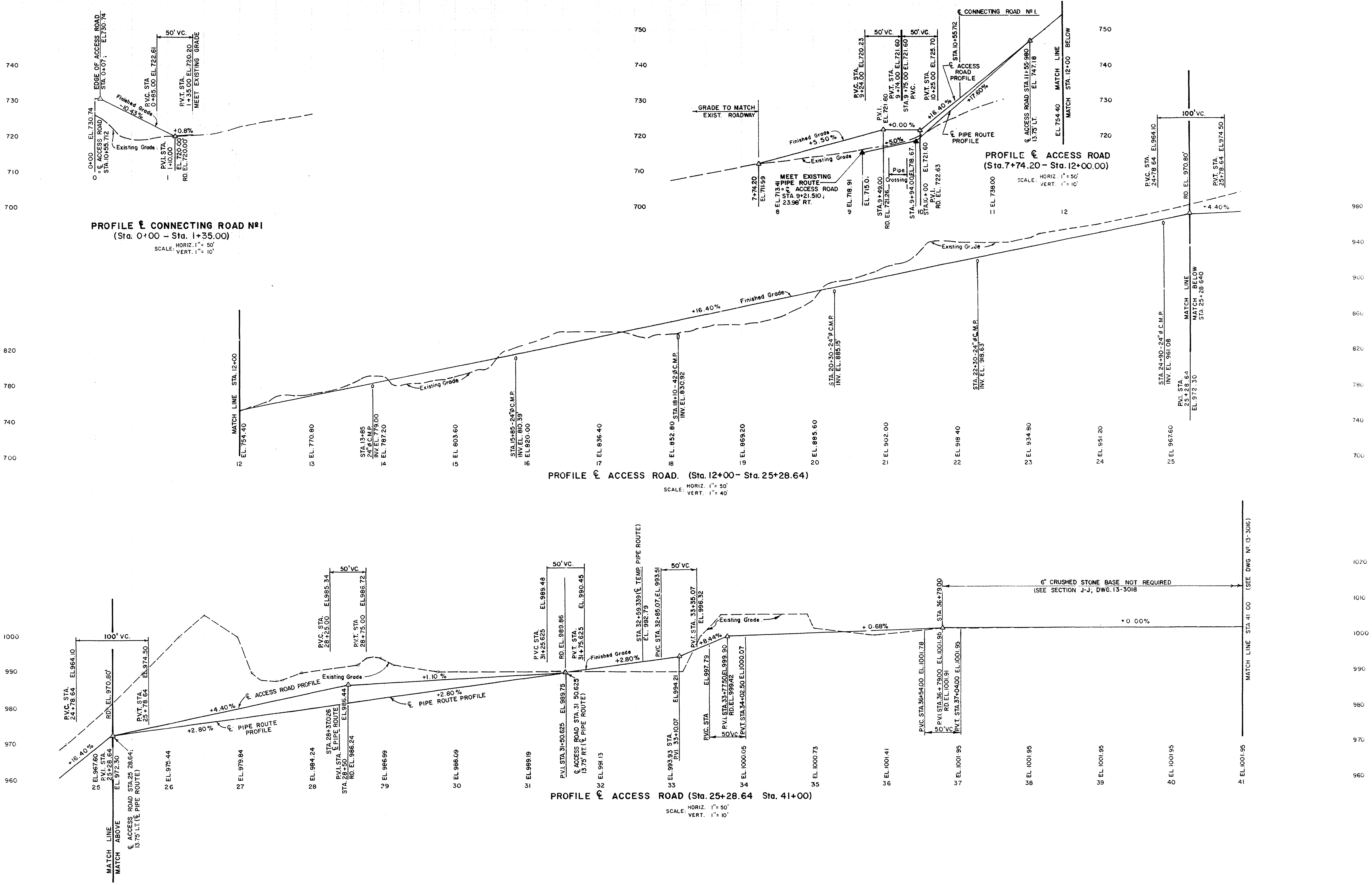
ARCH. – ELEC – MECH. STR. TS

SCALE: 1"=50'
DR. M.Zengler
CH. T.S.
SQ. LDR T.S.
DATE: 3-21-84

APPROVED BY
ENGINEERING DIV PHA
DESIGN DIV V. A. Legere

AMERICAN ELECTRIC POWER SERVICE CORP.

Dr. No. 13-3015



PROFILE ℄ CONNECTING ROAD Nº1
(Sta. 0+00 – Sta. 1+35.00)
SCALE: HORIZ. 1" = 50' VERT. 1" = 10'

PROFILE ℄ ACCESS ROAD
(Sta. 7+74.20 – Sta. 12+00.00)
SCALE: HORIZ. 1" = 50' VERT. 1" = 10'

PROFILE ℄ ACCESS ROAD. (Sta. 12+00 – Sta. 25+28.64)
SCALE: HORIZ. 1" = 50' VERT. 1" = 40'

PROFILE ℄ ACCESS ROAD (Sta. 25+28.64 Sta. 41+00)
SCALE: HORIZ. 1" = 50' VERT. 1" = 10'

GENERAL NOTES

FOR GENERAL NOTES, SEE DWG 13-3012

REFERENCE DRAWINGS

FOR REFERENCE DRAWINGS, SEE DWG 13-3013

DATE	NO.	DESCRIPTION	APP'D

REVISIONS

BUCKEYE POWER CO.
OHIO POWER CO.
CARDINAL FLY ASH DAM 2
BRILLIANT, OHIO

ACCESS ROAD
&
PIPE ROUTE PROFILE
SHEET 1 of 2

DR. NO. 13-3015

ARCH	–	ELEC	–	MECH		STR	

SCALE AS SHOWN
DR. S.L.P.
CH. R.D.
SQ. LDR.
DATE
APPROVED BY
ENGINEERING DIV.
DESIGN DIV.

AMERICAN ELECTRIC POWER SERVICE CORP.

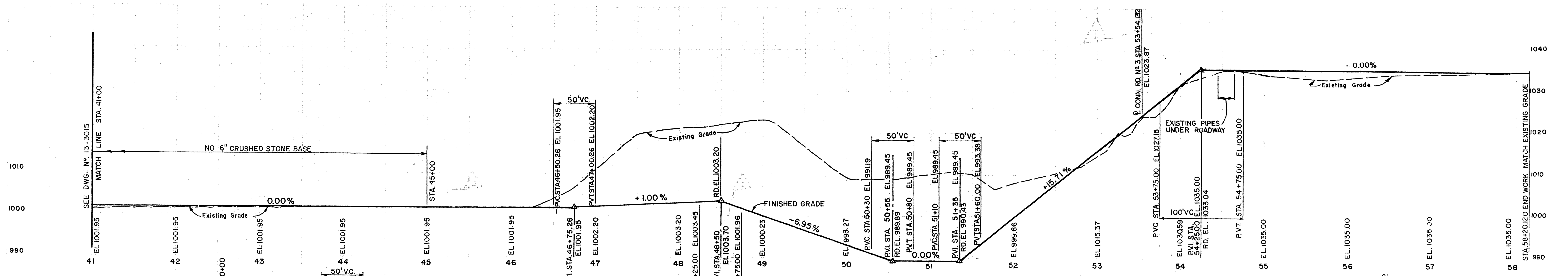
DR. No 13-3016

GENERAL NOTES

FOR GENERAL NOTES AND LEGEND SEE DWG. 13-3012.

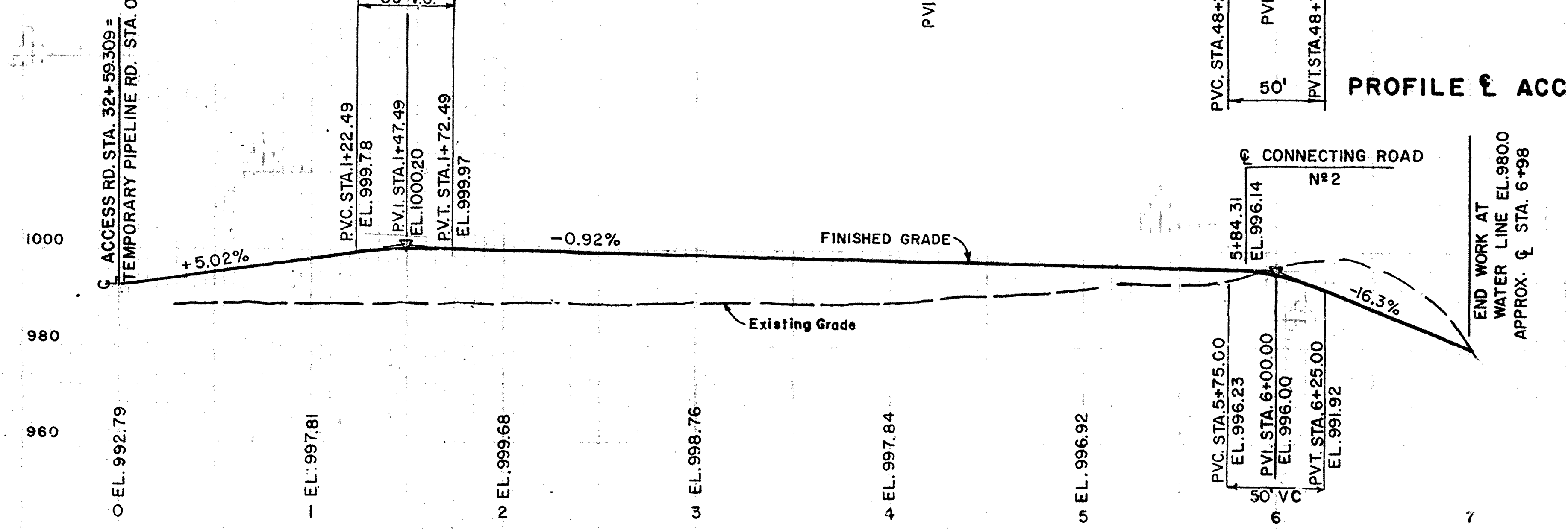
REFERENCE DRAWINGS

FOR REFERENCE DRAWINGS SEE DWG 13-3013



PROFILE ℄ ACCESS ROAD (Sta.41+00-Sta.58+20.120)

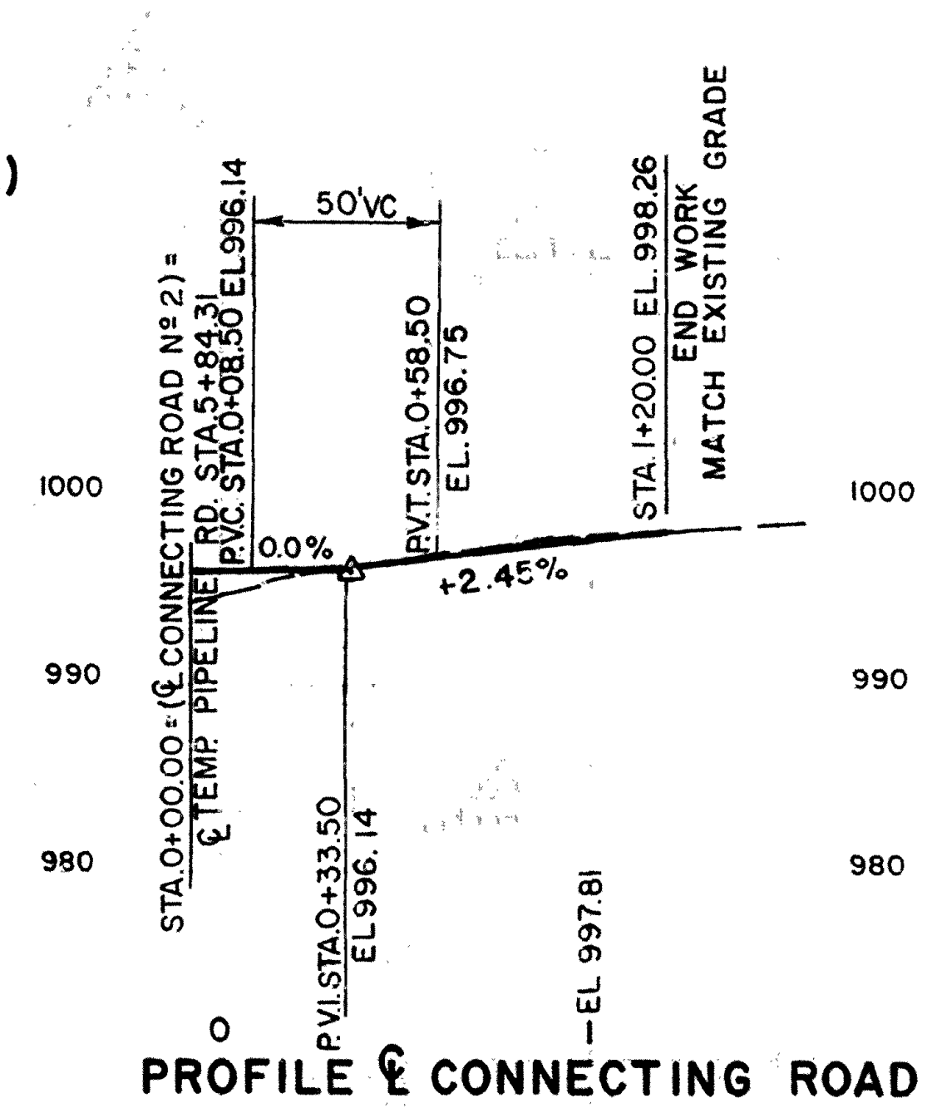
SCALE: HORZ. 1"=50'
VERT. 1"=10'



PROFILE ℄ TEMPORARY PIPELINE ROAD

(Sta.0+00-Approx. Sta.6+98)

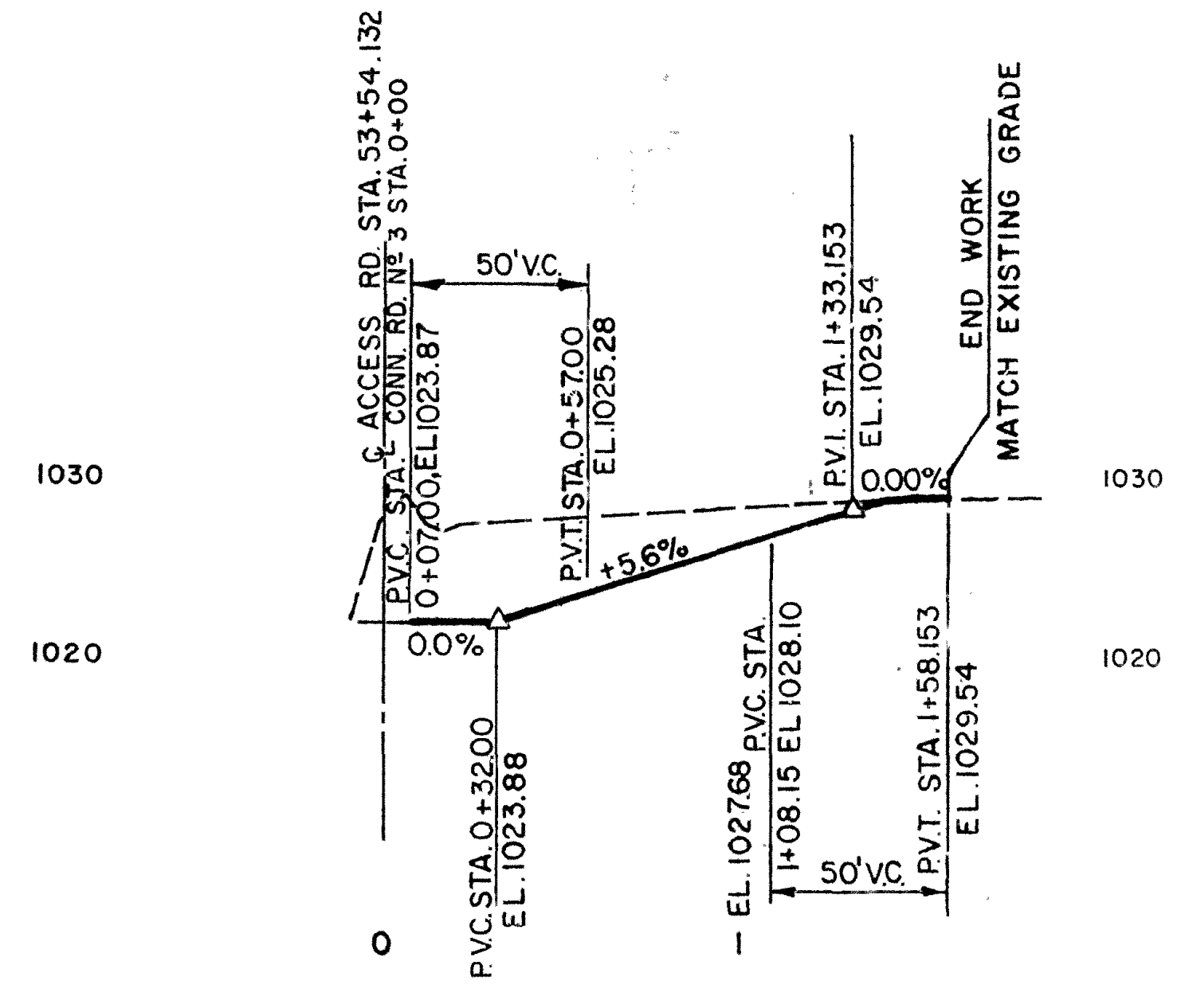
SCALE HORZ. 1"=50'
VERT. 1"=20'



PROFILE ℄ CONNECTING ROAD Nº 2

(Sta.0+00.00-Sta.1+20.00)

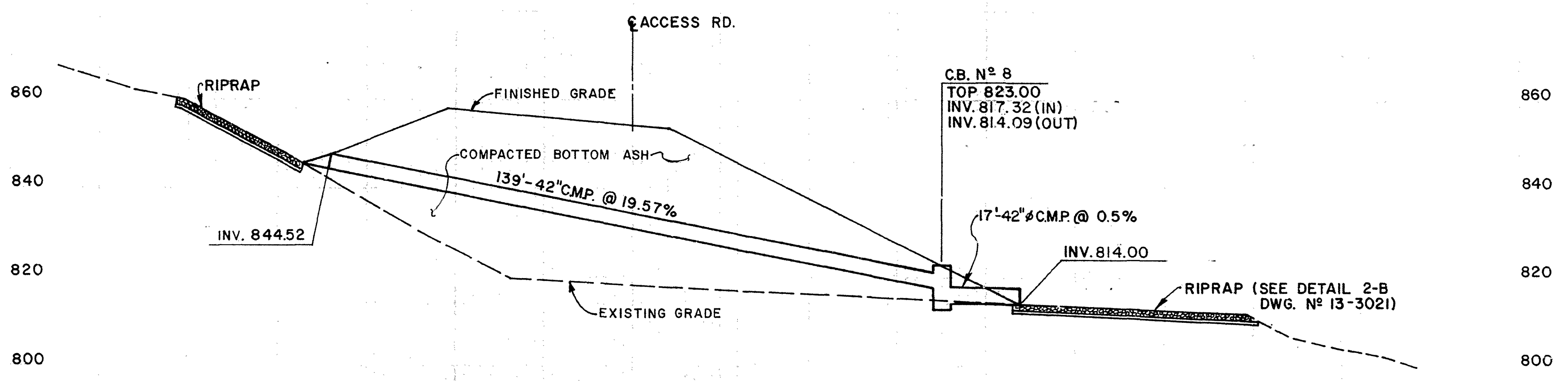
SCALE HORZ. 1"=50'
VERT. 1"=10'



PROFILE ℄ CONNECTING ROAD Nº 3

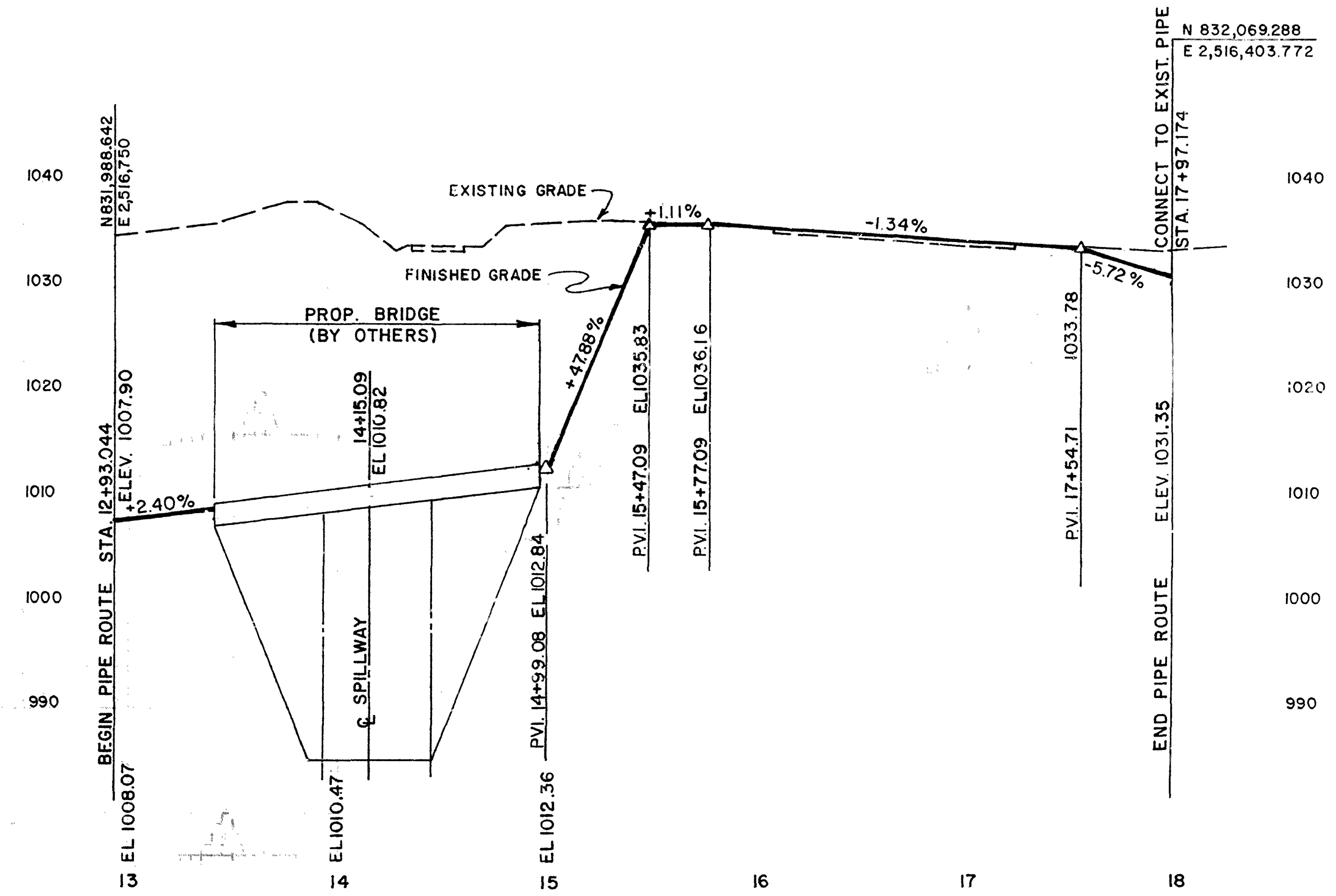
(Sta.0+00.00-Sta.1+58.153)

SCALE HORZ. 1"=50'
VERT. 1"=10'



42" ø C.M.P. PROFILE

SCALE: HORZ. 1"=20'
VERT. 1"=20'



PIPE ROUTE PROFILE

(Sta.12+93.044-Sta.17+97.174)

SCALE: HORZ. 1"=50'
VERT. 1"=10'

DATE	NO	DESCRIPTION	APPR.
[illegible]	1	CORRECTED STATIONS & ELEVS.	[illegible]

REVISIONS

BUCKEYE POWER CO.
OHIO POWER CO.

CARDINAL FLY ASH DAM 2
BRILLIANT, OHIO

ACCESS ROAD
&
PIPE ROUTE PROFILE
SHEET 2 of 2

DR. NO. 13-3016-1

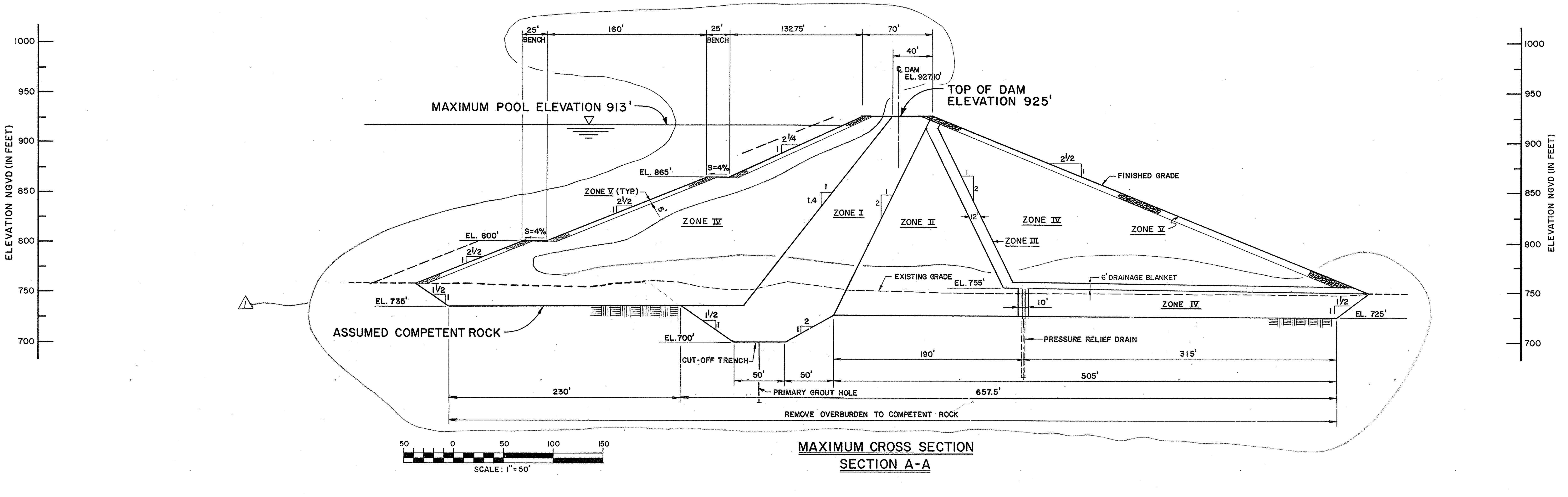
ARCH - ELEC - MECH STR.
SCALE SHOWN
DR RMC GARVEY
CH R D.
SQ LDR
DATE 7/2/84
APPROVED BY
ENGINEERING DIV
DESIGN DIV

AMERICAN ELECTRIC POWER SERVICE CORP.

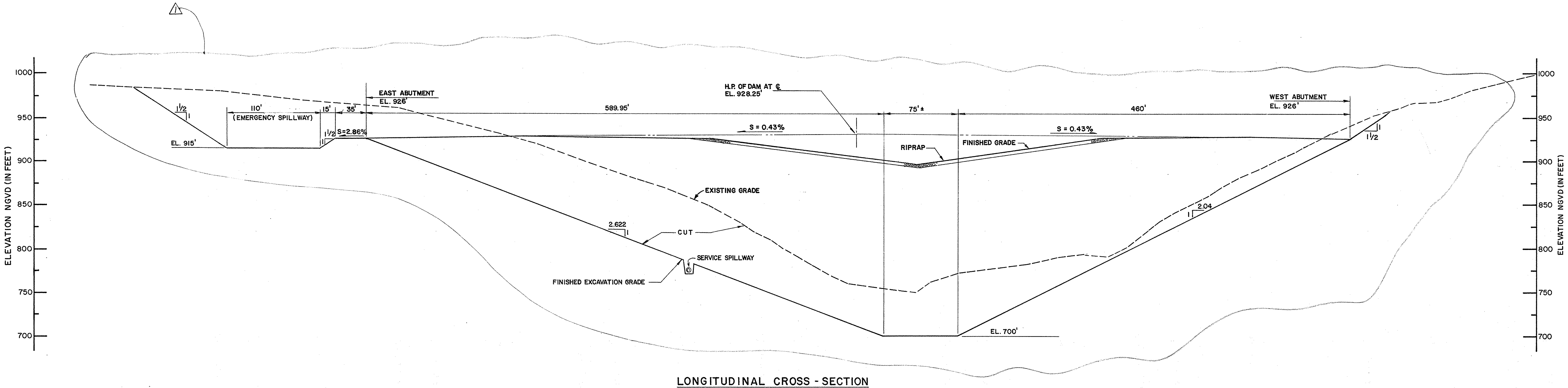
AEP3046XY

DR. No. 13-3008

ZONE	DESCRIPTION	SOURCE
I	IMPERVIOUS CLAY CORE	BORROW AREA I
II	MINE SPOIL (FRACTION ≤ 6 in.)	BORROW AREA II
III	BOTTOM ASH DRAIN	BOTTOM ASH POND
IV	MINE SPOIL (FRACTION ≤ 9 in.)	BORROW AREA II AND EMERGENCY SPILLWAY
V	OVERSIZE STONE	BORROW AREA II AND QUARRY



MAXIMUM CROSS SECTION
SECTION A-A



LONGITUDINAL CROSS - SECTION
SECTION B-B

DATE	NO.	DESCRIPTION	APPD.
1/14/85	1	REVISED SECTIONS "A-A" & "B-B" REVISED FILL MATERIAL DESCRIPTION.	SF/TS

REVISIONS

"THIS DRAWING IS THE PROPERTY OF THE AMERICAN ELECTRIC POWER SERVICE CORP. AND IS LOANED UPON CONDITION THAT IT IS NOT TO BE REPRODUCED OR COPIED, IN WHOLE OR IN PART, OR USED FOR FURNISHING INFORMATION TO ANY PERSON WITHOUT THE WRITTEN CONSENT OF THE AEP SERVICE CORP., OR FOR ANY PURPOSE DETRIMENTAL TO THEIR INTEREST, AND IS TO BE RETURNED UPON REQUEST"

BUCKEYE POWER CO.
OHIO POWER CO.
CARDINAL FLY ASH DAM 2
BRILLIANT OHIO

DAM SECTIONS

DR. NO. 13-3008-1

ARCH.	–	ELEC	–	MECH.		STR.	TS

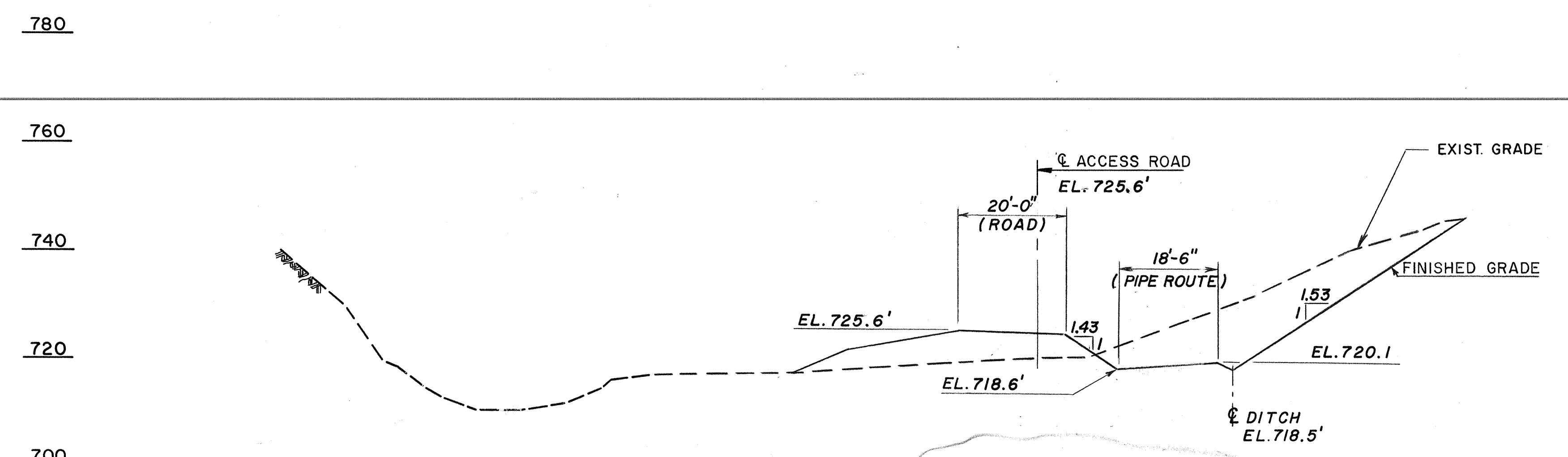
SCALE: 1" = 50'
DR. M. Zengler
CH. T.S.
SQ. LDR T. S.
DATE: 3-21-84

APPROVED BY
ENGINEERING DIV PHA
DESIGN DIV

AMERICAN ELECTRIC POWER SERVICE CORP.

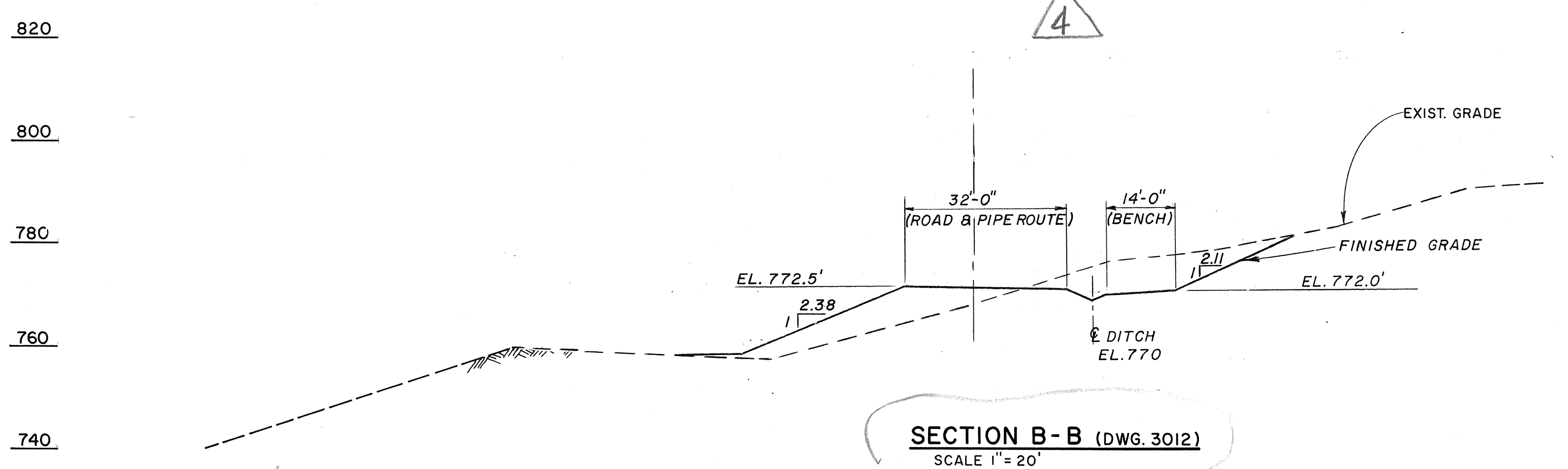
AEP3046XY

DR. No. 13-3017



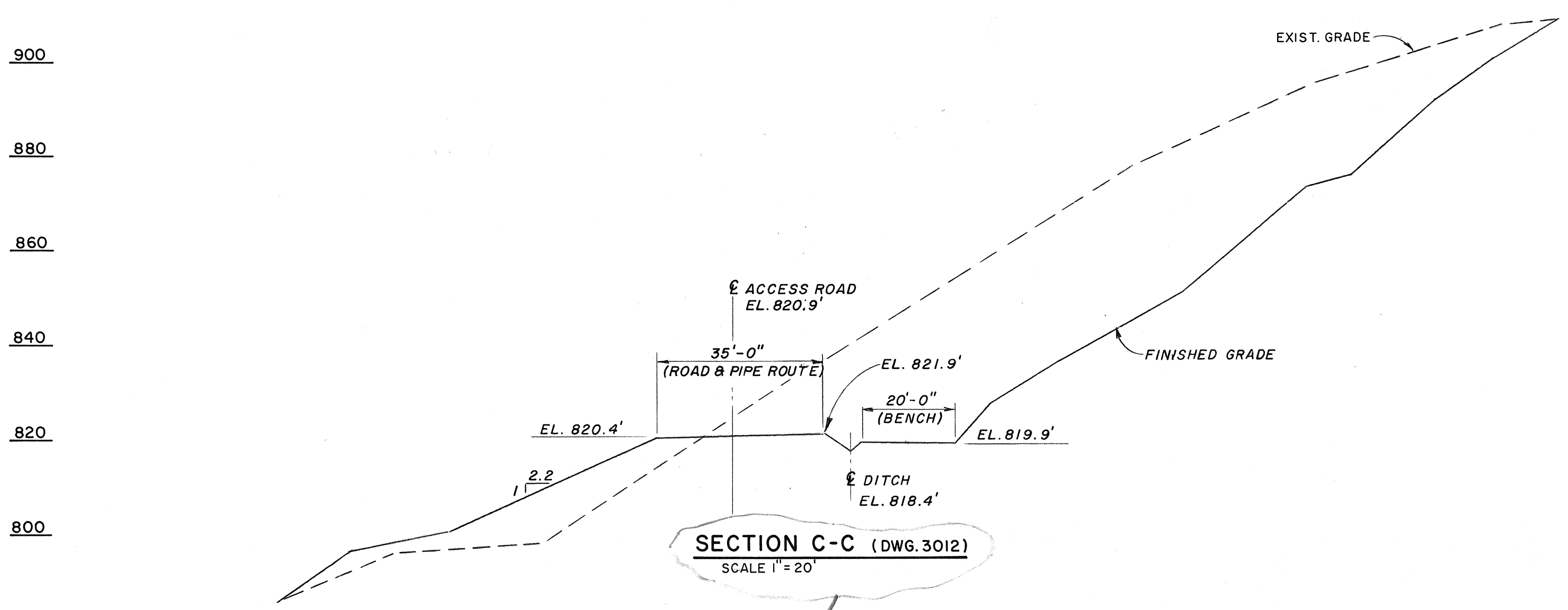
SECTION A-A (DWG. 3012)
SCALE 1"=20'

4



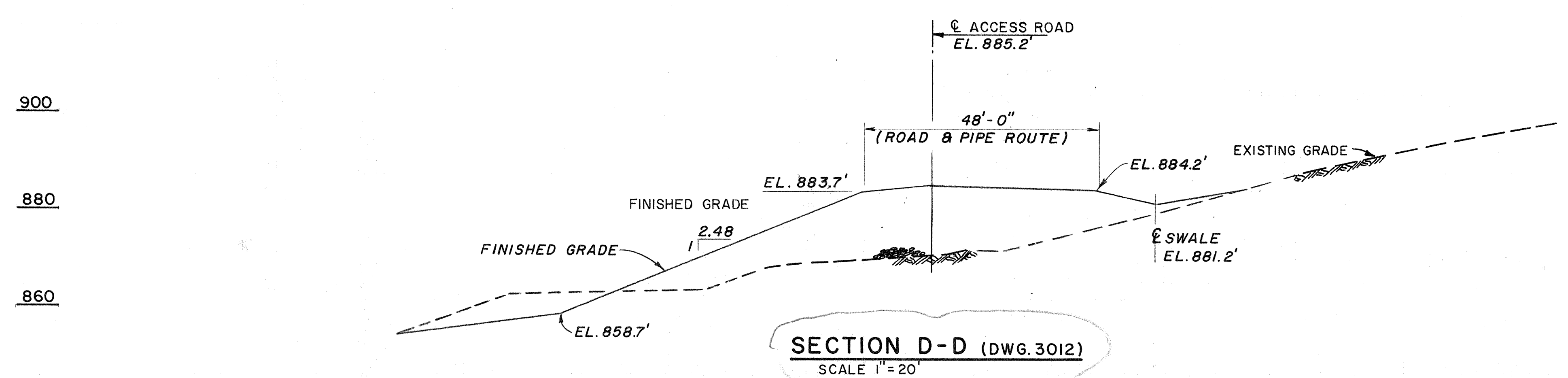
SECTION B-B (DWG. 3012)
SCALE 1"=20'

4



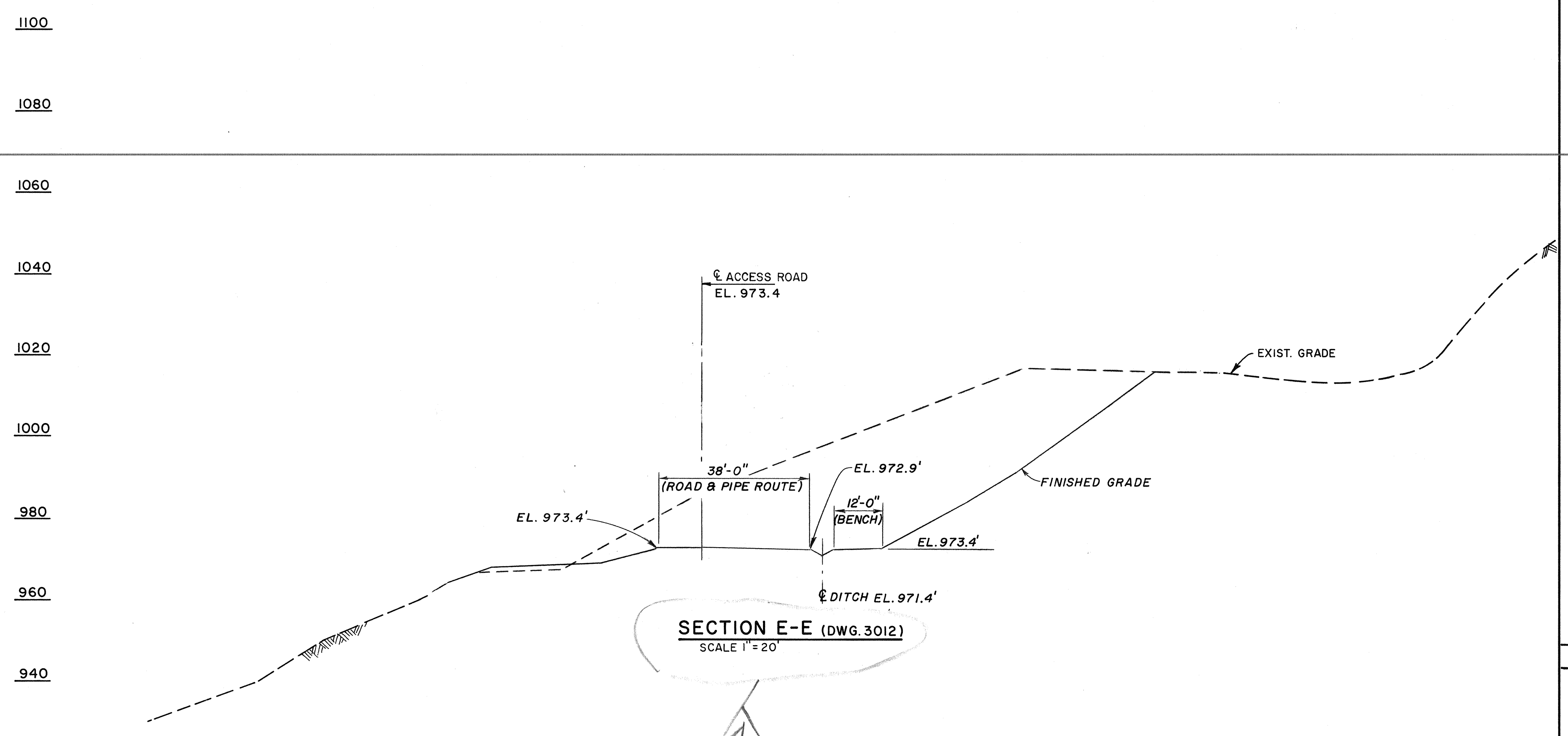
SECTION C-C (DWG. 3012)
SCALE 1"=20'

4



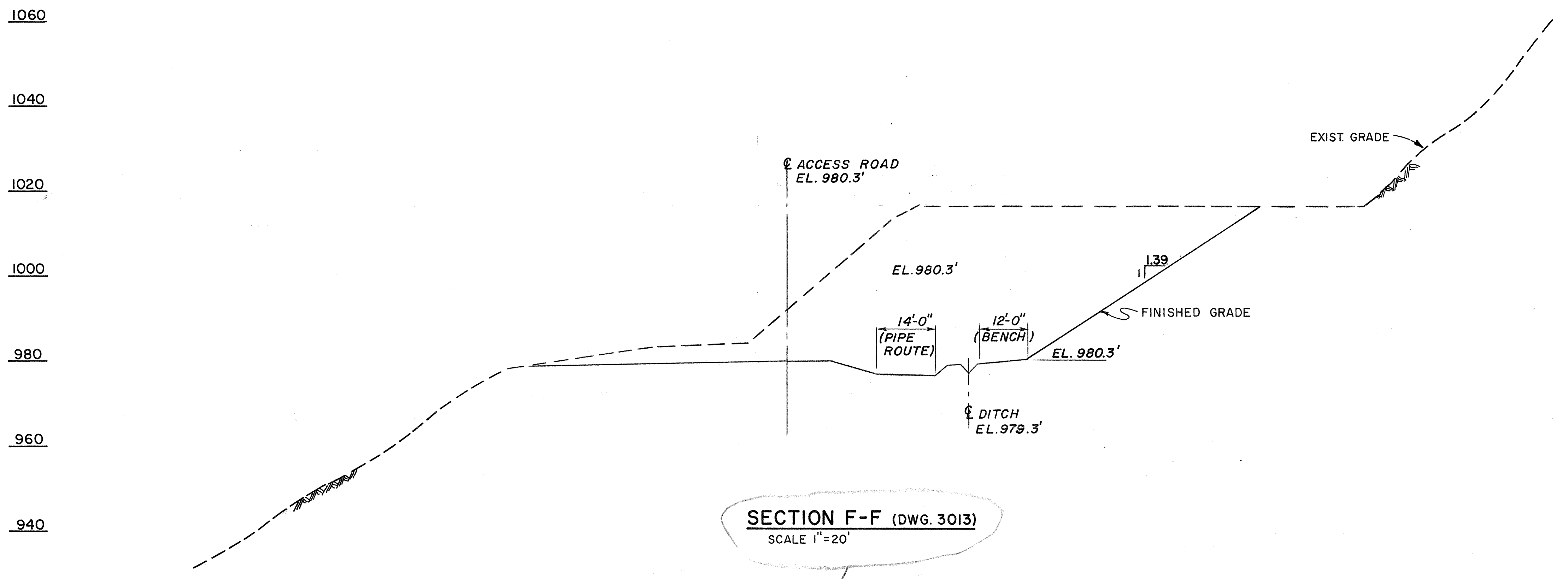
SECTION D-D (DWG. 3012)
SCALE 1"=20'

4



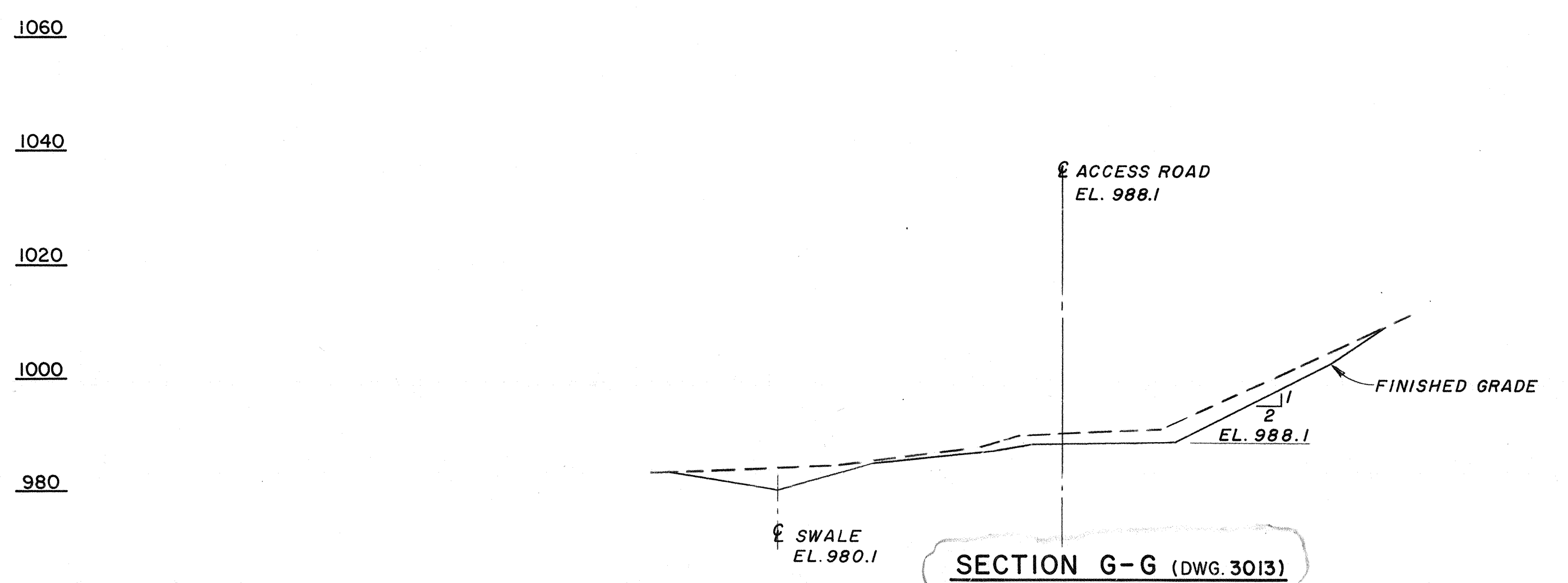
SECTION E-E (DWG. 3012)
SCALE 1"=20'

4



SECTION F-F (DWG. 3013)
SCALE 1"=20'

4



SECTION G-G (DWG. 3013)
SCALE 1"=20'

4

GENERAL NOTES

FOR GENERAL NOTES AND LEGEND SEE DWG. 13-3012.

FOR TYPICAL SECTION OF ACCESS ROAD, PIPE ROUTE & DRAINAGE, SEE DWG. 13-3019

REFERENCE DRAWINGS

FOR REFERENCE DRAWINGS SEE DWG. 13-3013

DATE	NO.	DESCRIPTION	APPD.
5/4/86	4	REV. SECTIONS TO REFLECT "AS-BUILT" INFO. R.M./RD	
1/8/85	3	REV. DIM. AT AREA "D-2", 29'-10 5/8" WAS 24'-3". DELETED DIMS. 30'-6" & 11'.	SF/TS
11/12/84	2	IN SECTIONS "E-E" & "F-F" DIM. 14'-0" WAS 12'-6"	SF/TS
9/19/84	1	DIM. 13'-0" WAS 12'-6" ADDED DITCH ON SECTION "D-D".	SF/TS

REVISIONS

MF

MF

MF

BUCKEYE POWER CO.
OHIO POWER CO.
CARDINAL FLY ASH DAM 2
BRILLIANT, OHIO

FLY ASH LINES
ACCESS ROAD & DRAINAGE
SECTIONS & DETAILS
SHEET № 1

DR. NO. 13-3017-4

ARCH.	–	ELEC	–	MECH.		STR.	TS

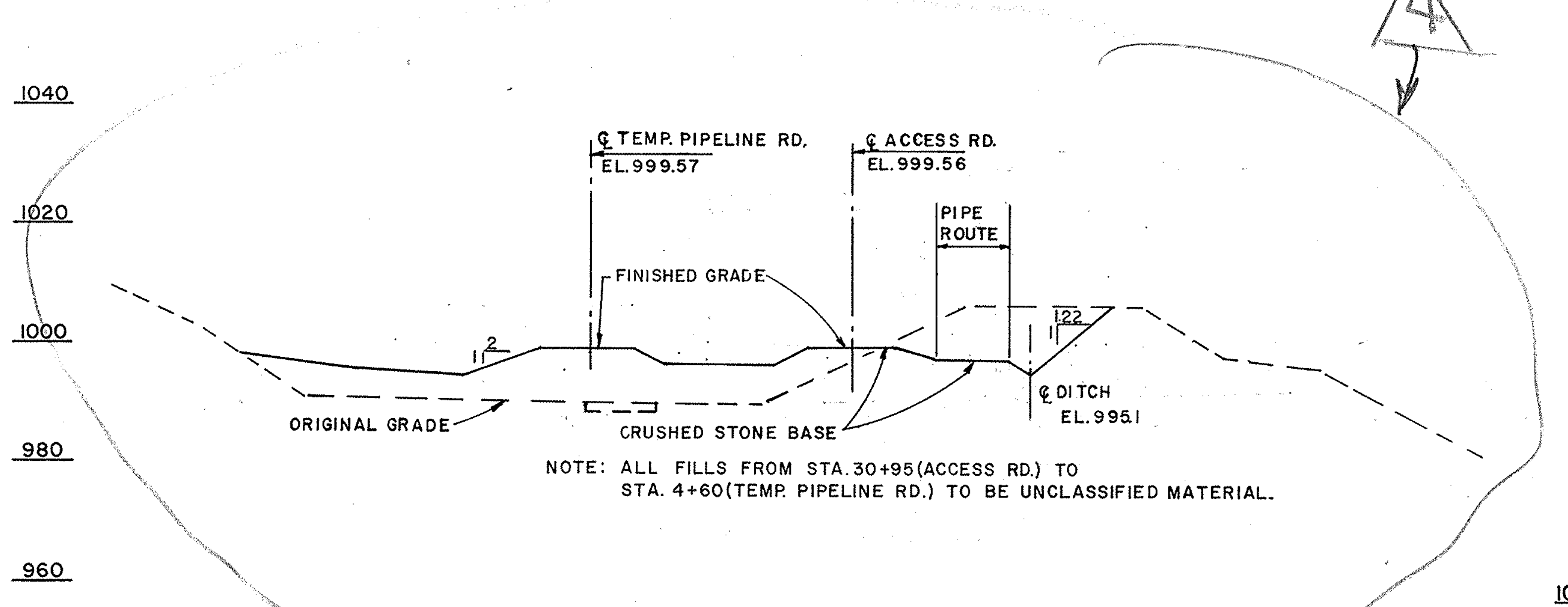
SCALE: 1"=20'
DR. K.S. Davies M.Zengler
CH. R.D.
SQ. LDR T.S.
DATE: 7/2/84

APPROVED BY
ENGINEERING DIV
DESIGN DIV
JA DiBella

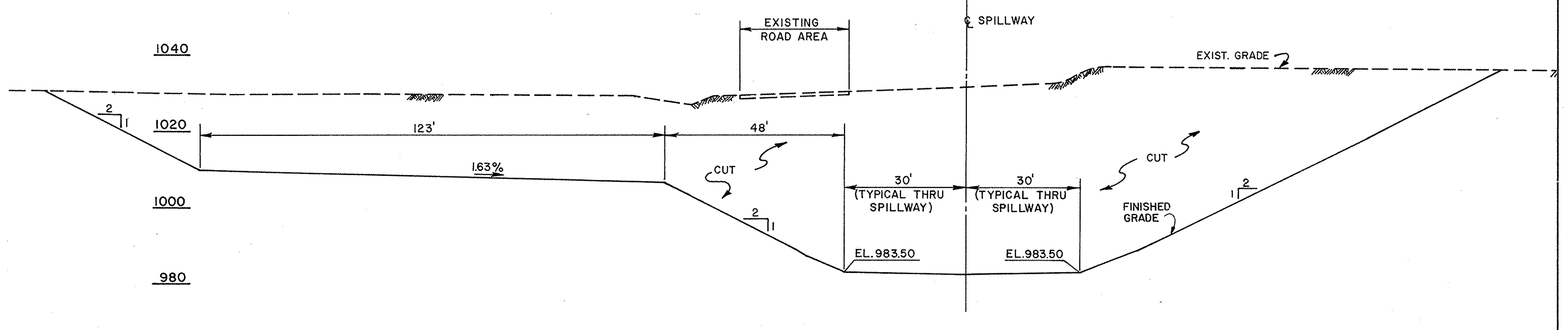
AMERICAN ELECTRIC POWER SERVICE CORP.

AEP3046XY

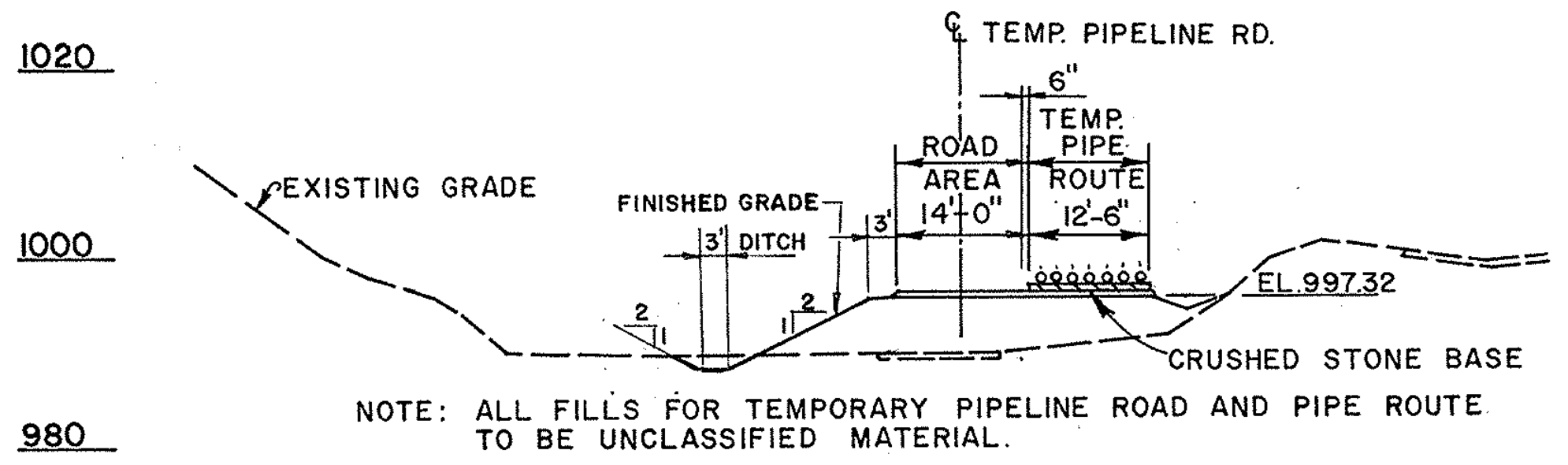
DR. No. 13-3018



SECTION H-H (DWG. 3013)
SCALE 1" = 20'

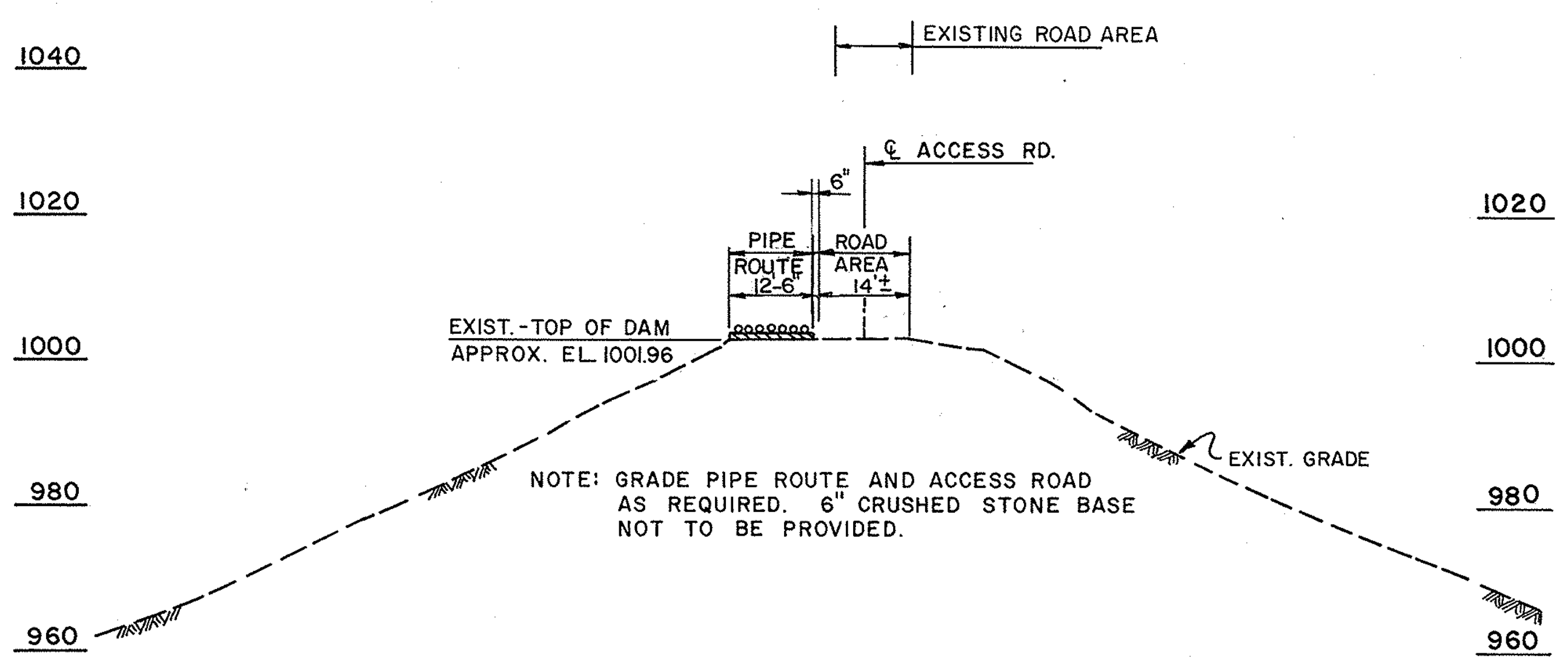


SECTION Z-Z (DWG. 3014)
SCALE 1" = 20'

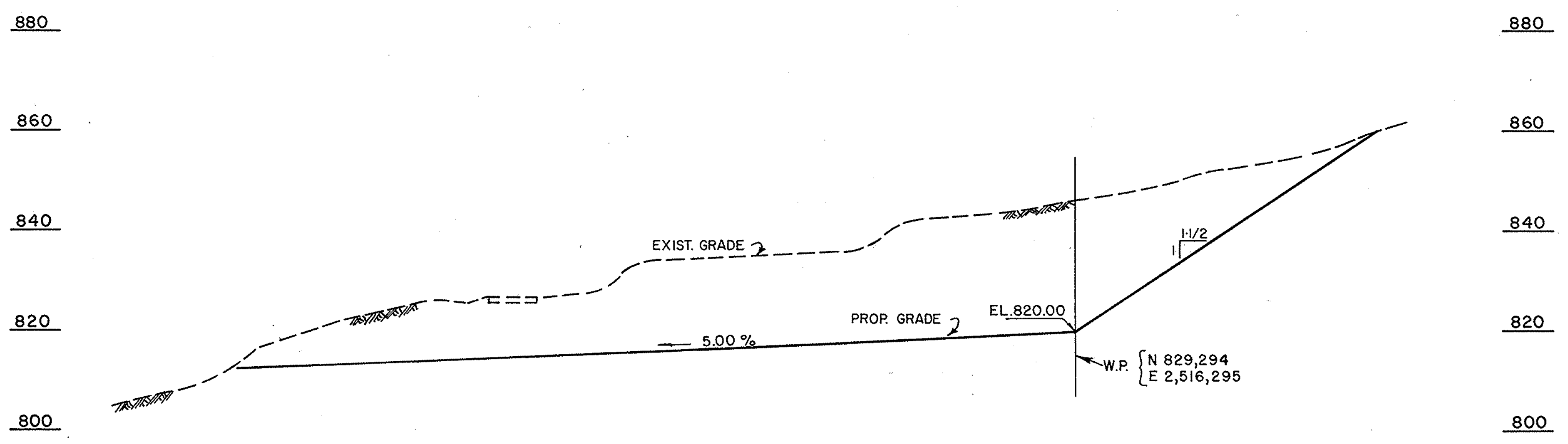


NOTE: ALL FILLS FOR TEMPORARY PIPELINE ROAD AND PIPE ROUTE TO BE UNCLASSIFIED MATERIAL.

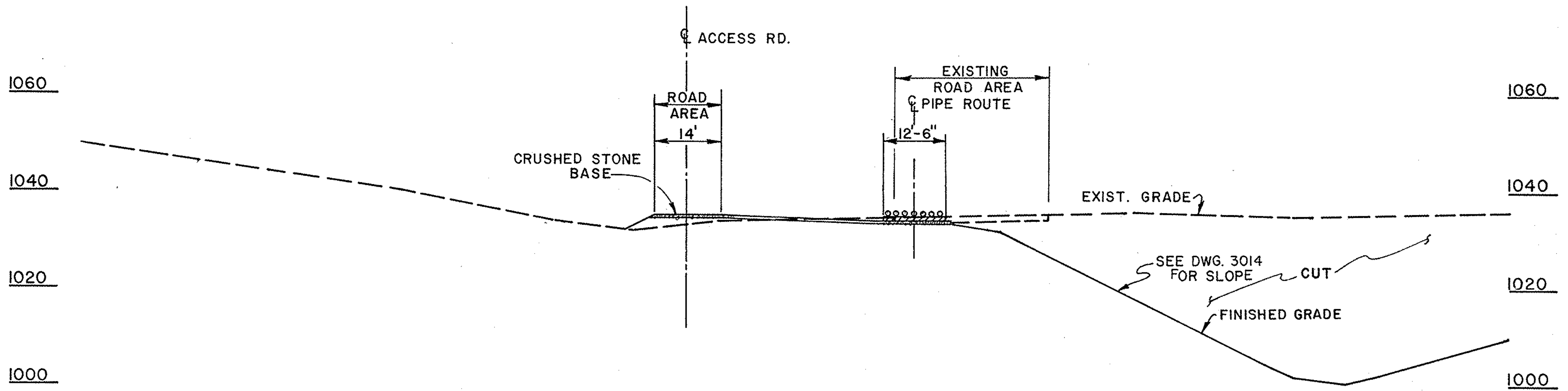
SECTION L-L (DWG. 3013)
SCALE 1" = 20'



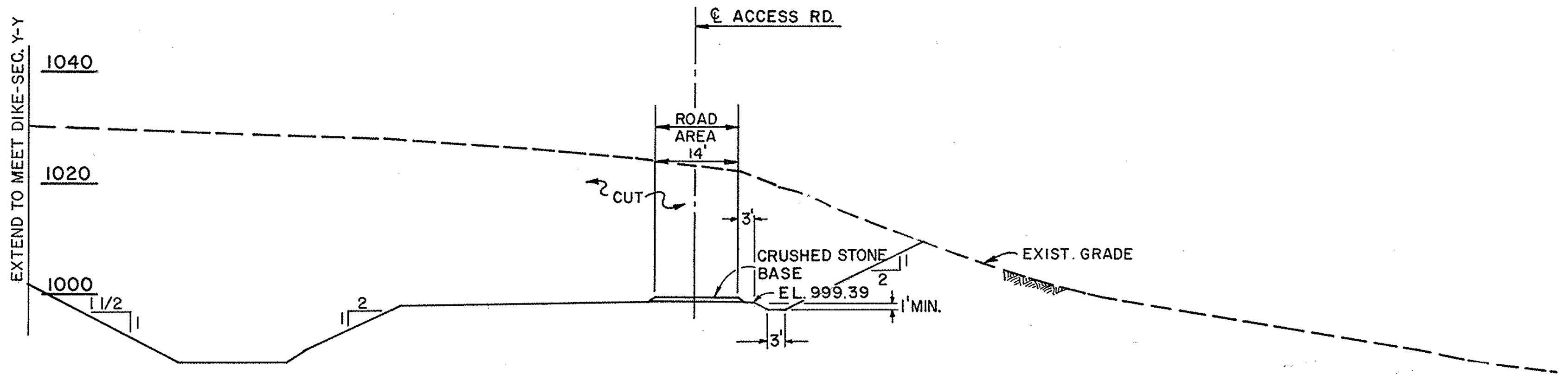
SECTION J-J (DWG. 3013)
SCALE 1" = 20'
(TYPICAL - STA. 36+79.00 THRU STA. 45+00.00)



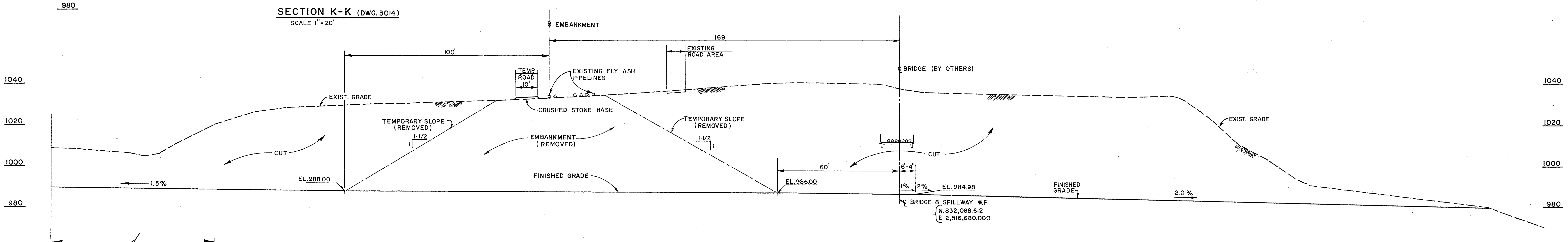
SECTION W-W (DWG. 3020)
SCALE 1" = 20'



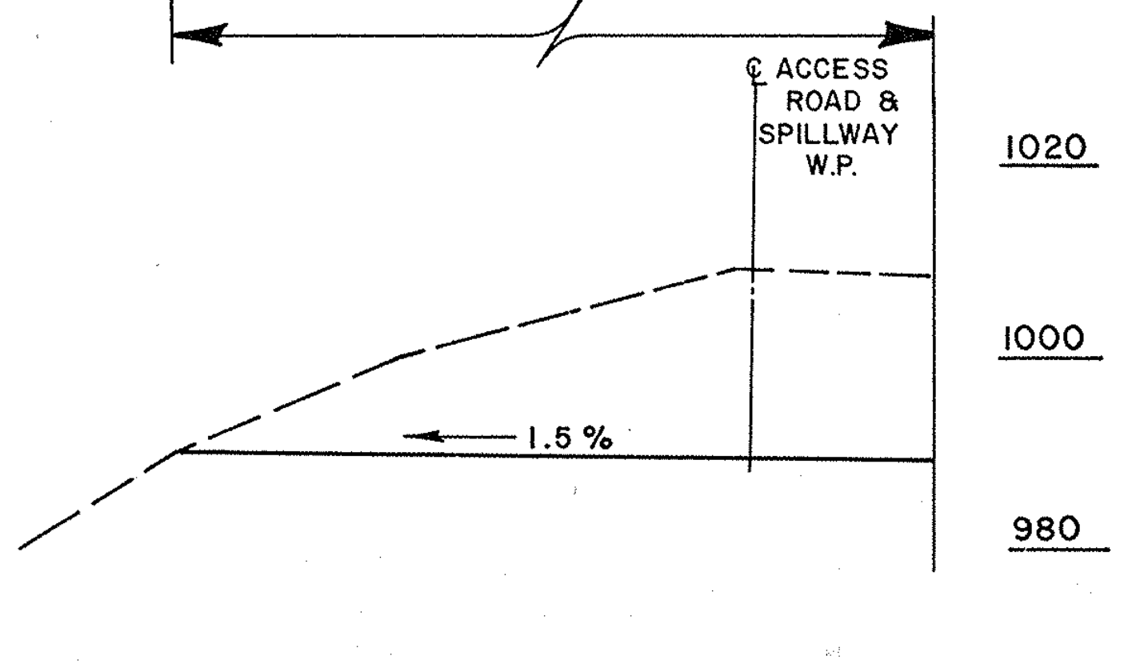
SECTION V-V (DWG. 3014)
SCALE 1" = 20'



SECTION K-K (DWG. 3014)
SCALE 1" = 20'



SECTION Y-Y (DWG. 3014)
SCALE 1" = 20'



GENERAL NOTES

1.- FOR GENERAL NOTES, SEE DWG. 13-3012.
2.- FOR TYPICAL SECTION OF ACCESS ROAD, PIPE ROUTE & DRAINAGE, SEE DWG. 13-3019.

REFERENCE DRAWINGS

FOR REFERENCE DRAWINGS, SEE DWG. 13-3013.

DATE	NO.	DESCRIPTION	APPD.
7/7/87	4	REVISED SECTION "H-H" AS PER HORIZ. & VERT ALIGNMENT CHANGE E.F.1 7-4-8	
5/4/86	3	REV. SECT. "H-H" & "Y-Y" AS PER FIELD INFO., "AS BUILT." R.D.	
1/8/84	2	CLARIFIED DIMS. AT AREAS "C-2", "C-4" & "G-3".	SF/TS
9/10/84	1	REV. & REVERSE SLOPE ON SEC. "Y-Y" REV. SEC. "K-K" & PIPE ROUTE 13'-0" WAS 12'-6"	SF/TS

REVISIONS

MF
MF

BUCKEYE POWER CO.
OHIO POWER CO.
CARDINAL FLY ASH DAM 2
BRILLIANT, OHIO

FLY ASH LINES
ACCESS ROAD & DRAINAGE
SECTIONS & DETAILS
SHEET № 2

DR. NO. 13-3018-3

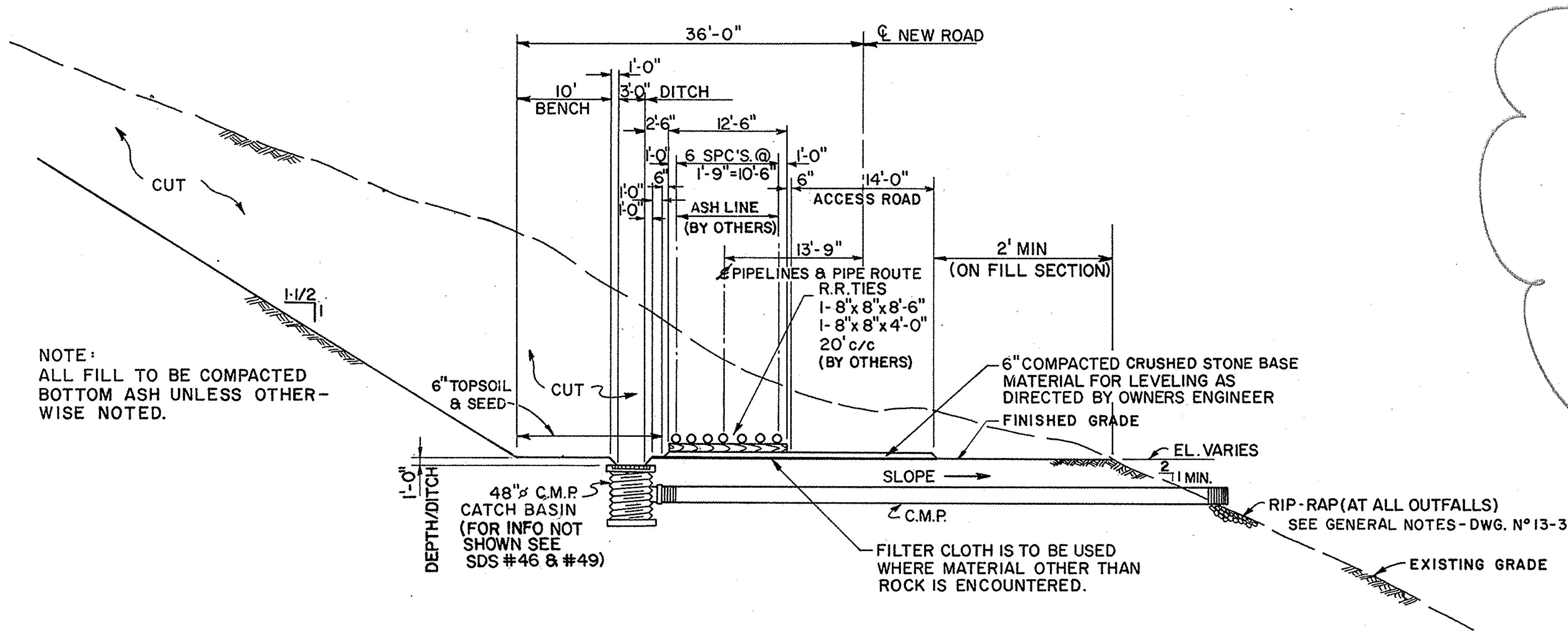
ARCH.	—	ELEC	—	MECH.		STR.	TS

SCALE: SHOWN
DR. K.S. DAVIES
CH. R.D.
SQ. LDR T.S.
DATE: 7/2/84
APPROVED BY
ENGINEERING DIV
DESIGN DIV
JA DiBella/VDH

AMERICAN ELECTRIC POWER SERVICE CORP.

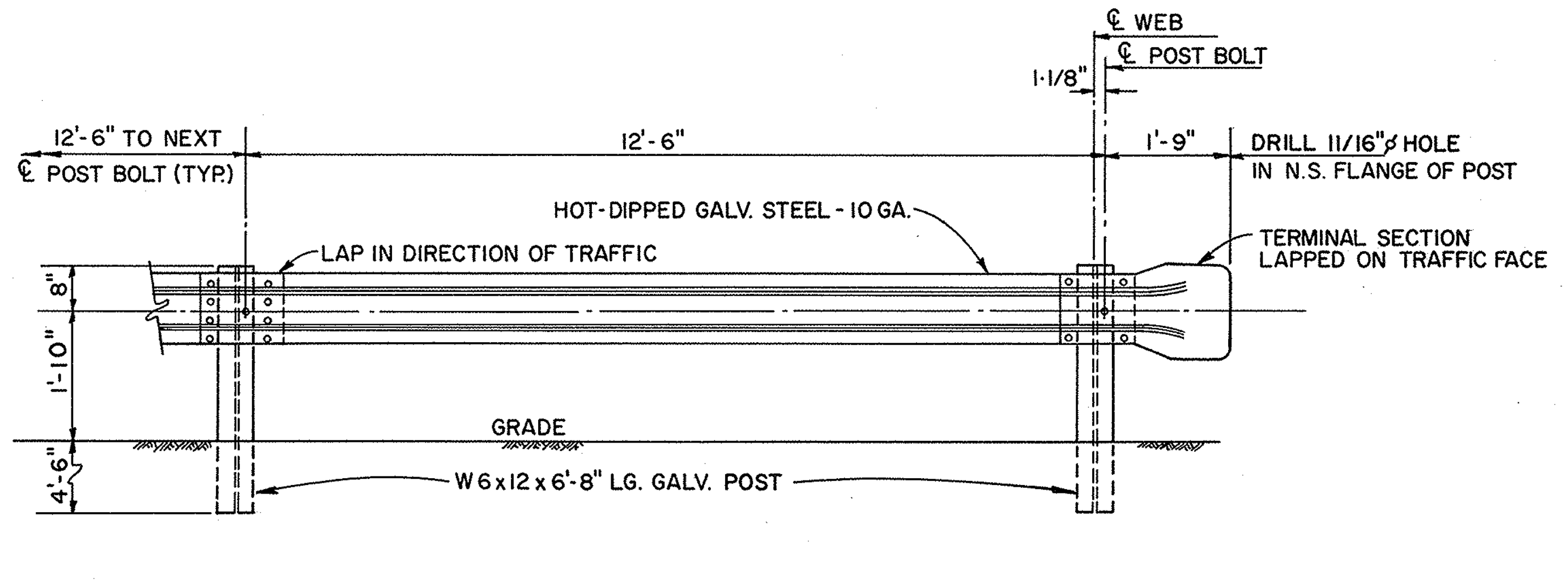
AEP3046XY

DR. No. 13-3019

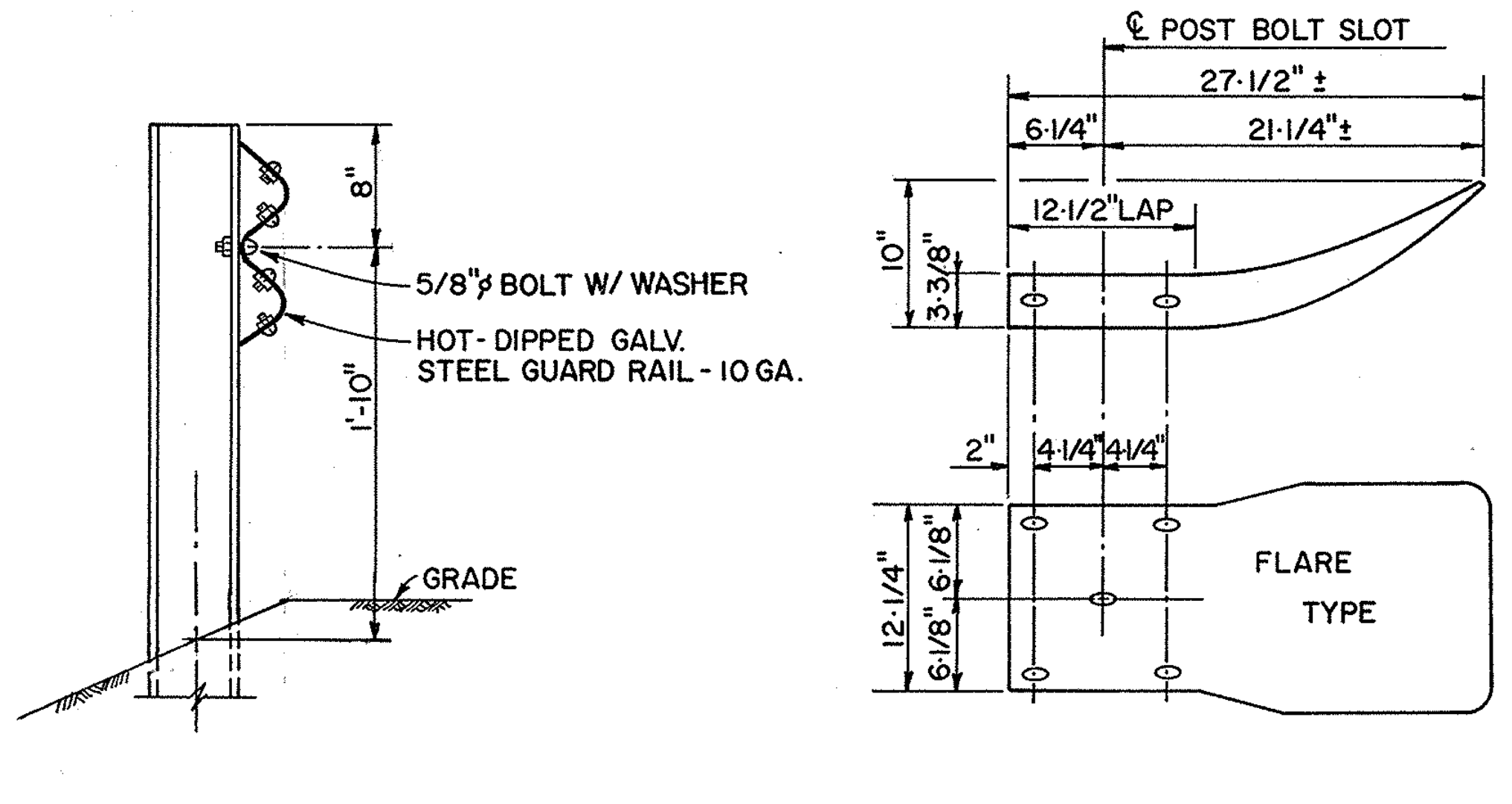


TYPICAL SECTION OF ACCESS ROAD, PIPE ROUTE & DRAINAGE (LOOKING NORTH)

SCALE: 1" = 10'



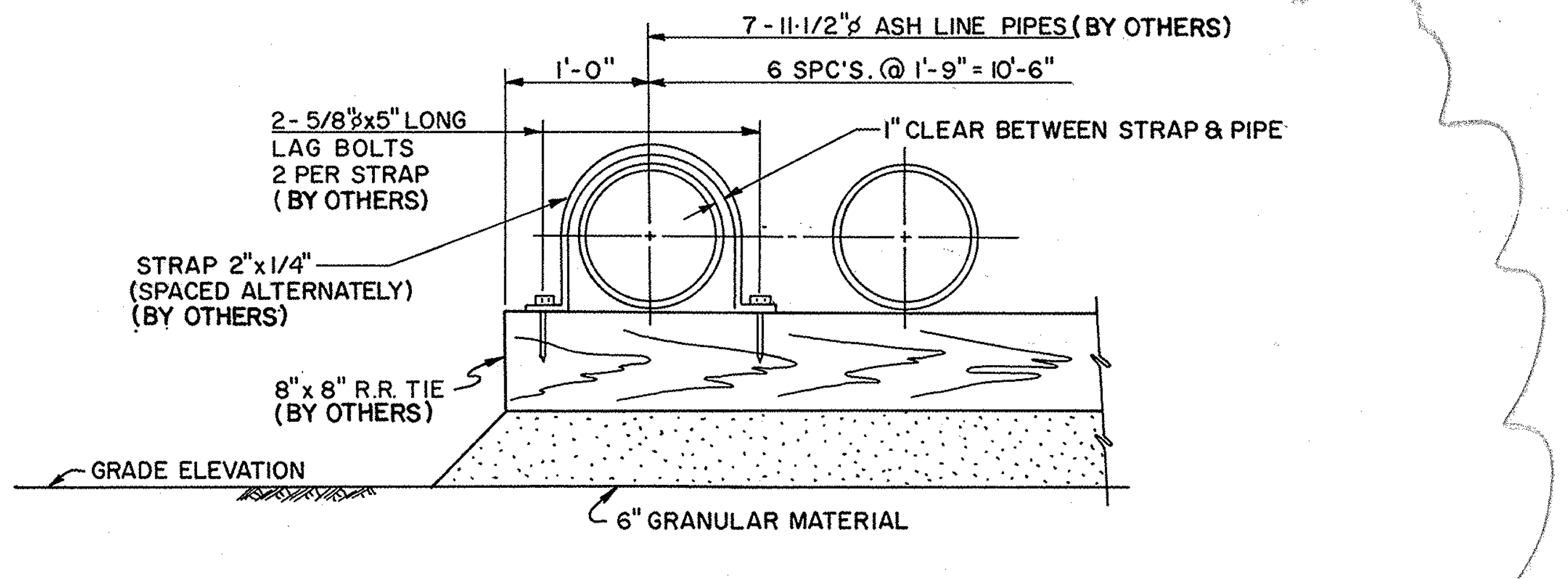
ELEVATION



ARRG'T. AT POST

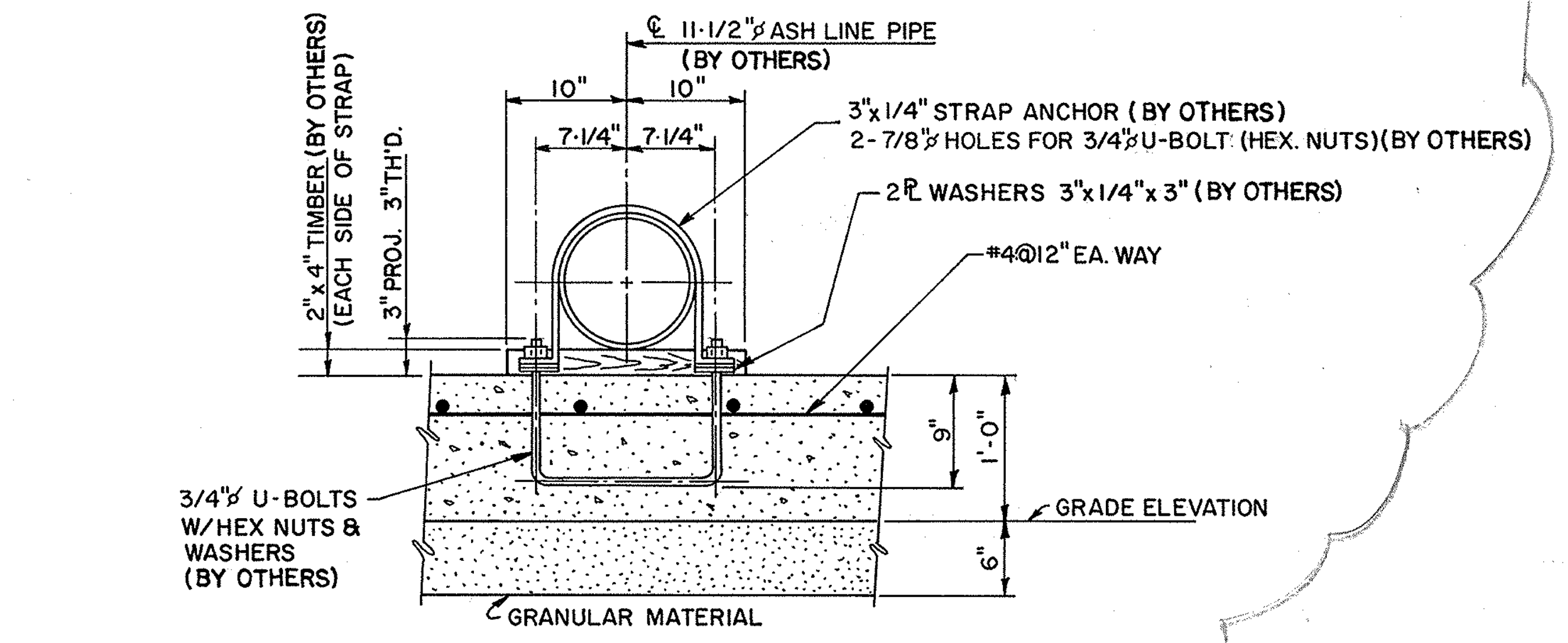
TERMINAL SECTION

GUARD RAIL DETAILS



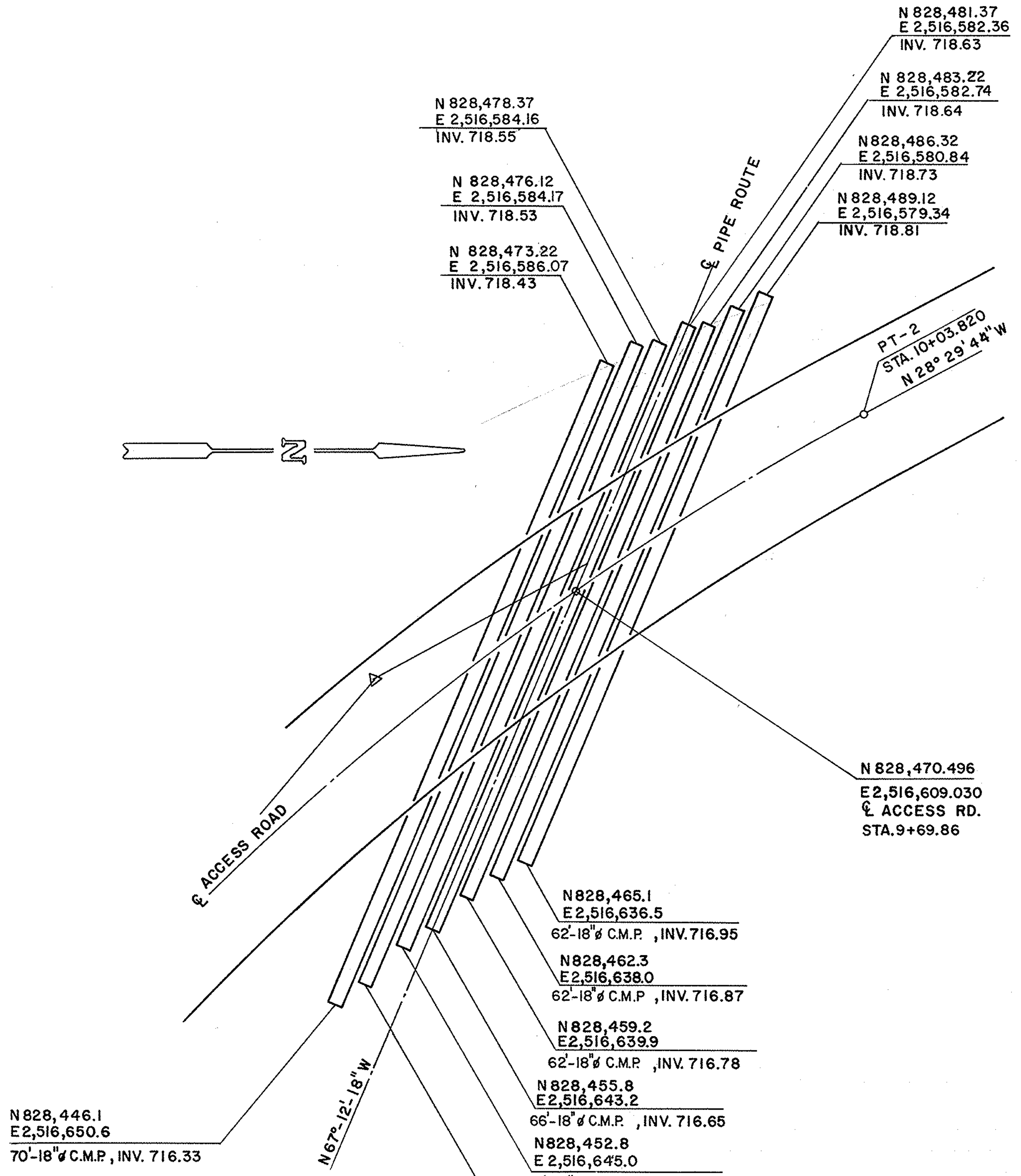
DETAIL OF NON-RESTRAINT SUPPORT

SCALE 1"=1'-0"



DETAIL OF RESTRAINT SUPPORT

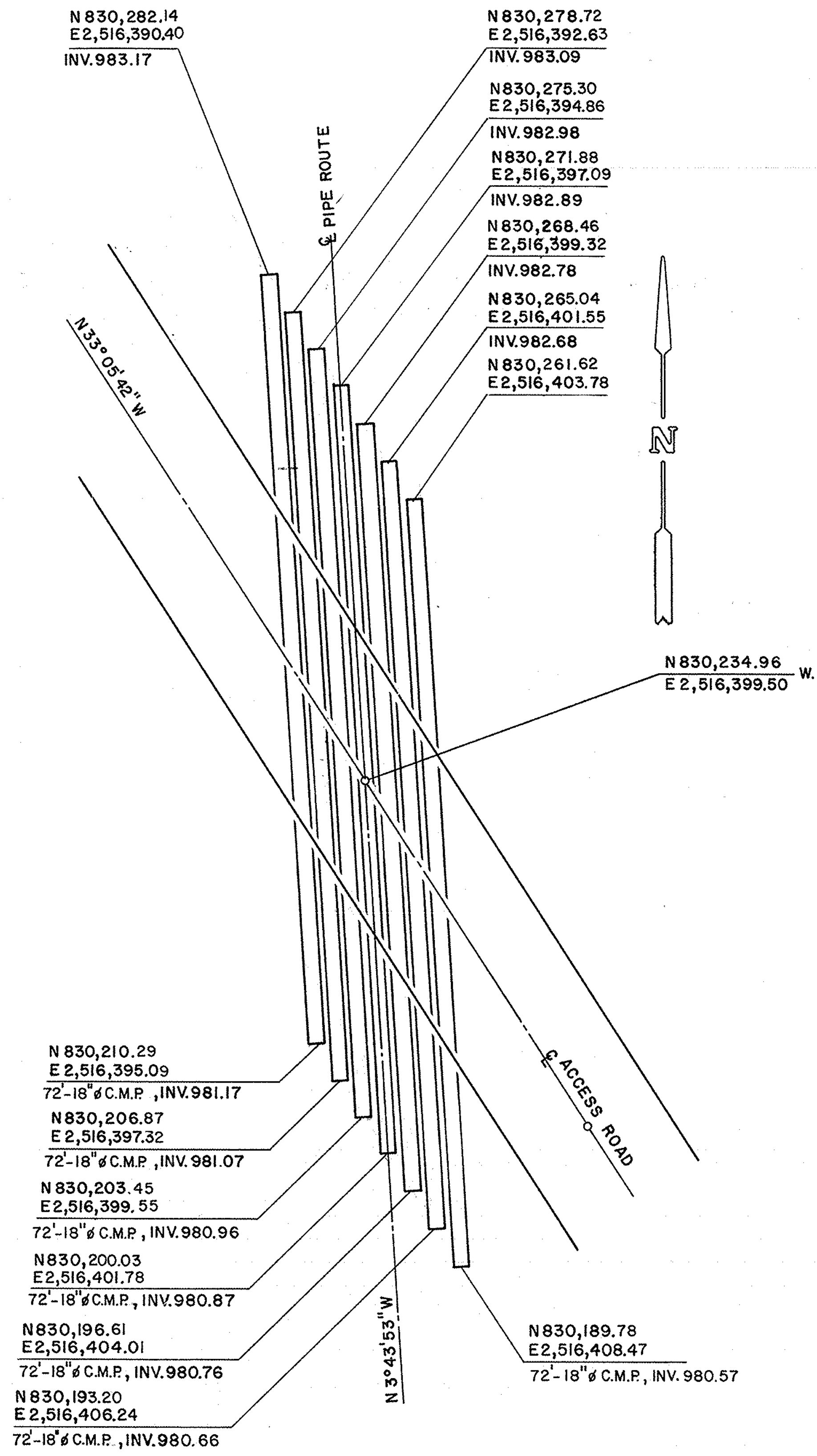
SCALE 1"=1'-0"



PIPE CROSSING Nº 1 DETAIL

SCALE 1"= 10'

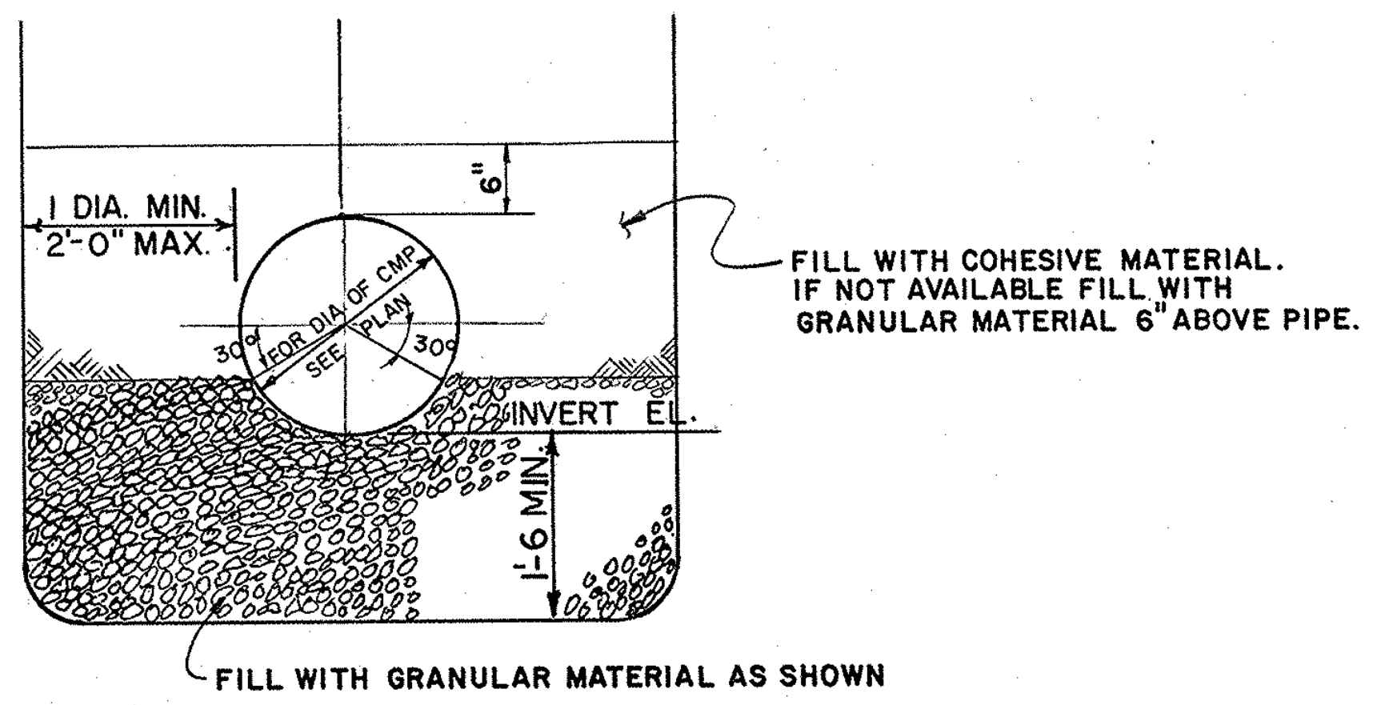
(FOR LOCATION, SEE DWG.13-3012)



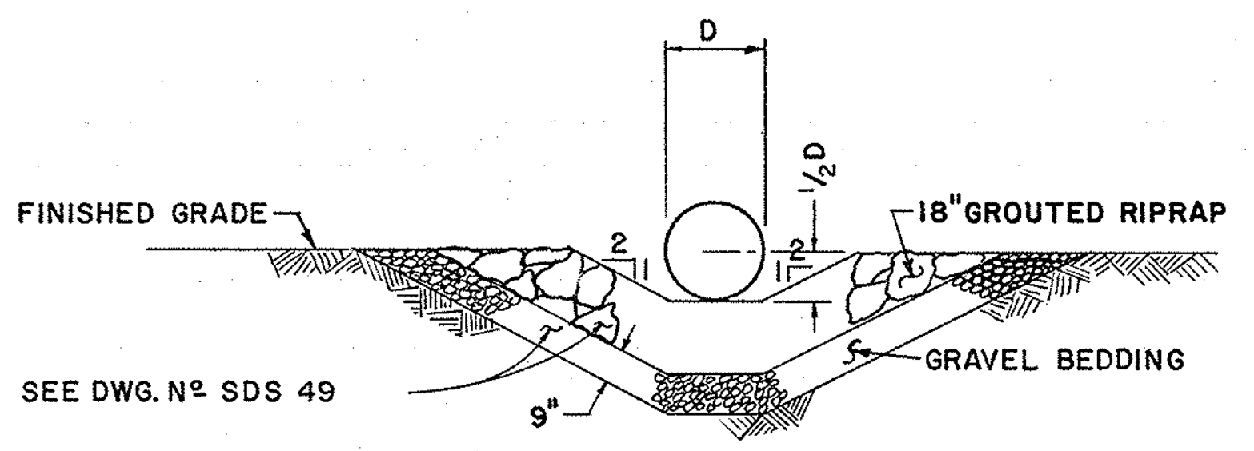
PIPE CROSSING Nº 2 DETAIL

SCALE: 1"= 10'

(FOR LOCATION, SEE DWG. 13-3013)

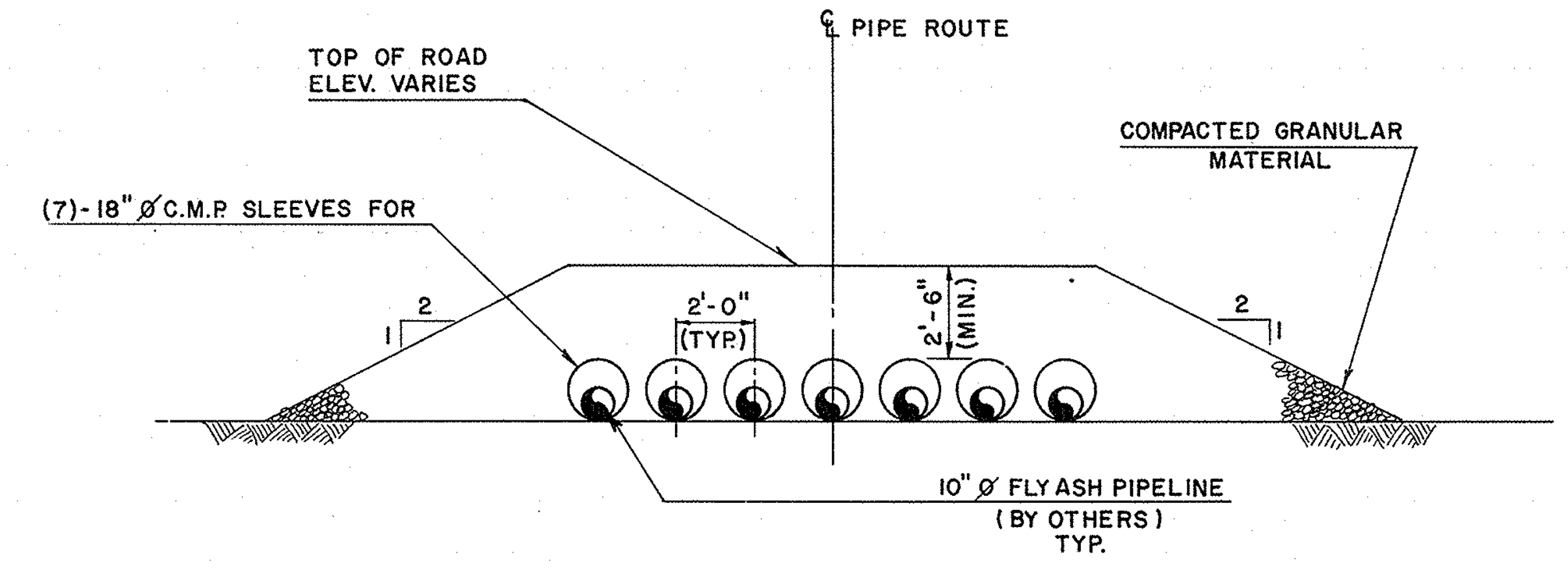


TYPICAL SECTION THRU· DRAINAGE PIPE·



TYPICAL SECTION OF RIPRAP AT C.M.P. DRAIN PIPE OUTFALLS

NO SCALE



PIPE CROSSING TYPICAL SECTION

SCALE: 1" = 4'-0"

GENERAL NOTES

1.- FOR GENERAL NOTES, SEE DWG. 13-3012.
2.- FOR MATERIALS LIST AND REFERENCE DRAWINGS, SEE DWG. 13-3013.

MATERIAL LIST
FOR GUARD RAIL ONLY

1538-L.F. HOT DIPPED GALV. GUARD RAIL
124-EA. HOT DIPPED GALV. GUARD RAIL POST
1125-EA. 5/8"ø BOLTS W/WASHERS

DATE	NO.	DESCRIPTION	APPD.
4/23/87	1	REVISED DIM'S AT G-2 & H-4. REMOVED HOLD ON GUARD RAIL RESTRAINT & NON-RESTRAINT SUPPORTS G-0 & I-4 JAR J.HILL	JEP

REVISIONS

BUCKEYE POWER CO.
OHIO POWER CO.
CARDINAL FLY ASH DAM 2
BRILLIANT OHIO

FLY ASH LINES
ACCESS ROAD & DRAINAGE
SECTIONS & DETAILS
SHEET Nº 3

DR. NO. 13-3019-1

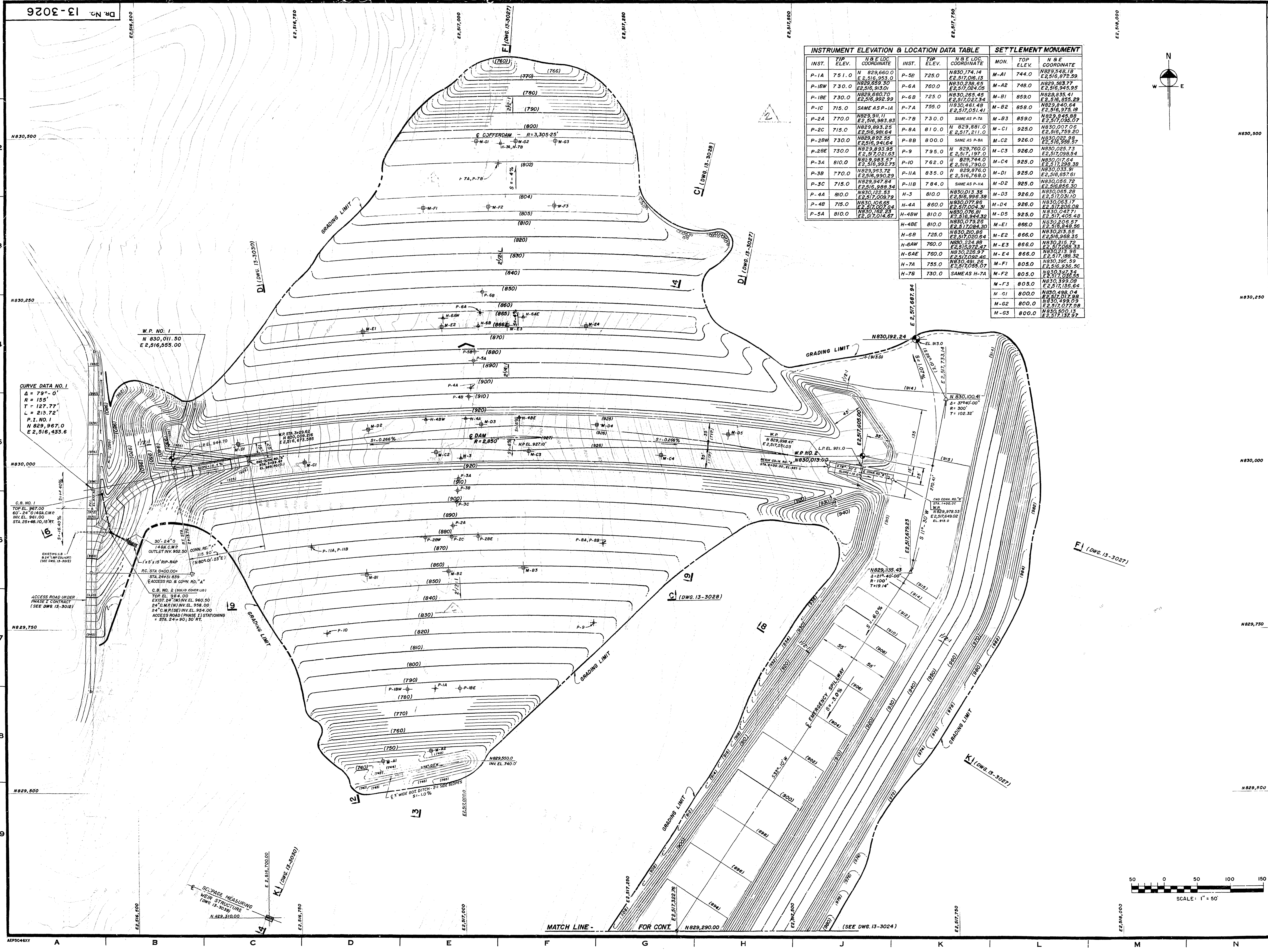
ARCH.	–	ELEC	–	MECH.		STR.	75

SCALE: As Noted
DR. M. Zengler
CH. R.D.
SQ. LDR T.S.
DATE: 7/2/84

APPROVED BY
ENGINEERING DIV
DESIGN DIV JA D. Bellavay

AMERICAN ELECTRIC POWER SERVICE CORP.

AEP3046XY



INSTRUMENT ELEVATION & LOCATION DATA TABLE

INST.	TIP ELEV.	N & E LOC. COORDINATE	INST.	TIP ELEV.	N & E LOC. COORDINATE
P-1A	751.0	N 829,660.0 E 2,516,953.0	P-5B	725.0	N830,174.14 E2,517,016.13
P-1BW	730.0	N829,659.30 E2,516,913.01	P-6A	760.0	N830,238.65 E2,517,024.05
P-1BE	730.0	N829,660.70 E2,516,992.99	P-6B	725.0	N830,265.45 E2,517,027.34
P-1C	715.0	SAME AS P-1A	P-7A	755.0	N830,461.48 E2,517,051.41
P-2A	770.0	N829,911.11 E2,516,983.83	P-7B	730.0	SAME AS P-7A
P-2C	715.0	N829,893.25 E2,516,981.64	P-8A	810.0	N 829,881.0 E 2,517,211.0
P-2BW	730.0	N829,892.55 E2,516,941.64	P-8B	800.0	SAME AS P-8A
P-2BE	730.0	N829,893.95 E2,517,021.63	P-9	795.0	N 829,760.0 E 2,517,197.0
P-3A	810.0	N829,983.57 E2,516,992.73	P-10	762.0	N 829,744.0 E 2,516,790.0
P-3B	770.0	N829,963.72 E2,516,990.29	P-11A	835.0	N 829,876.0 E 2,516,768.0
P-3C	715.0	N829,947.84 E2,516,988.34	P-11B	784.0	SAME AS P-11A
P-4A	810.0	N830,122.53 E2,517,009.79	H-3	810.0	N830,013.35 E2,516,996.38
P-4B	715.0	N830,106.65 E2,517,007.84	H-4A	860.0	N830,077.86 E2,517,004.31
P-5A	810.0	N830,162.23 E2,517,014.67	H-4BW	810.0	N830,076.81 E2,516,944.32
			H-4BE	810.0	N830,079.26 E2,517,084.30
			H-6B	725.0	N830,210.86 E2,517,020.64
			H-6AW	760.0	N830,224.88 E2,516,972.47
			H-6AE	760.0	N830,226.97 E2,517,092.46
			H-7A	755.0	N830,491.26 E2,517,055.07
			H-7B	730.0	SAME AS H-7A

SETTLEMENT MONUMENT

MON.	TOP ELEV.	N & E COORDINATE
M-A1	744.0	N829,548.18 E2,516,872.59
M-A2	748.0	N829,563.77 E2,516,945.95
M-B1	859.0	N829,835.41 E2,516,855.29
M-B2	858.0	N829,840.64 E2,516,975.18
M-B3	859.0	N829,845.88 E2,517,095.07
M-C1	925.0	N830,007.06 E2,516,759.20
M-C2	926.0	N830,022.98 E2,516,958.57
M-C3	926.0	N830,025.73 E2,517,098.54
M-C4	925.0	N830,017.64 E2,517,298.38
M-D1	925.0	N830,033.91 E2,516,657.61
M-D2	925.0	N830,056.72 E2,516,856.30
M-D3	926.0	N830,065.28 E2,517,031.10
M-D4	926.0	N830,063.17 E2,517,206.08
M-D5	925.0	N830,047.71 E2,517,405.48
M-E1	866.0	N830,206.57 E2,516,848.56
M-E2	866.0	N830,213.55 E2,516,968.35
M-E3	866.0	N830,215.72 E2,517,068.33
M-E4	866.0	N830,213.96 E2,517,188.32
M-F1	805.0	N830,395.59 E2,516,936.56
M-F2	805.0	N830,397.34 E2,517,036.55
M-F3	805.0	N830,399.08 E2,517,136.64
M-G1	800.0	N830,498.04 E2,517,017.98
M-G2	800.0	N830,499.09 E2,517,077.98
M-G3	800.0	N830,500.13 E2,517,137.97

GENERAL NOTES

1.- FOR FILL MATERIAL, PROCEDURE AND COMPACTION, SEE A.E.P. CIVIL ENGINEERING SPECIFICATIONS.

2.- ALL ELEVATIONS SHOWN ARE TO TOP OF FINISHED SURFACES.

——920——	EXISTING GRADE CONTOUR
(920)	FINISHED GRADE CONTOUR
⊡ M-A1	SETTLEMENT MONUMENT
⊖ P-1A	PNEUMATIC PIEZOMETER
△ H-3	HYDRAULIC PIEZOMETER

3.- DRAINAGE PIPES & CATCH BASINS SHALL BE CORRUGATED GALVANIZED STEEL INTERIOR & EXTERIOR AND FULLY FIBER BONDED. DRAINAGE PIPES SHALL HAVE ANNULAR ENDS WITH PAVED INVERT.

4.- FOR DETAILS OF CATCH BASINS AND PIPES, SEE DWGS. SDS-46 & SDS-47.

MATERIALS

24"Ø - 14 GA. C.M.P. —————— 90 L.F.
48"Ø - 14GA. C.M.P. C.B. NO. 1 & 2 —— 2 REQ'D.
BY: OWNER

FRAME: NEENAH FOUNDRY CO. R-6078A OR EQUAL —————— 2 REQ'D.
SOLID COVER: NEENAH R-6078A OR EQUAL - 1 REQ'D.
GRATE: NEENAH R-6117 OR EQUAL — 1 REQ'D.
BY: OWNER

REFERENCE DRAWINGS

13-3023 - DAM & EMERGENCY SPILLWAY EXCAVATION PLAN.
13-3027 - DAM & EMERGENCY SPILLWAY SECTIONS - SHEET 1.
13-3028 - DAM & EMERGENCY SPILLWAY SECTIONS - SHEET 2.
13-3024 - EMERGENCY & SERVICE SPILLWAYS CONTINUATION LAYOUT & GRADING PLAN.
13-3030 - DAM & EMERGENCY SPILLWAY SECTIONS SHEET 3.

REVISIONS

DATE	NO.	DESCRIPTION	APPD.
5/21/90	2	RELOCATED & ADDED SETTLEMENT MONUMENTS. ADDED PNEUMATIC & HYDRAULIC PIEZOMETERS. REV. LOCATION DATA TABLE. R.D.	
3/22/85	1.	AS PER RDR-CA-123057, RELOC. COFFERDAM & REV. GRADING. SLP/RD	

CARDINAL
OPERATING COMPANY
CARDINAL FLY ASH DAM 2
BRILLIANT, OHIO

DAM & EMERGENCY SPILLWAY LAYOUT & GRADING PLAN INSTRUMENTATION LOCATION

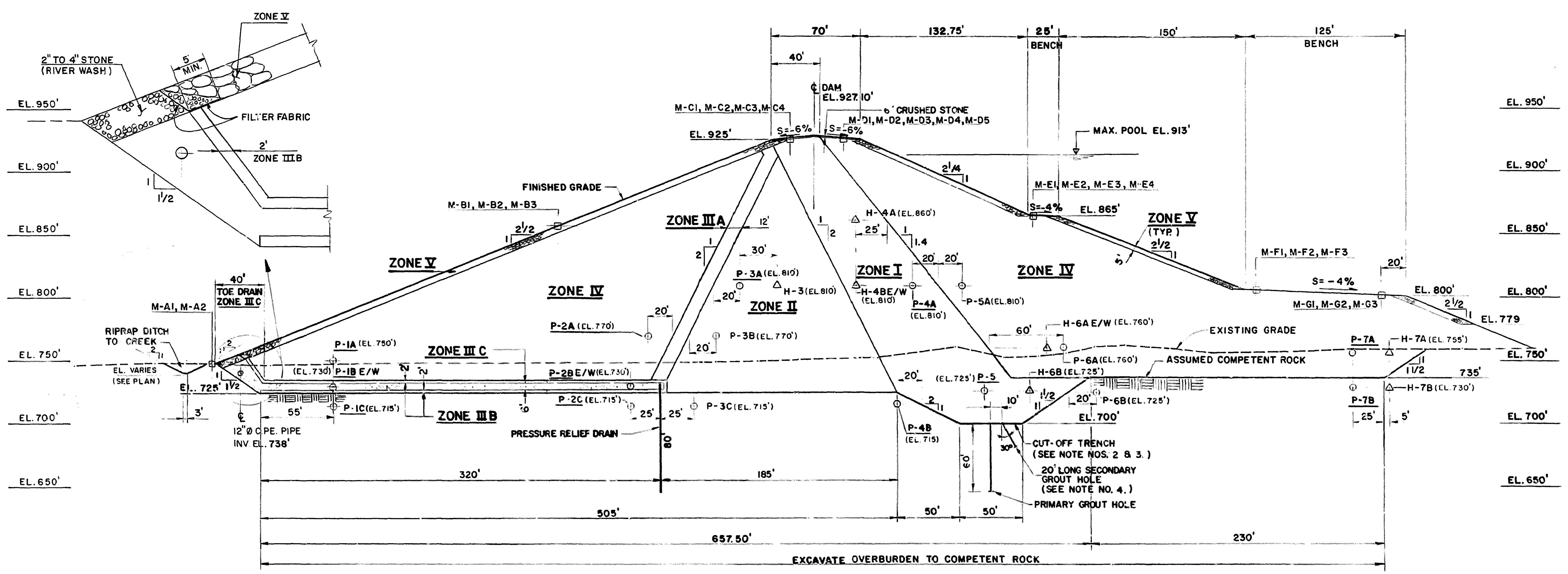
DR. NO. 13-3026-2

ARCH.	—	ELEC.	—	MECH.	—	STR.	75

SCALE: 1" = 50'
DR. R.G. DURO
CH 73
SQ LDR 75
DATE: 1-4-85
APPROVED BY
ENGINEERING DIV
DESIGN DIV JA Di Bella/vej

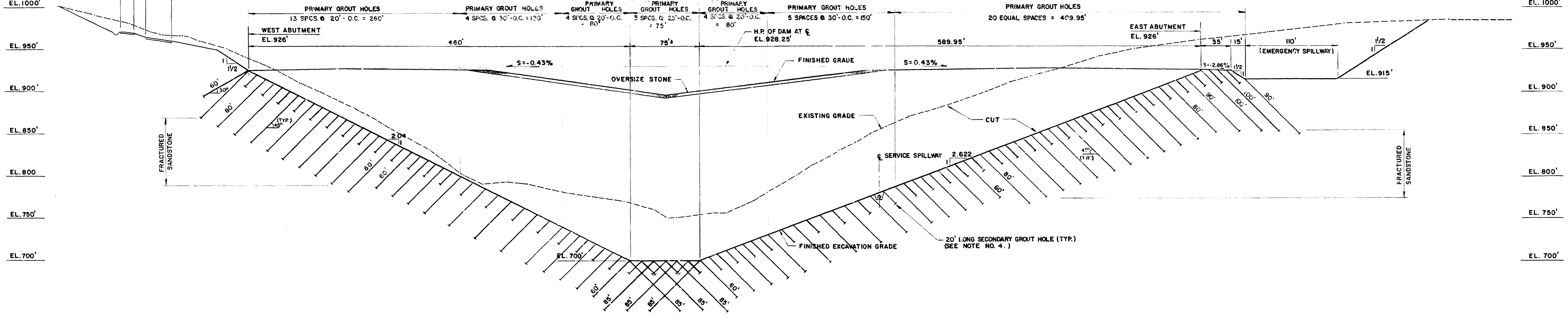
AMERICAN ELECTRIC POWER SERVICE CORP.

DR. No. 13-3027



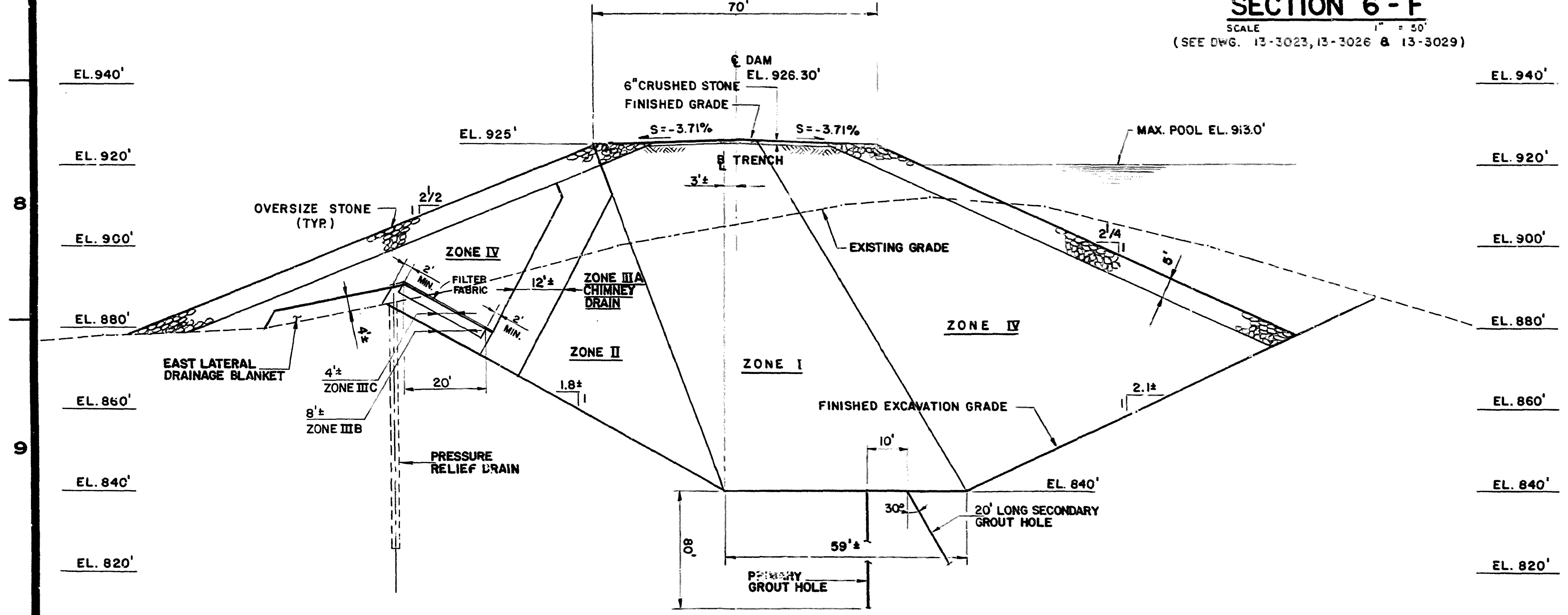
SECTION 3-F

SCALE 1" = 50'
(SEE DWG. 13-3023, 13-3026 & 13-3029)



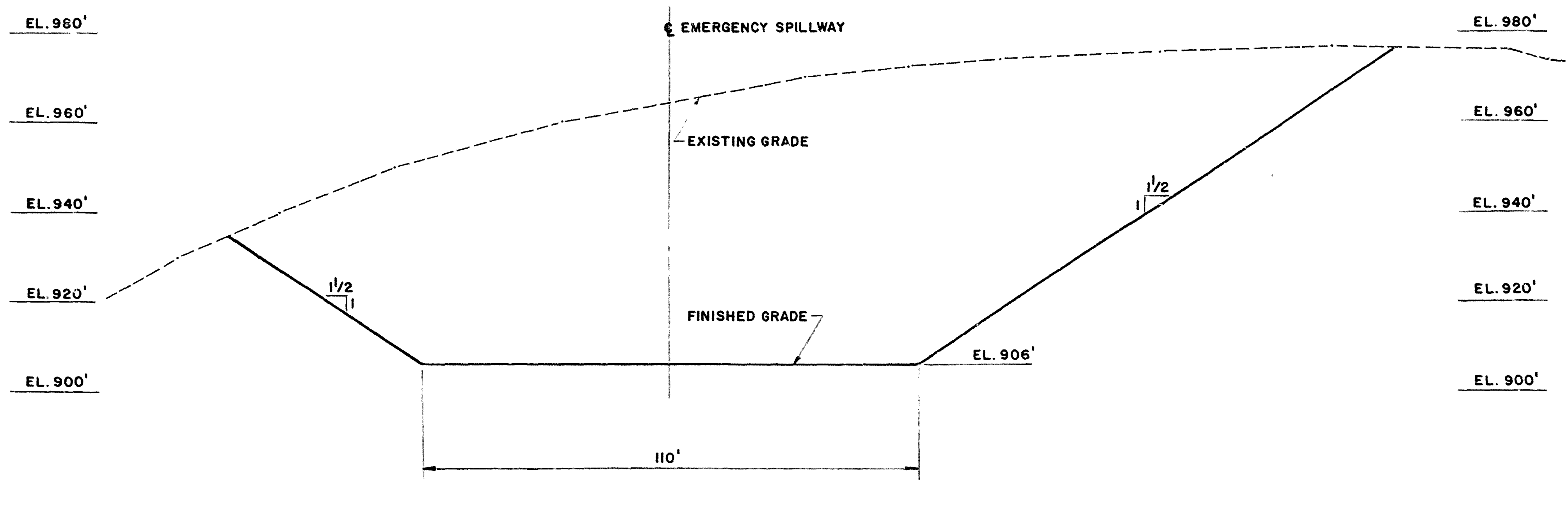
SECTION 6-F

SCALE 1" = 50'
(SEE DWG. 13-3023, 13-3026 & 13-3029)



SECTION 9-D

SCALE: 1" = 20'
(SEE DWG. 13-3023, 13-3026 & 13-3029)



SECTION 8-K

SCALE: 1" = 20'
(SEE DWG. 13-3026)

FILL MATERIALS

ZONE	DESCRIPTION	SOURCE	COMPACTION CONTROL	MOISTURE CONTROL	MAXIMUM LOOSE LIFT THICKNESS	ROLLER TYPE
I	CLAY CORE (≤ 6")	BORROW AREA I	$\gamma_D \geq 100\%$ (γ_D) MAX.	-1% TO +2% OF OPTIMUM	6"	SHEEPS FOOT
II	MINE SPOIL (FRACTION ≤ 6")	BORROW AREA II EMERGENCY SPILLWAY & ABUTMENT EXCAVATION	$\gamma_D \geq 100\%$ (γ_D) MAX.	-1% TO +2% OF OPTIMUM	9"	VIBRATORY SHEEPS FOOT
III A	UNITS 1, 2 & 3 BOTTOM ASH	BOTTOM ASH POND	70% OR HIGHER RELATIVE DENSITY	SATURATED	9"	LIGHT VIBRATORY
III B	UNITS 1 & 2 BOTTOM ASH	BOTTOM ASH POND	70% OR HIGHER RELATIVE DENSITY	SATURATED	9"	LIGHT VIBRATORY
III C	SAND & GRAVEL	LOCAL SUPPLIER	70% OR HIGHER RELATIVE DENSITY	SATURATED	9"	VIBRATORY
IV	MINE SPOIL (FRACTION ≤ 12")	BORROW AREA II EMERGENCY SPILLWAY & ABUTMENT EXCAVATION	$\gamma_D \geq 100\%$ (γ_D) MAX.	-2% TO +1% OF OPTIMUM	12"	VIBRATORY SHEEPS FOOT
V	OVERSIZE STONE	BORROW AREA II & QUARRY	END DUMPED OR PUSHED WITH DOZER		24"	

(SEE GEN. NOTE NO. 5)

LEGEND

⊟	SETTLEMENT MONUMENT
○	PNEUMATIC PIEZOMETER
△	HYDRAULIC PIEZOMETER

GENERAL NOTES

1. FOR GENERAL NOTES, SEE DWG. 13-3032.
2. THE BOTTOM OF CUT-OFF TRENCH SHALL BE SLUSH GROUTED AS DIRECTED BY OWNER'S ENGINEER.
3. THE SIDE SLOPES OF THE CUT-OFF TRENCH SHALL BE REPAIRED WITH DENTAL CONCRETE AS DIRECTED BY OWNER'S ENGR.
4. PRIMARY & SECONDARY GROUT HOLES SHALL BE SPLIT SPACED & STAGGERED AS SHOWN IN DRAWING 13-3029. SECONDARY GROUT HOLES SHALL BE BATTERED 30° WITH RESPECT TO VERTICAL.
5. MAXIMUM DRY UNIT WEIGHT, (γ_D)MAX., & OPTIMUM MOISTURE CONTENT AS DETERMINED BY ASTM D698-78 METHOD A FOR ZONE I AND METHOD C FOR ZONES II & IV. RELATIVE DENSITY AS DETERMINED BY ASTM D2049-69.

REFERENCE DRAWINGS

13-3023 - DAM & EMERGENCY SPILLWAY EXCAVATION PLAN.
13-3025 - DAM UNDERDRAIN SYSTEM - PLAN.
13-3026 - DAM & EMERGENCY SPILLWAY LAYOUT & GRADING PLAN.
13-3029 - EXCAVATION, FILL, RELIEF WALLS, GROUT & SPILLWAYS - COMPOSITE PLAN.

DATE	NO.	DESCRIPTION	APPD.
5/19/90	4	REV. SECT. "3-F", ADDED SETTLEMENT MONUMENTS. RELOCATED & ADDED PNEUMATIC & HYDRAULIC PIEZOMETER RN/R.D.	[illegible]
5/2/86	3	REV. SECT. "3-F", ADDED ENLARGED DETAIL OF TOE DRAIN. REV. SECT. "9-D", ADDED ZONES IIIB, IIIC & FILTER FABRIC. R.D.	ASP/GS
9/22/85	2	AS PER RDR-CA-123057 REV. SECT. 3-F [illegible]/RD	[illegible]
3/8/85	1	REV. SECTION 3-F & FILL MATERIALS TABLE.	SF

REVISIONS

CARDINAL
OPERATING COMPANY
CARDINAL FLY ASH DAM 2
BRILLIANT, OHIO

DAM & EMERGENCY SPILLWAY
SECTIONS - SHEET 1

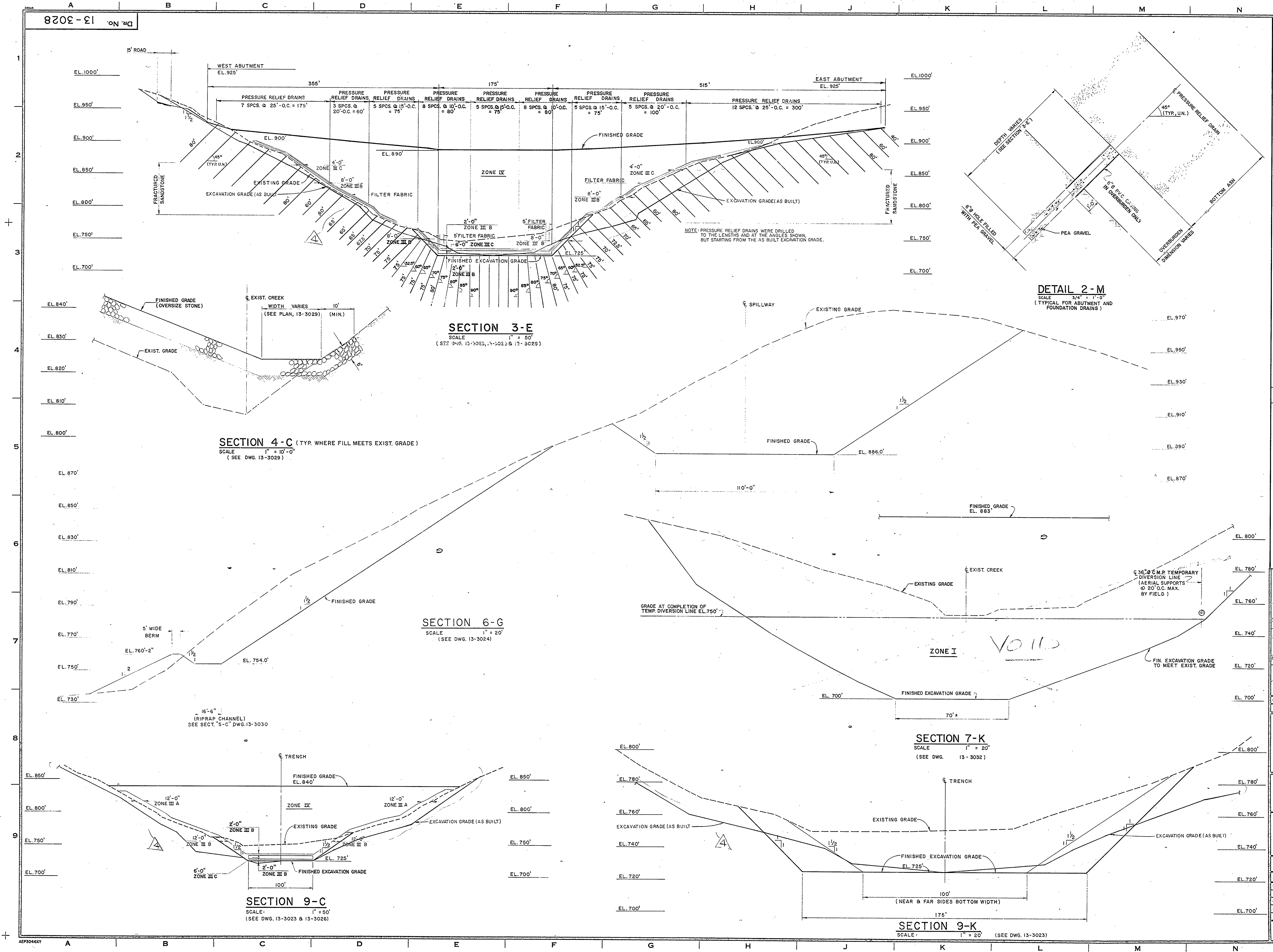
DR. NO. 13 - 3027 - 4

ARCH.	–	ELEC	–	MECH.	-	STR.	TS

SCALE: AS NOTED
DR. R. G. DURO
CH. TS
SQ. LDR TS
DATE: 1-1-85
APPROVED BY
ENGINEERING DIV
DESIGN DIV J.A. DiBello

AMERICAN ELECTRIC POWER SERVICE CORP.

DR. No. 13-3028



SECTION 3-E

SCALE 1" = 50'
(SEE DWG. 13-3023, 13-3026 & 13-3029)

DETAIL 2-M

SCALE 3/4" = 1'-0"
(TYPICAL FOR ABUTMENT AND FOUNDATION DRAINS)

SECTION 4-C (TYP. WHERE FILL MEETS EXIST. GRADE)

SCALE 1" = 10'-0"
(SEE DWG. 13-3029)

SECTION 6-G

SCALE 1" = 20'
(SEE DWG. 13-3024)

SECTION 7-K

SCALE 1" = 20'
(SEE DWG. 13-3032)

SECTION 9-C

SCALE: 1" = 50'
(SEE DWG. 13-3023 & 13-3026)

SECTION 9-K

SCALE: 1" = 20' (SEE DWG. 13-3023)

GENERAL NOTES

1.- FOR GENERAL NOTES, SEE DWG. 13-3032
2.- LATERAL DRAINAGE BLANKET SHALL BE INSTALLED ON TOP OF ROCK WHERE OVERBURDEN IS REMOVED.
3- RELIEF DRAINS SHALL BE INSTALLED ONLY WHERE OVERBURDEN IS NOT REMOVED OR AS DIRECTED BY THE OWNER'S ENGINEER.

REFERENCE DRAWINGS

13-3023	DAM & EMERGENCY SPILLWAY EXCAVATION PLAN.
13-3024	EMERGENCY & PRINCIPAL SPILLWAYS. LAYOUT & GRADING PLAN.
13-3026	DAM & EMERGENCY SPILLWAY LAYOUT & GRADING PLAN.
13-3029	EXCAVATION, FILL, RELIFE WELL, GROUT & SPILLWAYS COMPOSITE PLAN.
13-3025	DAM UNDERDRAIN SYSTEM - PLAN.
13-3032	SERVICE SPILLWAY & COFFERDAM LAYOUT & GRADING PLAN.

DATE	NO.	DESCRIPTION	APPD.
1/7/88	4	ADDED NOTE, PRESSURE RELIEF DRAINS VOIDED SECT. "7-K" ADDED AS BUILT EXCAVATION TO SECTS. "3-E", "9-C" & "9-K"	
5/2/86	3	REV. SECT. "3-E", ZONE III B WAS IIIA & DIM. 8'-0" WAS 12'-0". ADDED ZONE IIIC & FILTER FABRIC. R.M./R.D.	ASP / GS
8.22.85	2	ADDED NOTE ON SECT. "4-C".	
3/8/85	1	ADDED ZONE III C ON SECT. "9-C", ADDED PRESSURE RELIEF DRAINS & ZONE III C ON SECTION "3-E", REV. SECTION "7-K" & ADDED 6"Ø P.V.C. CASING ON DET. 2-M."	SF

REVISIONS

CARDINAL
OPERATING COMPANY
CARDINAL FLY ASH DAM 2
BRILLIANT, OHIO

DAM & EMERGENCY SPILLWAY
SECTIONS - SHEET 2
AS BUILT

DR. NO. 13-3028-4

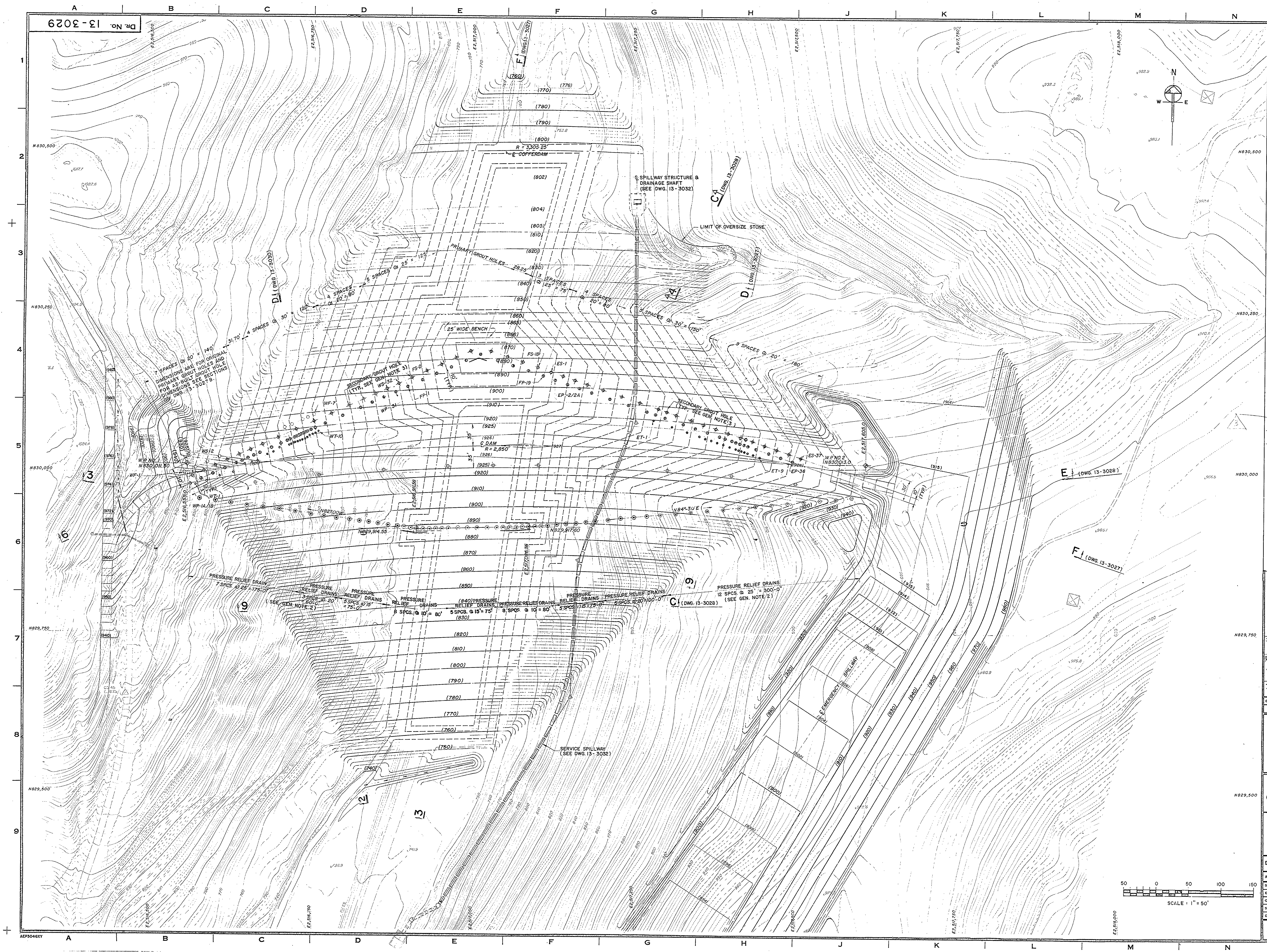
ARCH. – ELEC – MECH. – STR. 75

SCALE: AS NOTED
DR. R.M./ACY
CH. R.D.
SQ. LDR T.S.
DATE: 1-4-85

APPROVED BY
ENGINEERING DIV PH Q
DESIGN DIV JA DeBella

AMERICAN ELECTRIC POWER SERVICE CORP.

AEP3046XY



GENERAL NOTES

1. FOR GENERAL NOTES, SEE DWG. 13-3032.
2. RELIEF DRAINS SHALL BE INSTALLED ONLY WHERE OVERBURDEN IS NOT REMOVED OR AS DIRECTED BY THE OWNER'S ENGINEER.
3. PRIMARY & SECONDARY GROUT HOLES SHALL BE SPLIT SPACED & STAGGERED AS SHOWN. SECONDARY GROUT HOLES SHALL BE BATTERED 30° WITH RESPECT TO VERTICAL.

LEGEND:

—900—	EXISTING GRADE
- - - -	EXCAVATION GRADE
(900)	FINISHED GRADE
⊙	PRESSURE RELIEF DRAIN
◎	PRIMARY GROUT HOLE
○	MONUMENT
⊕	SECONDARY GROUT HOLE
•	TERTIARY GROUT HOLE
○	FOURTH GROUT HOLE

REFERENCE DRAWINGS:

13-3023	DAM & EMERGENCY SPILLWAY EXCAVATION PLAN.
13-3024	EMERGENCY & SERVICE SPILLWAYS LAYOUT & GRADING PLAN.
13-3025	DAM UNDERDRAIN SYSTEM - PLAN.
13-3026	DAM & EMERGENCY SPILLWAY LAYOUT & GRADING PLAN.
13-3027	DAM & EMERGENCY SPILLWAY SECTIONS SHEET 1.
13-3028	DAM & EMERGENCY SPILLWAY SECTIONS SHEET 2.
13-3030	DAM & EMERGENCY SPILLWAY SECTIONS SHEET 3.
13-3032	SERVICE SPILLWAY, TEMPORARY ACCESS ROAD & COFFERDAM LAYOUT & GRADING PLAN.

DATE	NO.	DESCRIPTION	APPD.
[illegible]	3	ADDED AS BUILT CONDITIONS. A.C.Y. / T.M.K.	[illegible]
8/22/85	2	ADDED COORDINATES, BEARINGS & DIMENSIONS. AS PER RDR.-CA.-123057, RELOC COFFERDAM & REV. GRADING. SLP/KD	[illegible]
3/8/85	1	ADDED PRESSURE RELIEF DRAINS. AREA "E-7".	SF

REVISIONS

CARDINAL
OPERATING COMPANY
CARDINAL FLY ASH DAM 2
BRILLIANT, OHIO

EXCAVATION, FILL, RELIEF WELLS, GROUT & SPILLWAYS COMPOSITE PLAN
AS BUILT

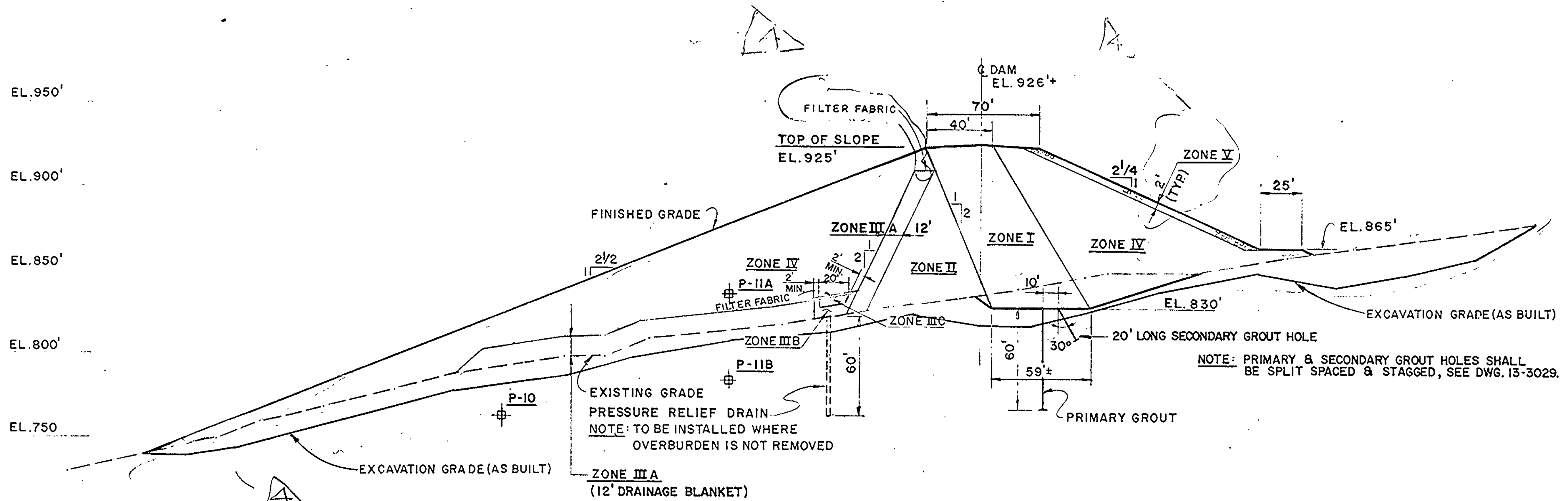
DR. NO. 13-3029-3

ARCH.	–	ELEC	–	MECH.	–	STR.	TS

SCALE: 1" = 50'
DR. R. G. DURO
CH. TS
SQ. LDR TS
DATE: 1-4-85
APPROVED BY
ENGINEERING DIV
DESIGN DIV

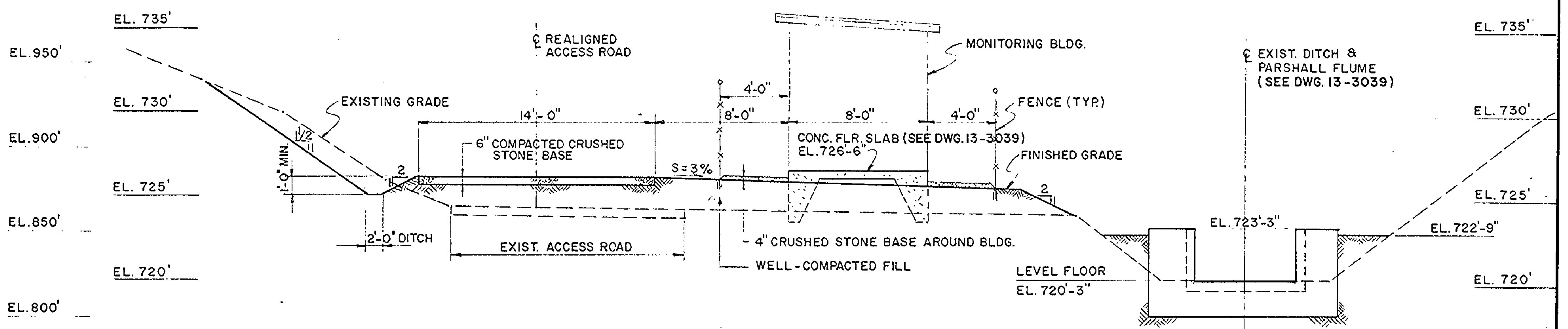
AMERICAN ELECTRIC POWER SERVICE CORP

DR. No. 13-3030



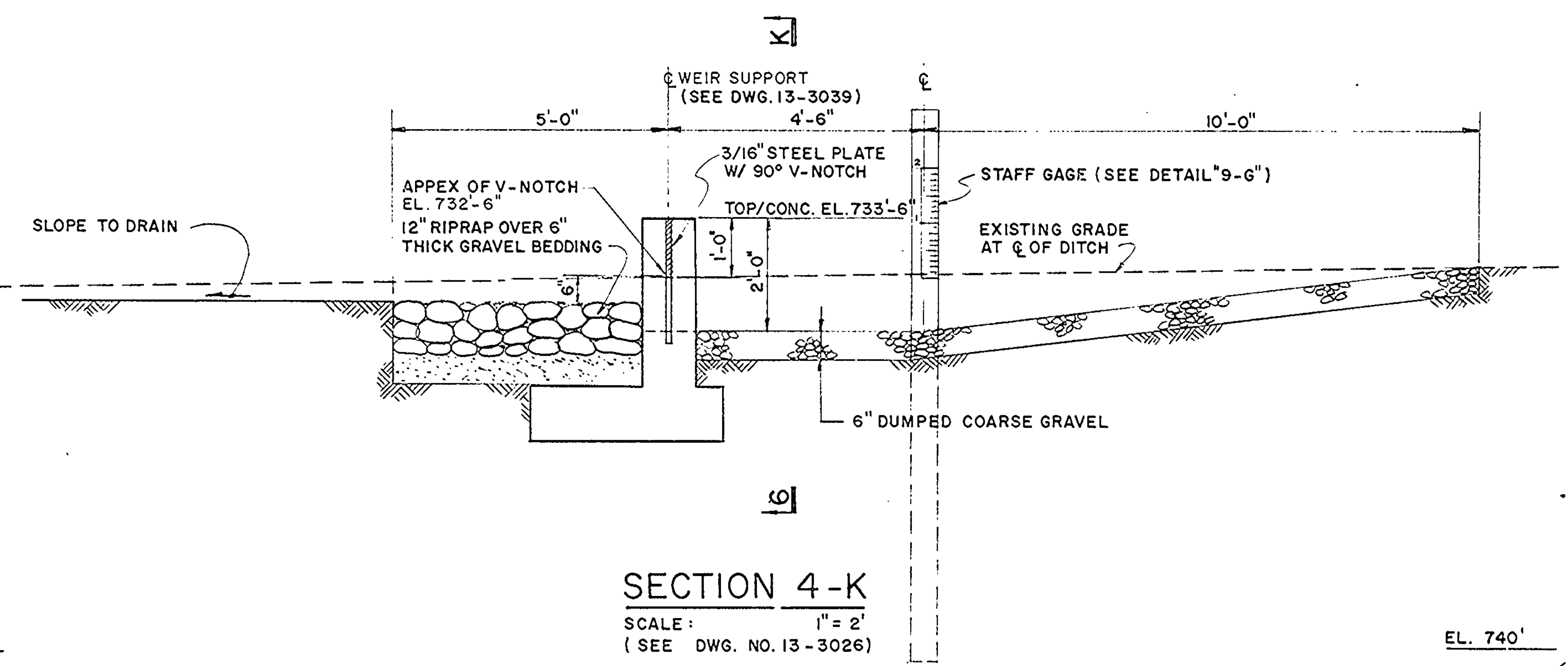
SECTION 2-D

SCALE: 1" = 50'
(SEE DWG. 13-3026 & 13-3029)
NOTE: FOR ZONE DESCRIPTION & SOURCE, SEE DWG. 13-3027.



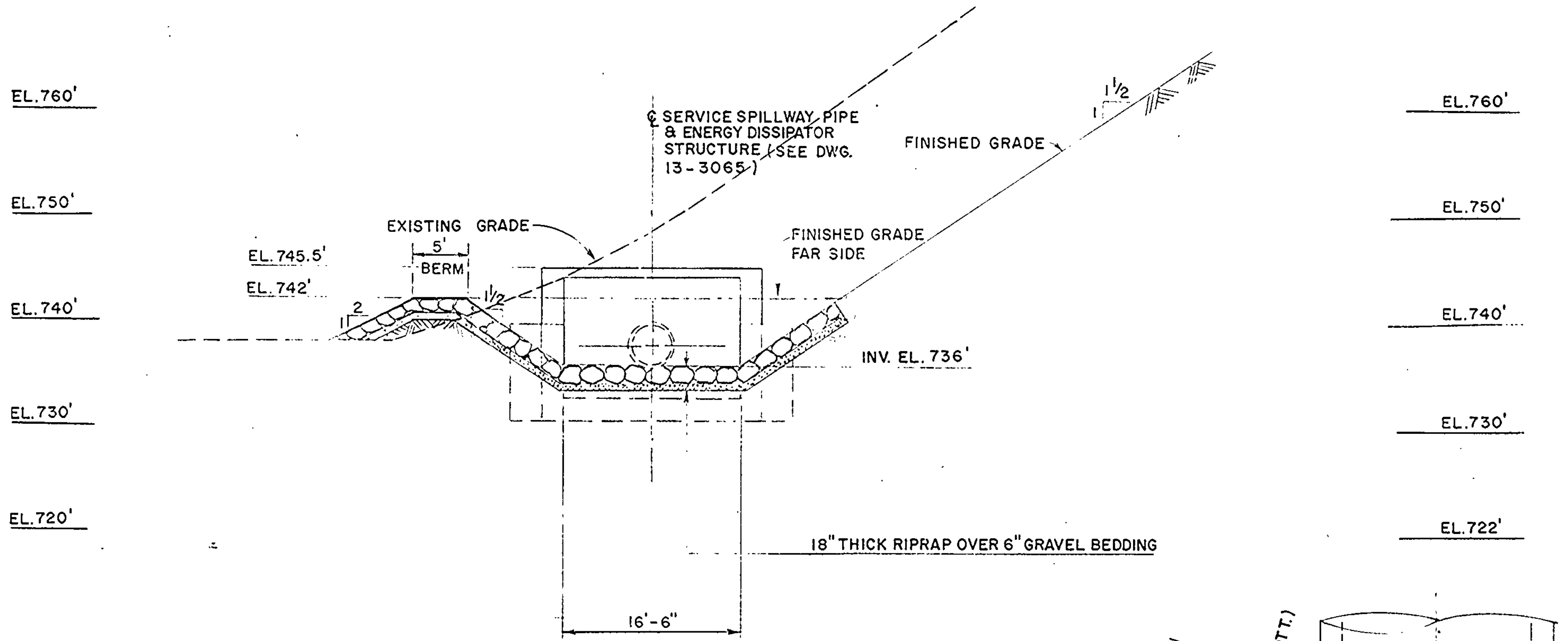
SECTION 2-K

SCALE: 1" = 5'
(SEE DWG. NO. 13-3024)



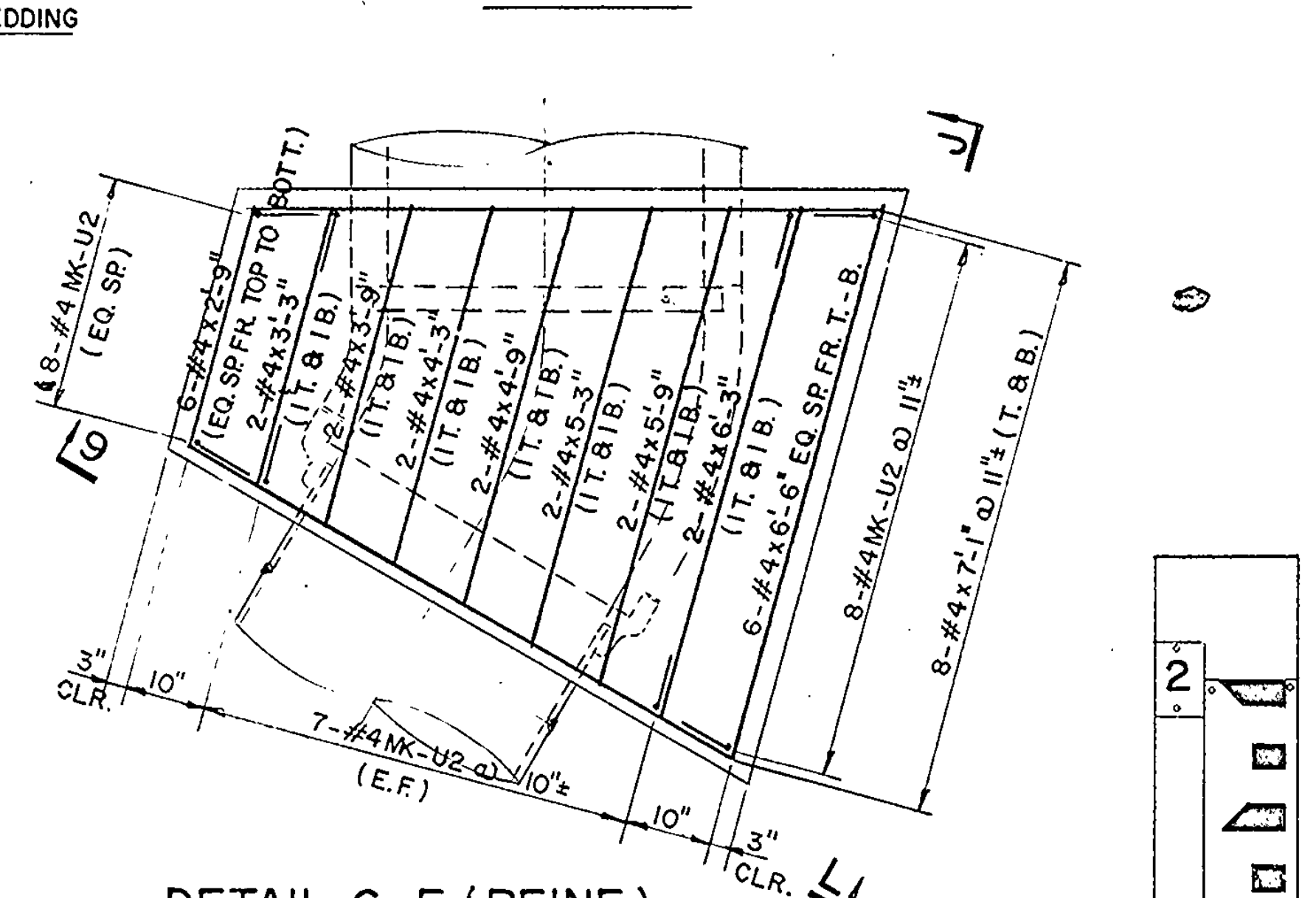
SECTION 4-K

SCALE: 1" = 2'
(SEE DWG. NO. 13-3026)



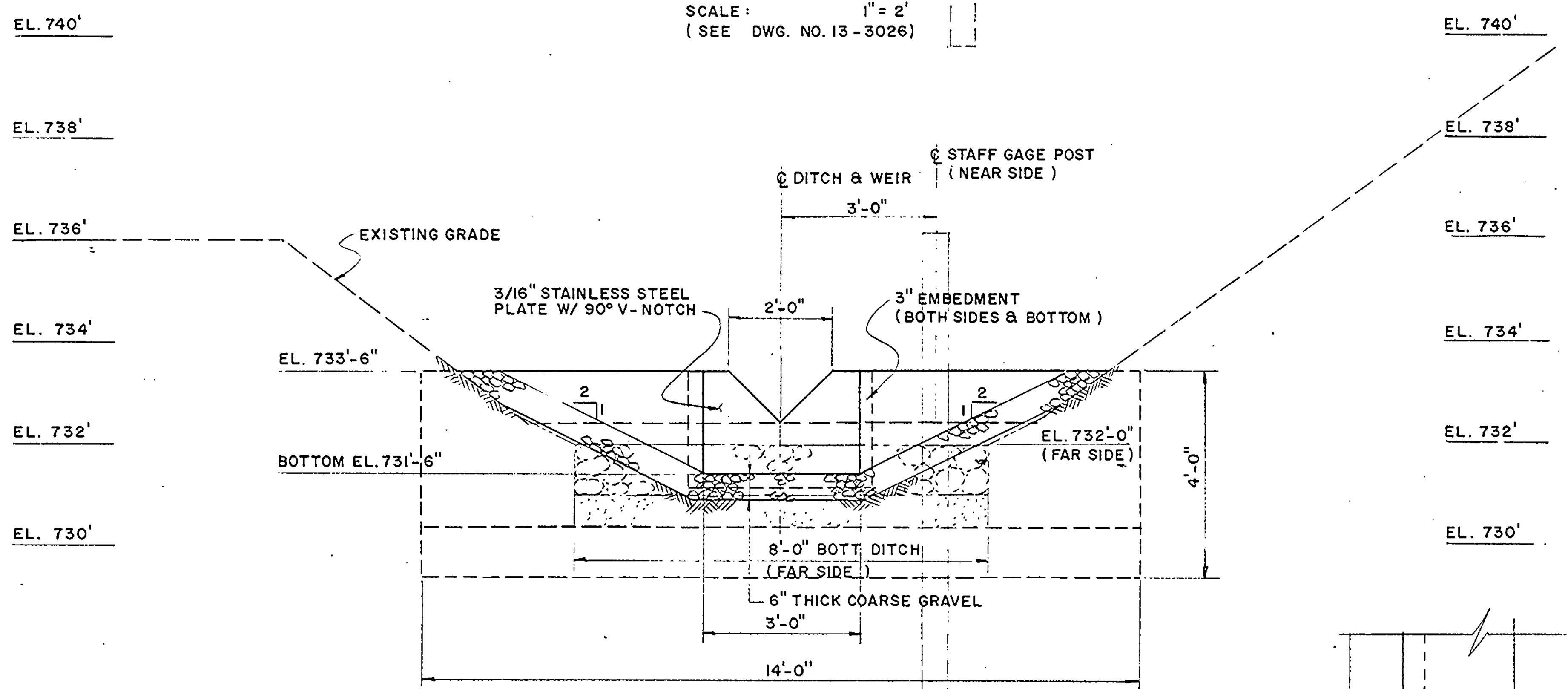
SECTION 5-C

SCALE: 1" = 10'
(SEE DWG. NO. 13-3024)



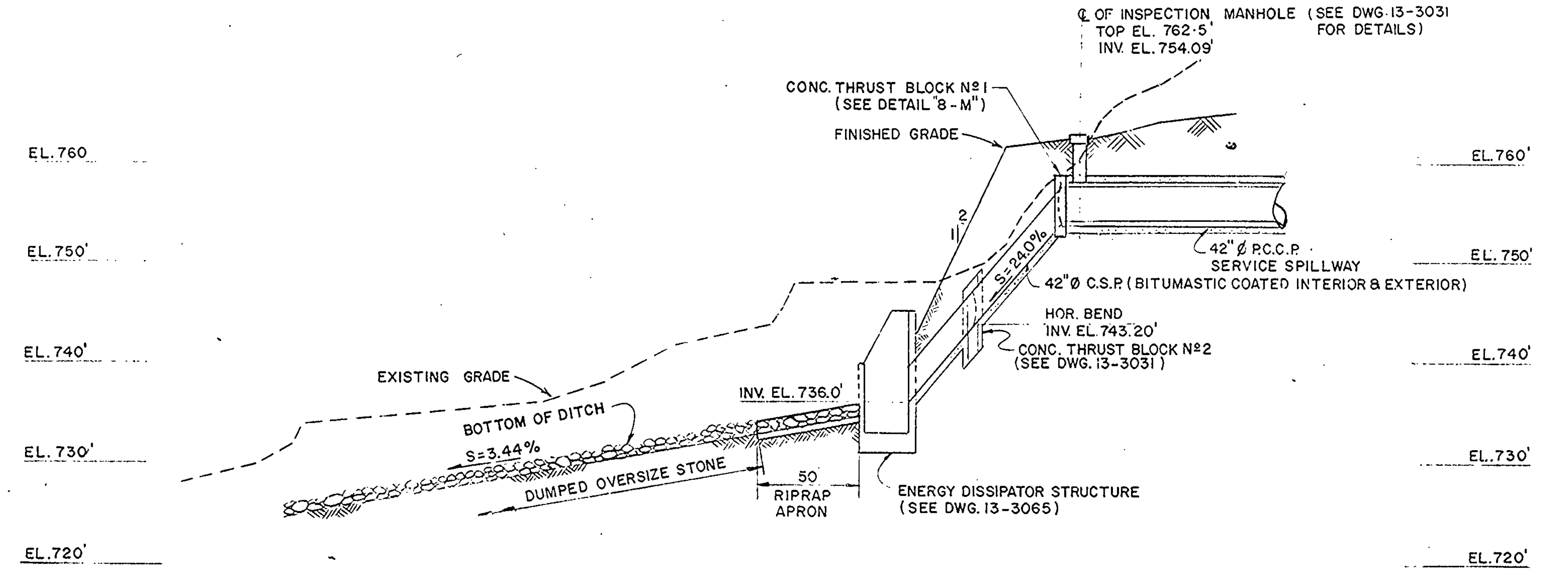
DETAIL 6-E (REINF.)

SCALE: 1/2" = 1'-0"



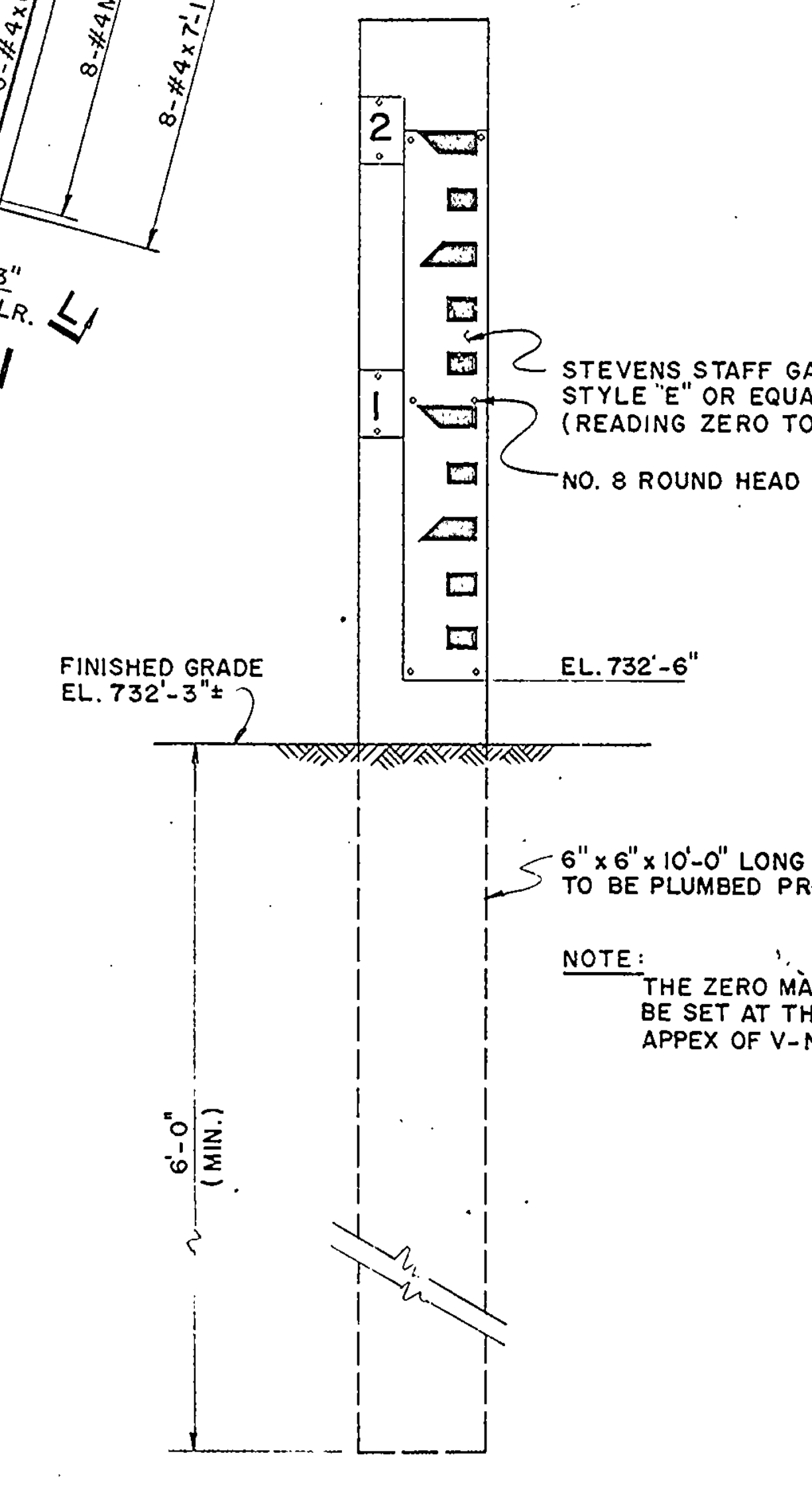
SECTION 6-K

SCALE: 1" = 2'
(LOOKING DOWNSTREAM)



℄ PROFILE-DISCHARGE LINE

SCALES: 1" = 50' HOR.
1" = 10' VERT.
(SEE DWG. 13-3024)



NOTE:
THE ZERO MARK OF THE STAFF GAGE SHOULD BE SET AT THE SAME ELEVATION AS THE APPEX OF V-NOTCH OF THE WEIR.

DETAIL 9-G

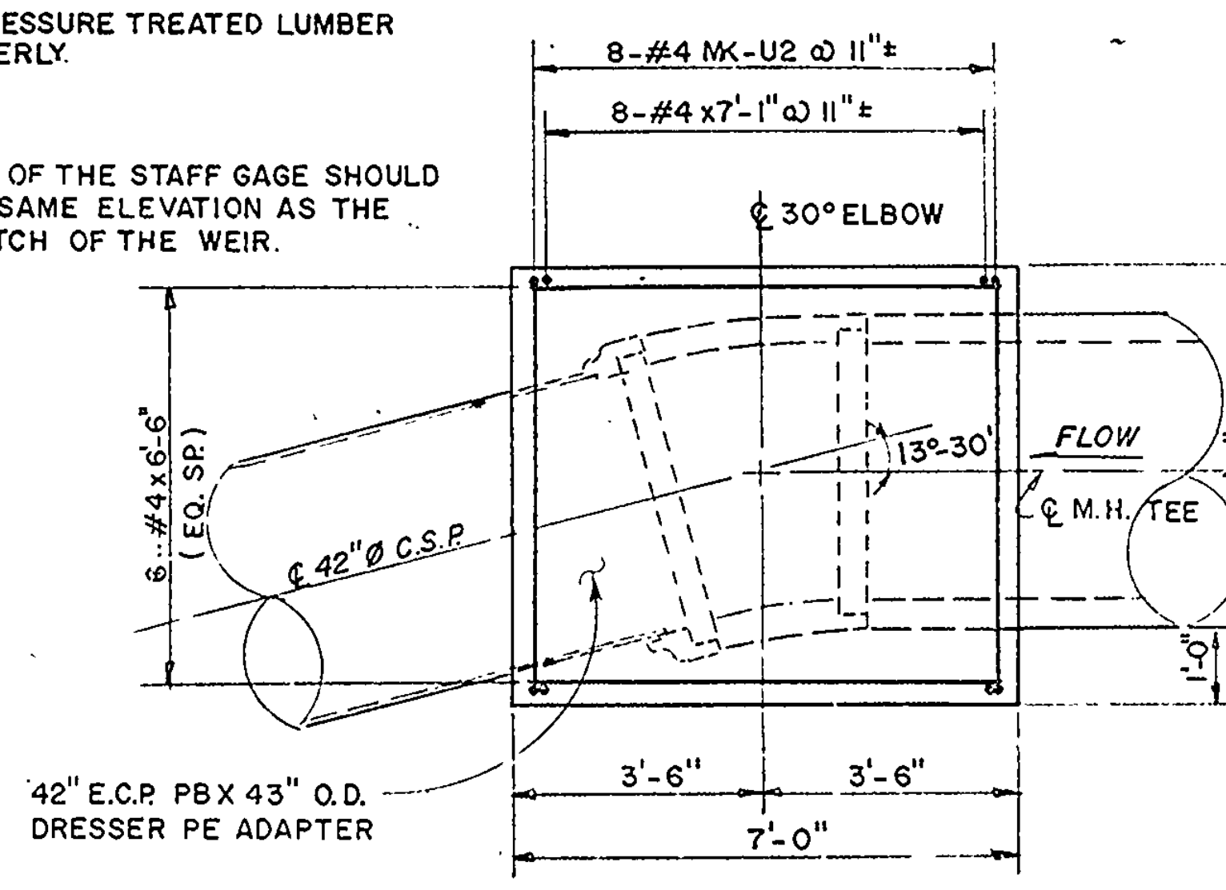
N.T.S.

THRUST BLOCK REINF. SCHED.: A-615 STEEL GR. 60

B | C | D TYPE (17)

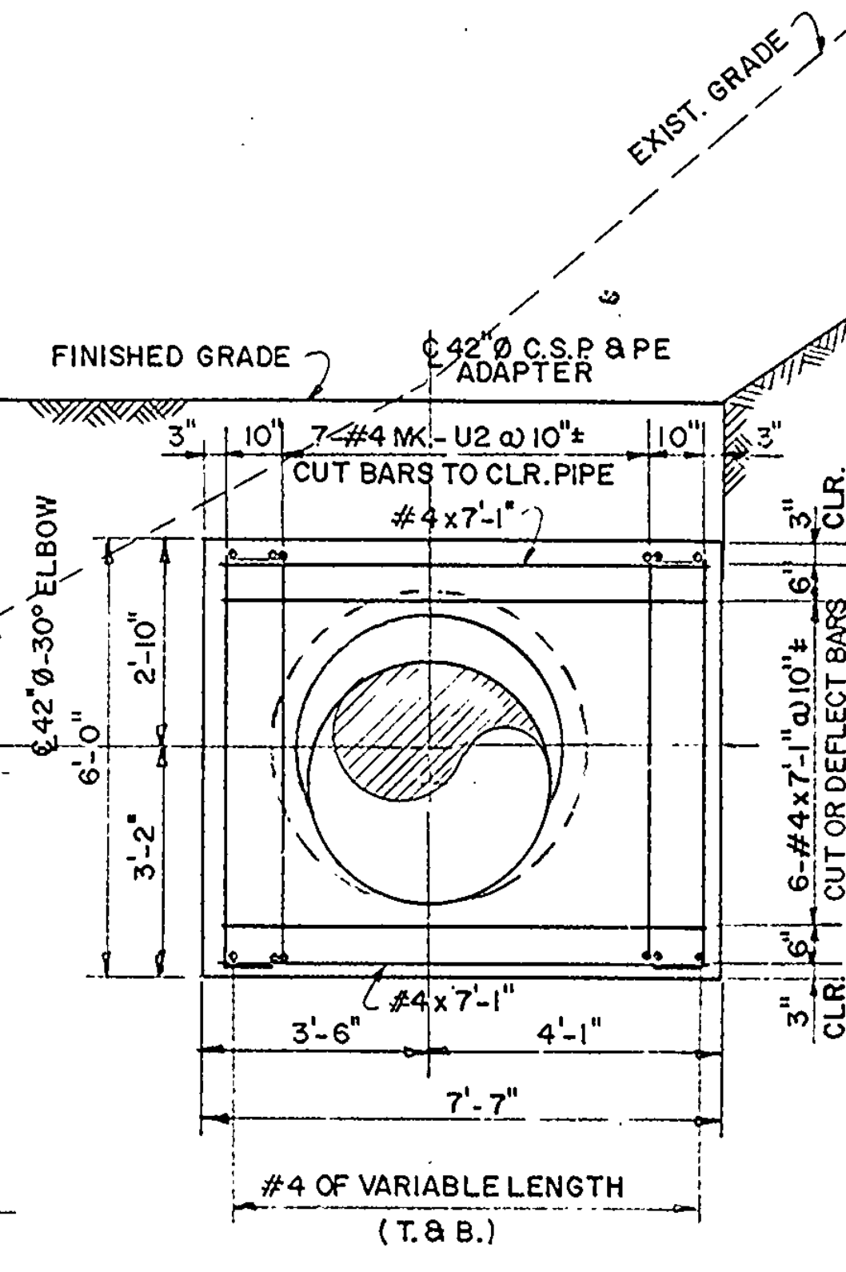
MK.	NO.	SIZE	LENGTH	TYPE	A	B	C	D
U2	30	#4	6'-10"	17		0'-8"	5'-6"	0'-8"
	28		7'-1"	STR.				
	6		6'-6"					
	2		6'-3"					
	2		5'-9"					
	2		5'-3"					
	2		4'-9"					
	2		4'-3"					
	2		3'-9"					
	2		3'-3"					
	6	#4	2'-9"	STR.				

WEIGHT = 350.92# (BY: FIELD)



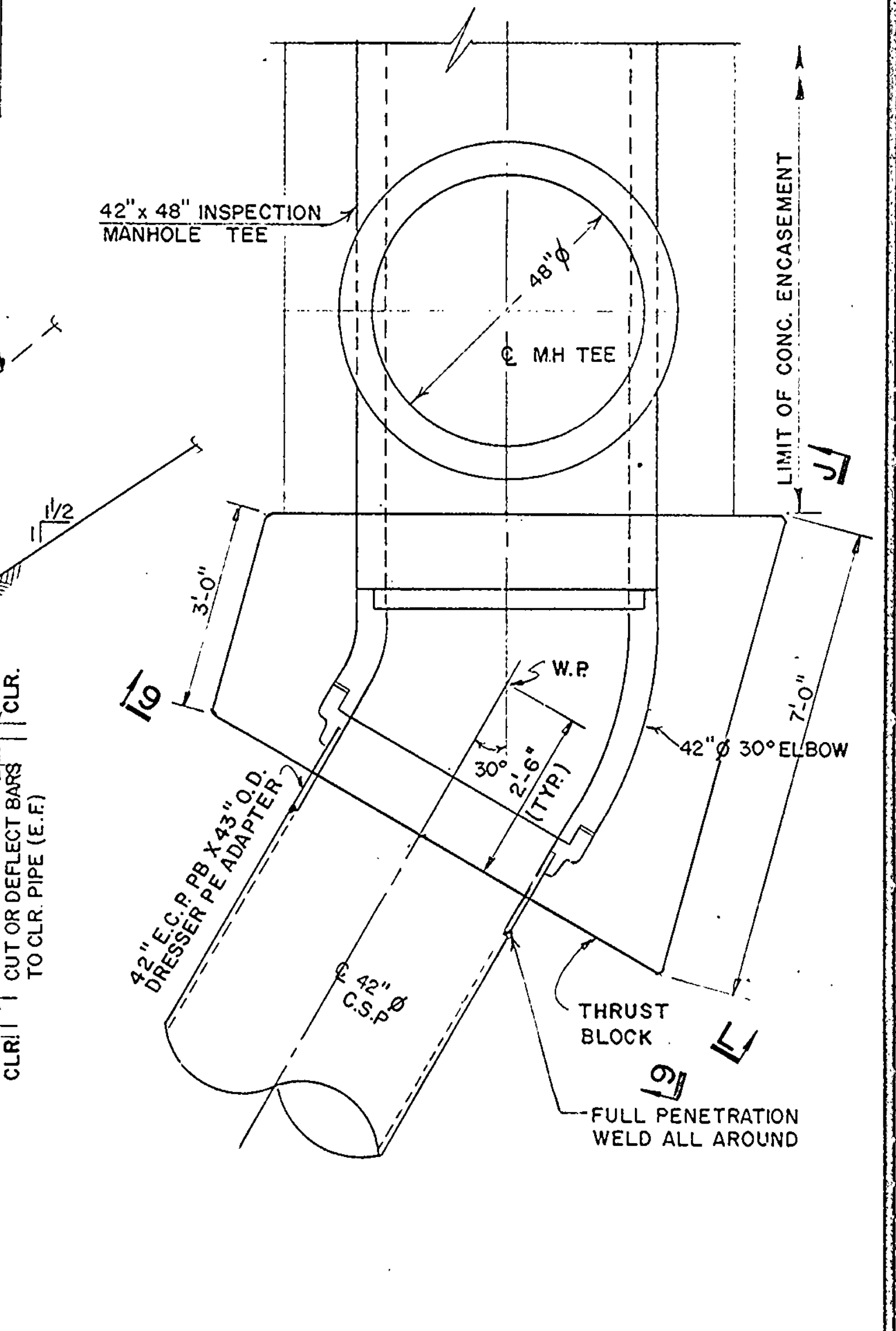
SECTION 9-J

SCALE: 3/8" = 1'-0"



SECTION 9-L

SCALE: 3/8" = 1'-0"



DETAIL 8-M

SCALE: 1/2" = 1'-0"

GENERAL NOTES

1.- FOR GENERAL NOTES, SEE DWG. 13-3032.

REFERENCE DRAWINGS

13-3023- DAM & EMERGENCY SPILLWAY EXCAVATION PLAN
13-3024- EMERGENCY & SERVICE SPILLWAYS CONTINUATION-LAYOUT & GRADING PLAN
13-3025- DAM UNDERDRAIN SYSTEM PLAN.
13-3026- DAM & EMERGENCY SPILLWAY LAYOUT & GRADING PLAN.
13-3027- DAM & EMERGENCY SPILLWAY SECTIONS SHEET 1.
13-3029- EXCAVATION, FILL, RELIEF WELLS, GROUT & SPILLWAYS COMPOSITE PLAN.
13-3032- SERVICE SPILLWAY & COFFERDAM LAYOUT & GRADING PLAN.
13-3039- MONITORING BLDG. FDN., PARSHALL FLUME & SEEPAGE MEASUREMENT WEIR PLANS, SECTIONS & DETAILS.

DATE	NO.	DESCRIPTION	APPD.
1/8/88	4	REVISED SECT "2-D" AS BUILT JAH/EF	28P
5/13/87	3	REMOVED "TEXT" BY OTHERS L-1 m.a.	28P
5/2/86	2	REV. SECT. "2-D", ADDED ZONES IIIB, IIIC & FILTER FABRIC. R.D.	ASP/95
8/22/85	1	AS PER RDR-CA-123056, ADDED DET. "8-M", "6-E", SECT. "9-J", "9-L" & R.S. SCHED. REV. SECT. "5-C" & ℄ PROFILE DISCH. LINE. S.P/R.D.	28P

REVISIONS

CARDINAL
OPERATING COMPANY
CARDINAL FLY ASH DAM 2
BRILLIANT, OHIO

DAM & EMERGENCY SPILLWAY
SECTIONS & DETAILS
AS BUILT

DR. NO. 13-3030-4

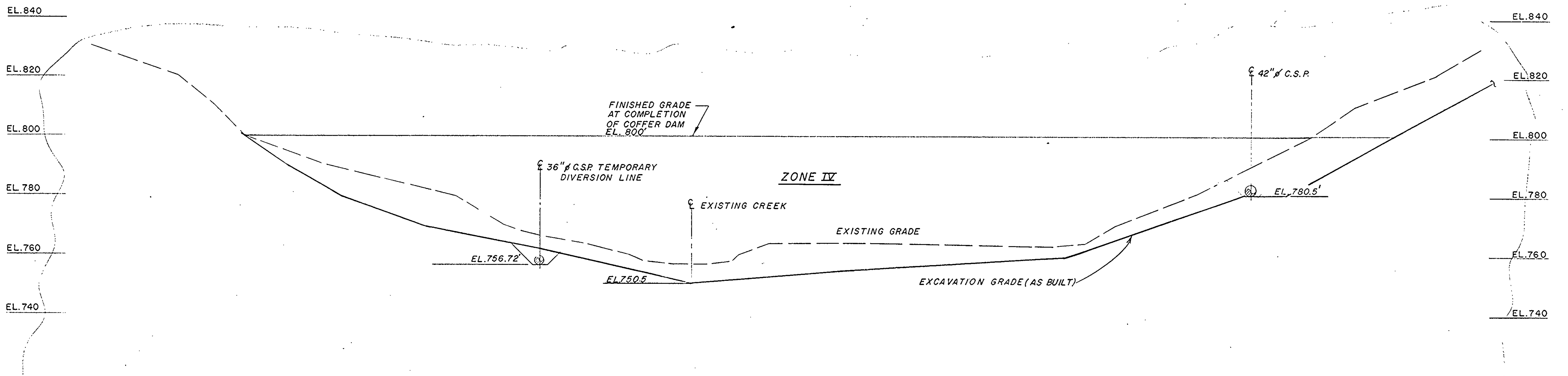
ARCH.	—	ELEC	—	MECH.	—	STR.	TS

SCALE: AS NOTED
DR. R.G. DURO
CH. TS
SQ. LDR TS
DATE: 1-4-85
APPROVED BY
ENGINEERING DIV
DESIGN DIV JA D. Bella

AMERICAN ELECTRIC POWER SERVICE CORP.

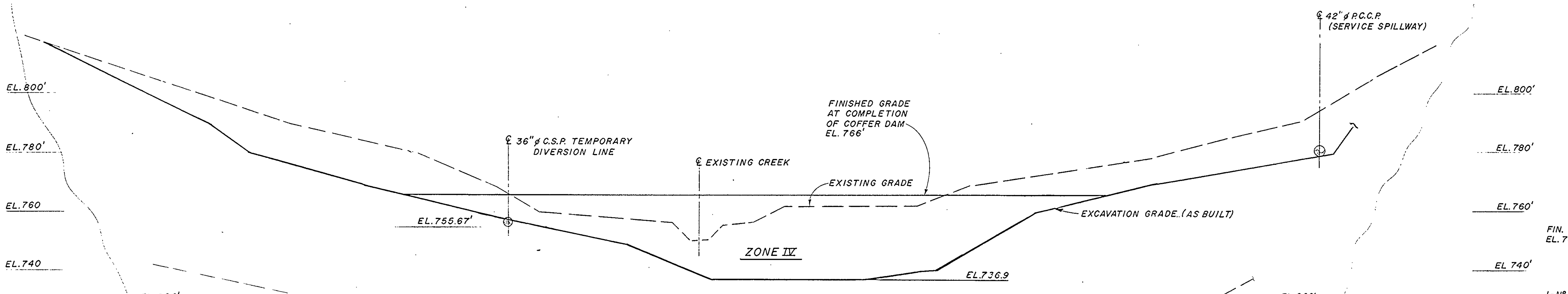
AEP3046XY

DR. No. 13-3031



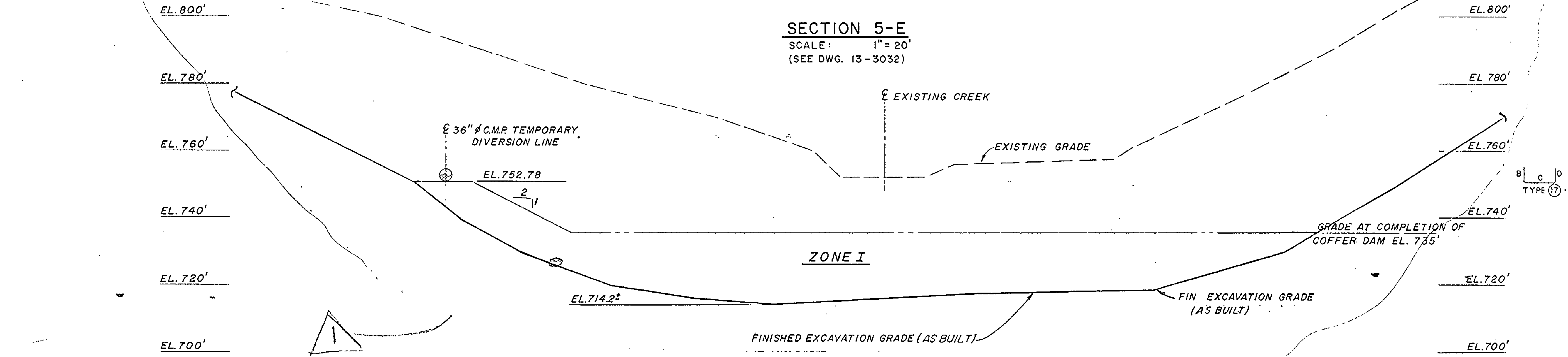
SECTION 2-E (℄ PROFILE COFFER DAM)

SCALE: 1" = 20'
(SEE DWG. 13-3032)



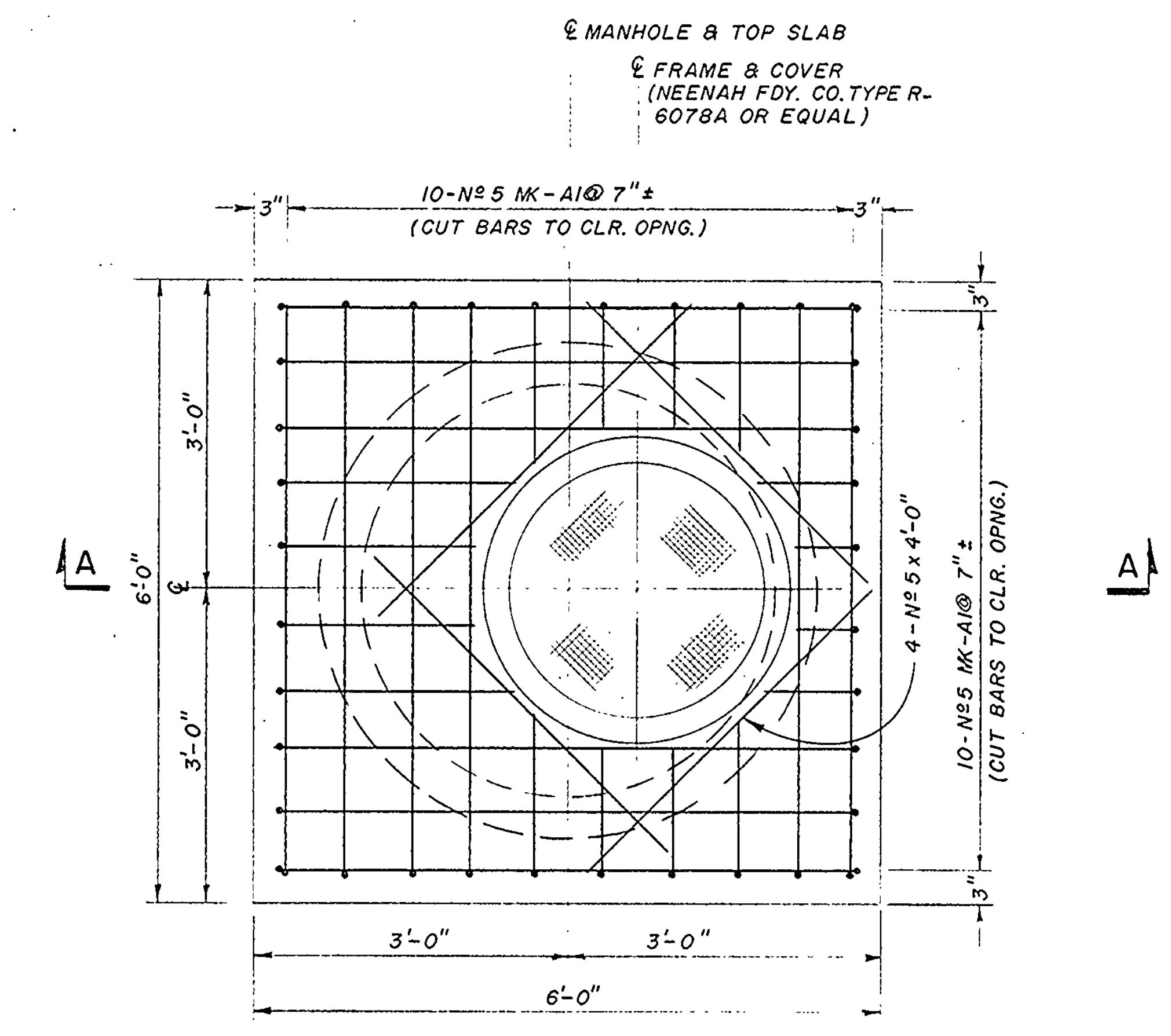
SECTION 5-E

SCALE: 1" = 20'
(SEE DWG. 13-3032)



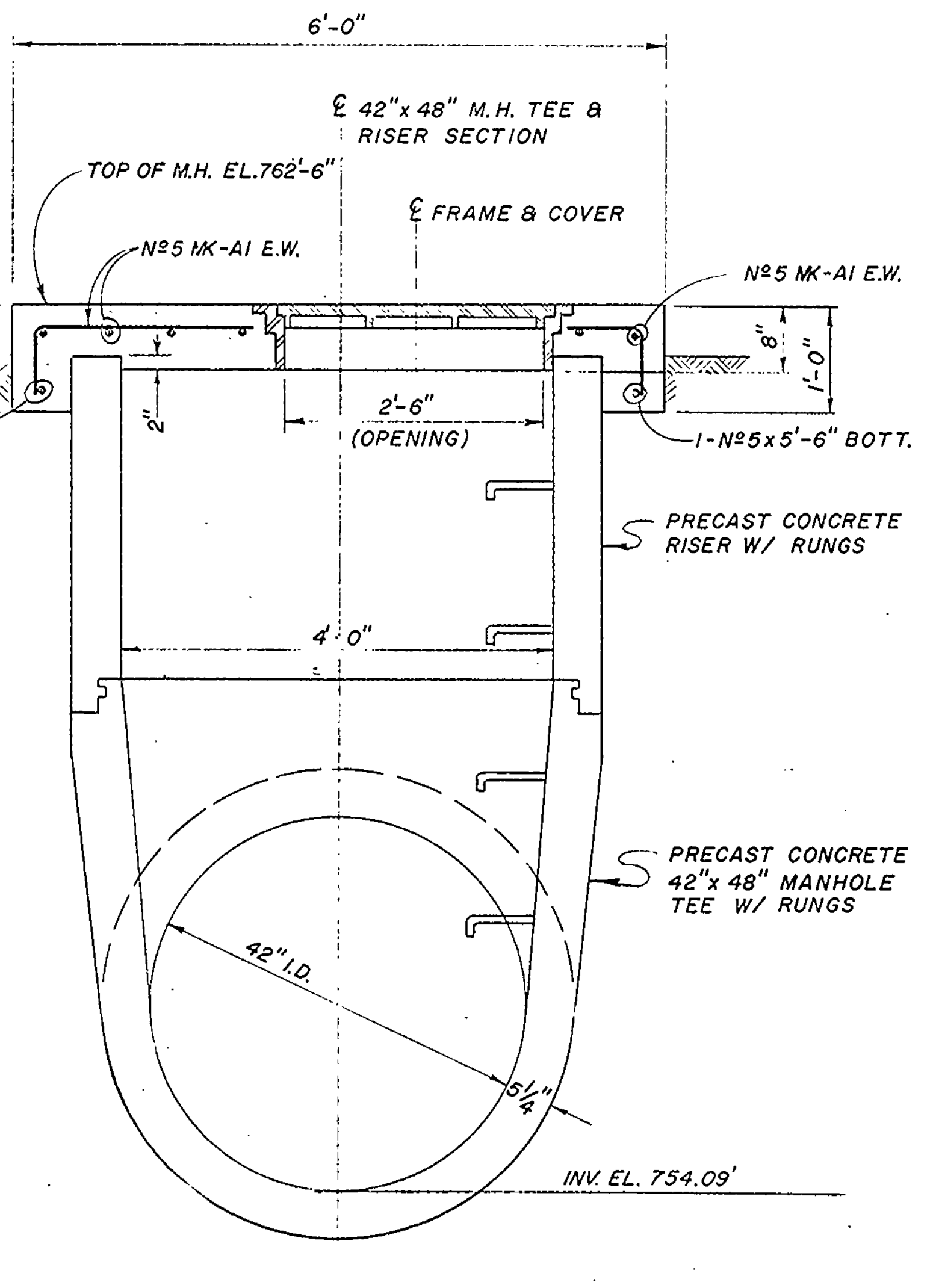
SECTION 7-E

SCALE: 1" = 20'
(SEE DWG. - 13-3032)



TOP SLAB REINF. PLAN

SCALE: 3/4" = 1'-0"



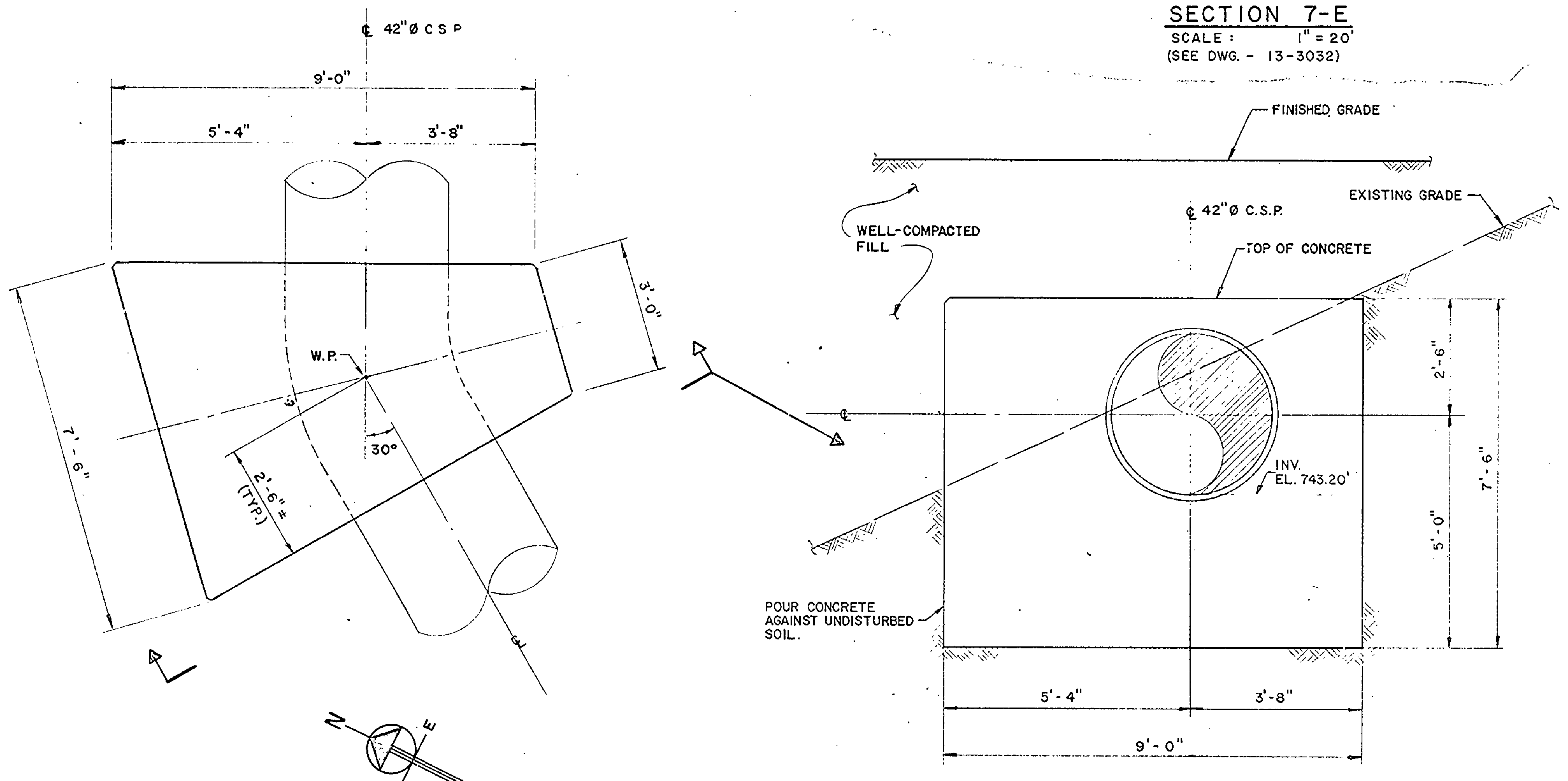
SECTION A-A

SCALE: 3/4" = 1'-0"

B C D TYPE ⑰

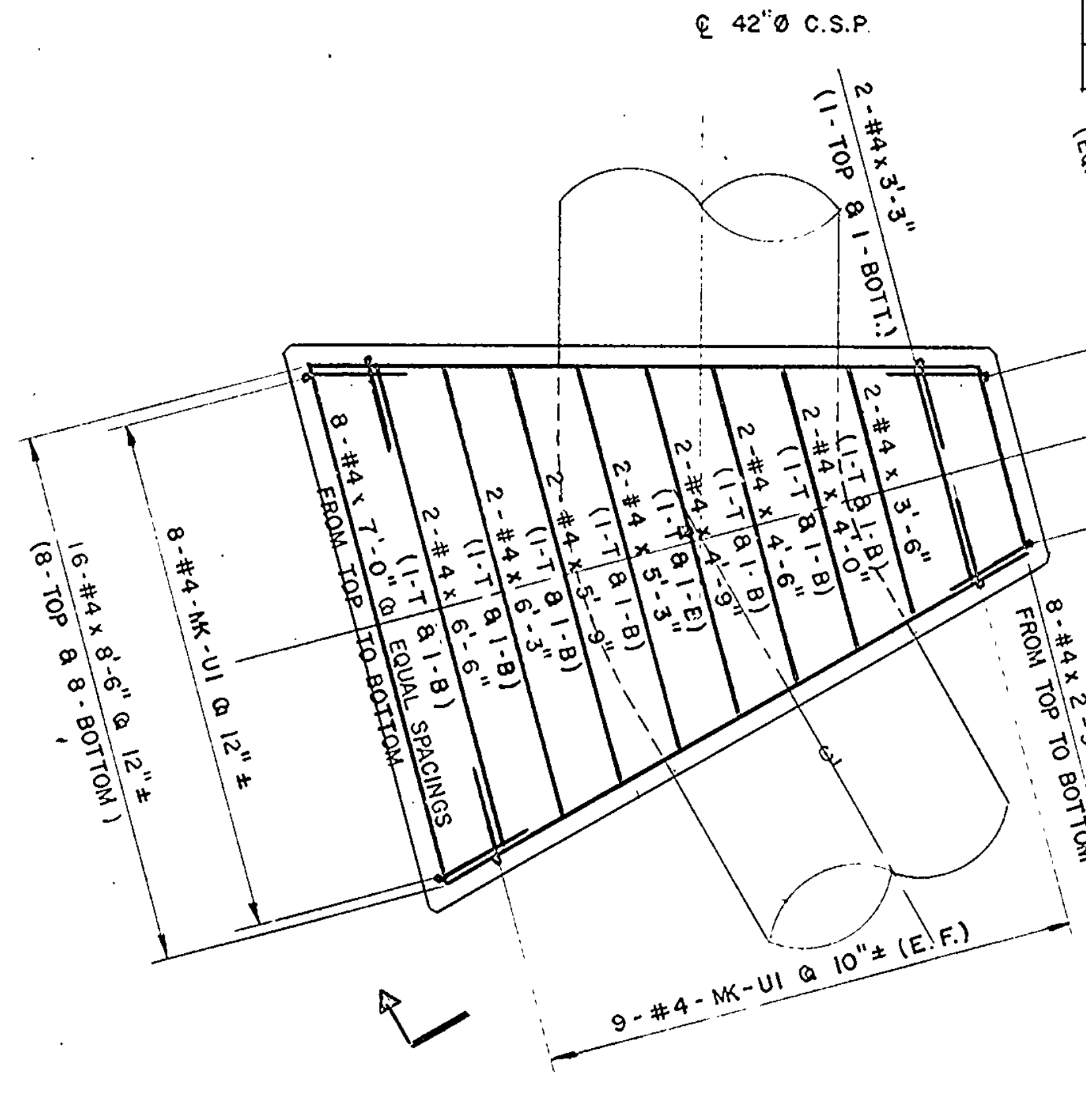
REINF. SCHEDULE: A-615 STEEL - GRADE 60								
MARK	No.	SIZE	LENGTH	TYPE	A	B	C	D
- INSPECTION M.H. TOP SLAB REINF. :								
A1	20	5	6'-6"	17		6"	5'-6"	6"
	4	5	5'-6"	STR.				
	4	5	4'-0"	STR.				
		WEIGHT	175.24#	(BY:	FIELD)			
- THRUST BLOCK REINF. =								
U1	34	4	8'-4"	17		8"	7'-0"	8"
	28	↑	8'-6"	STR.				
	8		7'-0"	↑				
	2		6'-6"					
	2		6'-3"					
	2		5'-9"					
	2		5'-3"					
	2		4'-9"					
	2		4'-6"					
	2		4'-0"					
	2		3'-6"					
	2	↓	3'-3"	↓				
	8	4	2'-9"	STR.				

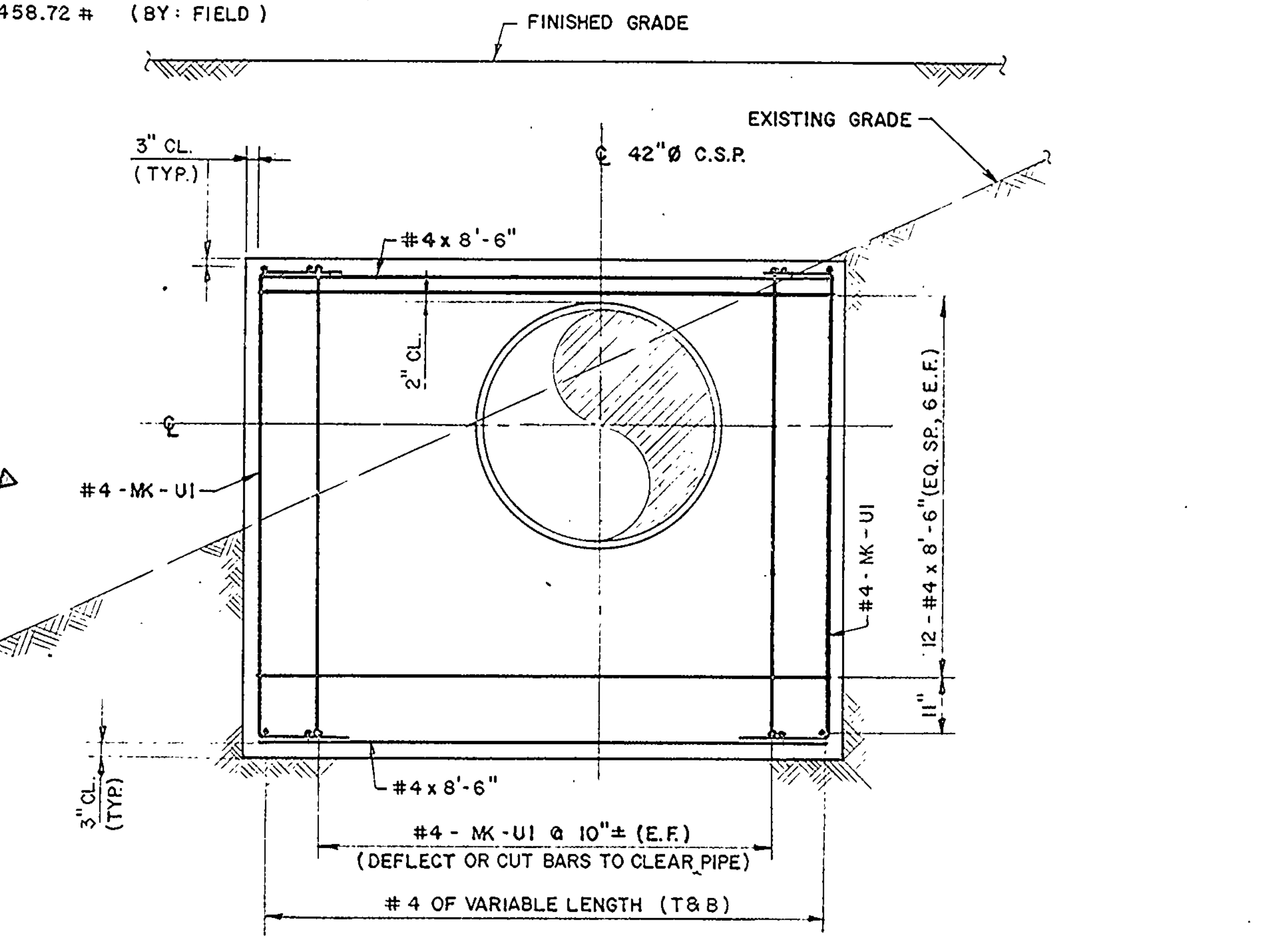
WEIGHT: 458.72 # (BY: FIELD)



CONC. THRUST BLOCK Nº2
MASONRY DETAIL 8-C

SCALE 1/2" = 1'-0"
(FOR LOCATION SEE DRAWING NO. 13 - 3032)





REINF. DETAIL 8-J

SCALE 1/2" = 1'-0"

GENERAL NOTES

FOR GENERAL NOTES, SEE DRAWING NO. 13 - 3032.

ALL CONCRETE MATERIALS AND WORKMANSHIP SHALL CONFORM TO A.E.P.S.C. CORP. SPECIFICATIONS #1000A

DIMENSIONS GIVEN FOR REINFORCING STEEL ARE TO CENTER LINE OF BARS.

CONSTRUCTION JOINTS ARE NOT TO BE ADDED, OMITTED OR RELOCATED, EXCEPT WITH THE WRITTEN APPROVAL OF THE COLUMBUS OFFICE, AND FURTHER PROVIDED THAT THE CONTRACTOR'S MIXING AND PLACING EQUIPMENT IS PROPERLY SIZED SO THAT NO COLD JOINTS WILL RESULT IN THE CONCRETE.

EXPANSION JOINTS MUST BE LOCATED AS SHOWN.

ALL EXPOSED EDGES SHALL HAVE A 1 INCH BEVEL.

FLOOR FINISH SHALL CONFORM TO THE A.E.P.'s CORP'S SPEC. AND TYPE OF SAME WILL BE DECIDED UPON BY CONCRETE LABORATORY.

ALL EXPOSED VERTICAL EXTERIOR CONCRETE SURFACES TO HAVE RUBBED FINISH.

CONCRETE, f'c. = 3,500 P.S.I.

MATERIALS

CONCRETE :	14.0	CU. YD.
FRAME & COVER : NEENAH FDY. CO. TYPE R-6078A OR EQUAL BY - ORDER NO	1	REQ'D.
42" P.J. x 48" M.H. TEE	1	REQ'D.
42"Ø RISER BY: PRICE BROTHERS CO. PO. Nº 01764-221-5X	3	LN. FT.

REFERENCE DRAWINGS

13-3030 - DAM & EMERGENCY SPILLWAY - SECTIONS SHT. 3.

13-3032 - SERVICE SPILLWAY & COFFERDAM LAYOUT & GRADING PLAN.

DATE	NO.	DESCRIPTION	APPO.
12/18/87	1	REVISED SECTIONS 2-E, 5-E & 7-E AS BUILT JM/E.F.	

REVISIONS

CARDINAL
OPERATING COMPANY
CARDINAL FLY ASH DAM 2
BRILLIANT, OHIO

COFFERDAM, CREEK DIVERSION,
INSPECTION M.H. &
THRUST BLOCK Nº2
SECTIONS & DETAILS
AS BUILT

DR. NO. 13-3031-1

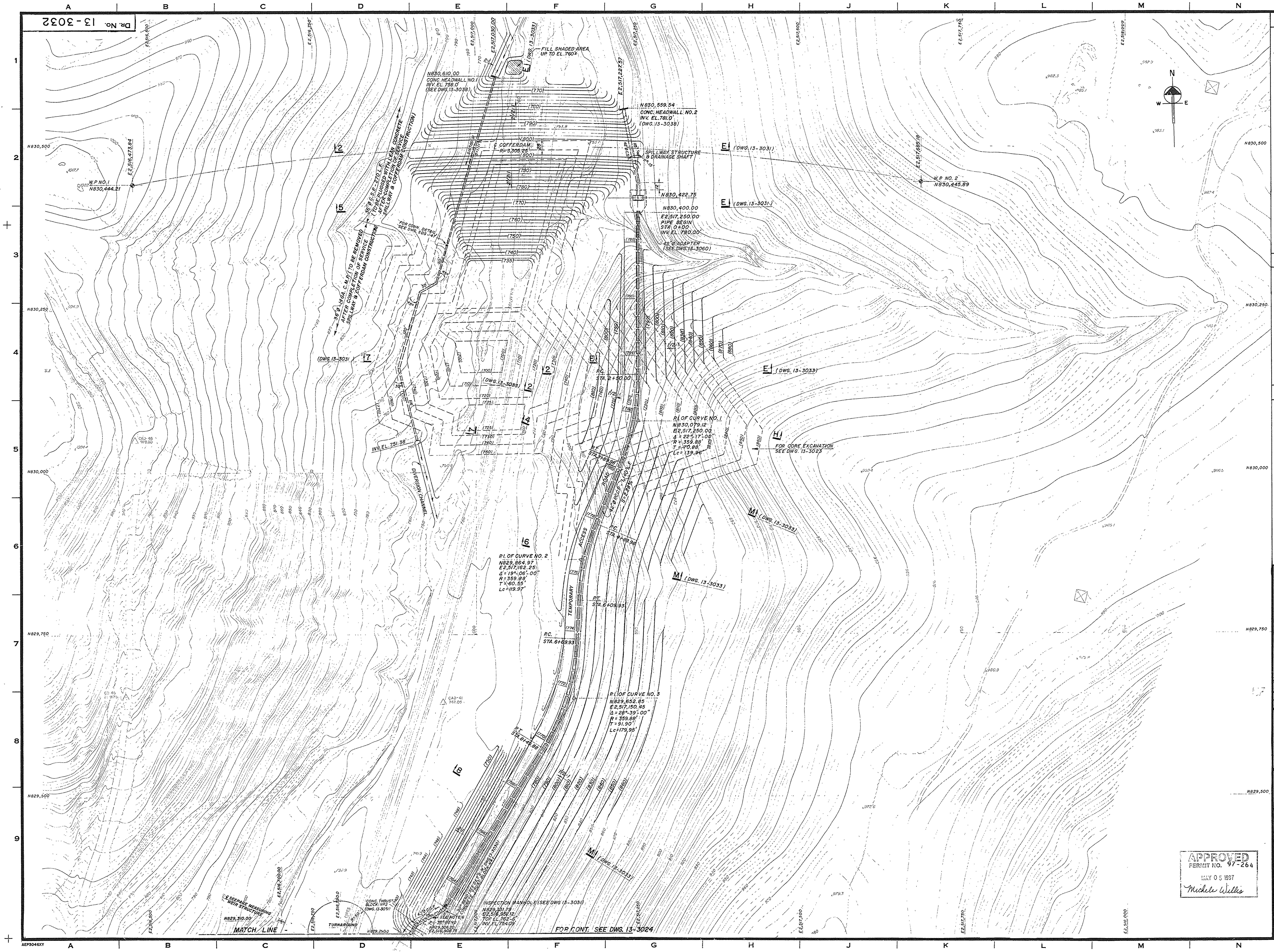
ARCH.		ELEC		MECH.		STR.	G.S.

SCALE: AS NOTED
DR. KM/ACY
CH. RD/GN
SQ. LDR G.S.
DATE: 8.22.85

APPROVED BY
ENGINEERING DIV R.H. Bennett RHA
DESIGN DIV J.A. DiBellaluz

AMERICAN ELECTRIC POWER SERVICE CORP.

RDR-CA-123056
RDR-CA-123057



APPROVED
PERMIT NO. 97-264
MAY 0 5 1997
Michele Willis

GENERAL NOTES

1. - FOR FILL MATERIAL, PROCEDURE AND COMPACTION, SEE A.E.P. CIVIL ENGINEERING SPECIFICATIONS.
2. - EXCAVATION FOR PIPE TRENCH IS SHOWN WITH A 15' ACCESS ROAD TO FACILITATE PIPE INSTALLATION. CONTRACTOR MAY PROPOSE ALTERNATE EXCAVATION/ACCESS SCHEMES.
3. - SIDE SLOPES AND EXCAVATION LIMITS SHOWN DEFINE THE MINIMUM SCOPE OF WORK. WHENEVER LOCALLY WEAK MATERIALS ARE ENCOUNTERED, THE SLOPES SHALL BE FLATTENED AS REQUIRED TO ASSURE SAFE WORKING CONDITIONS. SUCH SLOPES SHALL MEET ALL SAFETY REQUIREMENTS OF O.S.H.A., ET AL.
4. - CONTRACTOR MAY EXCAVATE FOR CORE PRIOR TO, AT THE SAME TIME AS OR AFTER EXCAVATING FOR PIPE TRENCH. MATERIAL EXCAVATED FROM THE PIPE TRENCH MAY BE PLACED AS BACKFILL IN THE TRENCH ABOVE THE PIPE EXCEPT IN THE CORE AREA. EXCESS MATERIAL EXCAVATED FROM THE PIPE TRENCH MAY BE USED IN ZONE IV OF THE DAM IF IT MEETS GRADATION & MOISTURE REQUIREMENTS.
5. - ALL C.S.P. (CARBON STEEL PIPE) SHALL BE A-134 STRUCTURAL ROLLED PLATE STEEL.
6. - 75 LIN. FT OF 42"Ø C.S.P. FROM INSPECTION MANHOLE DOWN TO THE DISSIPATOR SHALL RECEIVED ONE FIELD COAT OF BITUMASTIC PAINT INTERIOR & EXTERIOR.

MATERIALS

42"Ø I.D. P.C.C.P. (STD. WALL) —— 1140 L.F.
CONCRETE (f'_c = 4,500 P.S.I.) —— 1840 C.Y.
BY: PRICE BROTHERS CO. P.O. Nº 01764-221-5X

42"Ø C S P —— 202 L.F.
36"Ø C.S.P. —— 270 L.F.
BY: BI-CON SERVICES INC.
P.O. NO. 01109-221-5X

36"Ø - 14 GA. C.M.P. —— 335 L.F.
36"Ø - 14 GA. (30°) 2-PIECE ELBOW - 1 REQ'D.
36"Ø - 14 GA. (45°) 2-PIECE ELBOW - 2 REQ'D.
BY: LANE METAL PRODUCTS CO., INC.
P.O. NO. 01108-221-5X

REFERENCE DRAWINGS

13-3023 - DAM & EMERGENCY SPILLWAY EXCAVATION PLAN.
13-3024 - EMERGENCY & SERVICE SPILLWAYS CONTINUATION LAYOUT AND GRADING PLAN.
13-3026 - DAM & EMERGENCY SPILLWAY LAYOUT & GRADING PLAN.
13-3030 - DAM & EMERGENCY SPILLWAY SECTIONS & DETAILS SHEET 3.
13-3033 - SERVICE SPILLWAY & COFFERDAM PROFILE, SECTIONS & DETAILS.
13-3060 - SERVICE SPILLWAY STRUCTURE MASONRY SHEET 1 OF 2.
13-3065 - EMERGENCY DISSIPATOR MASONRY & REINF.
13-3038 - CONCRETE HEADWALLS NO. 1, 2 & 3 PLANS, SECTIONS & DETAILS.
13-3080 - WATER DIVERSION SEQUENCE PHASE I EXCAVATION PLAN.
13-3082 - WATER DIVERSION SEQUENCE PHASE II EXCAVATION PLAN.

RECEIVED
APR 2 8 1997
ODNR/DIVISION OF WATER
ENGINEERING GROUP

DATE	NO.	DESCRIPTION	APPD.
9/24/85	1	RDR - CA - 123057 GENERAL REVISION. R.D	

REVISIONS

CARDINAL
OPERATING COMPANY
CARDINAL FLY ASH DAM 2
BRILLIANT, OHIO

SERVICE SPILLWAY & COFFERDAM
LAYOUT & GRADING PLAN
AS BUILT

DR. NO. 13-3032-1

ARCH.	-	ELEC	-	MECH.	-	STR.	TS

SCALE: 1"=50'
DR. R.G.DURO
CH. TS
SQ. LDR TS
DATE: 1-4-85

APPROVED BY
ENGINEERING DIV
DESIGN DIV

AMERICAN ELECTRIC POWER SERVICE CORP.

DR. No. 13-3033

DETAIL 2-B (TYPICAL)
N.T.S.
(FOR JOINT DETAIL, SEE DET."1-M")

SECTION 2-E
SCALE: 1" = 20'

SECTION 2-H
SCALE: 1" = 20'

SECTION 4-H (TYPICAL)
SCALE: 1" = 5'

DETAIL 1-M
N.T.S.
(TYP. JOINT DETAIL)

SECTION 4-M
SCALE: 1" = 20'

℄ PROFILE - SERVICE SPILLWAY

SCALES: 1" = 50' HOR.
1" = 20' VERT.

SECTION 6-M
SCALE: 1" = 20'

SECTION 7-E
SCALE: 1" = 20'

NOTE: FOR ZONE DESCRIPTION & SOURCE SEE DWG. 13-3027.

SECTION 8-M
SCALE: 1" = 20'

OMIT
(SEE SECTION "5-E" DWG. 13-3031)

SECTION 9-E (℄ PROFILE COFFER DAM)
SCALE: 1" = 20'

AEP3046XY

GENERAL NOTES

FOR GENERAL NOTES, SEE DRAWING 13-3032.

REFERENCE DRAWNIGS

13-3023	DAM & EMERGENCY SPILLWAY EXCATION PLAN.
13-3027	DAM & EMERGENCY SPILLWAY SECTIONS SHEET 1.
13-3032	SERVICE SPILLWAY & COFFERDAM LAYOUT & GRADING PLAN.
13-3038	CONCRETE HEADWALLS NO. 1, 2 & 3 PLANS, SECTIONS & DETAILS.
13-3065	ENERGY DISSIPATOR STRUCTURE PLAN & SECTIONS - MASONRY & REINF
13-3060	SERVICE SPILLWAY STRUCTURE MASONRY.
13-3031	COFFERDAM, CREEK DIVERSION, INSPECTION M.H. & THRUST BLOCK SECTIONS & DETAILS.

DATE	NO.	DESCRIPTION	APPD.
12/22/87	2	ADDED AS BUILT EXCAVATION SECTS. "2-E", "2-H", "4-M", "4-H" "6-M", "8-M" & "7-E" ma/E.F.	
8/22/85	1	ADDED DET. "1-M" & SECT. "4-H". CHG'D. DIM. ON DET. "2-B", SECTS. "2-E", "2-H", "4-M", "6-M" & "8-M" REV. SECT. "7-E" & ℄ PROFILE OMITTED SECT. "9-E" & ADDED REF. DWG. R.H./R.D.	

REVISIONS

CARDINAL
OPERATING COMPANY
CARDINAL FLY ASH DAM 2
BRILLIANT, OHIO

SERVICE SPILLWAY & COFFERDAM
PROFILE SECTION & DETAILS
AS BUILT

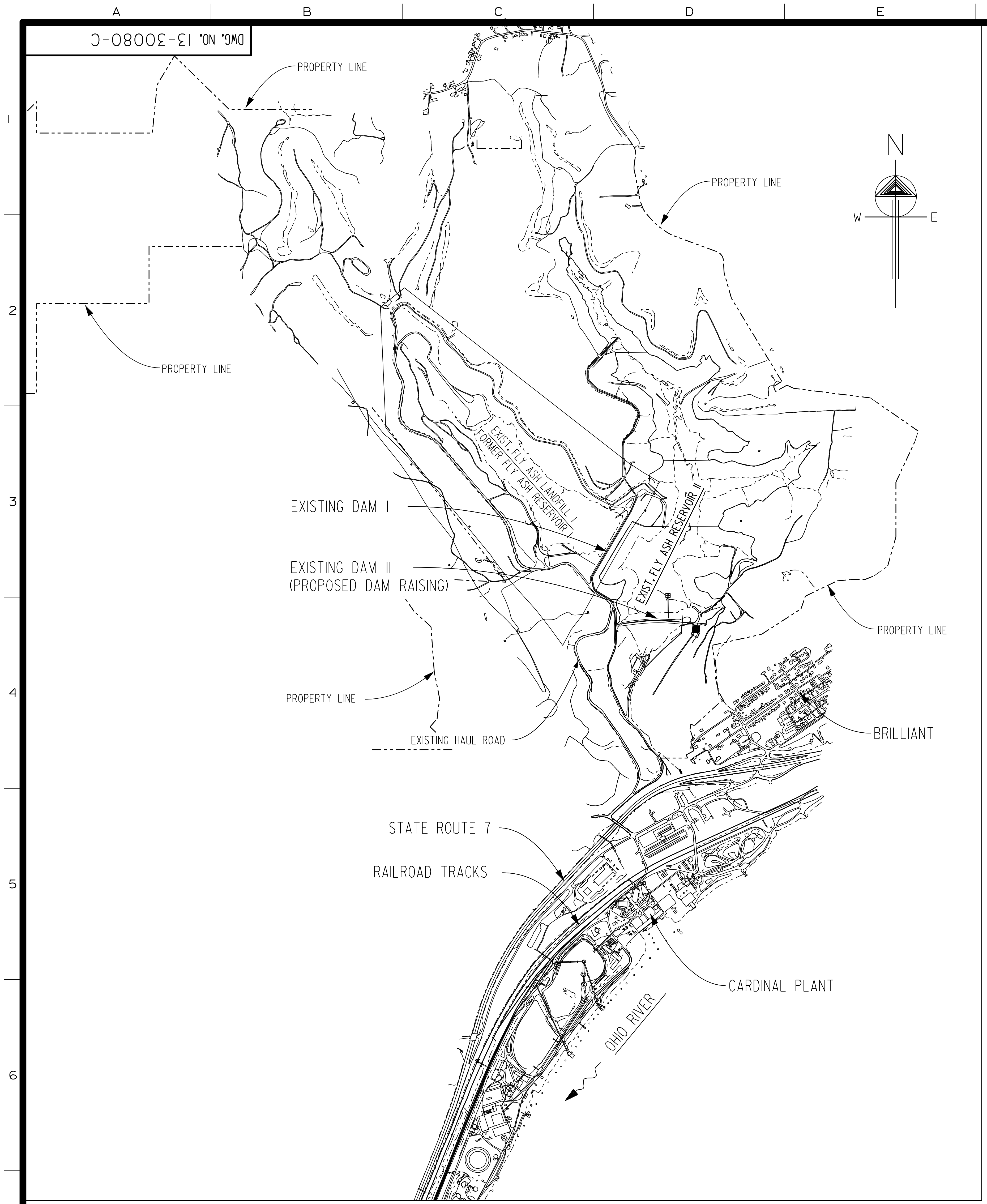
DR. NO. 13-3033-2

ARCH.	-	ELEC	-	MECH.	-	STR.	TS

SCALE: AS SHOWN
DR. S.L.P.
CH. R.D.
SQ. LDR TS
DATE: 1-4-85

APPROVED BY
ENGINEERING DIV
P.H. Q
DESIGN DIV
J.A. De Bella / Nag

AMERICAN ELECTRIC POWER SERVICE CORP.



VACINITY MAP
SCALE 1"=1000'

PERMIT APPLICATION
FOR THE
RAISING
OF
FLY ASH RETENTION DAM II
CLASS I DAM
AT THE
CARDINAL PLANT
BRILLIANT, OHIO

SECTION 5, WELLS TOWNSHIP
JEFFERSON COUNTY

PREPARED FOR:
CARDINAL OPERATING COMPANY
SUBMITTED TO:
OHIO DEPARTMENT OF
NATURAL RESOURCES

PREPARED BY:
S&ME, INC.
6190 ENTERPRISE COURT, DUBLIN, OHIO 43016

AND

AMERICAN ELECTRIC POWER SERVICE CORP.
1 RIVERSIDE PLAZA, COLUMBUS, OHIO 43215

JANUARY 2013

DRAWING INDEX

I3-30080-A	COVER SHEET
I3-30081-A	RESERVOIR GENERAL ARRANGEMENT
I3-30082-A	DAM CONTROL PLAN AND PROFILE
I3-30083-A	SITE PLAN
I3-30084-A	TRENCH EXCAVATION, SLURRY WALL, AND SHEET PILE WALL
I3-30085-A	MSE WALL ELEVATION
I3-30086-A	MSE WALL DETAILS
I3-30087-A	DAM RAISING TYPICAL SECTIONS
I3-30088-A	RIGHT ABUTMENT PLAN
I3-30089-A	EMERGENCY SPILLWAY PLAN
I3-30090-A	EMERGENCY SPILLWAY PROFILE AND SECTIONS
I3-30091-A	SERVICE SPILLWAY GENERAL PLAN, PROFILE, AND ELEVATION
I3-30092-A	S1.0 SERVICE SPILLWAY INLET STRUCTURE DETAILS
I3-30093-A	S1.1 SERVICE SPILLWAY INLET STRUCTURE DETAILS
I3-30094-A	S1.2 SERVICE SPILLWAY INLET STRUCTURE DETAILS
I3-30095-A	S2.0 EMERGENCY SPILLWAY RETAINING WALL PLAN
I3-30096-A	S2.1 EMERGENCY SPILLWAY RETAINING WALL SECTIONS
I3-30097-A	EROSION AND SEDIMENTATION PLAN
I3-30098-A	INSTRUMENTATION PLAN

REFERENCE DRAWINGS

I3-3004-1	GEOLOGIC PROFILE N-S AXIS
I3-3005-1	GEOLOGIC PROFILE E-W AXIS
I3-30040-5	GRADING AND DRAINAGE PLAN
I3-30042-3	SECTIONS AND DETAILS SHEET 1
I3-30043-5	SERVICE SPILLWAY
I3-30053-3	GROUT HOLES PLAN AND SECTION
I3-32000	DRAINAGE SHAFT MASONRY AND REINFORCEMENT - PLANS, SECTIONS, AND DETAILS
I3-30099-A	BORING PLAN

S&ME, INC.
6190 ENTERPRISE COURT
DUBLIN, OH 43016
PHONE: 614-793-2226
FAX: 614-793-24103
www.smeinc.com

MICHAEL GILBERT ROWLAND
E-65559

Michael G. Rowland
MICHAEL GILBERT ROWLAND

E-65559
NUMBER

JAN 16, 2012
DATE

PROJECT NUMBER: 011-11497-042
DRAWING DATE: 9/28/12
LAST UPDATED: 1/16/13
DRAWN BY: EDV
ENGINEER: MTR
APPROVED BY: MGR
SCALE:

S&ME

DATE	NO.	DESCRIPTION	APPD.
1/16/13	C	REVISED PER ODNR COMMENTS REC'D 12/4/2012	MGR
9/12/12	B	REVISED PER ODNR COMMENTS REC'D 7/18/12	MGR
5/23/12	A	ISSUED FOR PERMIT INITIAL SUMMITTAL	MGR

REVISIONS

CARDINAL OPERATING COMPANY
CARDINAL PLANT
BRILLIANT OHIO

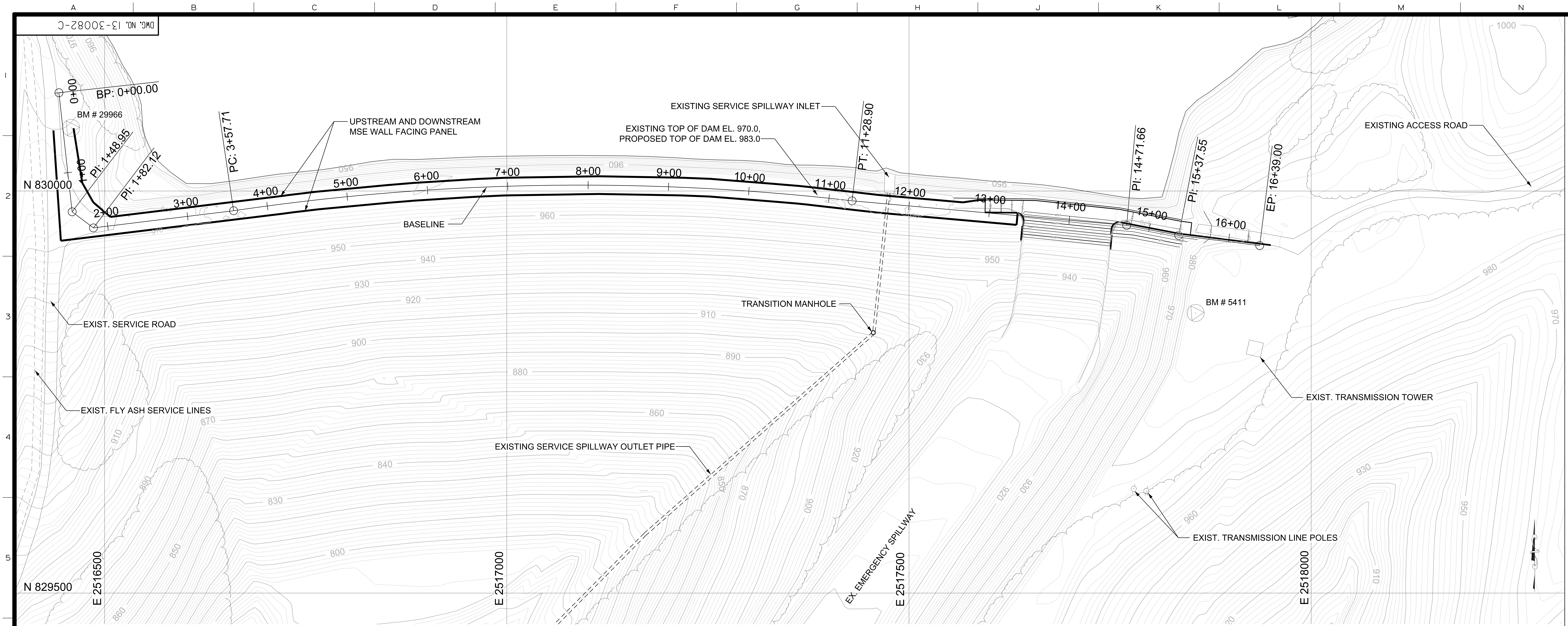
DAM RAISING
FLY ASH RETENTION DAM II
COVER SHEET

DWG. NO. I3-30080-C

ARCH — ELEC — MECH — STR

SCALE: AS SHOWN
DR:
CH:
SQD SUPV:
DATE:
APPROVED BY
CIVIL ENGINEERING DIVISION

AEP AMERICAN ELECTRIC POWER
1 RIVERSIDE PLAZA
COLUMBUS, OH 43215

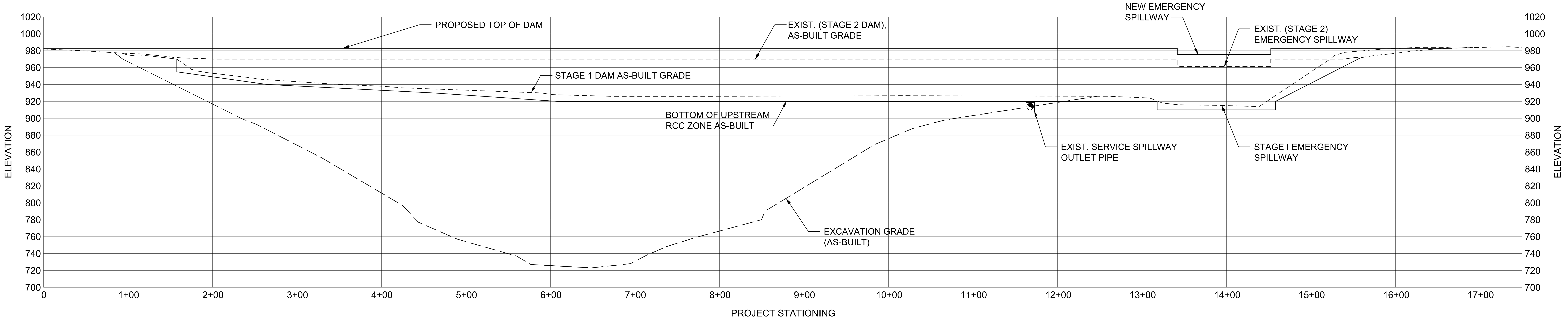


LEGEND

900	EXISTING CONTOUR 10FT INTERVAL
	EXISTING CONTOUR 2FT INTERVAL
	EXISTING ACCESS ROADS
	EXISTING FLY ASH SERVICE LINES
△	BENCHMARK
	EXISTING TREELINE

REFERENCE DRAWING

13-30043-5



GENERAL NOTES

1. TOPOGRAPHIC MAPPING DEVELOPED FROM AERIAL SURVEY PERFORMED ON 3/5/2009 AND 1986 AS-BUILT DATA.

NOTES: PROJECT STATIONING CORRESPONDS TO SHEET PILE CENTERLINE AND IS NOT THE C/L OF THE DAM.

STATION CONTROL POINT COORDINATES			
POINT	STATION	NORTHING	EASTING
BP	0+00.00	830,122.22	2,516,443.18
PI	1+48.95	829,974.18	2,516,459.63
PI	1+82.12	829,954.17	2,516,486.08
PC	3+57.71	829,975.46	2,516,660.37
PT	11+28.90	829,987.85	2,517,429.23
PI	14+71.66	829,957.32	2,517,770.63
PI	15+37.55	829,944.82	2,517,835.32
EP	16+39.00	829,932.17	2,517,935.98

CURVE DATA					
PI STATION	NORTHING	EASTING	△	L	R
7+45.55	830,032.84	2,517,043.98	15.0871	771.191	2928.733

BENCHMARK INFORMATION				
BM #	NORTHING	EASTING	ELEVATION	LOCATION
5411	829,848.39	2,517,856.35	984.70	FAR-II LEFT ABUTMENT
7117	832,977.28	2,514,971.06	1010.31	FAR-I NORTH ACCESS ROAD
7114	833,949.40	2,518,415.12	1020.08	FAR-II EAST ACCESS ROAD

DATUM: NAD27/NAVD 29 OHIO SOUTH

0 50 100
SCALE IN FEET

WWW.SMEINC.COM

S&ME, INC.
6190 ENTERPRISE COURT
DUBLIN, OH 43016
PHONE: 614-793-2226
FAX: 614-793-2410

MICHAEL GILBERT ROWLAND E-65559

Michael A. R

MICHAEL GILBERT ROWLAND	E-65559	JAN 16, 2012
	NUMBER	DATE

PROJECT NUMBER:	011-11497-042	DRAWN BY:	EDV
DRAWING DATE:	9/28/12	ENGINEER:	MTR
LAST UPDATED:	1/16/13	APPROVED BY:	MGR
		SCALE:	AS NOTED

S&ME

DATE	NO.	DESCRIPTION	APPD.
1/16/13	C	REVISED PER ODNR COMMENTS REC'D 12/4/2012	MGR
9/12/12	B	REVISED PER ODNR COMMENTS REC'D 7/18/12	MGR
5/23/12	A	ISSUED FOR PERMIT INITIAL SUMMITTAL	MGR

REVISIONS

"THIS DRAWING IS THE PROPERTY OF THE AMERICAN ELECTRIC POWER SERVICE CORP. AND IS LOANED UPON CONDITION THAT IT IS NOT TO BE REPRODUCED OR COPIED, IN WHOLE OR IN PART, OR USED FOR FURNISHING INFORMATION TO ANY PERSON WITHOUT THE WRITTEN CONSENT OF THE AEP SERVICE CORP. , OR FOR ANY PURPOSE DETRIMENTAL TO THEIR INTEREST, AND IS TO BE RETURNED UPON REQUEST"

CARDINAL OPERATING COMPANY

CARDINAL PLANT

BRILLIANT OHIO

DAM RAISING
FLY ASH RETENTION DAM II
DAM CONTROL PLAN AND PROFILE

DWG. NO. 13-30082-C

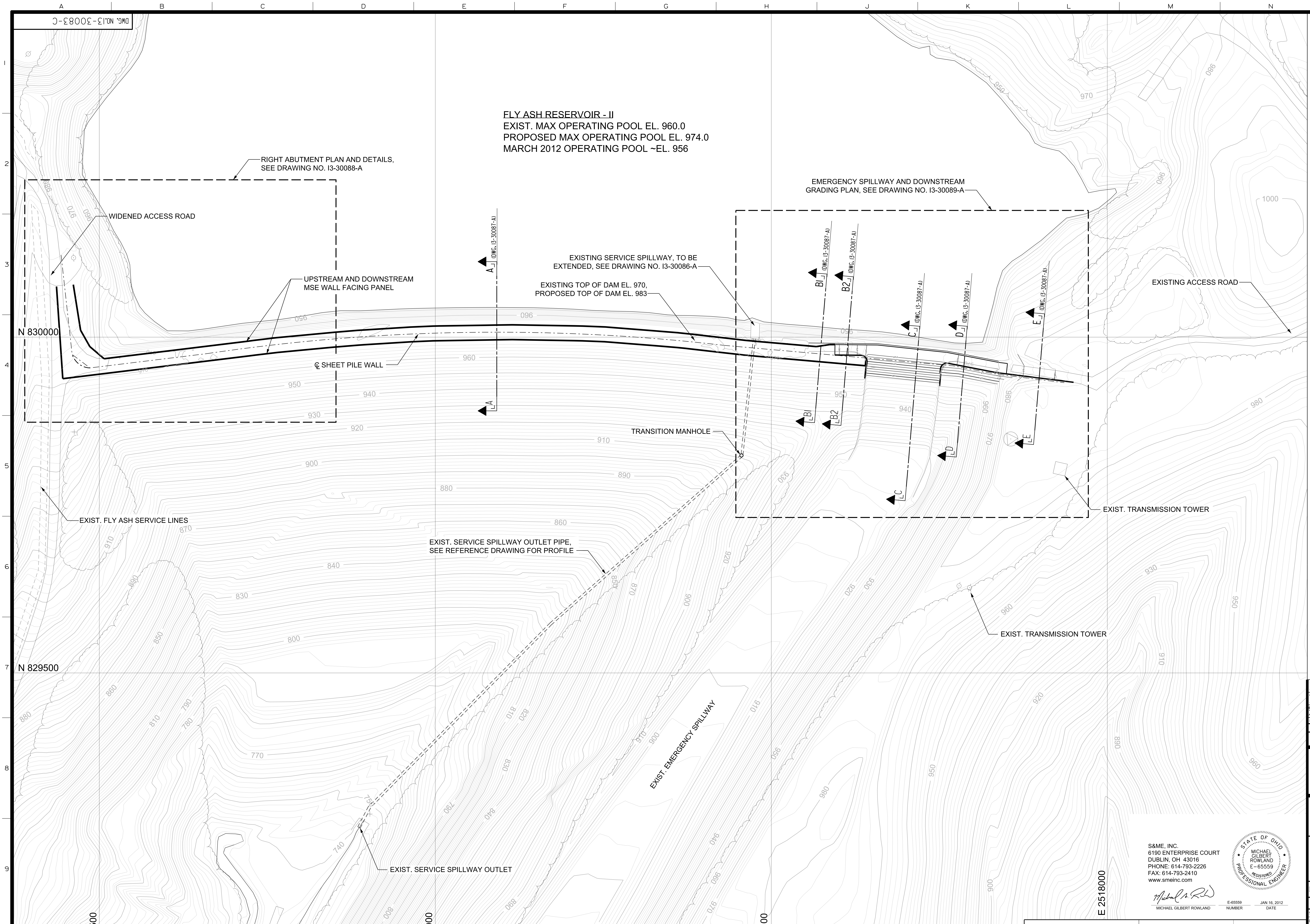
ARCH	—	ELEC	—	MECH	—	STR	GS

SCALE: AS SHOWN
DR:
CH:
SOD SUPV:
DATE:
APPROVED BY
CIVIL ENGINEERING DIVISION

AEP AMERICAN ELECTRIC POWER

1 RIVERSIDE PLAZA
COLUMBUS, OH 43215

SYSTEM DATE- DD-MMM-YYYY
SYSTEM TIME- HOUR:MINUTE
15Φ= FLR.CED



LEGEND

900	EXISTING CONTOUR 10FT INTERVAL
950	PROPOSED GRADE CONTOUR 10FT INTERVAL
	EXISTING ACCESS ROADS
	EXISTING FLY ASH SERVICE LINES
	BENCHMARK
	EXISTING TREELINE
A	SECTION NUMBER AND LOCATION

GENERAL NOTES

1. TOPOGRAPHIC MAPPING DEVELOPED FROM AERIAL SURVEY PERFORMED ON 3/5/2009

0 50 100
SCALE IN FEET

DATE	NO.	DESCRIPTION	APPD.
1/16/13	C	REVISED PER ODNR COMMENTS REC'D 12/4/2012	MGR
9/12/12	B	REVISED PER ODNR COMMENTS REC'D 7/18/12	MGR
5/23/12	A	ISSUED FOR PERMIT INITIAL SUMMITTAL	MGR

REVISIONS

CARDINAL OPERATING COMPANY
CARDINAL PLANT
BRILLIANT OHIO

DAM RAISING
FLY ASH RETENTION DAM II
DAM SITE PLAN

DWG. NO. I3-30083-C

ARCH — ELEC — MECH — STR

SCALE: AS SHOWN
DR:
CH:
SQD SUPV:
DATE:
APPROVED BY
CIVIL ENGINEERING DIVISION

AMERICAN ELECTRIC POWER
1 RIVERSIDE PLAZA
COLUMBUS, OH 43215

S&ME, INC.
6190 ENTERPRISE COURT
DUBLIN, OH 43016
PHONE: 614-793-2226
FAX: 614-793-2410
www.smeinc.com

MICHAEL GILBERT ROWLAND E-65559

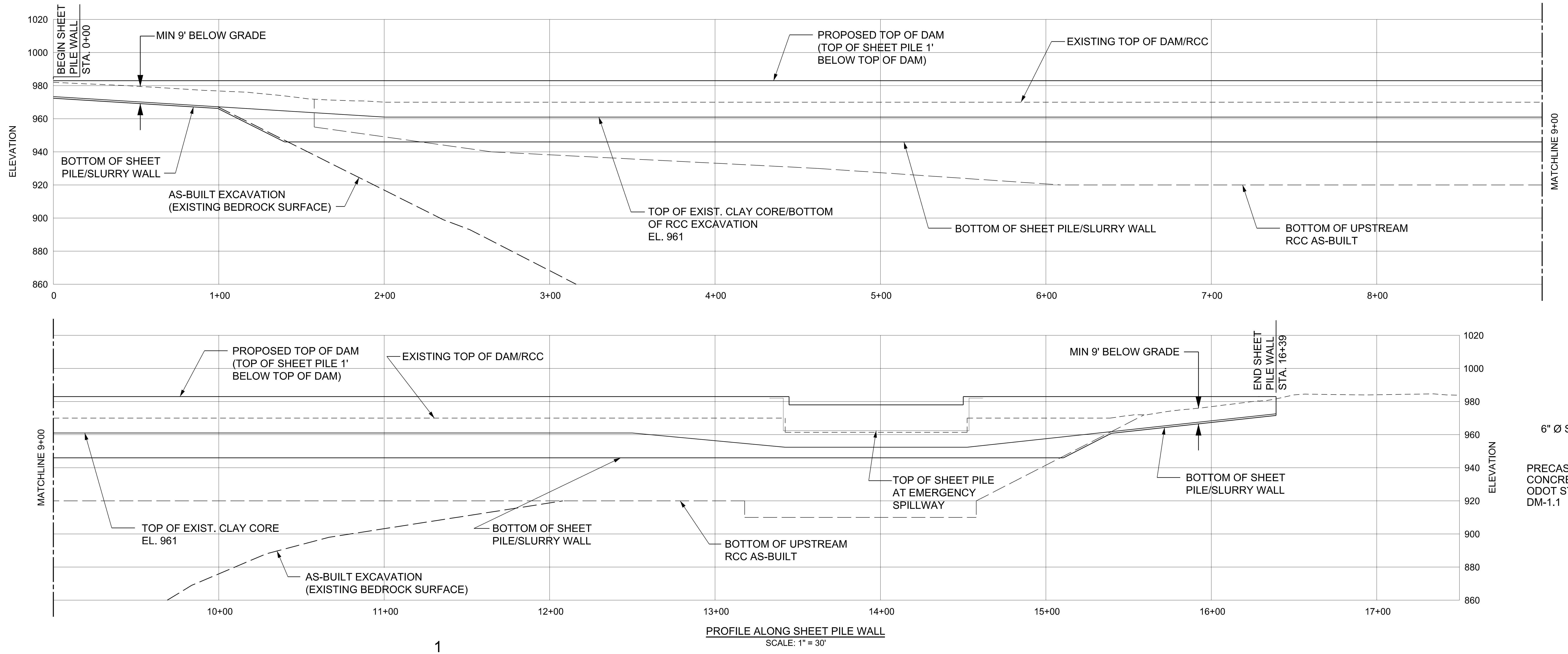
Michael G. Rowland		E-65559	JAN 16, 2012
MICHAEL GILBERT ROWLAND		NUMBER	DATE

PROJECT NUMBER:	011-11497-042	DRAWN BY:	EDV
DRAWING DATE:	9/28/12	ENGINEER:	MTR
LAST UPDATED:	1/16/13	APPROVED BY:	MGR
		SCALE:	1" = 50'

S&ME

SYSTEM DATE- DD-MMM-YYYY
SYSTEM TIME- HOUR:MINUTE
I5Φ≡ FLR.CED

DWG. NO. 13-30084-C



PROFILE ALONG SHEET PILE WALL

SCALE: 1" = 30'

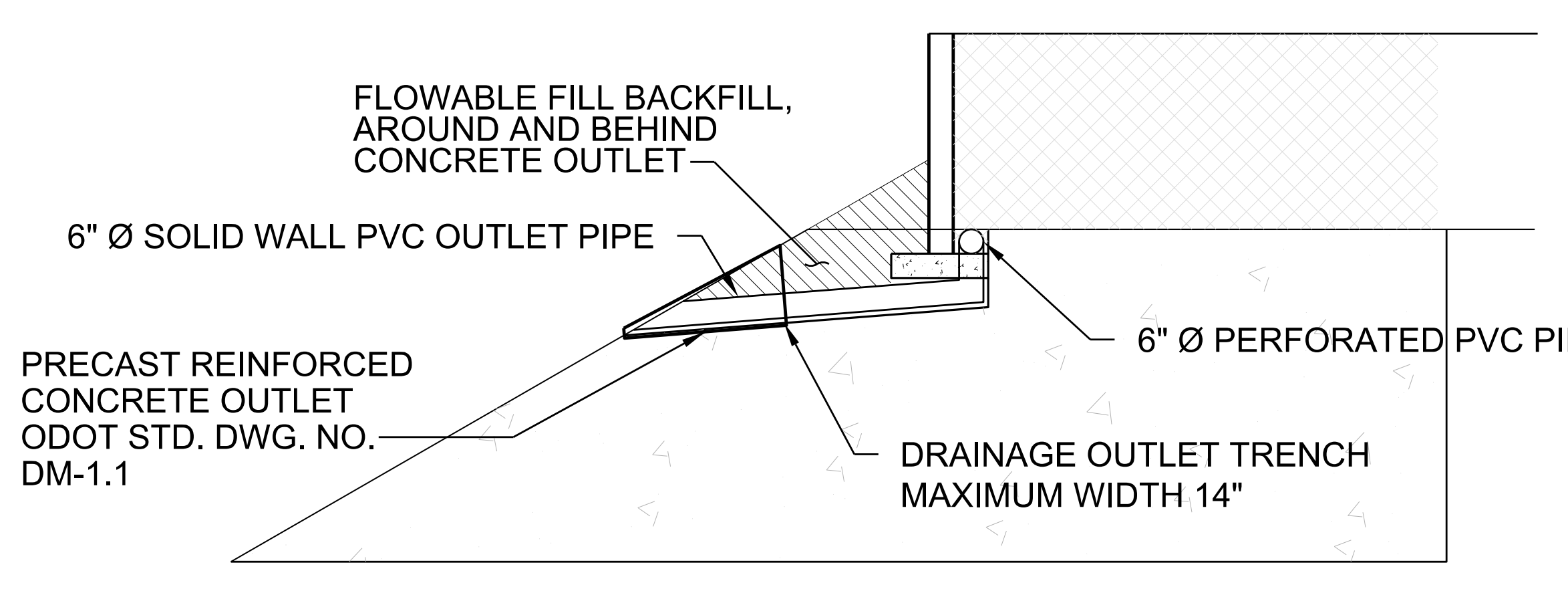
GENERAL SEQUENCE OF TRENCH EXCAVATION AND SHEET PILE INSTALLATION:

1. EXCAVATE RCC AND ABUTMENTS TO TOP OF CLAY CORE OR TO DEPTHS SHOWN WITH ROCK TRENCHER TO MINIMIZE DISTURBANCE OF RCC.

2. INSTALL CEMENT-BENTONITE SLURRY WALL TO BOTTOM OF SHEET PILE ELEVATION USING LONG-REACH EXCAVATOR. MINIMUM 3 FOOT PENETRATION INTO CLAY CORE BENEATH RCC

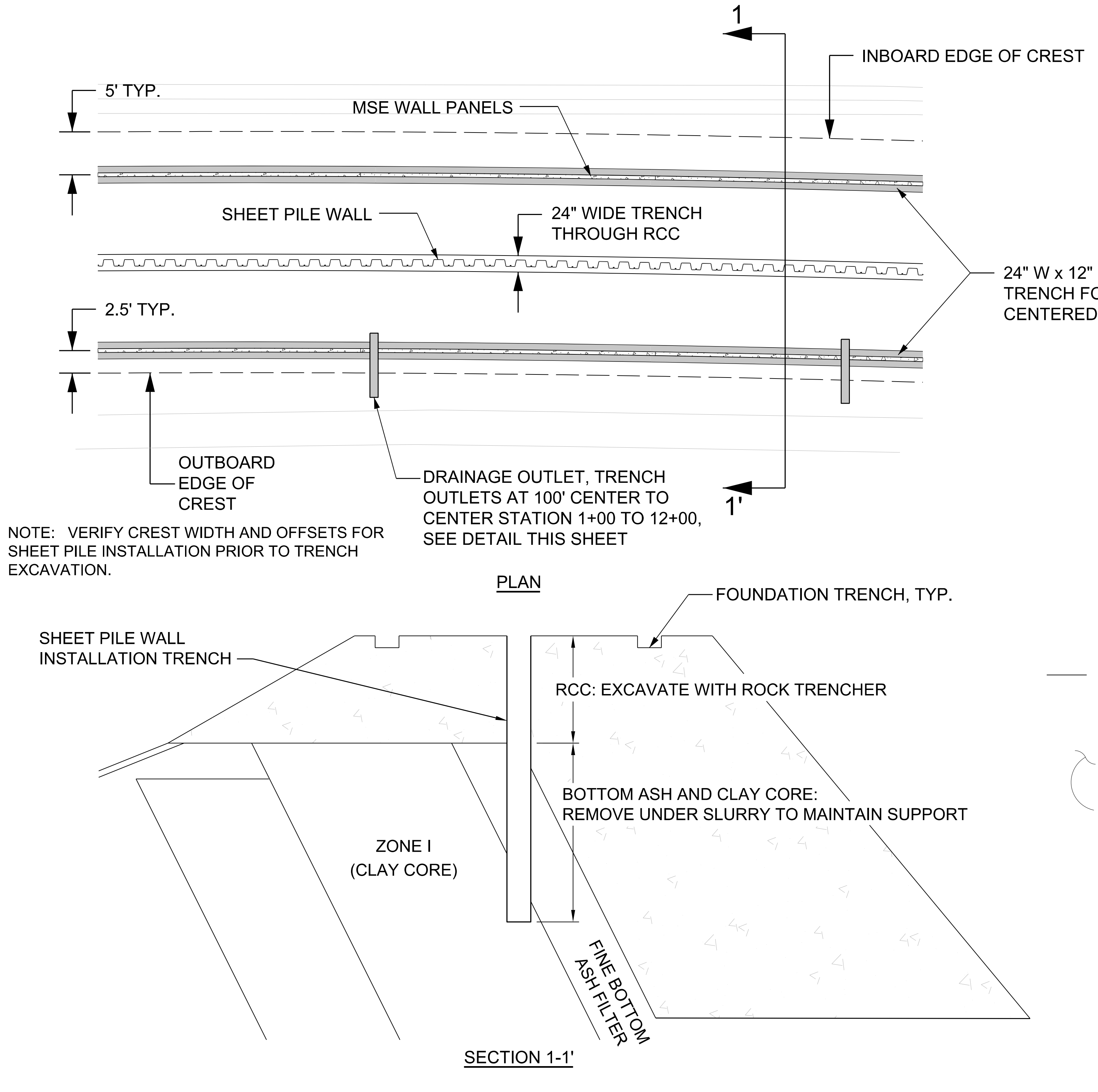
3. INSTALL SHEET PILE WALL THROUGH HARDENED SLURRY WALL. SHEET PILE TO EXTEND ABOVE TOP OF RCC TRENCH TO 1 FOOT BELOW TOP OF DAM.

4. APPLY SIKAFLEX CAULKING TO UPSTREAM JOINT FROM TOP OF SLURRY WALL TO TOP OF PILE



DRAINAGE OUTLET DETAIL

NOT TO SCALE



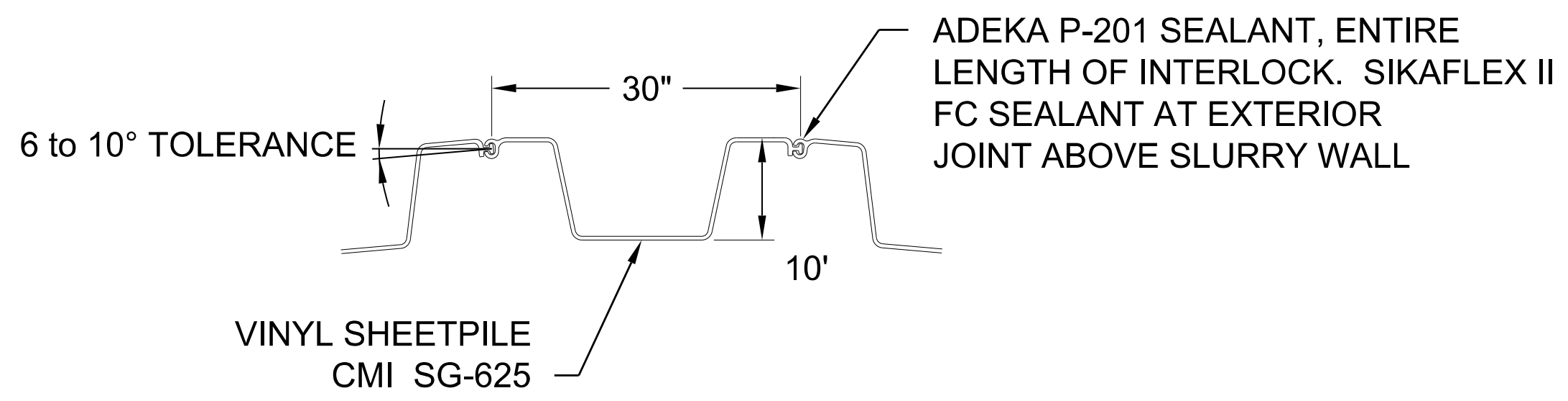
NOTE: VERIFY CREST WIDTH AND OFFSETS FOR SHEET PILE INSTALLATION PRIOR TO TRENCH EXCAVATION.

PLAN

SECTION 1-1'

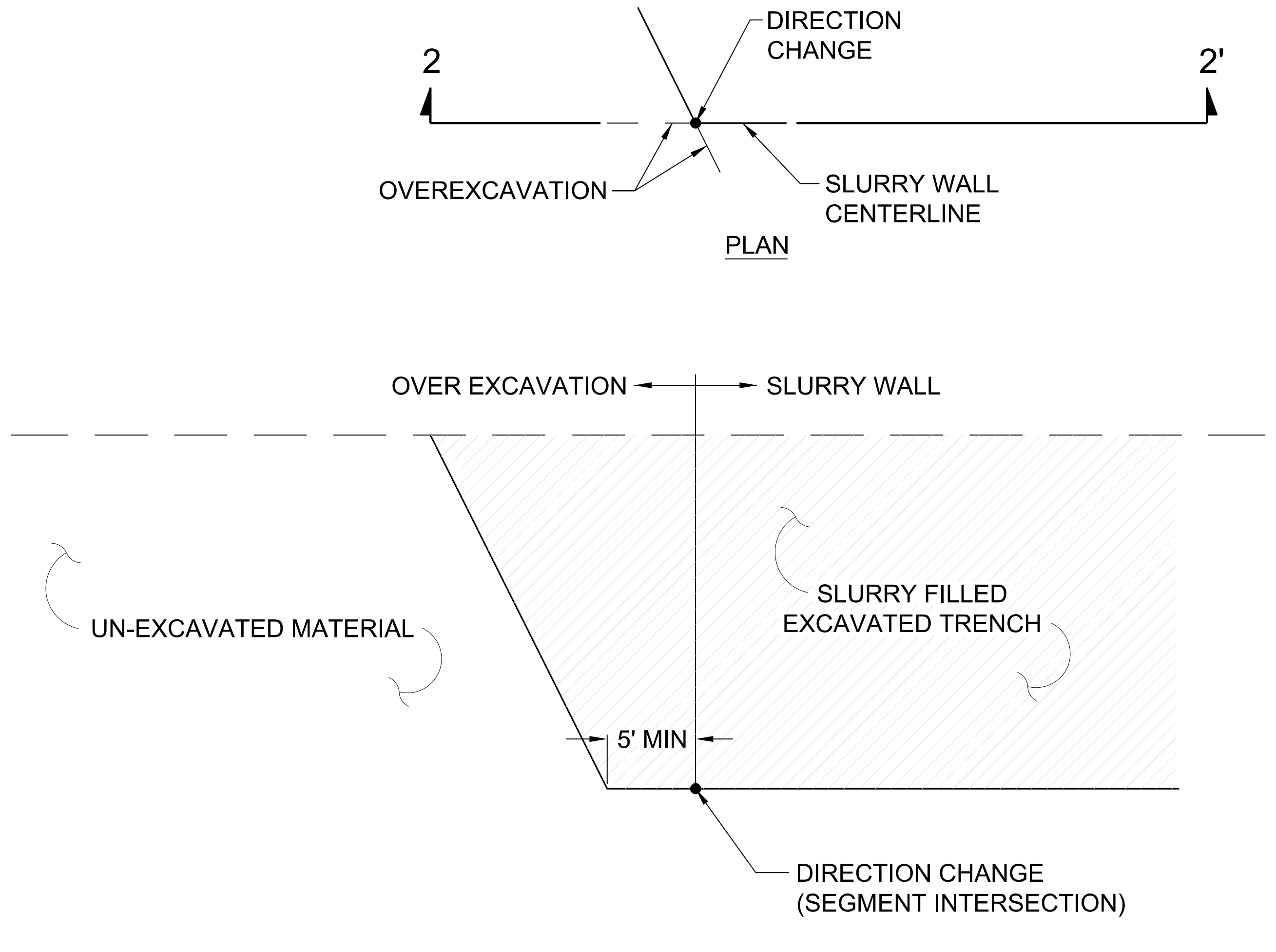
TRENCHING DETAIL

NOT TO SCALE



SHEET PILE DETAIL

NOT TO SCALE

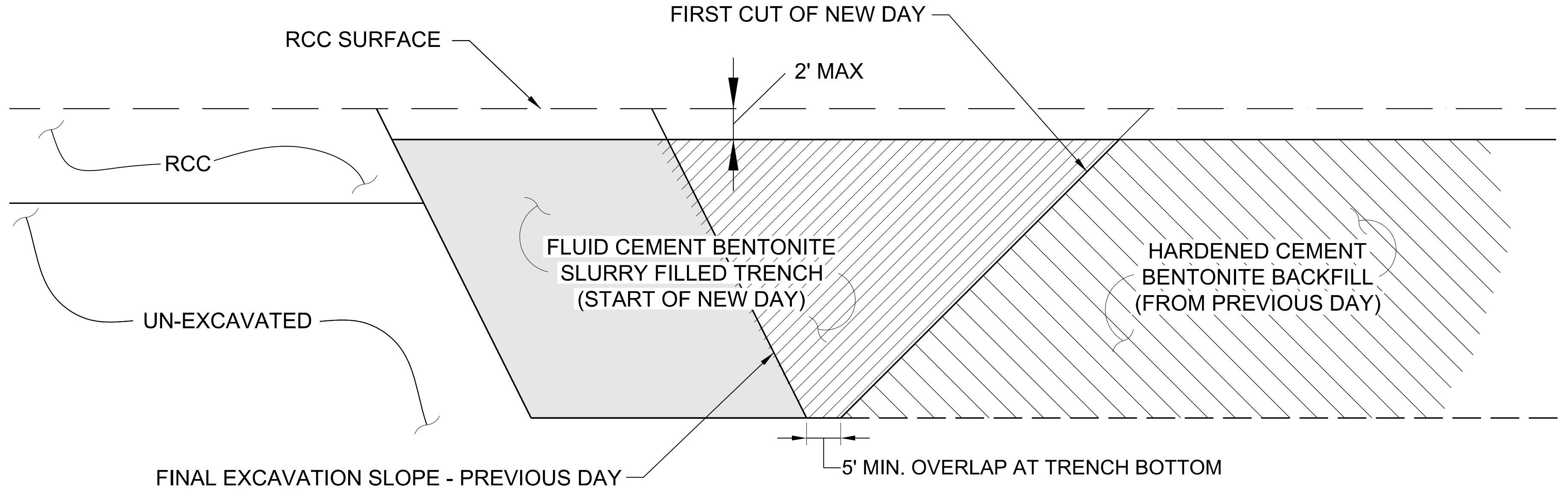


PLAN

PROFILE 2-2'

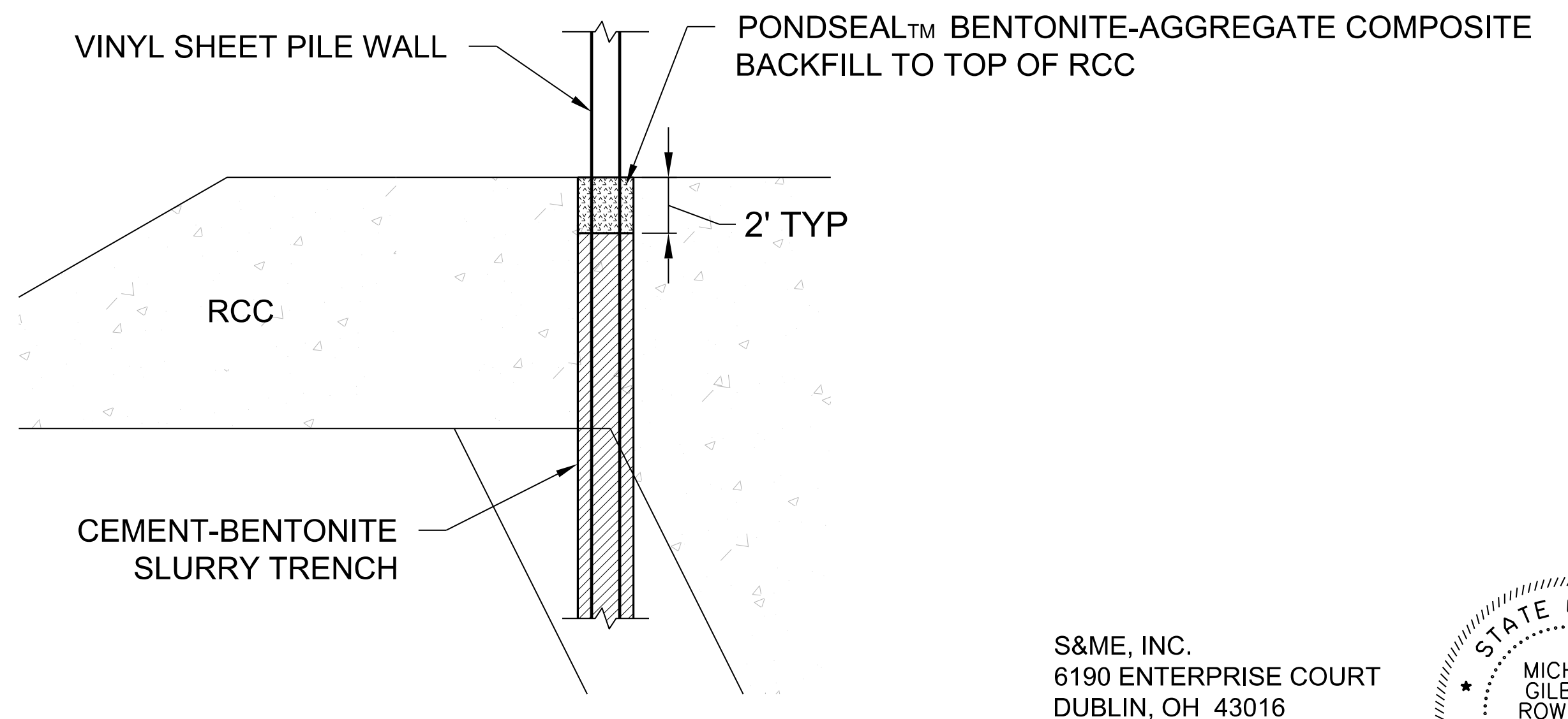
OVER EXCAVATION AT SLURRY WALL DIRECTION CHANGE

NOT TO SCALE



TYPICAL NEW DAY SLURRY WALL TIE-IN

NOT TO SCALE



SLURRY WALL BACKFILL

NOT TO SCALE

0 30 60
SCALE IN FEET

S&ME, INC.
6190 ENTERPRISE COURT
DUBLIN, OH 43016
PHONE: 614-793-2226
FAX: 614-793-2410
www.smeinc.com

MICHAEL GILBERT ROWLAND E-65559

Michael G. Rowland

MICHAEL GILBERT ROWLAND	E-65559 NUMBER	JAN 16, 2012 DATE

PROJECT NUMBER:	011-11497-042	DRAWN BY:	EDV
DRAWING DATE:	9/28/12	ENGINEER:	MTR
LAST UPDATED:	1/16/13	APPROVED BY:	MGR
		SCALE:	AS NOTED

S&ME

DATE	NO.	DESCRIPTION	APPD.
1/16/13	C	REVISED PER ODNR COMMENTS REC'D 12/4/2012	MGR
9/12/12	B	REVISED PER ODNR COMMENTS REC'D 7/18/12	MGR
5/23/12	A	ISSUED FOR PERMIT INITIAL SUMMITTAL	MGR

REVISIONS

CARDINAL OPERATING COMPANY

CARDINAL PLANT

BRILLIANT OHIO

DAM RAISING

FLY ASH RETENTION DAM II

TRENCH EXCAVATION, SLURRY WALL, AND SHEET PILE PLAN

DWG. NO. 13-30084-C

ARCH	—	ELEC	—	MECH	—	STR	GS

SCALE: AS SHOWN
DR:
CH:
SQD SUPV:
DATE:
APPROVED BY
CIVIL ENGINEERING DIVISION

AMERICAN ELECTRIC POWER

1 RIVERSIDE PLAZA
COLUMBUS, OH 43215

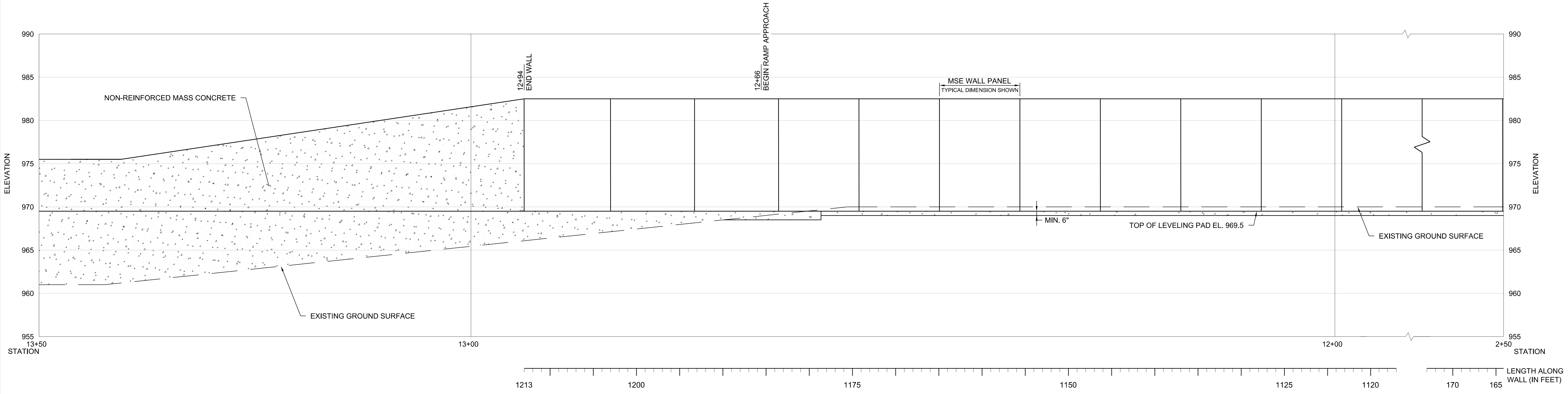
SYSTEM DATE- DD-MMM-YYYY
SYSTEM TIME- HOUR:MINUTE
I5Φ≡ FLR.CED

IGSI092

DWG. NO. 13-30085-C

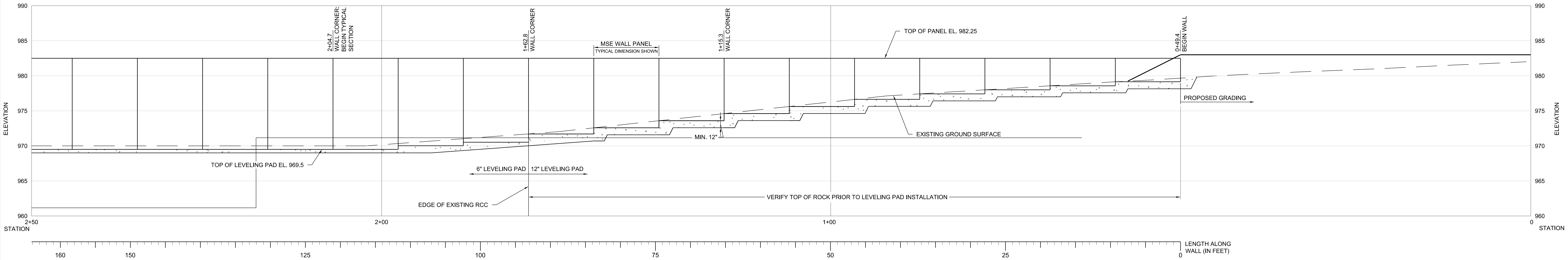
DOWNSTREAM MSE WALL SOUTH ELEVATION

STA 0+45.75 TO STA 13+35



UPSTREAM MSE WALL NORTH ELEVATION

STA 2+50 TO STA 12+94



UPSTREAM MSE WALL NORTH ELEVATION

STA 0+49.4 TO STA 2+50

NOTES: CONCRETE COPING NOT SHOWN
SEE DETAIL ON DRAWING 13-30086-A

NOTES:
1. LEVELING PAD TO BE SUPPORTED ON BEDROCK AS CONFIRMED BY ENGINEER. IF OVEREXCAVATION REQUIRED, BACKFILL WITH LEAN CONCRETE TO PROPOSED LEVELING PAD BASE ELEVATION.
2. CONCRETE COPING NOT SHOWN. PLAN TO BE SUBMITTED BY CONTRACTOR FOR APPROVAL. SEE TYPICAL COPING DETAIL ON DWG. 13-30086-A

DATE	NO.	DESCRIPTION	APPD.
1/16/13	C	REVISED PER ODNR COMMENTS REC'D 12/4/2012	MGR
9/12/12	B	REVISED PER ODNR COMMENTS REC'D 7/18/12	MGR
5/23/12	A	ISSUED FOR PERMIT INITIAL SUMMITTAL	MGR

REVISIONS

CARDINAL OPERATING COMPANY
CARDINAL PLANT
BRILLIANT OHIO

DAM RAISING
FLY ASH RETENTION DAM II
MSE WALL
ELEVATION

DWG. NO. 13-30085-C

ARCH	—	ELEC	—	MECH	—	STR

SCALE: AS SHOWN
DR:
CH:
SQD SUPV:
DATE:
APPROVED BY
CIVIL ENGINEERING DIVISION

AEP AMERICAN ELECTRIC POWER
1 RIVERSIDE PLAZA
COLUMBUS, OH 43215

S&ME, INC.
6190 ENTERPRISE COURT
DUBLIN, OH 43016
PHONE: 614-793-2226
FAX: 614-793-2410
www.smeinc.com

MICHAEL GILBERT ROWLAND
E-65559

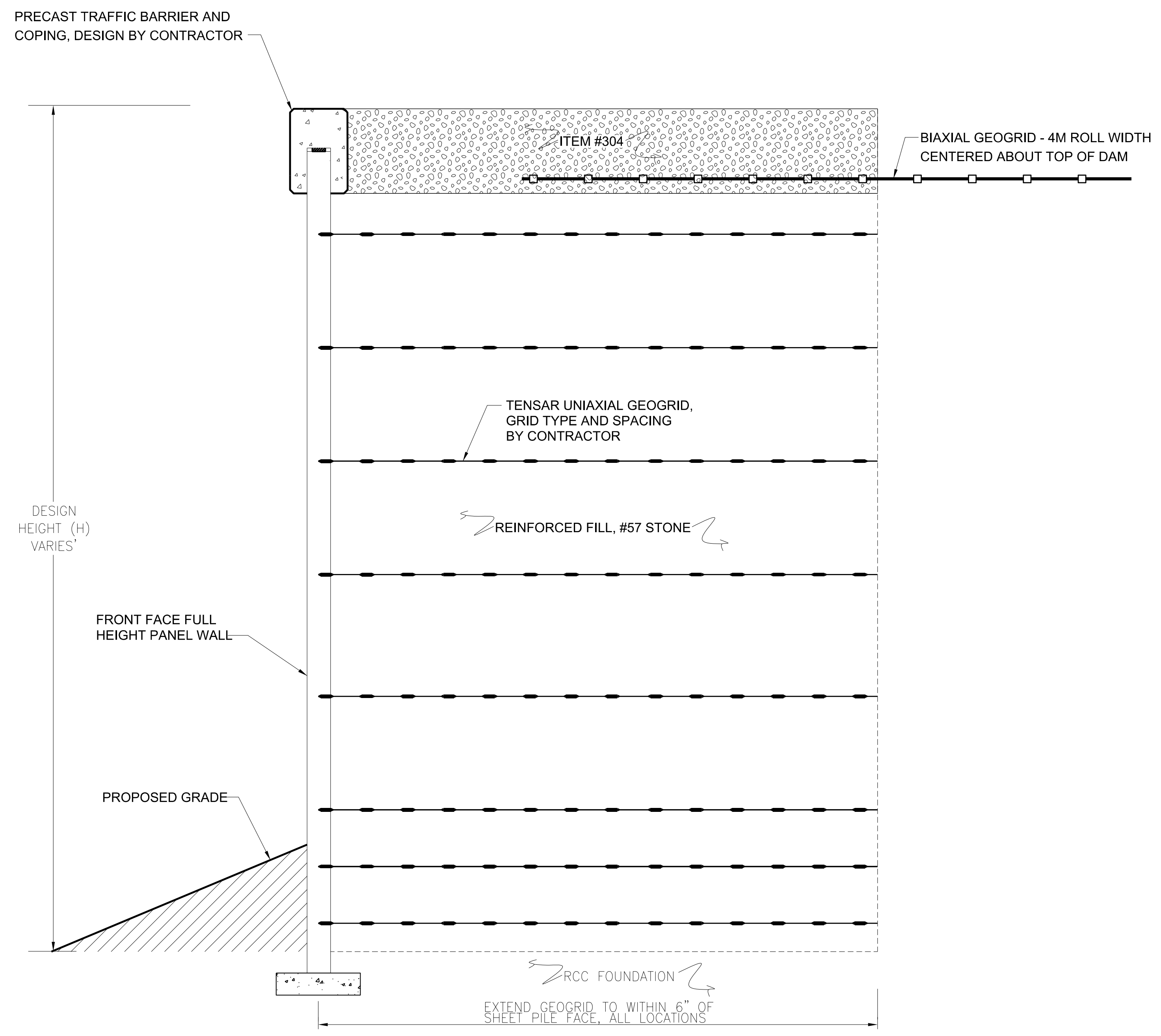
Michael G. Rowland

MICHAEL GILBERT ROWLAND	E-65559	JAN 16, 2012
	NUMBER	DATE

PROJECT NUMBER:	011-11497-042	DRAWN BY:	EDV
DRAWING DATE:	9/28/12	ENGINEER:	MTR
LAST UPDATED:	1/16/13	APPROVED BY:	MGR
		SCALE:	AS NOTED

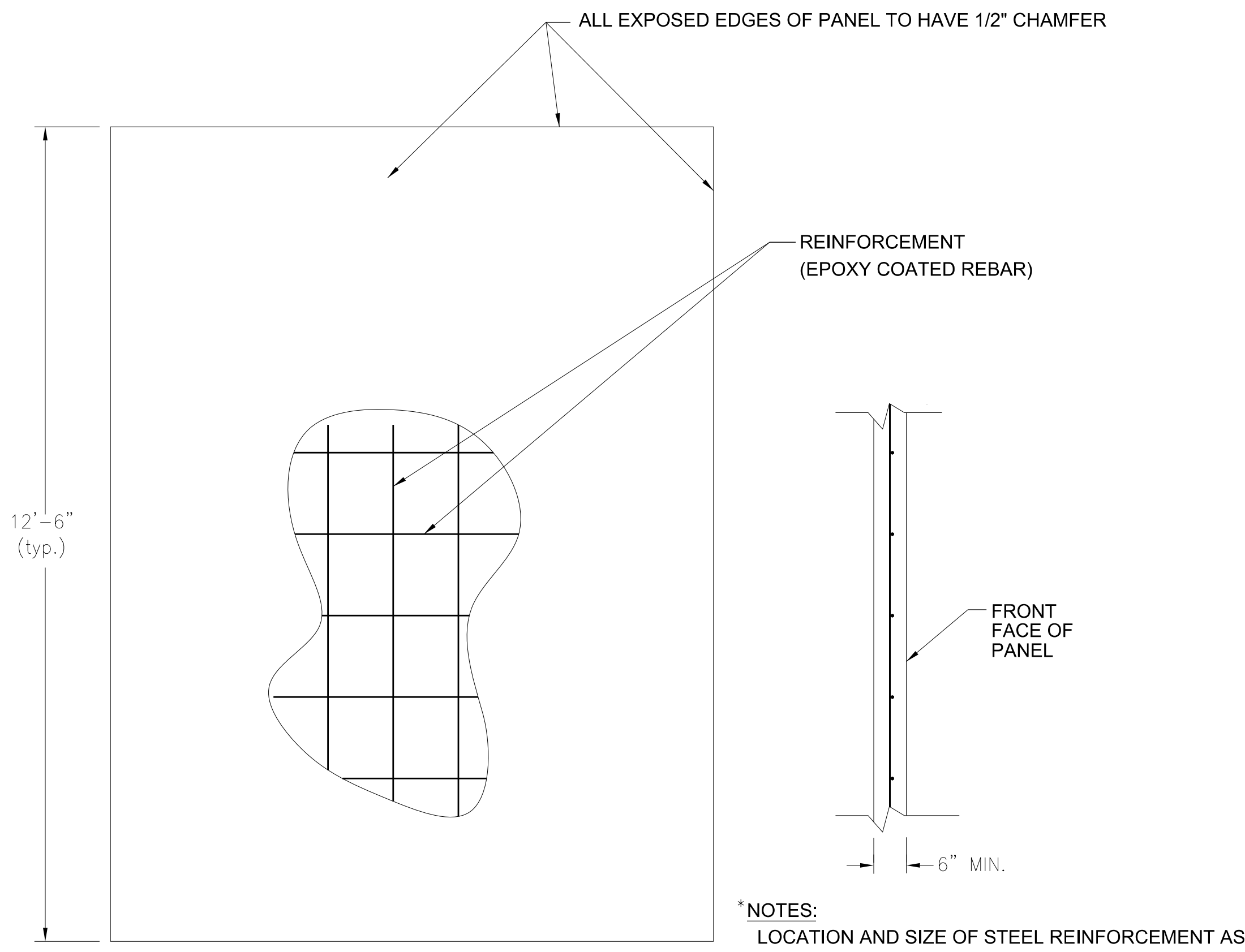
S&ME

DWG. NO. 13-30086-C



TYPICAL CROSS-SECTION-FULL HEIGHT PANEL WALL

NOT TO SCALE



* TYPICAL PANEL WIDTH, CONTRACTOR MAY ELECT TO USE ALTERNATE WIDTH IN CONSULTATION WITH TENSAR AND AS APPROVED BY OWNER.

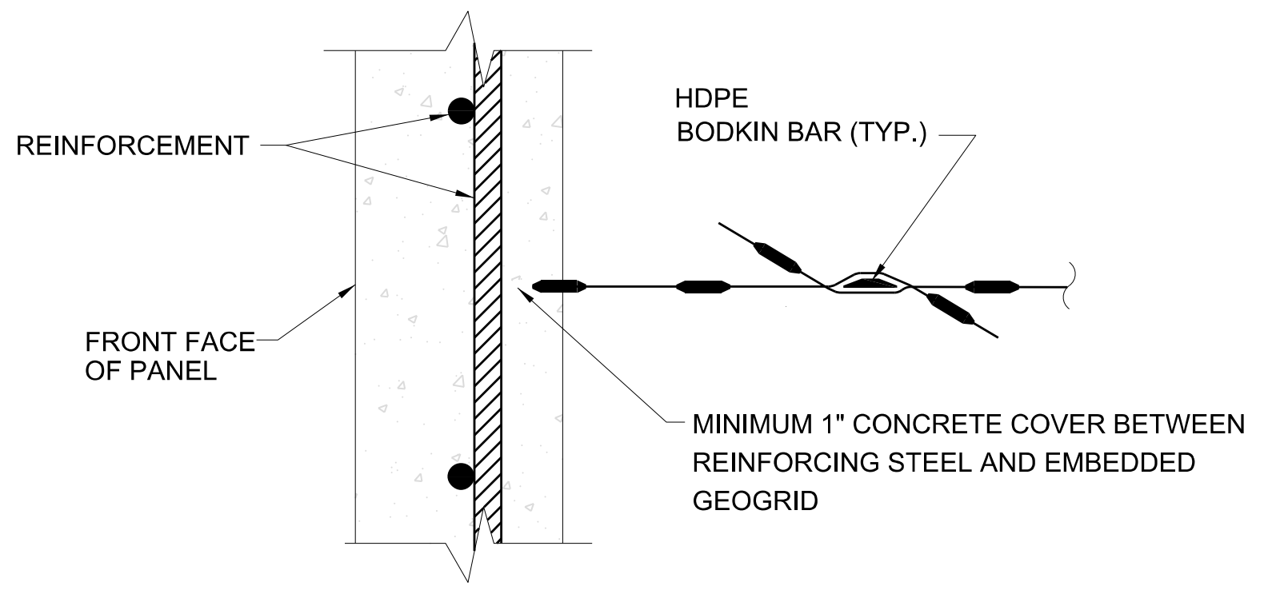
*NOTES:

LOCATION AND SIZE OF STEEL REINFORCEMENT AS WELL AS PANEL THICKNESS WILL VARY DEPENDING ON PROJECT SPECIFIC REQUIREMENTS.

FINAL DESIGN BY CONTRACTOR: SEE SPECIFICATIONS

PANEL REINFORCEMENT (TYP.)

NOT TO SCALE

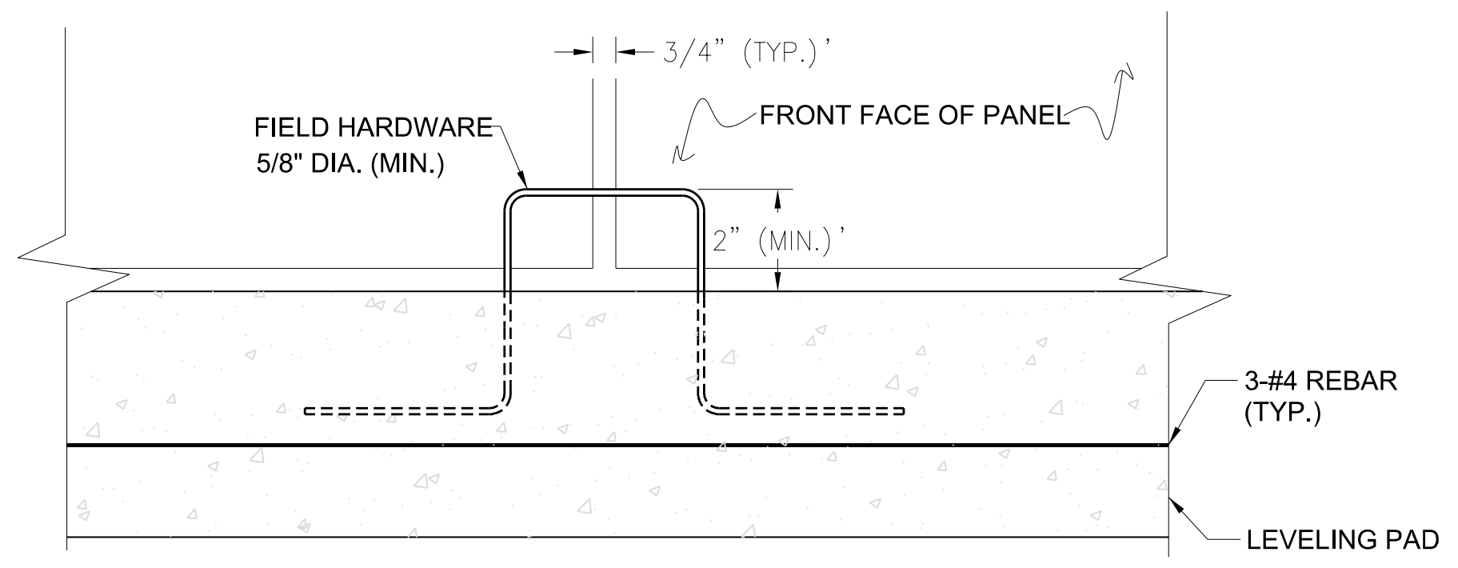


PANEL REINFORCEMENT DETAIL

NOT TO SCALE

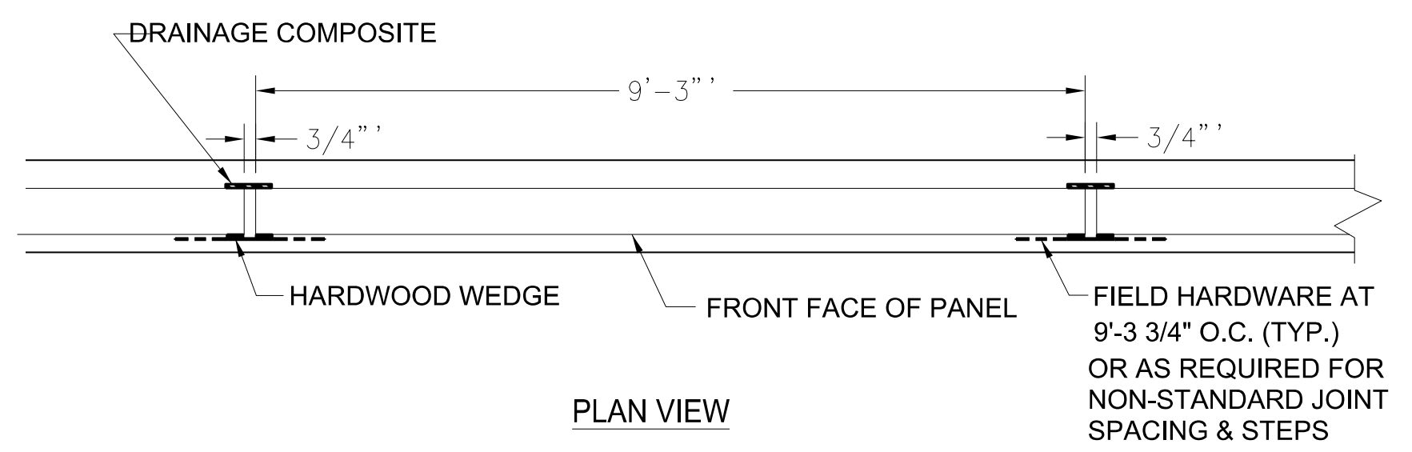
*NOTE:

LOCATION AND SIZE OF STEEL REINFORCEMENT AS WELL AS PANEL THICKNESS WILL VARY, TO BE DESIGNED BY CONTRACTOR.



PANEL PAD DETAIL - TYPE 1

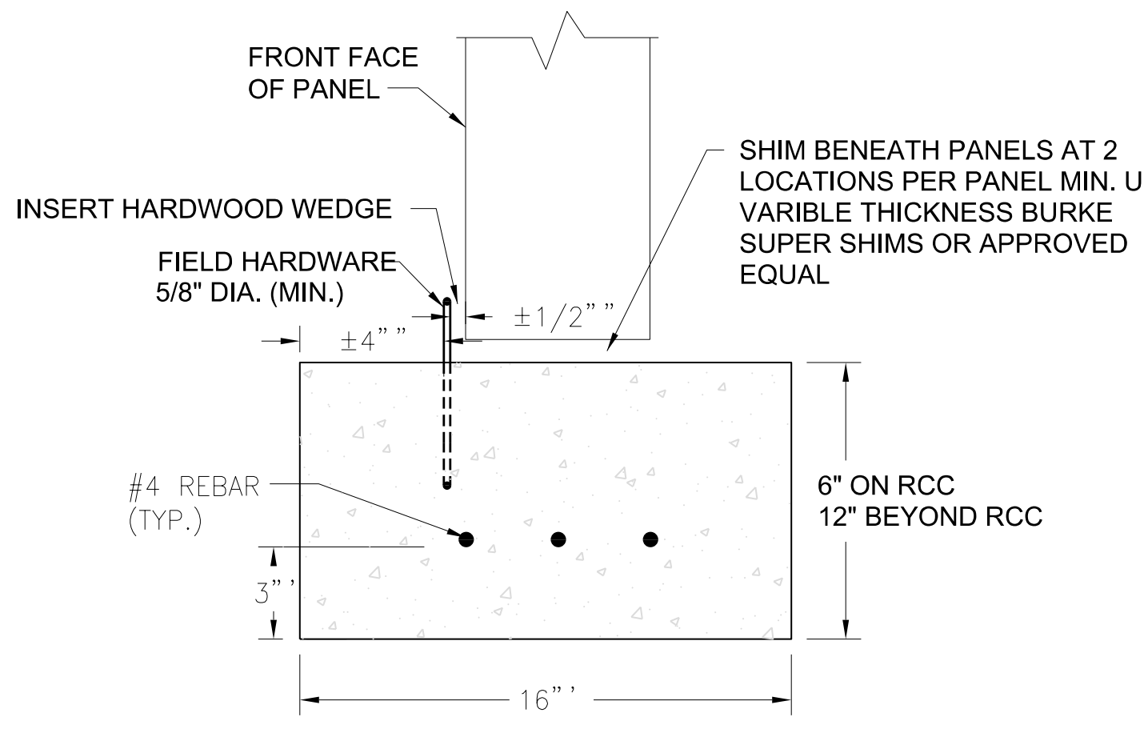
NOT TO SCALE



PLAN VIEW

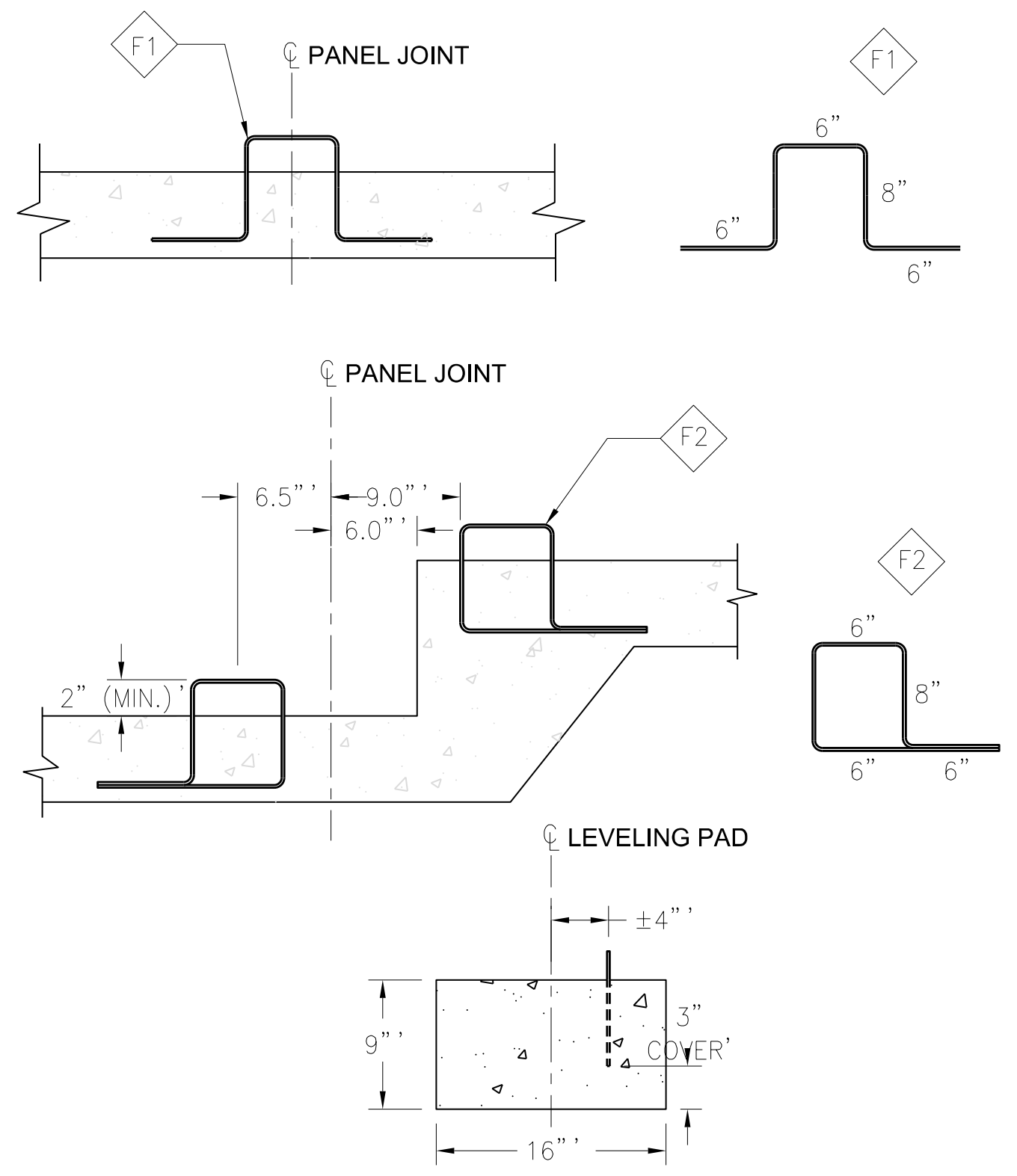
TYPE 1 LEVELING PAD

NOT TO SCALE



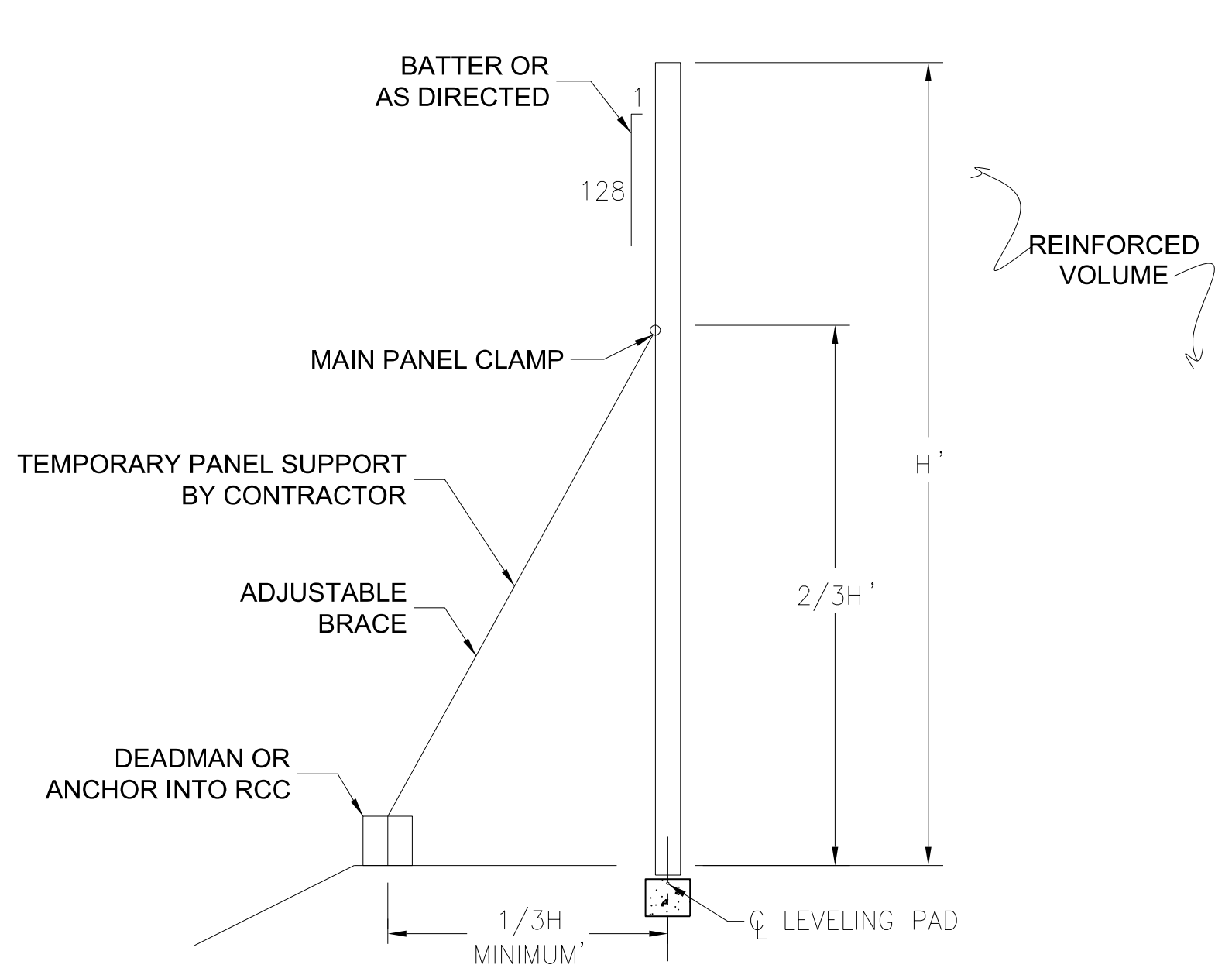
PANEL PAD DETAIL - TYPE 1

NOT TO SCALE



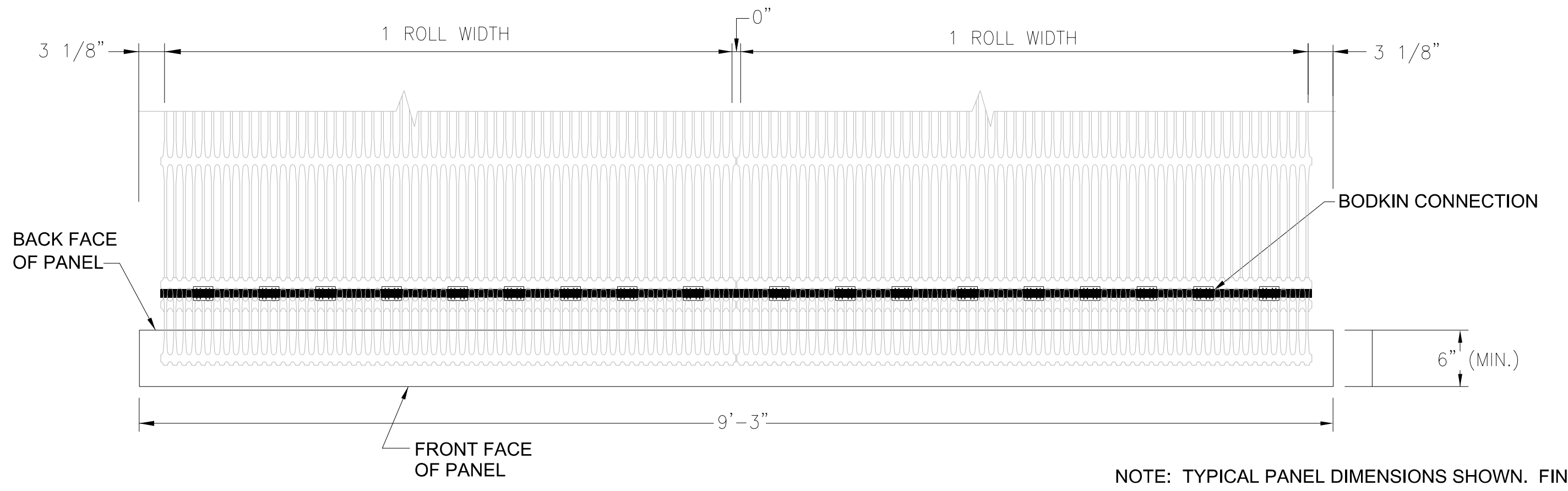
FIELD HARDWARE-TYPE 1 LEVELING PAD

NOT TO SCALE



PANEL BRACING DETAIL

NOT TO SCALE

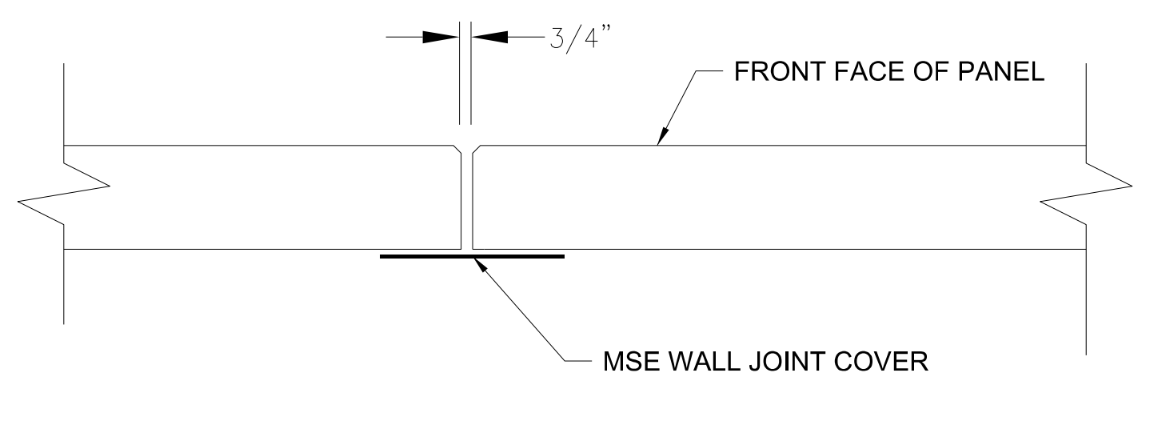


NOTE: TYPICAL PANEL DIMENSIONS SHOWN. FINAL DIMENSIONS TO BE SELECTED BY CONTRACTOR: MAX PANEL WIDTH NOT TO EXCEED 9'3"

CONNECTION DETAIL PLAN VIEW (86% COVERAGE)

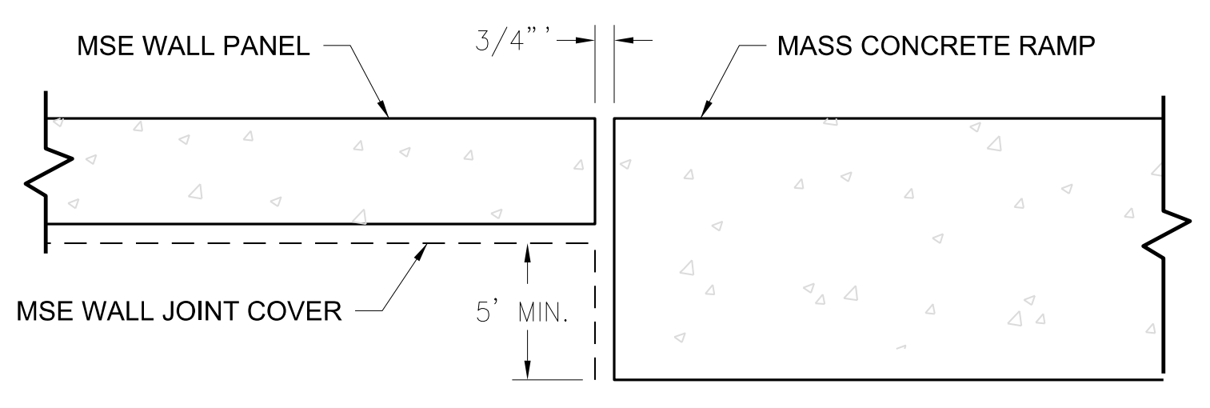
MAXIMUM COVERAGE

NOT TO SCALE



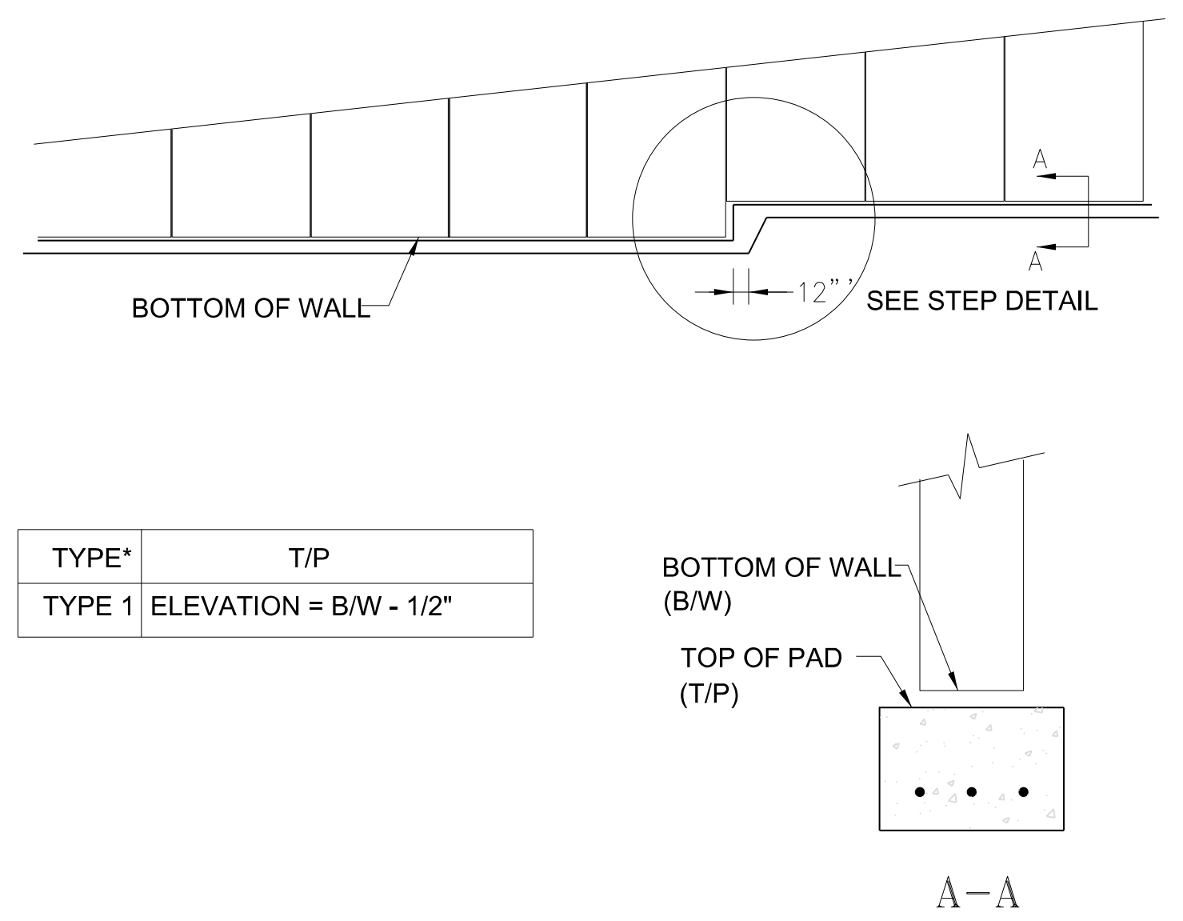
JOINT DETAIL

NOT TO SCALE



MSE WALL END DETAIL

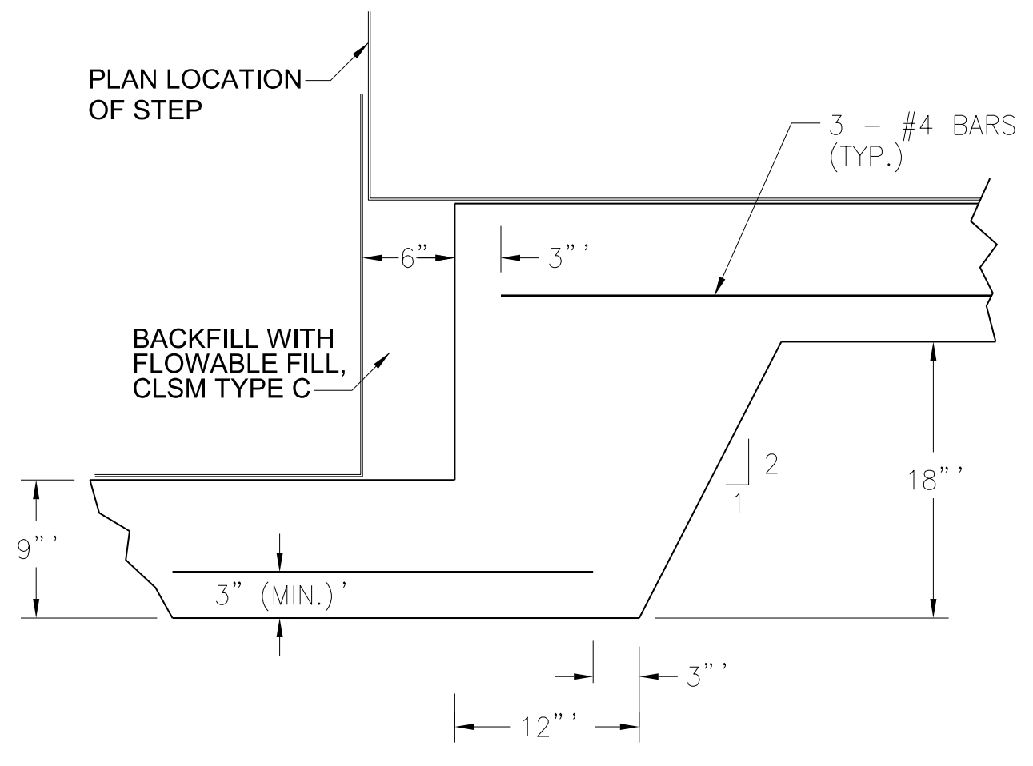
NOT TO SCALE



TYPE*	T/P
TYPE 1	ELEVATION = B/W - 1/2"

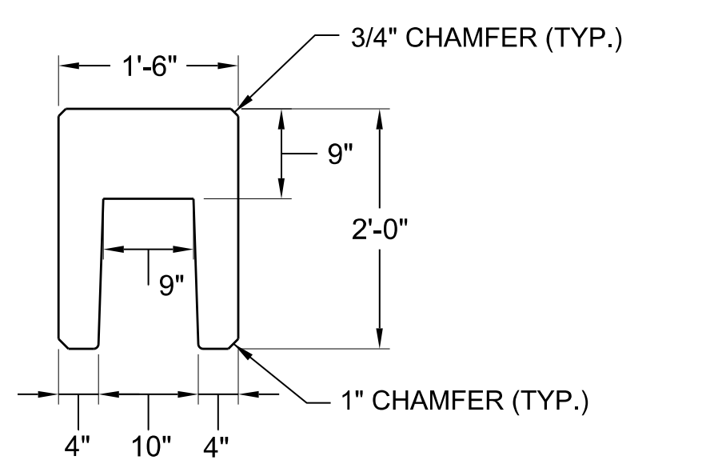
LEVELING PAD DETAILS

NOT TO SCALE



STEP DETAIL

NOT TO SCALE



NOTES: STEEL REINFORCEMENT LOCATION AND SIZE BY CONTRACTOR.

ACTUAL PRECAST COPING TO BE DESIGNED BY CONTRACTOR.

TYPICAL PRECAST COPING DETAIL

NOT TO SCALE

NOTE: THE DETAILS ON THIS SHEET ARE FOR GENERAL INFORMATION PUPOSES ONLY. ACTUAL DETAILS USED FOR CONSTRUCTION SHALL BE THOSE DEVELOPED BY TENSAR ON THEIR STAMPED DESIGN SUBMITTAL.

S&ME, INC.
6190 ENTERPRISE COURT
DUBLIN, OH 43016
PHONE: 614-793-2226
FAX: 614-793-2410
www.smeinc.com

MICHAEL GILBERT ROWLAND
E-65559
JAN 16, 2012

MICHAEL GILBERT ROWLAND	E-65559	JAN 16, 2012
	NUMBER	DATE

PROJECT NUMBER:	011-11497-042	DRAWN BY:	EDV
DRAWING DATE:	9/28/12	ENGINEER:	MTR
LAST UPDATED:	1/16/13	APPROVED BY:	MGR
		SCALE:	AS NOTED

S&ME

1/16 /13	C	REVISED PER ODNR COMMENTS REC'D 12/4/2012	MGR
9/12 /12	B	REVISED PER ODNR COMMENTS REC'D 7/18/12	MGR
5/23 /12	A	ISSUED FOR PERMIT INITIAL SUMMITTAL	MGR
DATE	NO.	DESCRIPTION	APPD.

REVISIONS

CARDINAL OPERATING COMPANY

CARDINAL PLANT

BRILLIANT OHIO

DAM RAISING

FLY ASH RETENTION DAM II

MSE WALL DETAILS

DWG. NO. 13-30086-C

ARCH — ELEC — MECH — STR

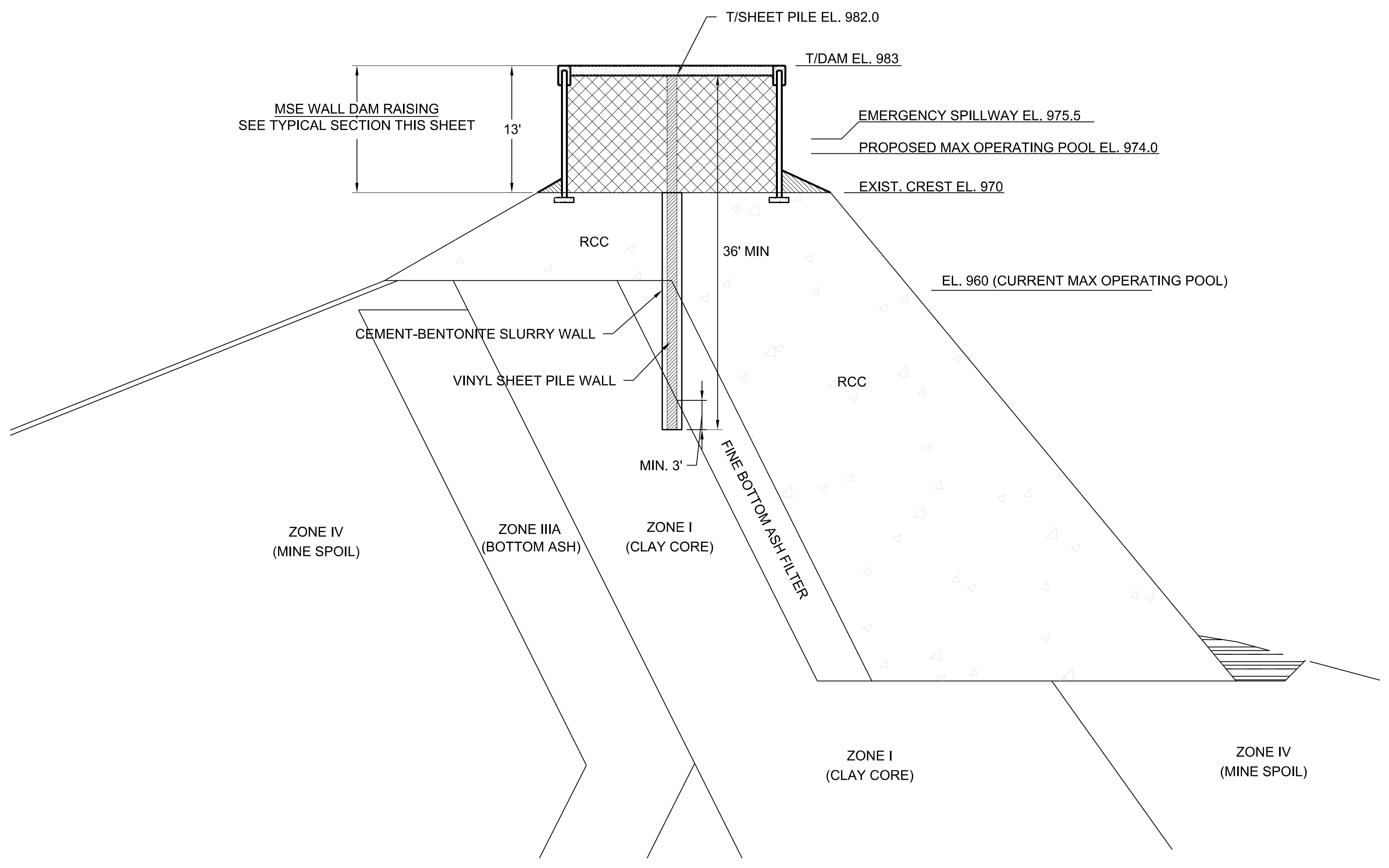
SCALE: AS SHOWN
DR:
CH:
SQD SUPV:
DATE:
APPROVED BY
CIVIL ENGINEERING DIVISION

AEP AMERICAN ELECTRIC POWER

1 RIVERSIDE PLAZA
COLUMBUS, OH 43215

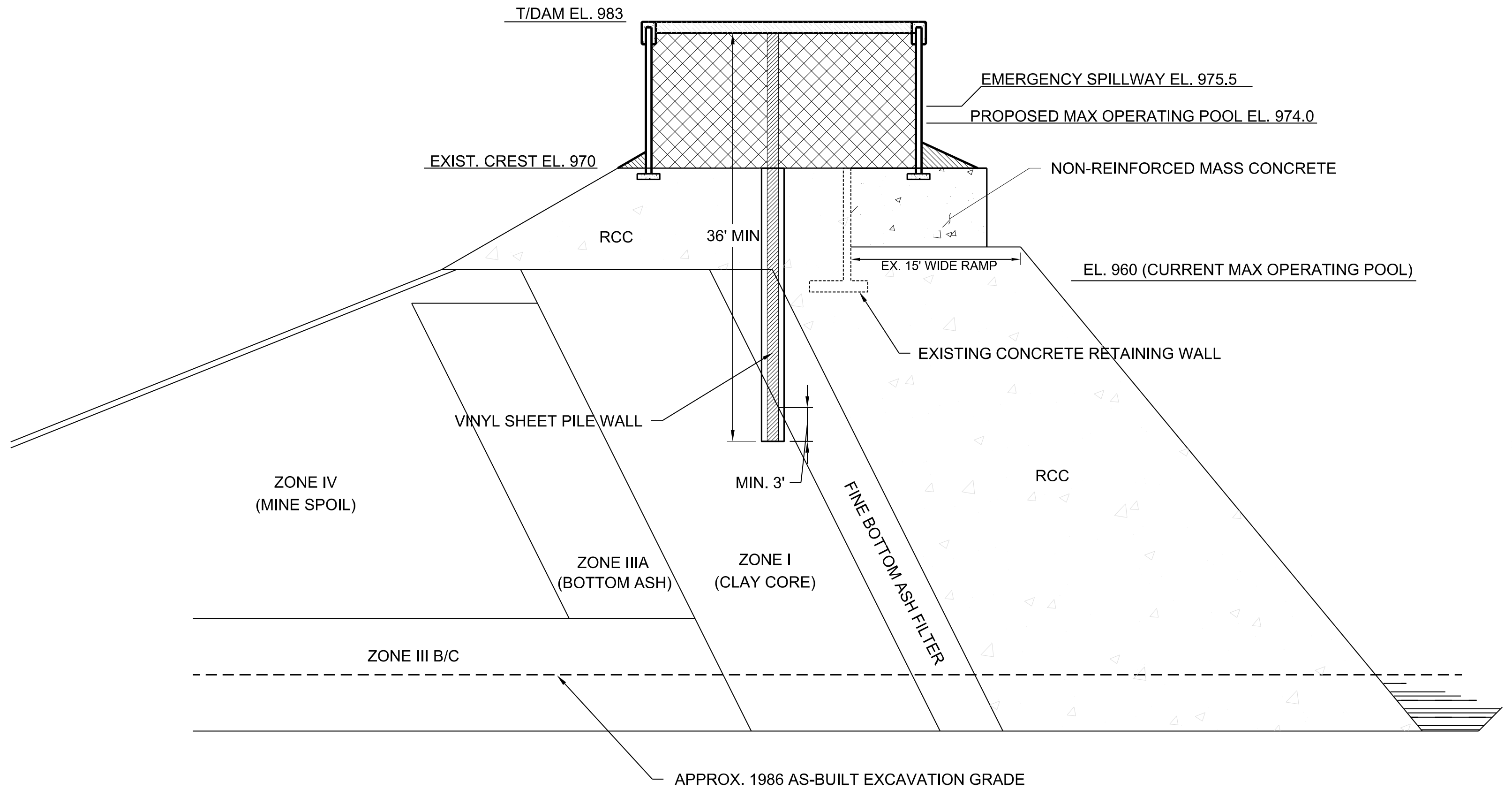
IGS092

SYSTEM DATE- DD-MMM-YYYY
SYSTEM TIME- HOUR:MINUTE
I5Φ≡ FLR.CED



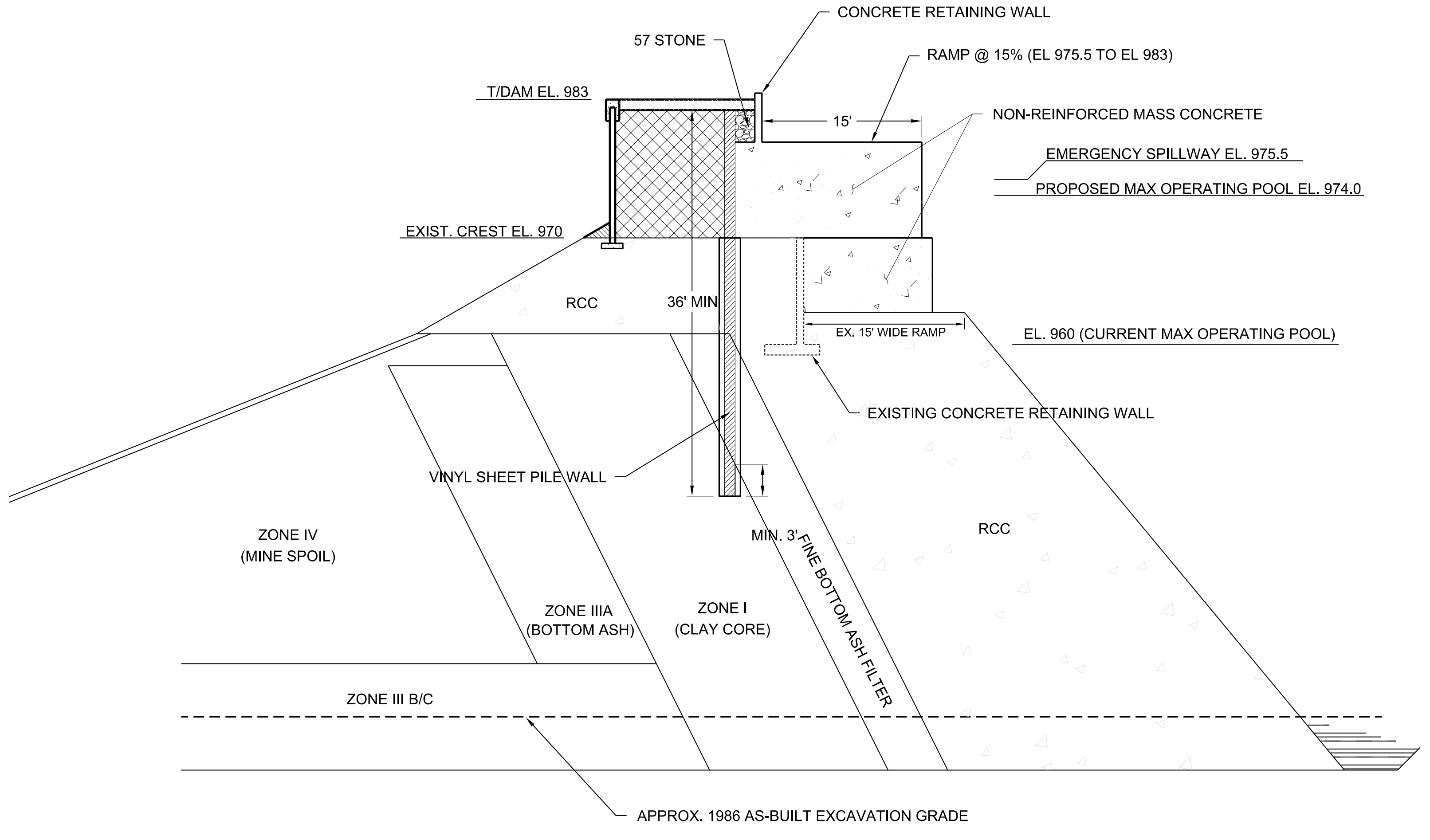
A TYPICAL SECTION - MAIN DAM

SCALE: 1" = 10'



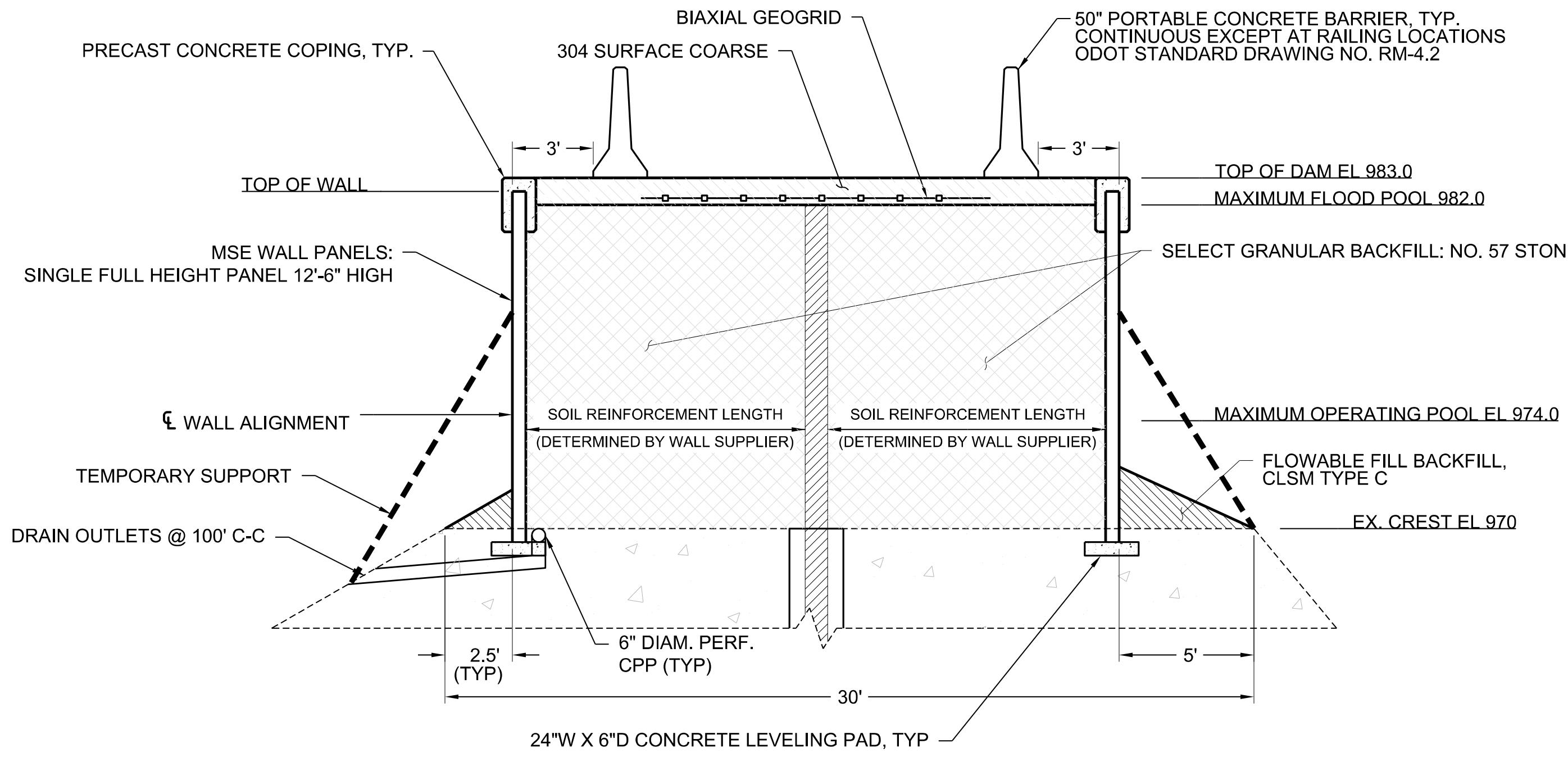
B1 SECTION THROUGH EXISTING EMERGENCY SPILLWAY RAMP

SCALE: 1" = 10'



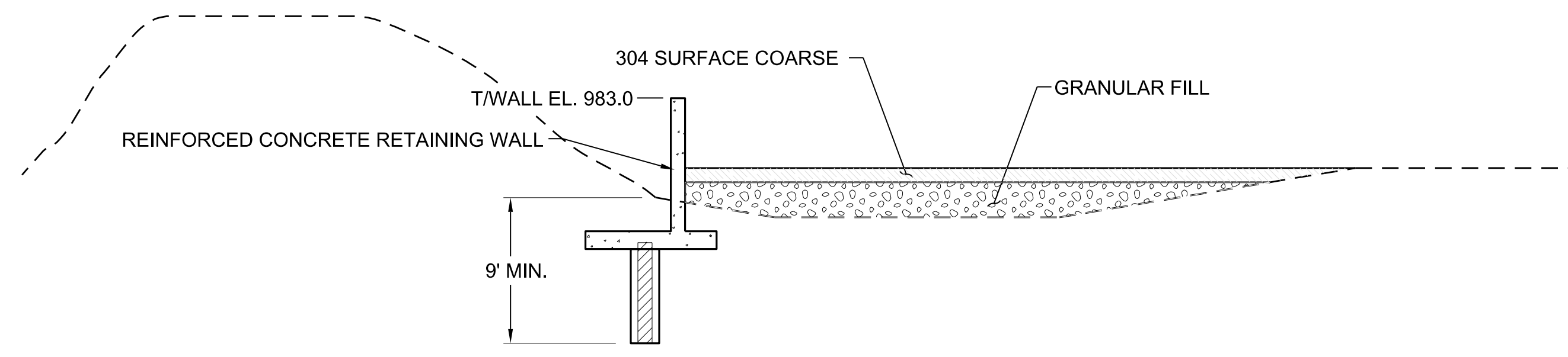
B2 SECTION THROUGH EXISTING EMERGENCY SPILLWAY RAMP

SCALE: 1" = 10'



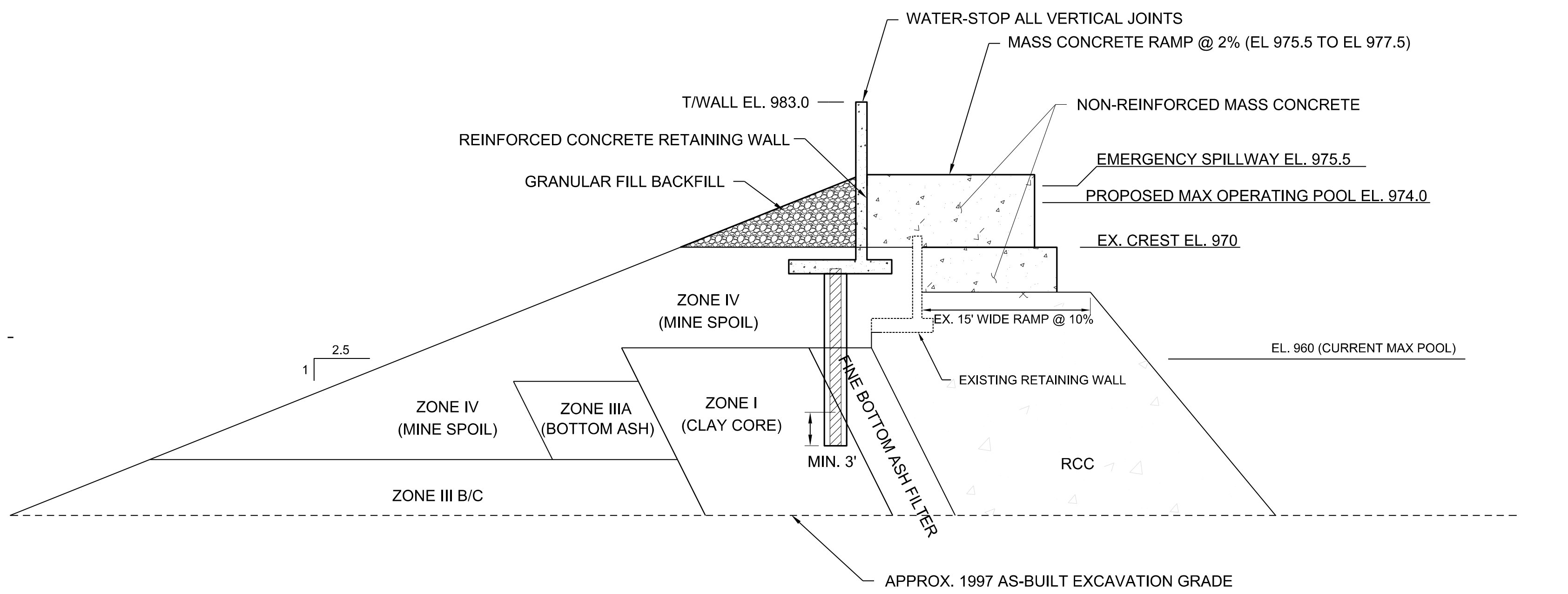
TYPICAL MSE WALL SECTION

SCALE: 1" = 5'



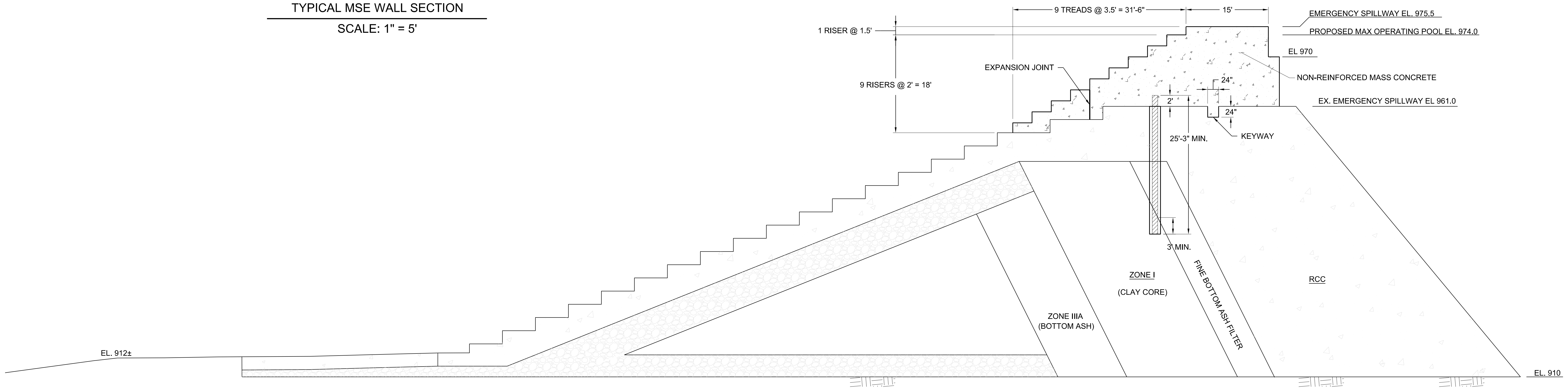
E SECTION E-E

SCALE: 1" = 10'



D SECTION THROUGH EXISTING EMERGENCY SPILLWAY RAMP

SCALE: 1" = 10'



C SECTION THROUGH EXISTING EMERGENCY SPILLWAY

SCALE: 1" = 10'

S&ME, INC.
6190 ENTERPRISE COURT
DUBLIN, OH 43016
PHONE: 614-793-2226
FAX: 614-793-2410
www.smeinc.com

Michael G. Rowland

MICHAEL GILBERT ROWLAND	E-65559 NUMBER	JAN 16, 2012 DATE

E-65559

PROJECT NUMBER:	011-11497-042	DRAWN BY:	EDV
DRAWING DATE:	9/28/12	ENGINEER:	MTR
LAST UPDATED:	1/16/13	APPROVED BY:	MGR
		SCALE:	AS NOTED

S&ME

DATE	NO.	DESCRIPTION	APPD.
1/16/13	C	REVISED PER ODNR COMMENTS REC'D 12/4/2012	MGR
9/12/12	B	REVISED PER ODNR COMMENTS REC'D 7/18/12	MGR
5/23/12	A	ISSUED FOR PERMIT INITIAL SUMMITTAL	MGR

REVISIONS

CARDINAL OPERATING COMPANY

CARDINAL PLANT

BRILLIANT OHIO

DAM RAISING
FLY ASH RETENTION DAM II
DAM RAISING
TYPICAL SECTIONS

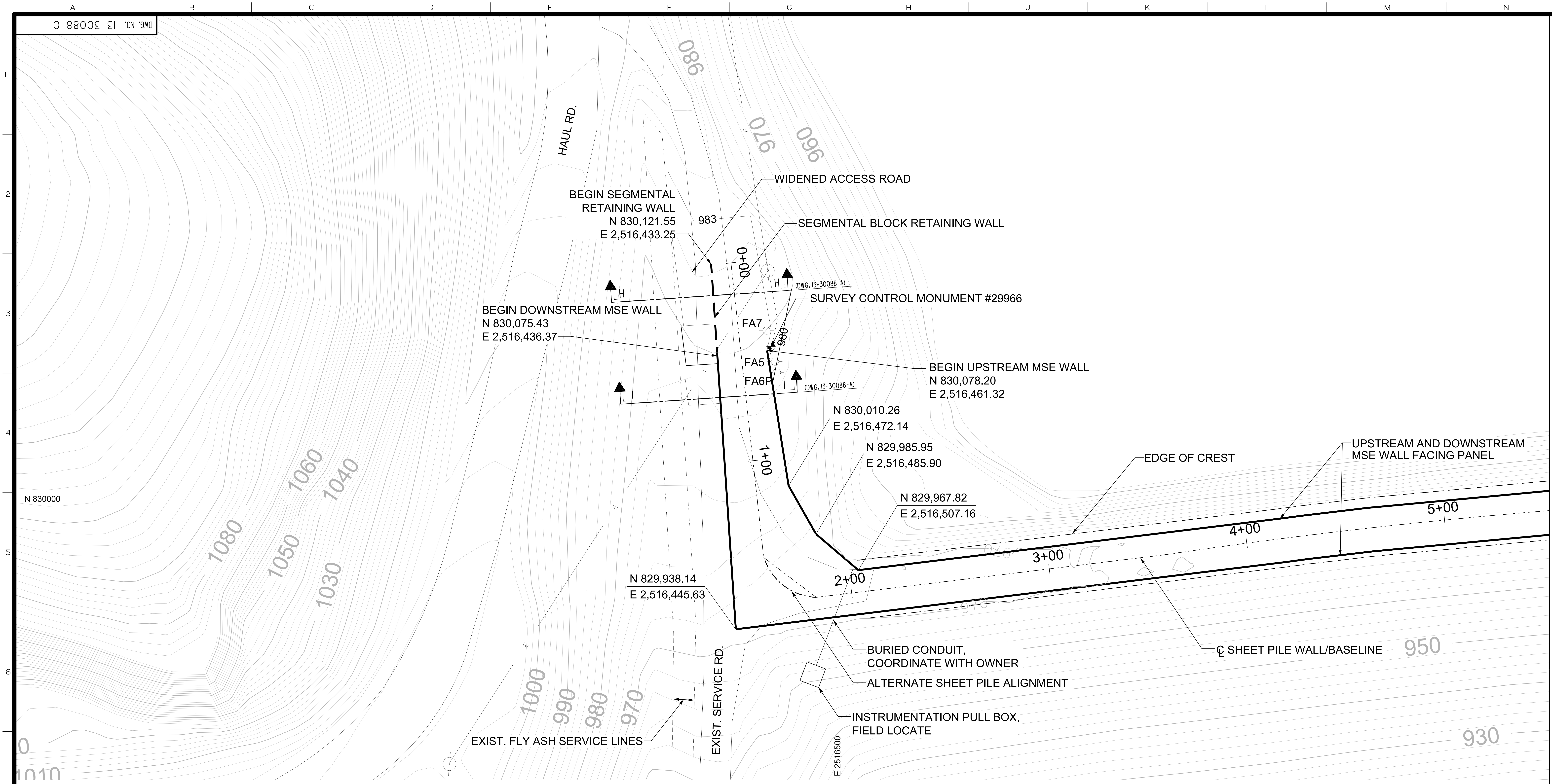
DWG. NO. 13-30087-C

ARCH — ELEC — MECH — STR

SCALE: AS SHOWN
DR:
CH:
SQD SUPV:
DATE:
APPROVED BY
CIVIL ENGINEERING DIVISION

AEP AMERICAN ELECTRIC POWER
1 RIVERSIDE PLAZA
COLUMBUS, OH 43215

DWG. NO. 13-30088-C

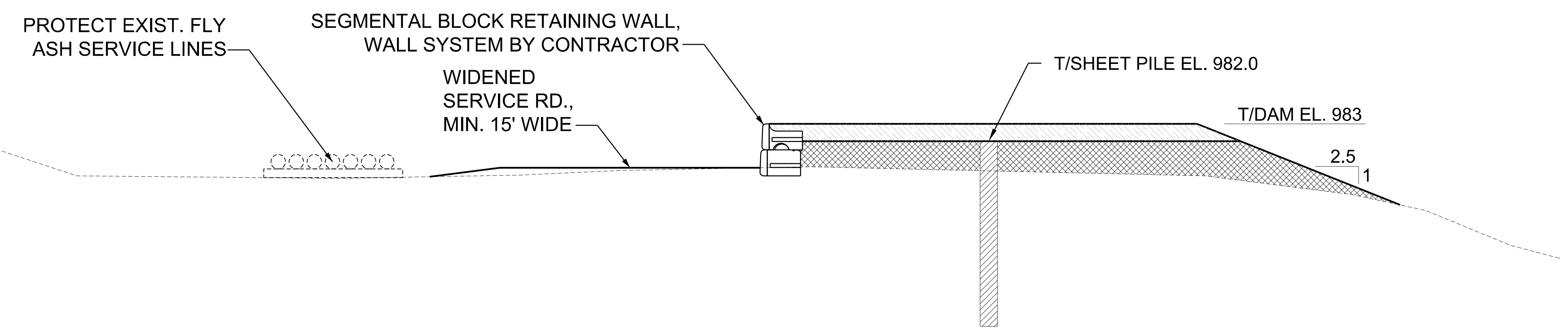


LEGEND

900	EXISTING CONTOUR 10FT INTERVAL
950	PROPOSED GRADE CONTOUR 10FT INTERVAL
	EXISTING ACCESS ROADS
	EXISTING FLY ASH SERVICE LINES
	BENCHMARK
	EXISTING TREELINE
E	SECTION NUMBER AND LOCATION
	UTILITY POLE
E	OVERHEAD ELECTRIC LINE
FA7	EXISTING MONITORING WELL

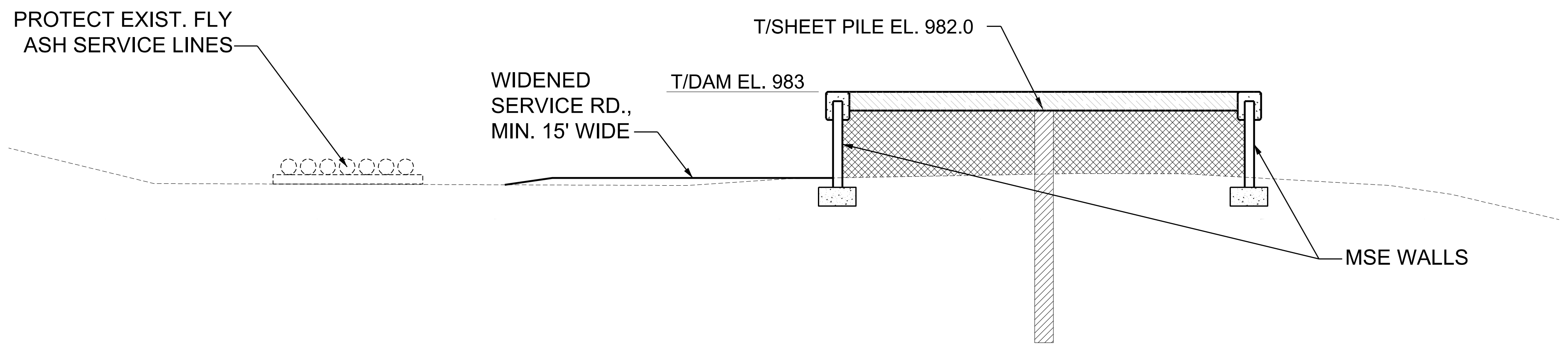
GENERAL NOTES

1. TOPOGRAPHIC MAPPING DEVELOPED FROM AERIAL SURVEY PERFORMED ON 3/5/2009 AND 1986 AS-BUILT DRAWINGS.



SECTION H-H

SCALE: N.T.S.



SECTION I-I

SCALE: N.T.S.

S&ME, INC.
6190 ENTERPRISE COURT
DUBLIN, OH 43016
PHONE: 614-793-2226
FAX: 614-793-2410
www.smeinc.com

MICHAEL GILBERT ROWLAND
E-65559

Michael A. R	E-65559	JAN 16, 2012
MICHAEL GILBERT ROWLAND	NUMBER	DATE

0 20 40
SCALE IN FEET

PROJECT NUMBER:	011-11497-042	DRAWN BY:	EDV
DRAWING DATE:	9/28/12	ENGINEER:	MTR
LAST UPDATED:	1/16/13	APPROVED BY:	MGR
		SCALE:	AS NOTED

S&ME

DATE	NO.	DESCRIPTION	APPD.
1/16/13	C	REVISED PER ODNR COMMENTS REC'D 12/4/2012	MGR
9/12/12	B	REVISED PER ODNR COMMENTS REC'D 7/18/12	MGR
5/23/12	A	ISSUED FOR PERMIT INITIAL SUMMITTAL	MGR

REVISIONS

CARDINAL OPERATING COMPANY

CARDINAL PLANT

BRILLIANT OHIO

DAM RAISING
FLY ASH RETENTION DAM II

RIGHT ABUTMENT PLAN

DWG. NO. 13-30088-C

ARCH	—	ELEC	—	MECH	—	STR	GS

SCALE: AS SHOWN
DR:
CH:
SQD SUPV:
DATE:
APPROVED BY
CIVIL ENGINEERING DIVISION

AEP AMERICAN ELECTRIC POWER

1 RIVERSIDE PLAZA
COLUMBUS, OH 43215

SYSTEM DATE- DD-MMM-YYYY
SYSTEM TIME- HOUR:MINUTE
15Φ≡ FLR.CED

IGSI092

DWG. NO.· 13-30089-C

S&ME, INC.
6190 ENTERPRISE COURT
DUBLIN, OH 43016
PHONE: 614-793-2226
FAX: 614-793-2410

STATE OF OHIO
MICHAEL GILBERT ROWLAND
E-65559
REGISTERED PROFESSIONAL ENGINEER

Michael G. Rowland

MICHAEL G. ROWLAND	E-45559 NUMBER	JAN 16, 2012 DATE

PROJECT NUMBER:	011-11497-042	DRAWN BY:	EDV
DRAWING DATE:	12-20-2012	ENGINEER:	MTR
LAST UPDATED:	1-16-2013	APPROVED BY:	MGR
		SCALE:	GRAPHIC 1:1

S&ME

DATE	NO.	DESCRIPTION	APPD.
1/16/13	C	REVISED PER ODNR COMMENTS REC'D 12/4/12	MGR
9/12/12	B	REVISED PER ODNR COMMENTS REC'D 7/18/12	MGR
5/23/12	A	ISSUED FOR PERMIT – INITIAL SUBMITTAL	MGR

REVISIONS

CARDINAL OPERATING COMPANY

CARDINAL PLANT

BRILLIANT OHIO

DAM RAISING
FLY AS RETENTION DAM II

EMERGENCY SPILLWAY PLAN

DWG. NO.· 13-30089-C

SCALE: •
DR: •
CH: •
•
•
DATE: •
APPROVED BY

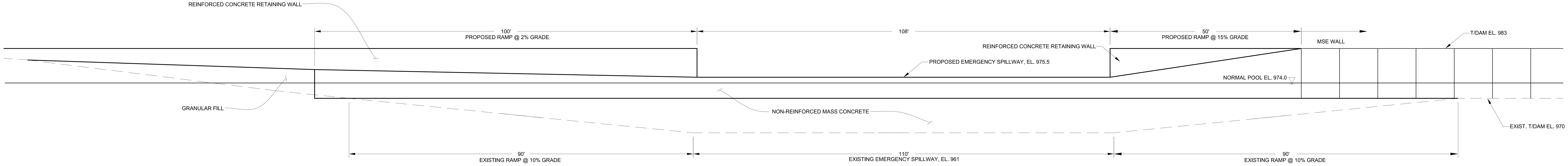
AEP AMERICAN ELECTRIC POWER

AEP SERVICE CORP.
1 RIVERSIDE PLAZA
COLUMBUS, OH 43215

IGS0700

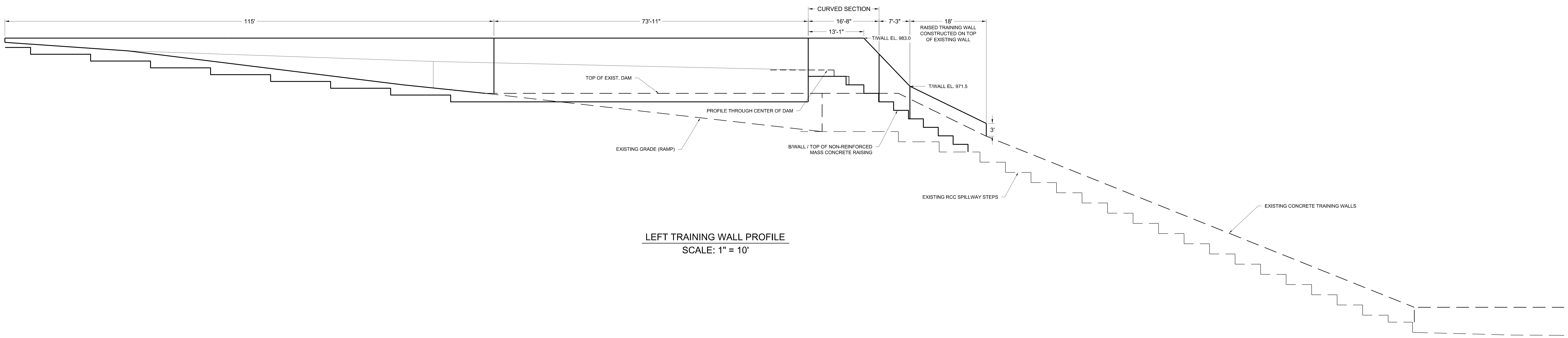
O Rev. No:
Plot Date:
Plotted by:

DWG. NO. 13-30090-C



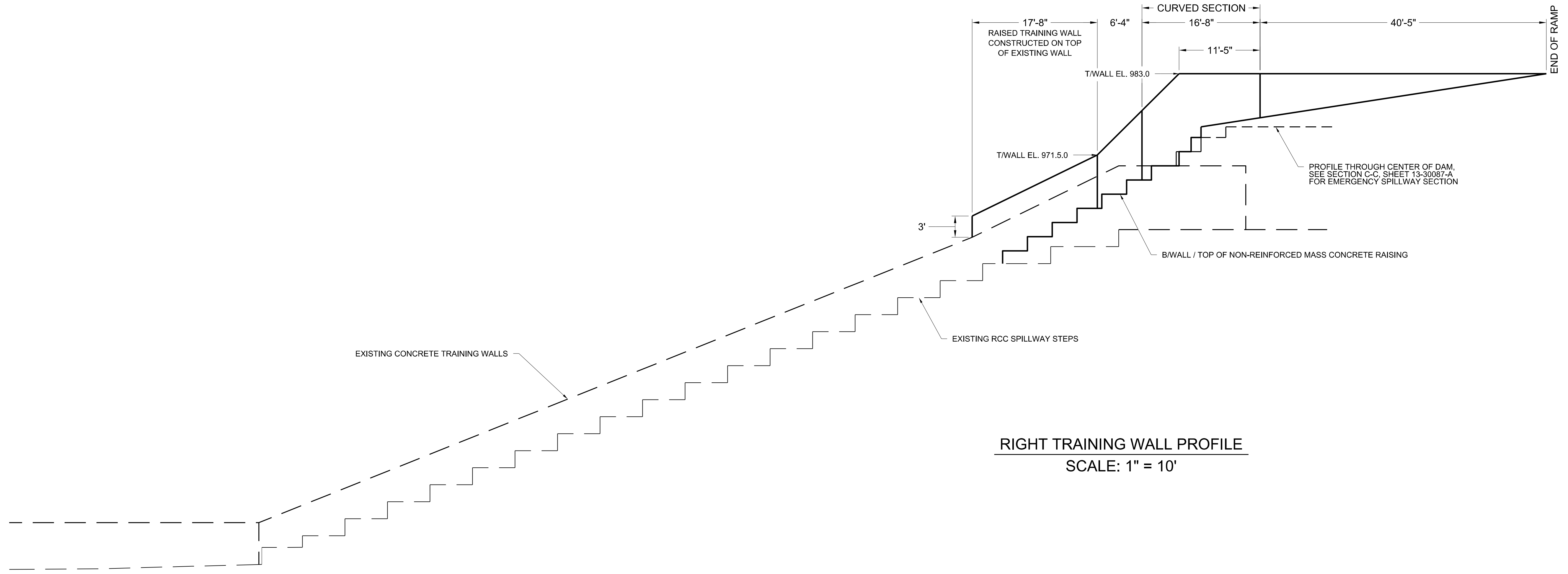
EMERGENCY SPILLWAY NORTH ELEVATION

SCALE: 1" = 10'



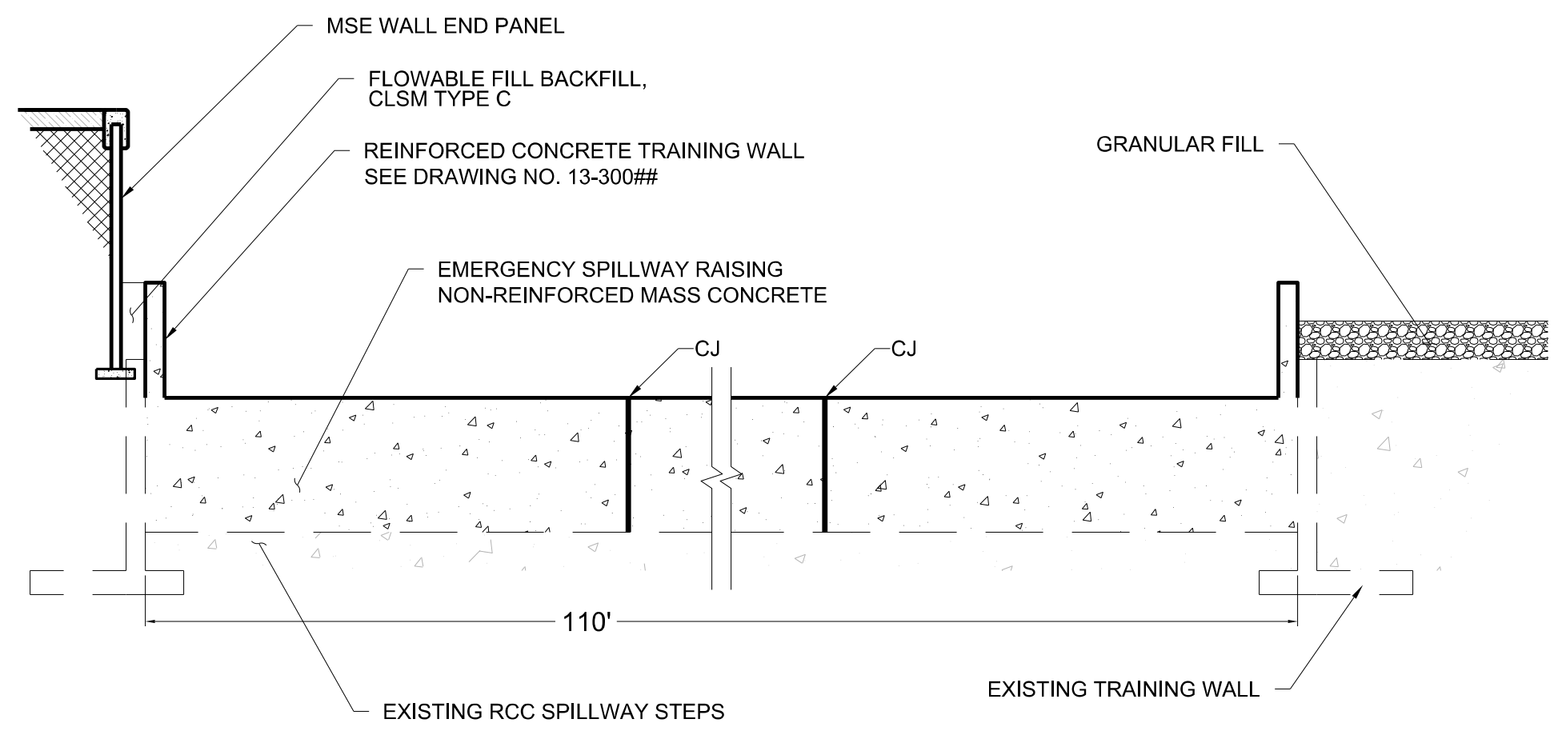
LEFT TRAINING WALL PROFILE

SCALE: 1" = 10'



RIGHT TRAINING WALL PROFILE

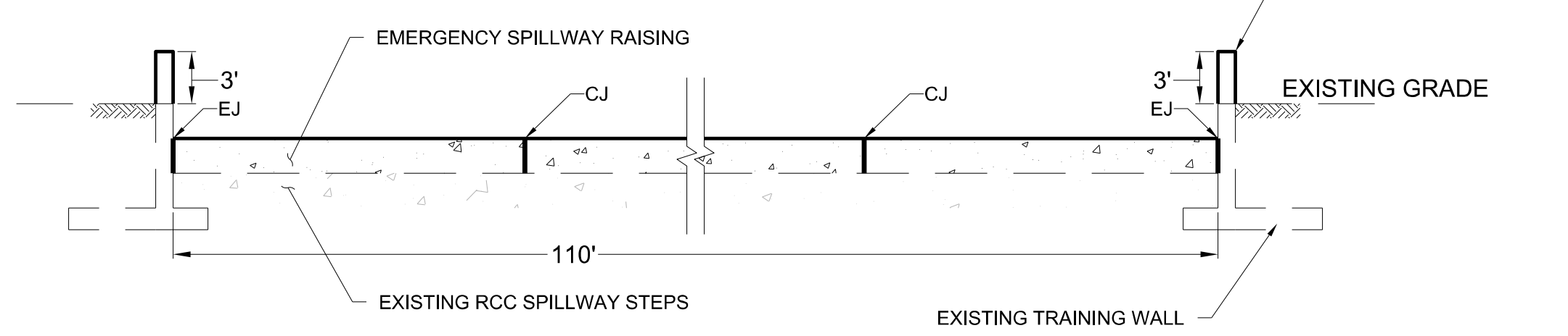
SCALE: 1" = 10'



F

SECTION F-F

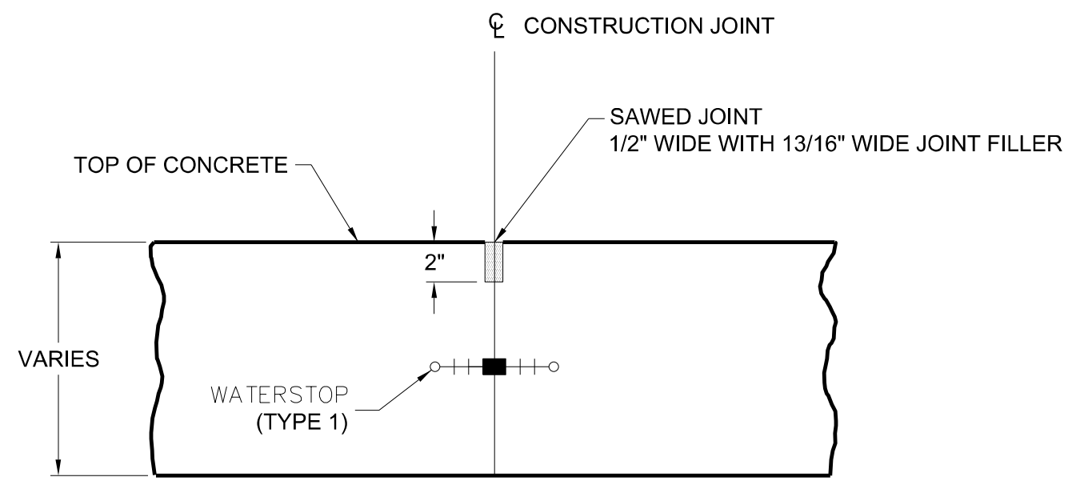
SCALE: N.T.S.



G

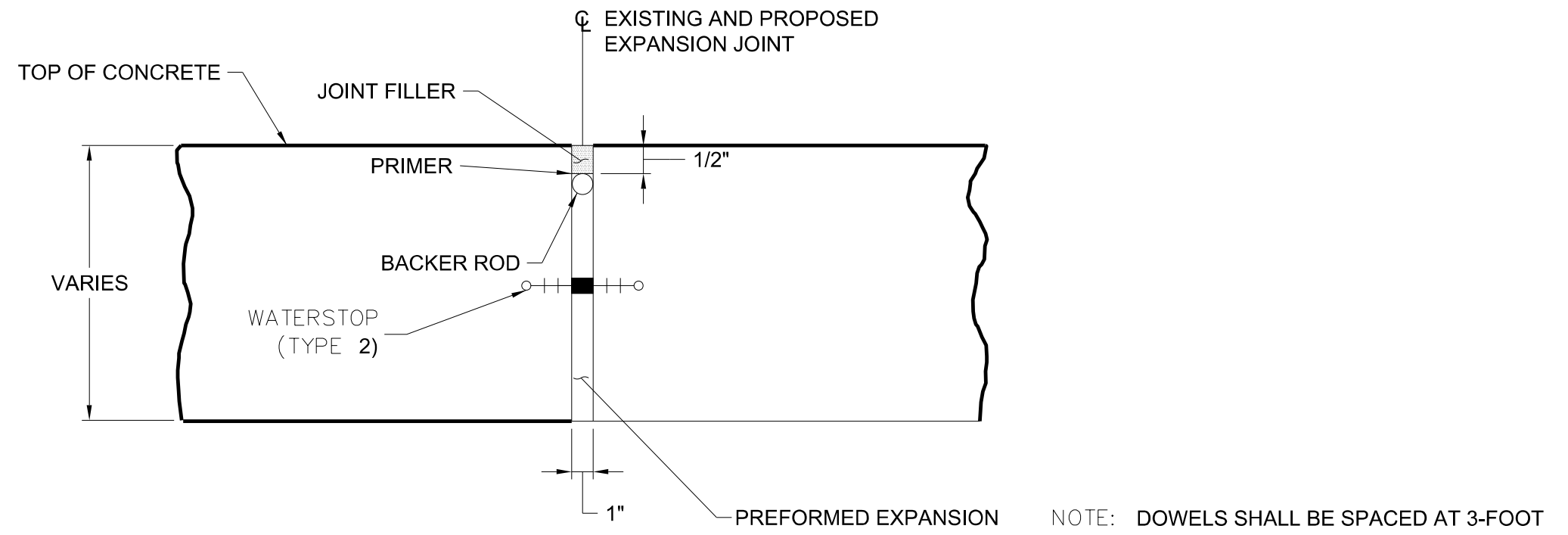
SECTION G-G

SCALE: N.T.S.



CONSTRUCTION JOINT DETAIL

SCALE: N.T.S.



NOTE: DOWELS SHALL BE SPACED AT 3-FOOT C/C ALONG ENTIRE WIDTH OF THE DAM

EXPANSION JOINT DETAIL

SCALE: N.T.S.

S&ME, INC.
6190 ENTERPRISE COURT
DUBLIN, OH 43016
PHONE: 614-793-2226
FAX: 614-793-2410
www.smeinc.com

MICHAEL GILBERT ROWLAND
E-65559

Michael G. Rowland

MICHAEL GILBERT ROWLAND	E-65559	JAN 16, 2012
	NUMBER	DATE

PROJECT NUMBER:	011-11497-042	DRAWN BY:	EDV
DRAWING DATE:	9/28/12	ENGINEER:	AJS
LAST UPDATED:	1/16/13	APPROVED BY:	MGR
		SCALE:	AS NOTED

S&ME

DATE	NO.	DESCRIPTION	APPD.
1/16 /13	C	REVISED PER ODNR COMMENTS REC'D 12/4/2012	MGR
9/12 /12	B	REVISED PER ODNR COMMENTS REC'D 7/18/12	MGR
5/23 /12	A	ISSUED FOR PERMIT INITIAL SUMMITTAL	MGR

REVISIONS

CARDINAL OPERATING COMPANY

CARDINAL PLANT

BRILLIANT OHIO

DAM RAISING

FLY ASH RETENTION DAM II

EMERGENCY SPILLWAY PROFILE AND SECTIONS

DWG. NO. 13-30090-C

ARCH	—	ELEC	—	MECH	—	STR

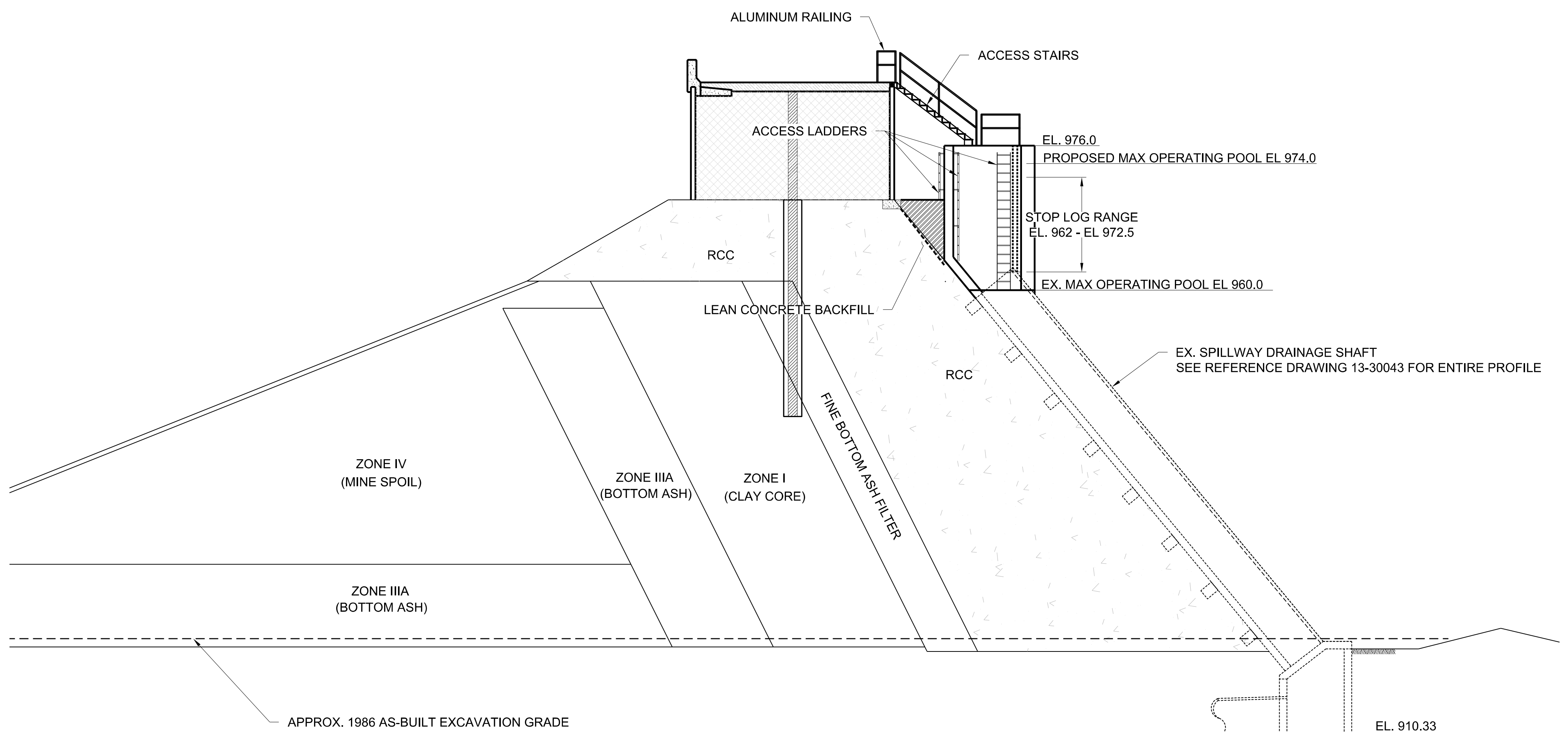
SCALE: AS SHOWN
DR:
CH:
SQD SUPV:
DATE:
APPROVED BY
CIVIL ENGINEERING DIVISION

AEP AMERICAN ELECTRIC POWER

1 RIVERSIDE PLAZA
COLUMBUS, OH 43215

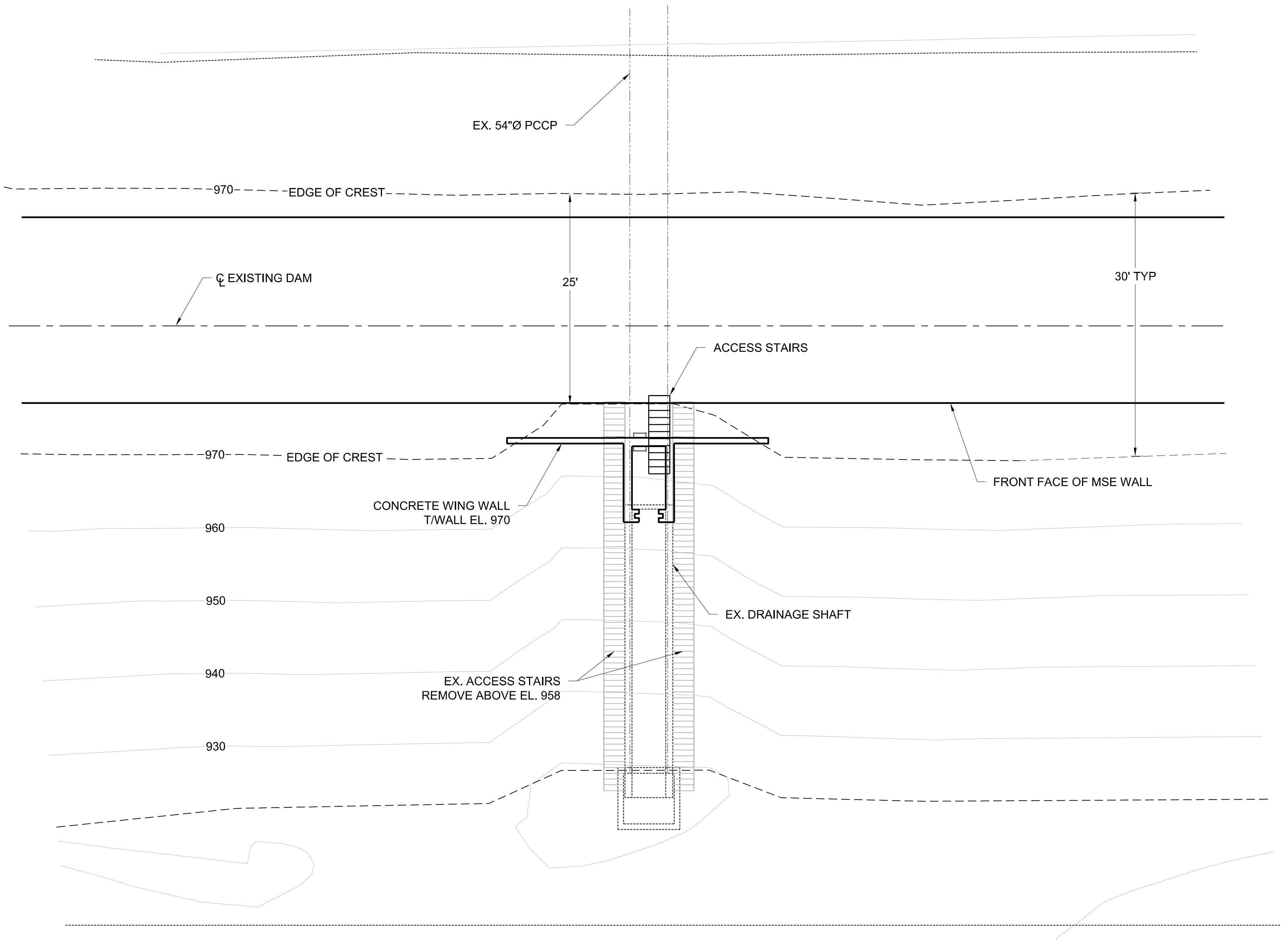
IGS1092

SYSTEM DATE- DD-MMM-YYYY
SYSTEM TIME- HOUR:MINUTE
I5Φ≡ FLR.CED



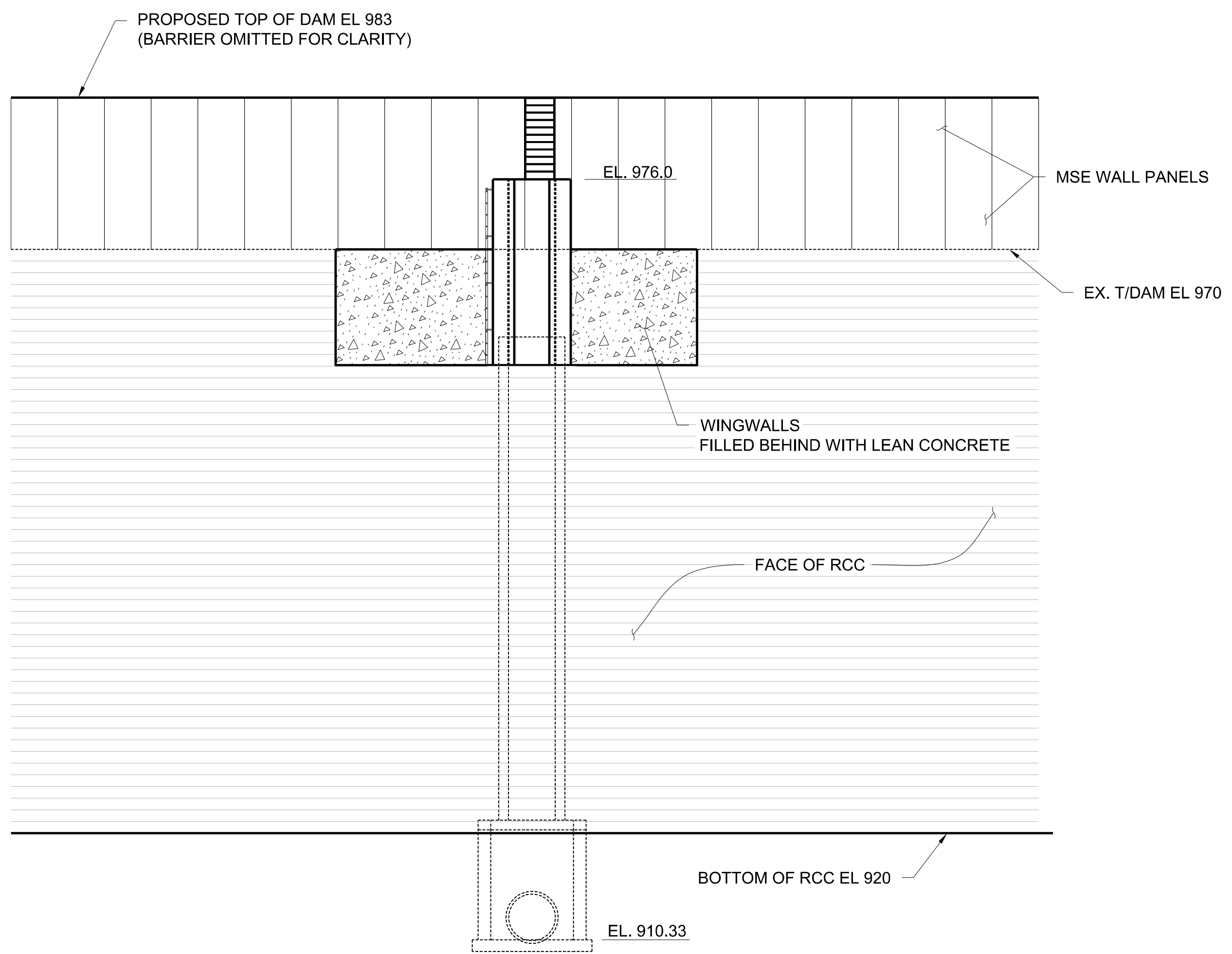
NOTE: SEE REFERENCE DRAWINGS 13-300-40 & 13-30043-5 FOR EXISTING PLAN AND PROFILE VIEWS

SERVICE SPILLWAY, PROFILE
SCALE: 1" = 10'



NOTE: EXISTING AND PROPOSED DRAINAGE SHAFT SKIMMER NOT SHOWN

SERVICE SPILLWAY, PLAN
SCALE: 1" = 10'



SERVICE SPILLWAY, ELEVATION
SCALE: 1" = 10'

DATE	NO.	DESCRIPTION	APPD.
1/16/13	C	REVISED PER ODNR COMMENTS REC'D 12/4/2012	MGR
9/12/12	B	REVISED PER ODNR COMMENTS REC'D 7/18/12	MGR
5/23/12	A	ISSUED FOR PERMIT INITIAL SUMMITTAL	MGR

REVISIONS

CARDINAL OPERATING COMPANY

CARDINAL PLANT

BRILLIANT OHIO

DAM RAISING
FLY ASH RETENTION DAM II
SERVICE SPILLWAY
PLAN/PROFILE/ELEVATION

DWG. NO. 13-30091-C

ARCH — ELEC — MECH — STR

SCALE: AS SHOWN
DR:
CH:
SQD SUPV:
DATE:
APPROVED BY
CIVIL ENGINEERING DIVISION

AMERICAN ELECTRIC POWER
1 RIVERSIDE PLAZA
COLUMBUS, OH 43215

S&ME, INC.
6190 ENTERPRISE COURT
DUBLIN, OH 43016
PHONE: 614-793-2226
FAX: 614-793-2410

MICHAEL GILBERT ROWLAND
E-65559

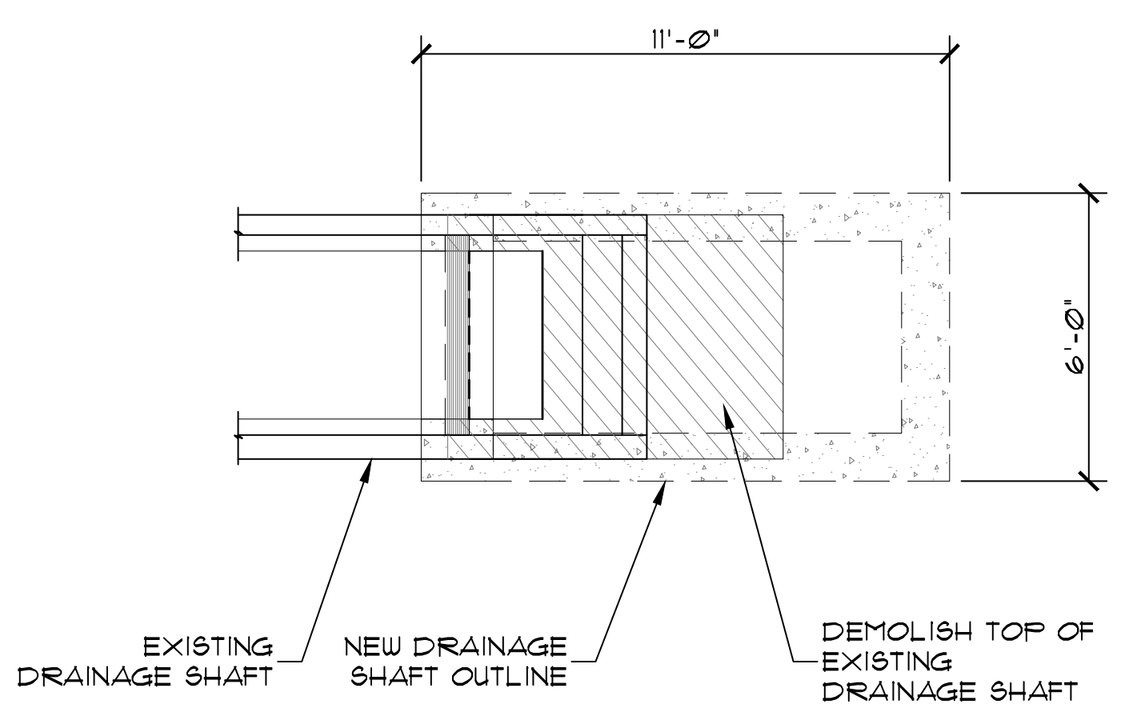
Michael A. R

MICHAEL GILBERT ROWLAND	E-65559	JAN 16, 2012	
	NUMBER	DATE	

PROJECT NUMBER:	011-11497-042	DRAWN BY:	EDV
DRAWING DATE:	9/28/12	ENGINEER:	MTR
LAST UPDATED:	1/16/13	APPROVED BY:	MGR
		SCALE:	AS NOTED

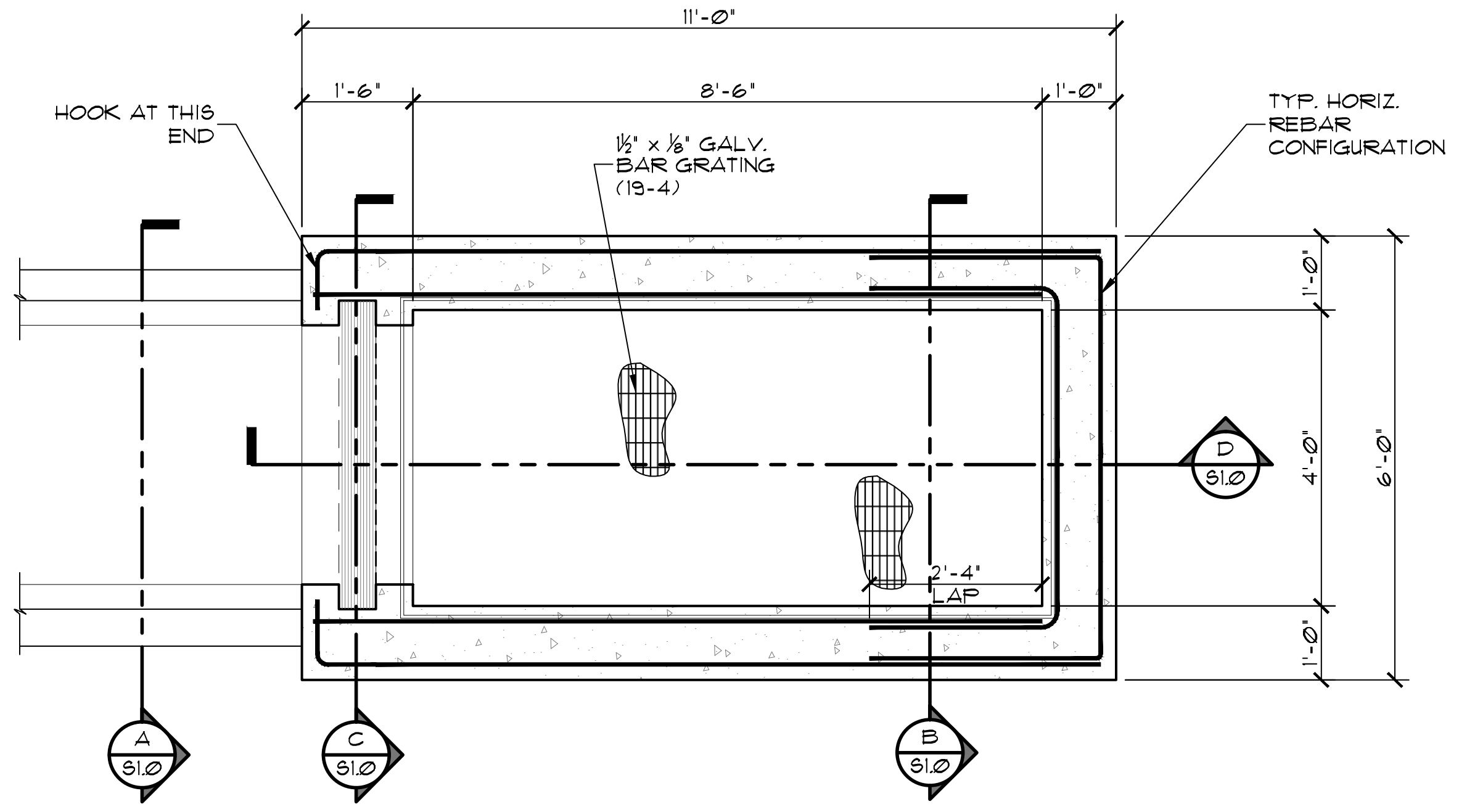
S&ME

DWG. NO. 13-30092-A



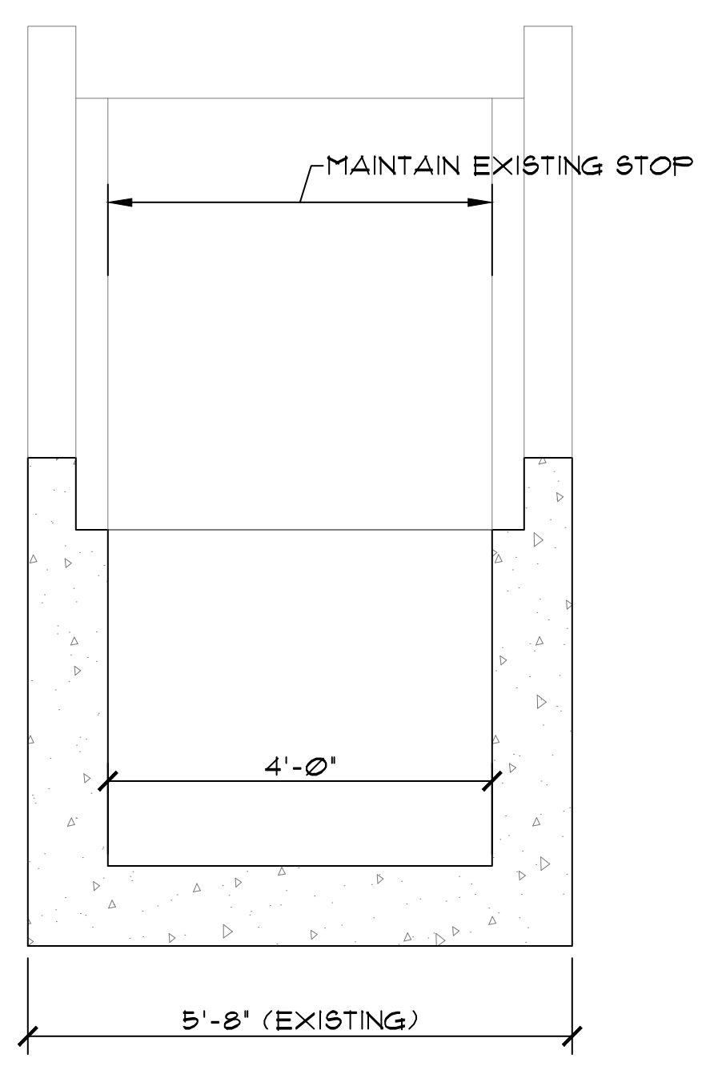
DEMOLITION PLAN - INLET STRUCTURE

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



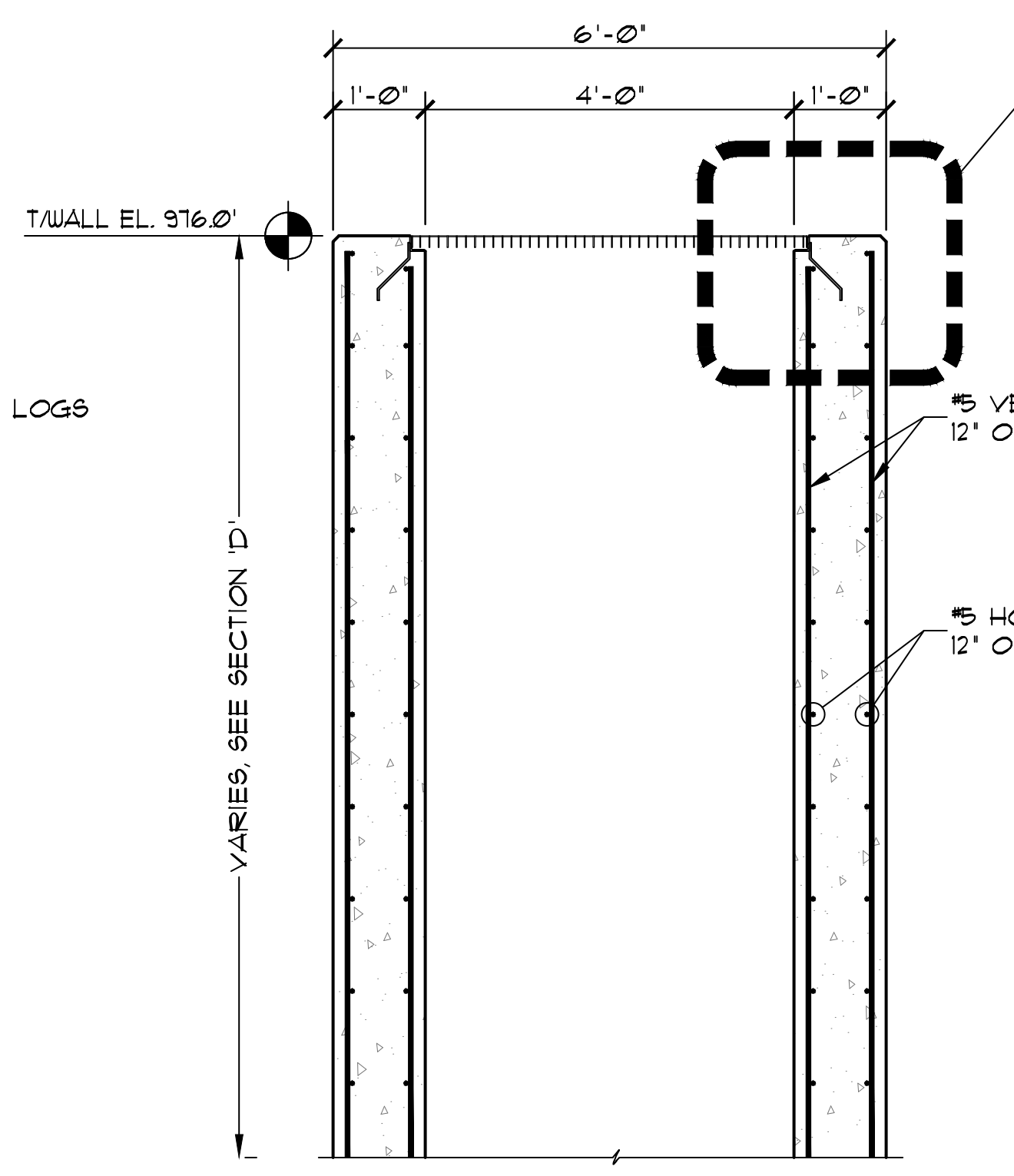
PLAN - INLET STRUCTURE

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



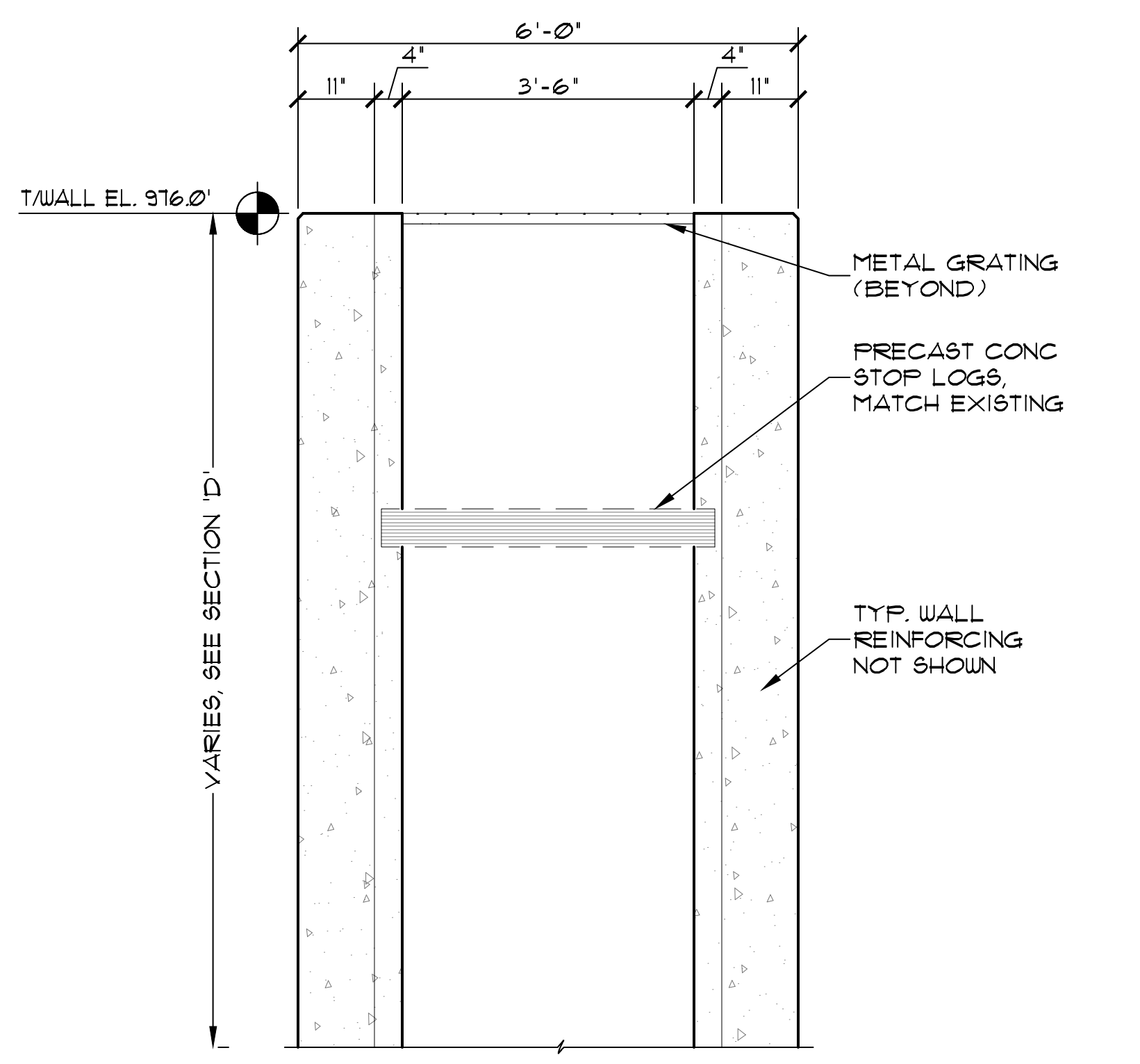
A S1.0 SECTION (existing)

SCALE: 1/2" = 1'-0"



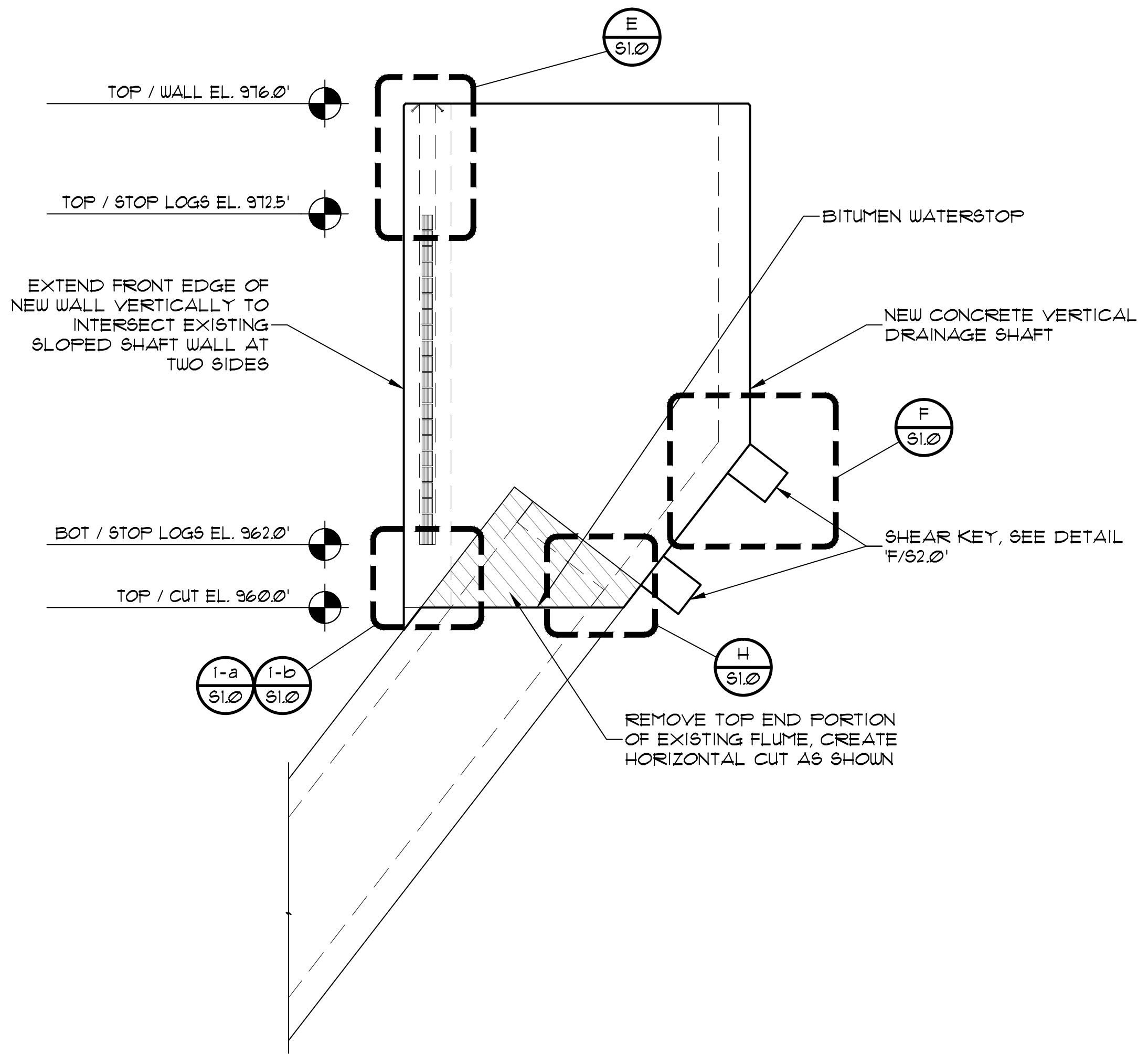
B S1.0 SECTION

SCALE: 1/2" = 1'-0"



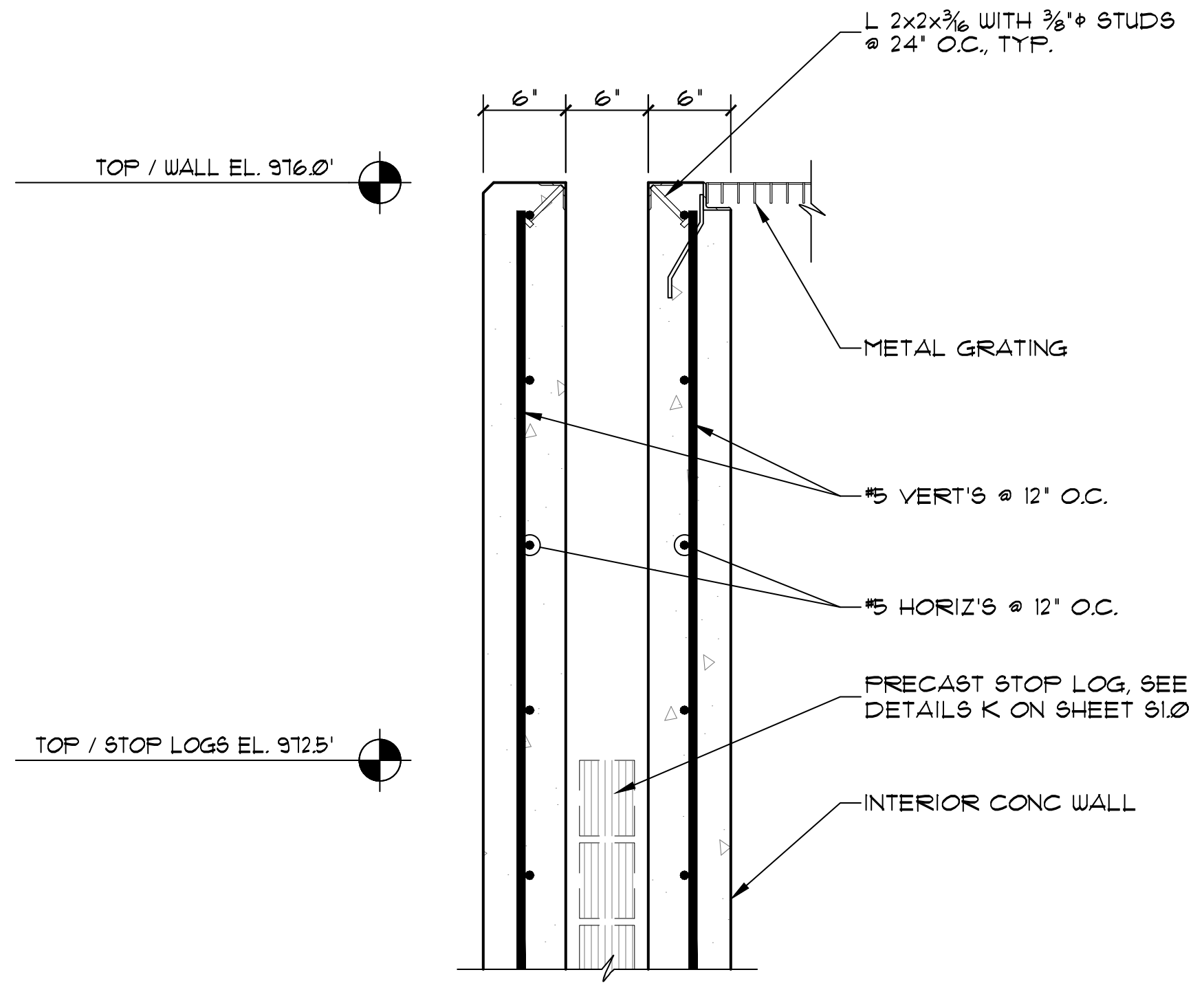
C S1.0 SECTION

SCALE: 1/2" = 1'-0"



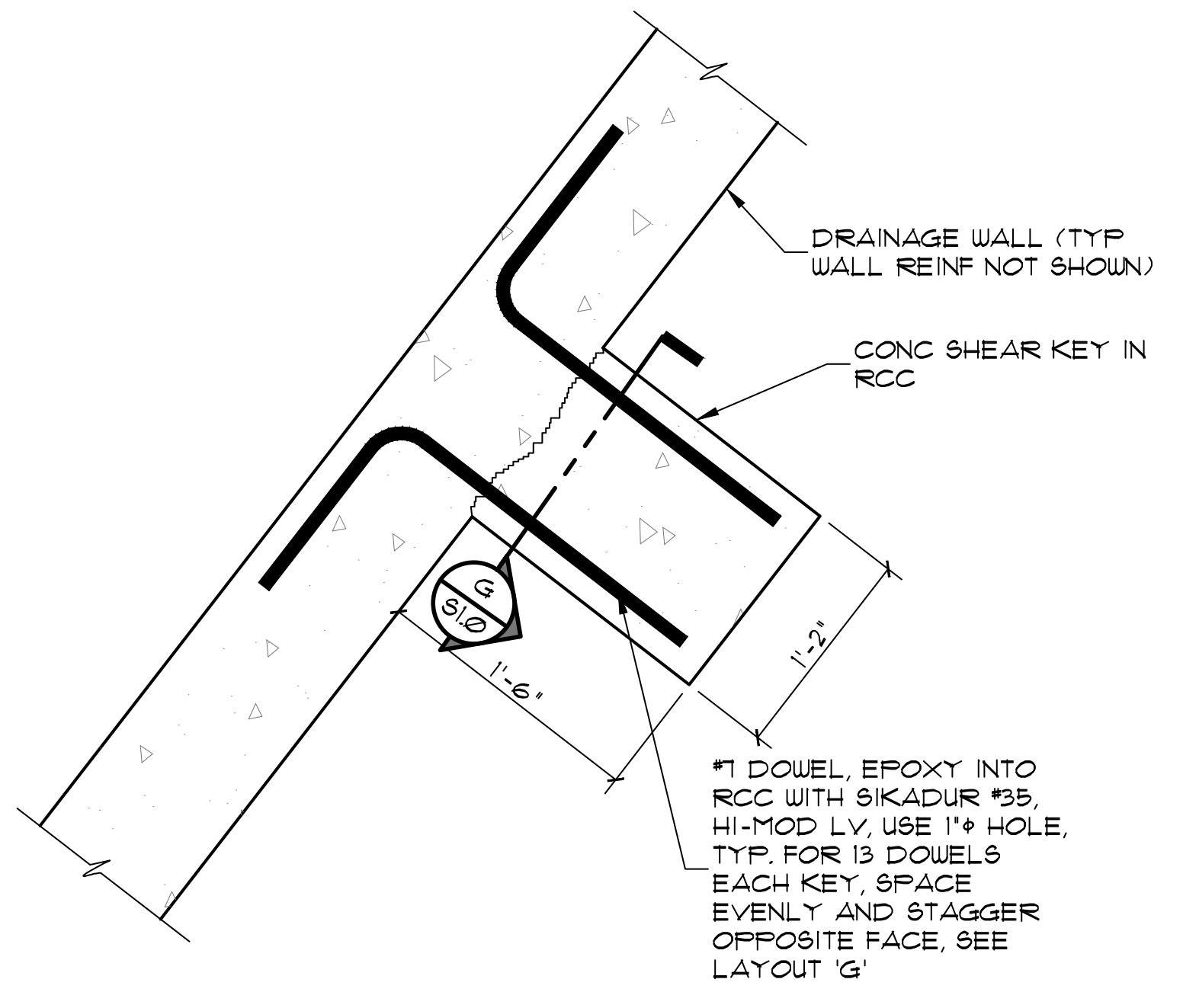
D S1.0 SECTION

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



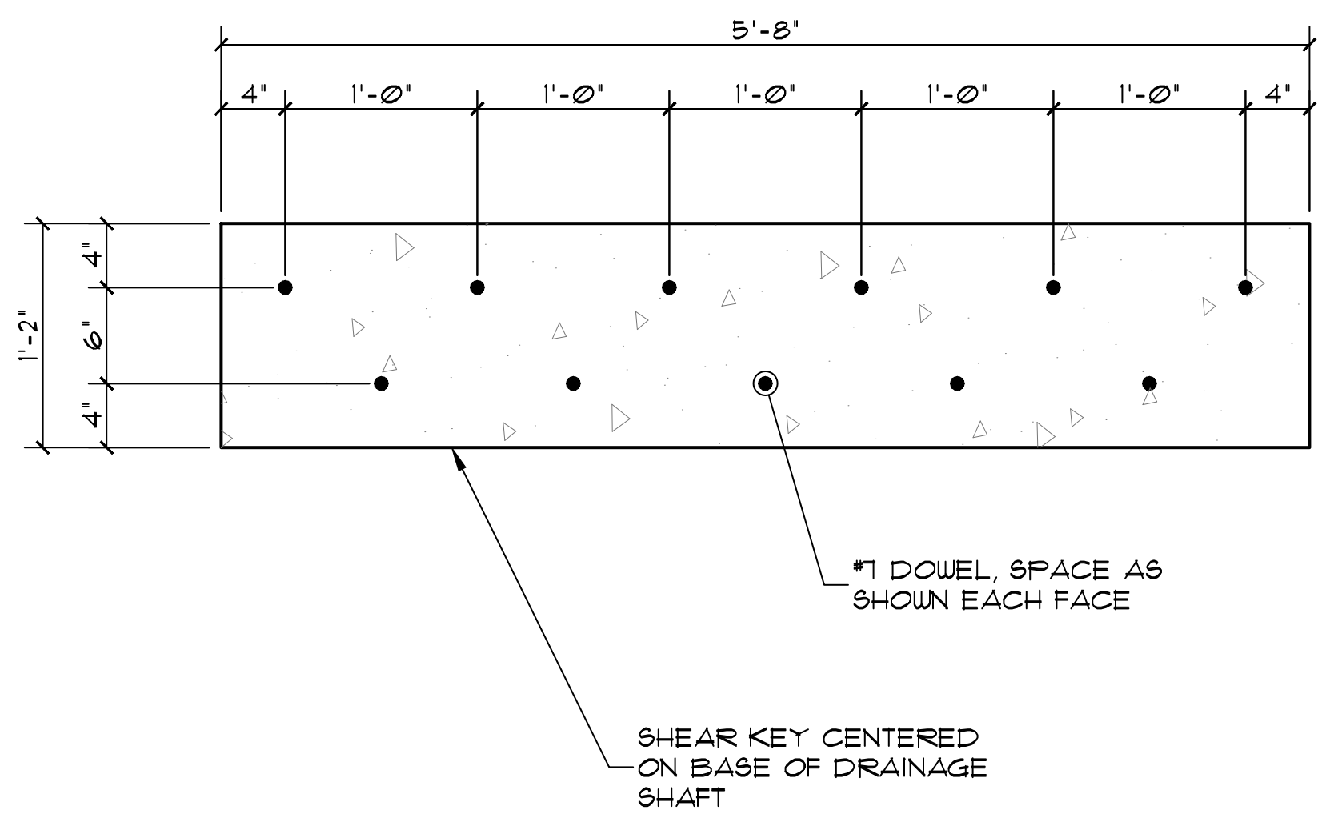
E S1.0 DETAIL @ STOP LOGS

SCALE: 1" = 1'-0"



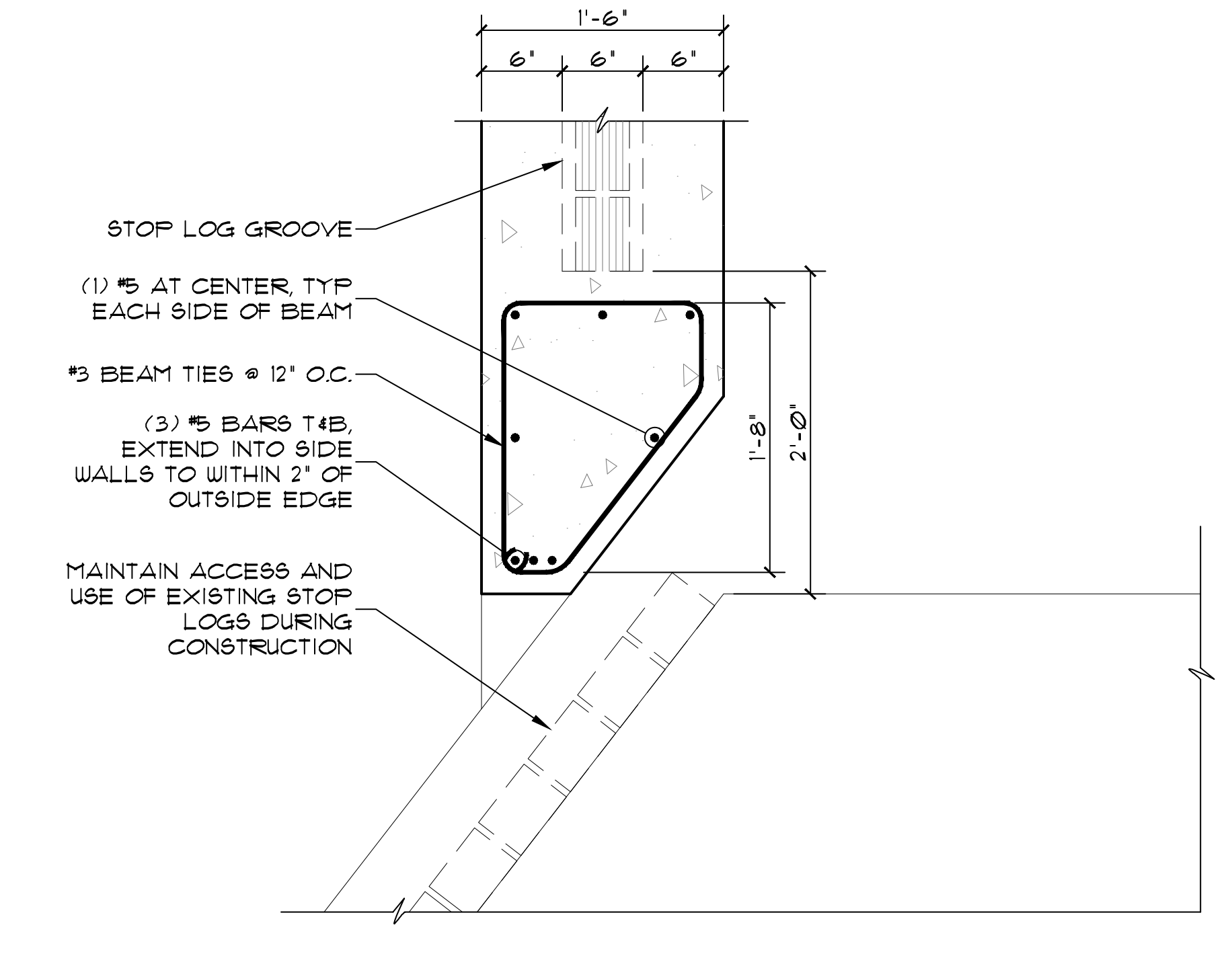
F S1.0 KEY DETAIL

SCALE: 1" = 1'-0"



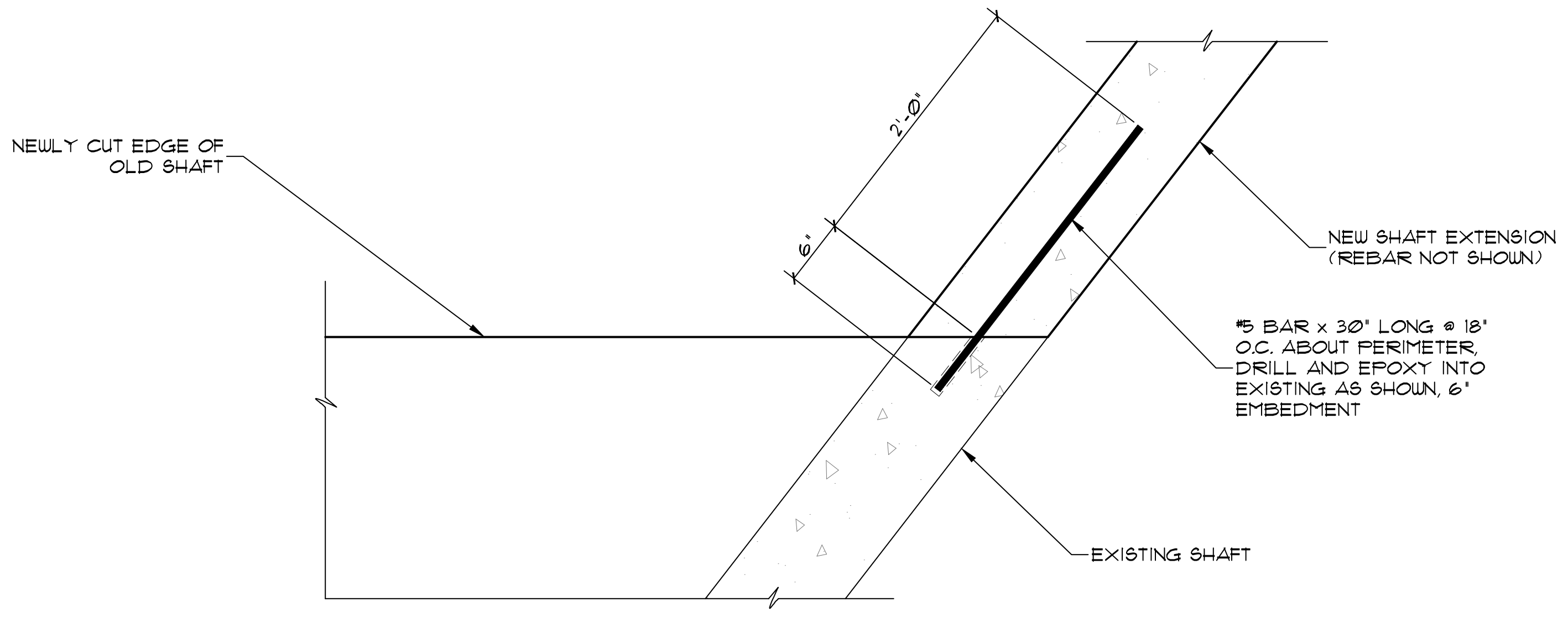
G S1.0 KEY DETAIL

SCALE: 1" = 1'-0"



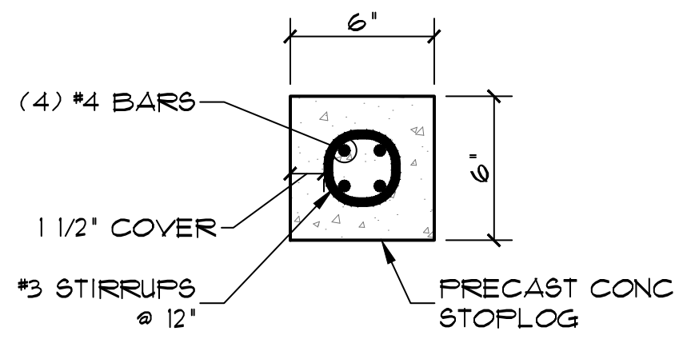
i-a S1.0 DETAIL

SCALE: 1" = 1'-0"



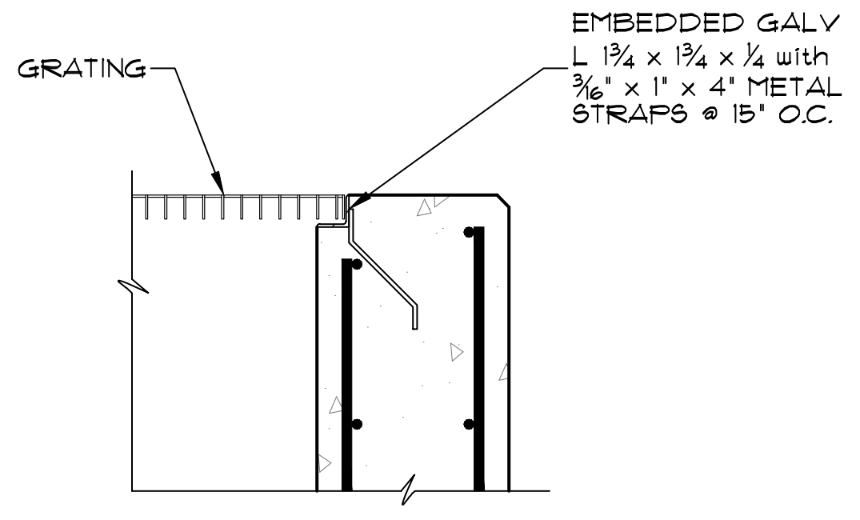
H S1.0 DETAIL

SCALE: 1" = 1'-0"



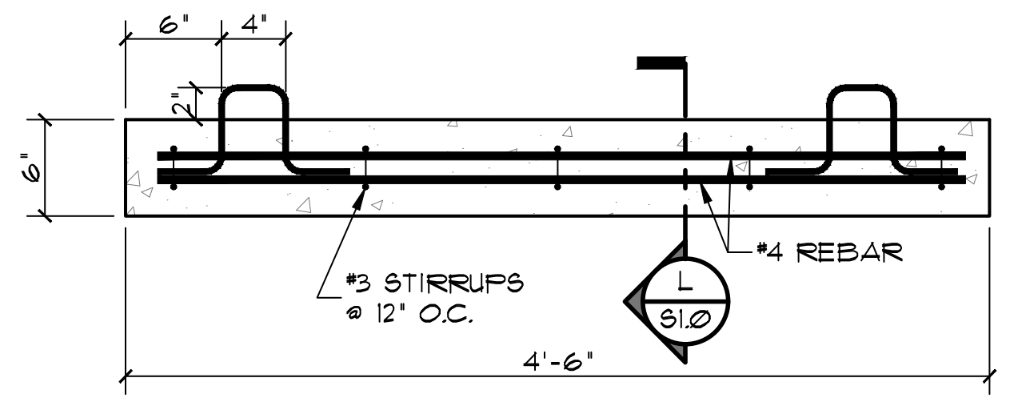
L S1.0 SECTION

SCALE: 1 1/2" = 1'-0"



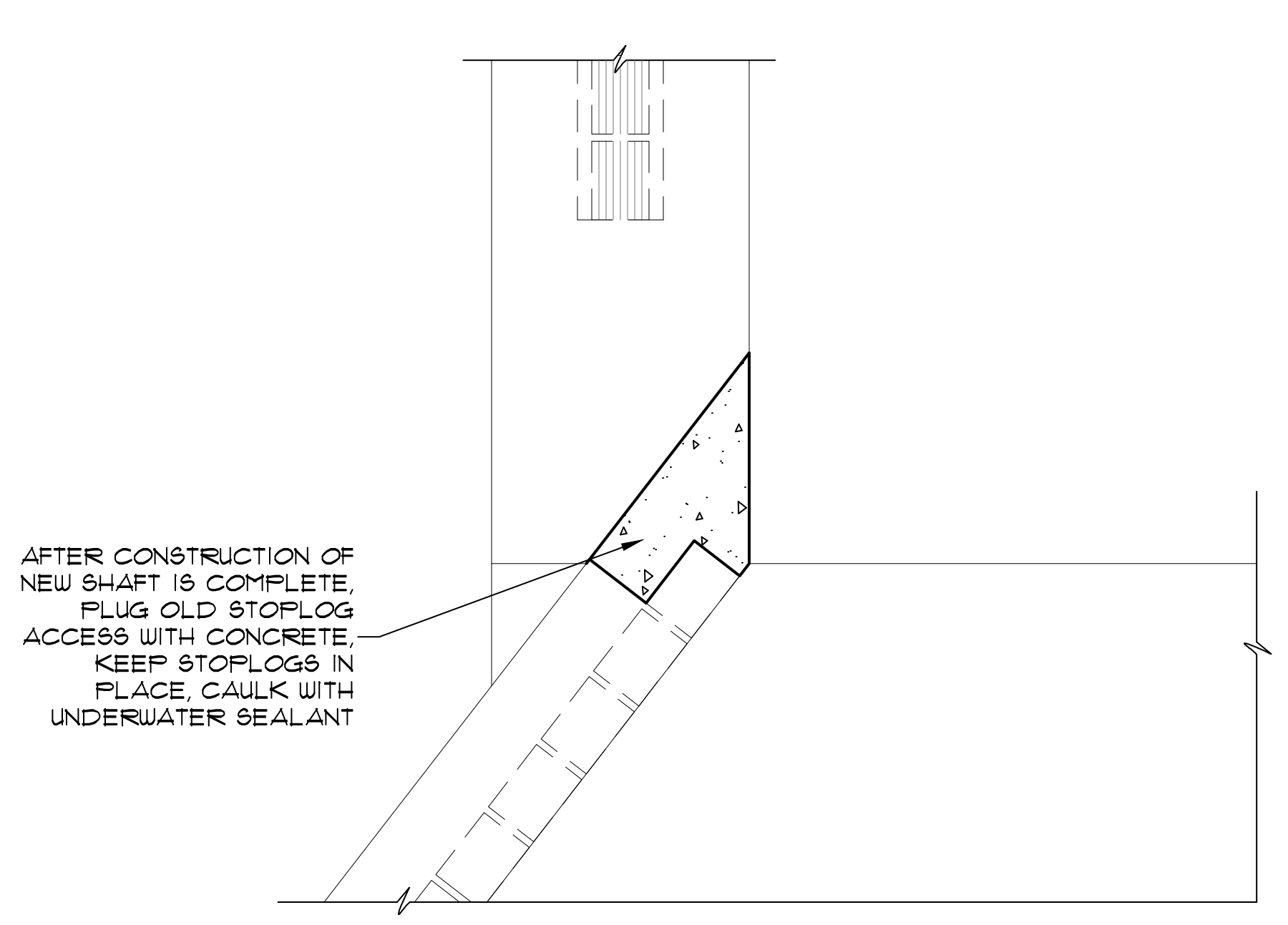
J S1.0 DETAIL

SCALE: 1" = 1'-0"



K S1.0 STOP LOG DETAIL

SCALE: 1" = 1'-0"



i-b S1.0 DETAIL

SCALE: 1" = 1'-0"

GENERAL NOTE:

1. SEE REFERENCE DRAWING 13-32000-4 FOR STRUCTURAL DETAILS OF EXISTING SERVICE SPILLWAY.

9/28/12	A	ISSUED FOR PERMIT	RWM
DATE	NO.	DESCRIPTION	APPD.

REVISIONS

CARDINAL OPERATING COMPANY
CARDINAL PLANT
BRILLIANT OHIO

DAM RAISING
FLY ASH RETENTION DAM II
S1.0 SERVICE SPILLWAY
INLET STRUCTURE DETAILS

DWG. NO. 13-30092-A

ROBERT W. MILLER E-58904

ARCH	—	ELEC	—	MECH	—	STR	

SCALE: AS SHOWN
DR:
CH:
SQD SUPV:
DATE:
APPROVED BY
CIVIL ENGINEERING DIVISION

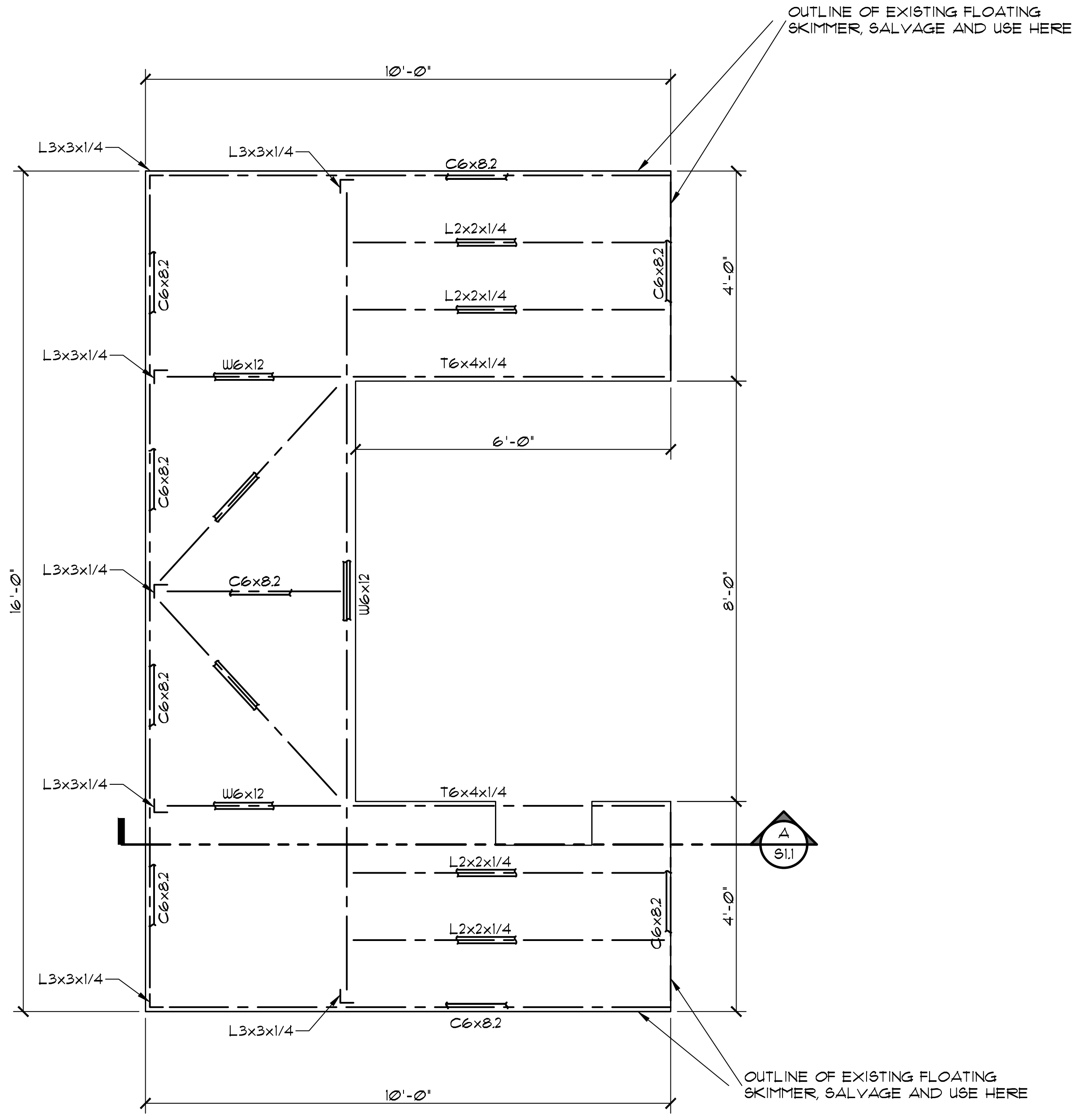
PROJECT NUMBER:	011-11497-042	DRAWN BY:	JCW
DRAWING DATE:	9/28/12	ENGINEER:	RWM
LAST UPDATED:	9/28/12	APPROVED BY:	GLM
		SCALE:	AS NOTED

NEW AVENUE architects . engineers
S&ME
AEP AMERICAN ELECTRIC POWER
1 RIVERSIDE PLAZA
COLUMBUS, OH 43215

IGS1092

SYSTEM DATE– DD–MMM–YYYY
SYSTEM TIME– HOUR:MINUTE
15ø≡ FLR.CED

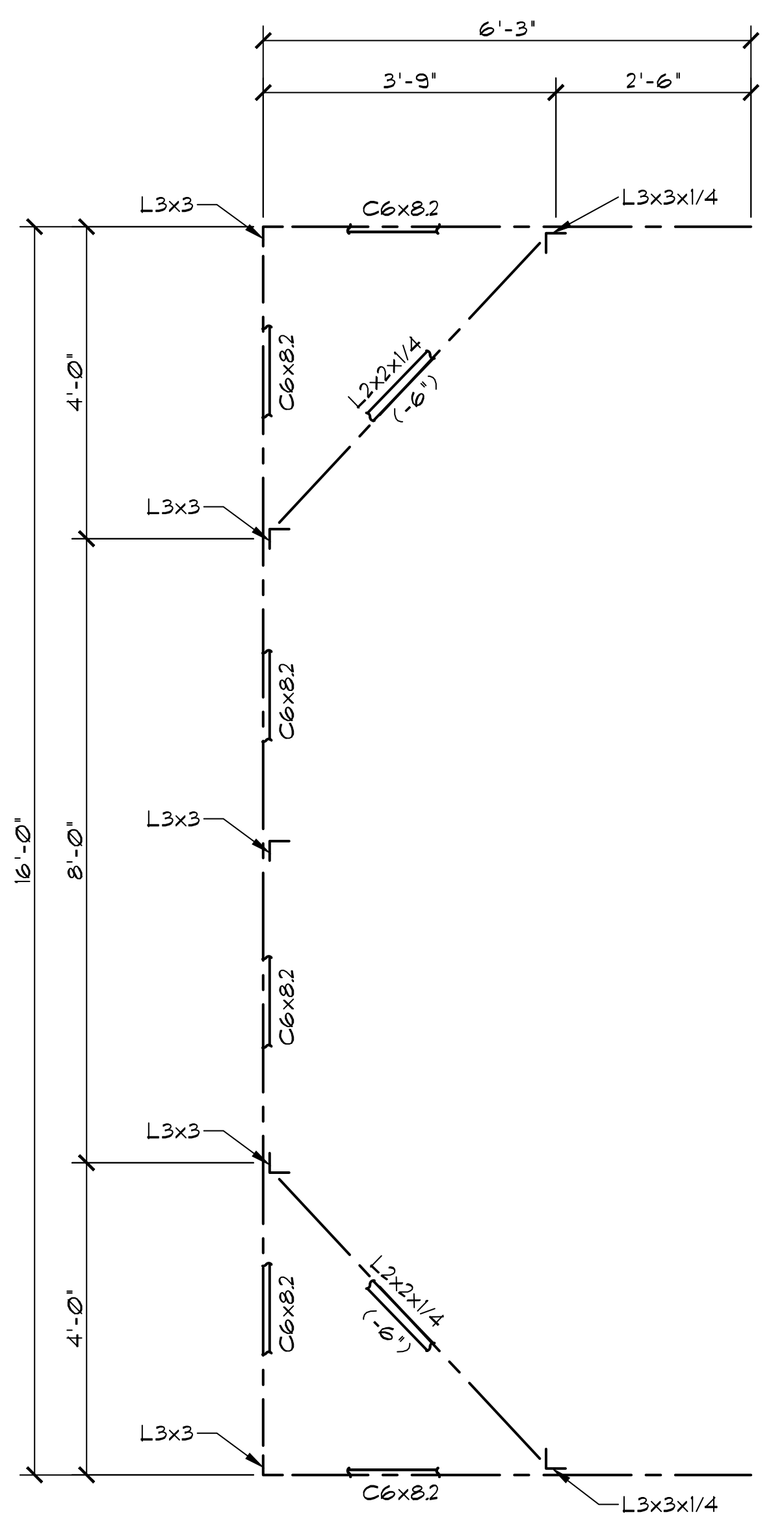
DWG. NO. 13-30093-A



RE-USE EXISTING PLATFORM AT NEW LOCATION, ATTACH PER NEW DETAIL 'X-X'. PONTOONS SHALL BE DISCARDED AND REPLACED WITH NEW ONES.

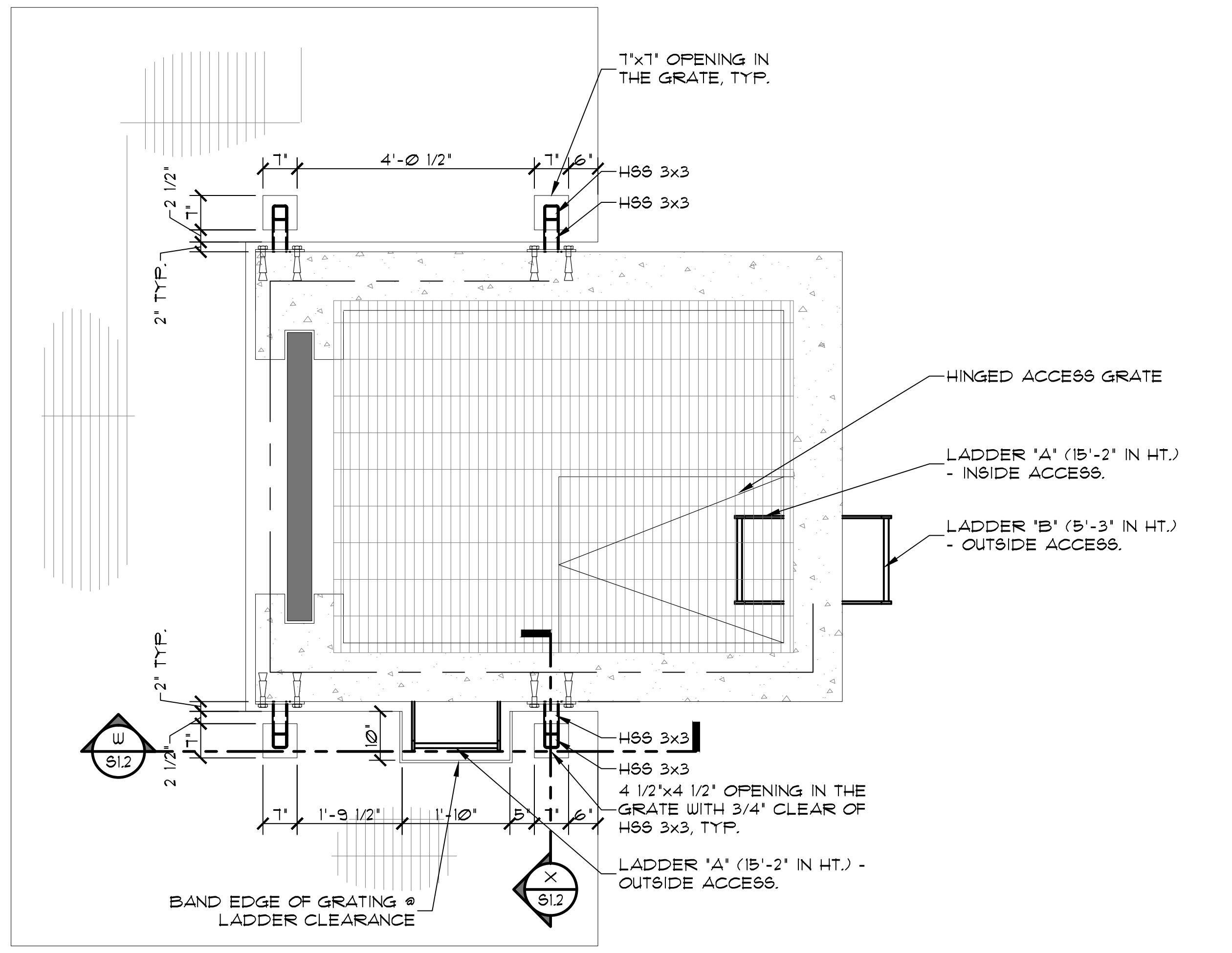
DECK PLATFORM FRAMING PLAN

SCALE: 1/2" = 1'-0"



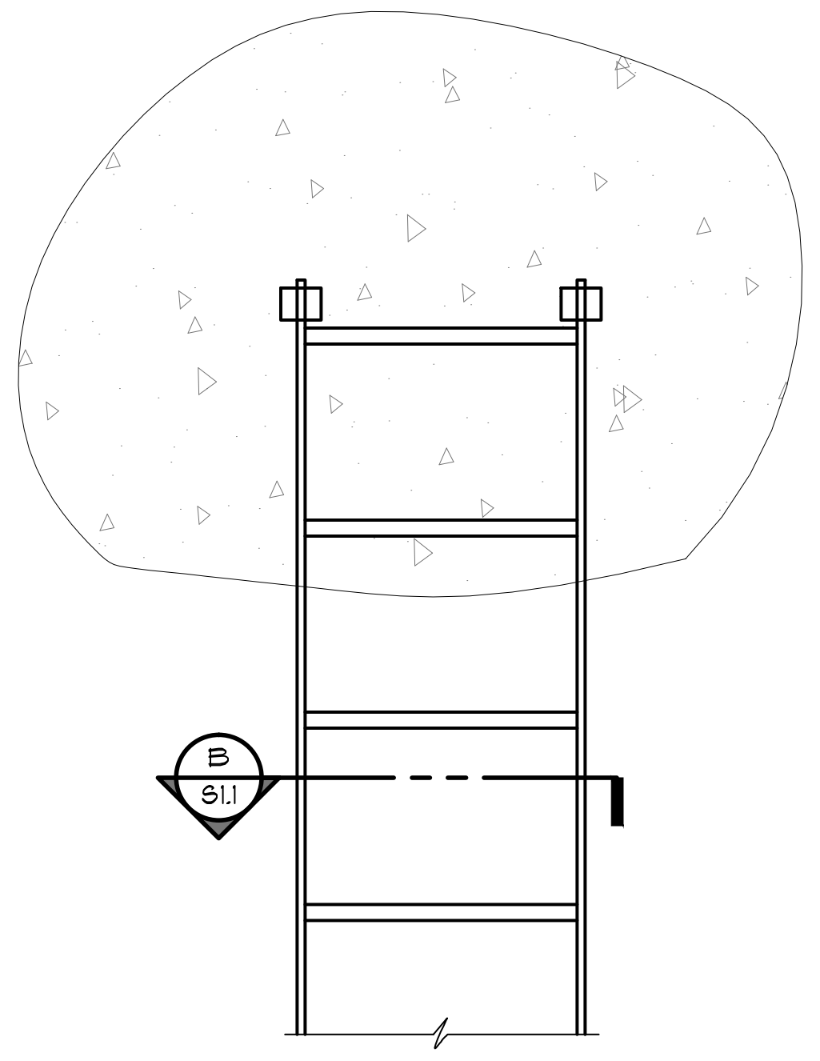
FLOATING SKIMMER STRUCTURAL PLAN

SCALE: 1/2" = 1'-0"



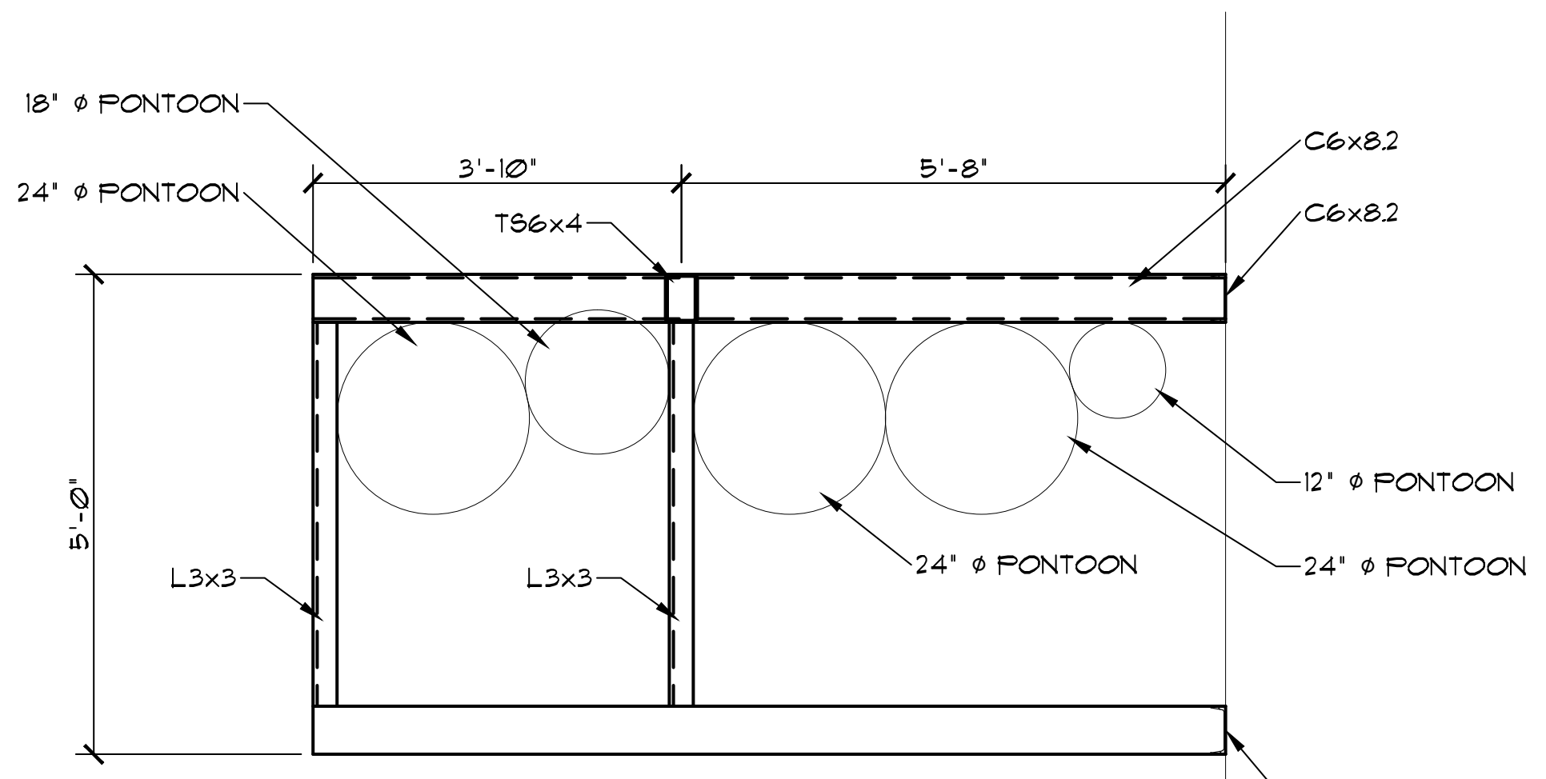
LADDER KEY PLAN

SCALE: 1/2" = 1'-0"



TYPICAL LADDER DETAIL

SCALE: 1" = 1'-0"

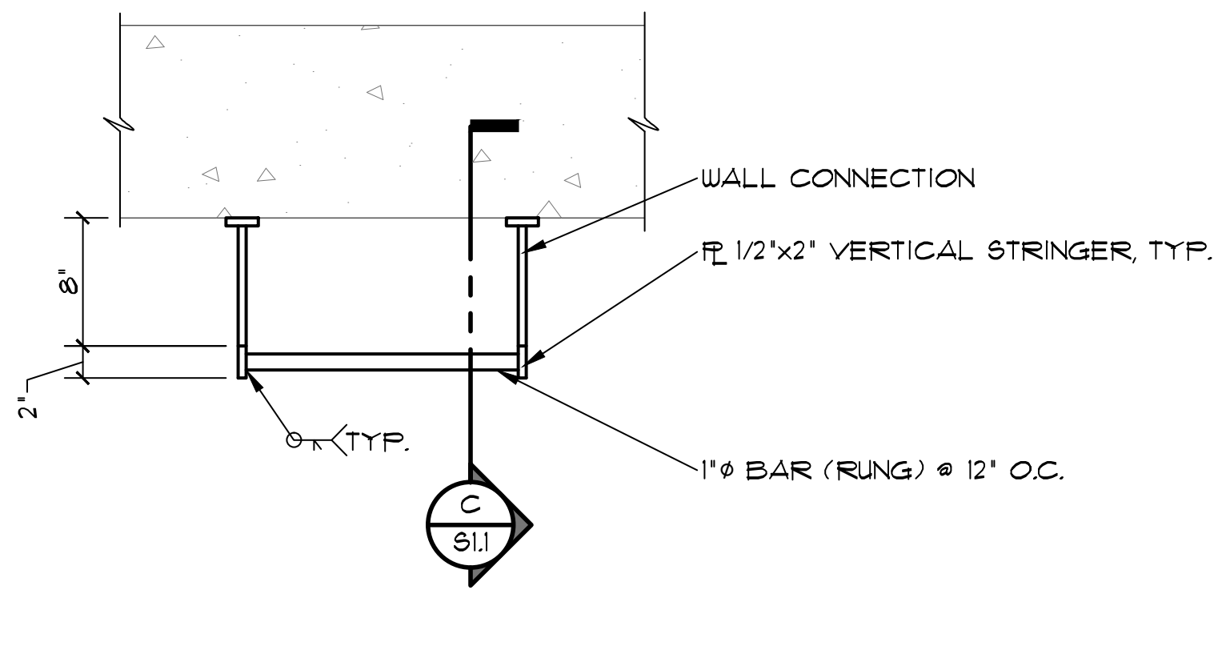


REPLACE ALL EXISTING PONTOONS FOR NEW LOCATION OF EXISTING PLATFORM.

FLOATING SKIMMER SECTION

A
S2.1

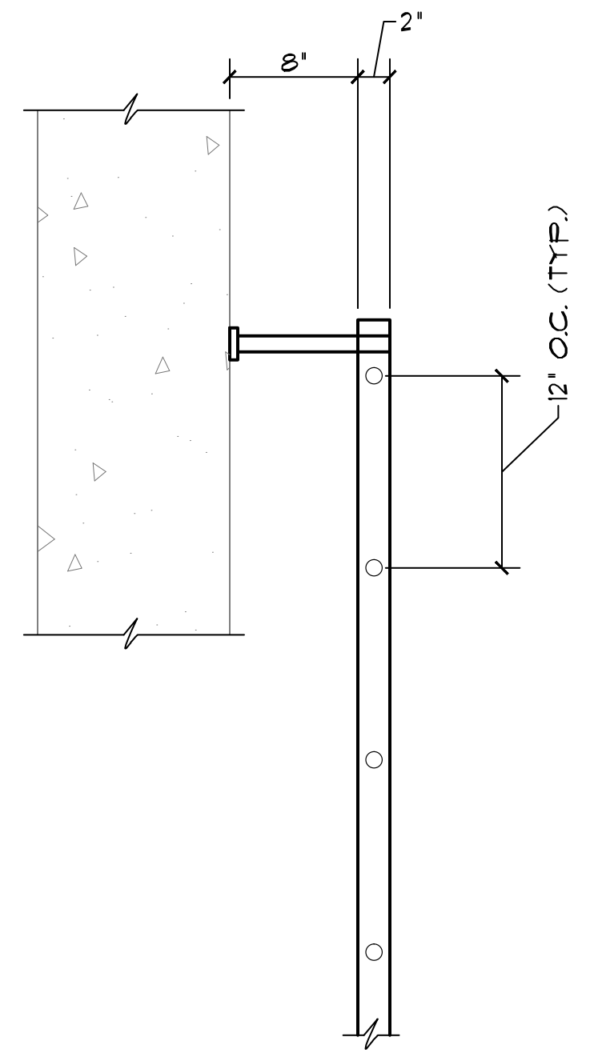
SCALE: 1/2" = 1'-0"



LADDER SECTION

B
S2.1

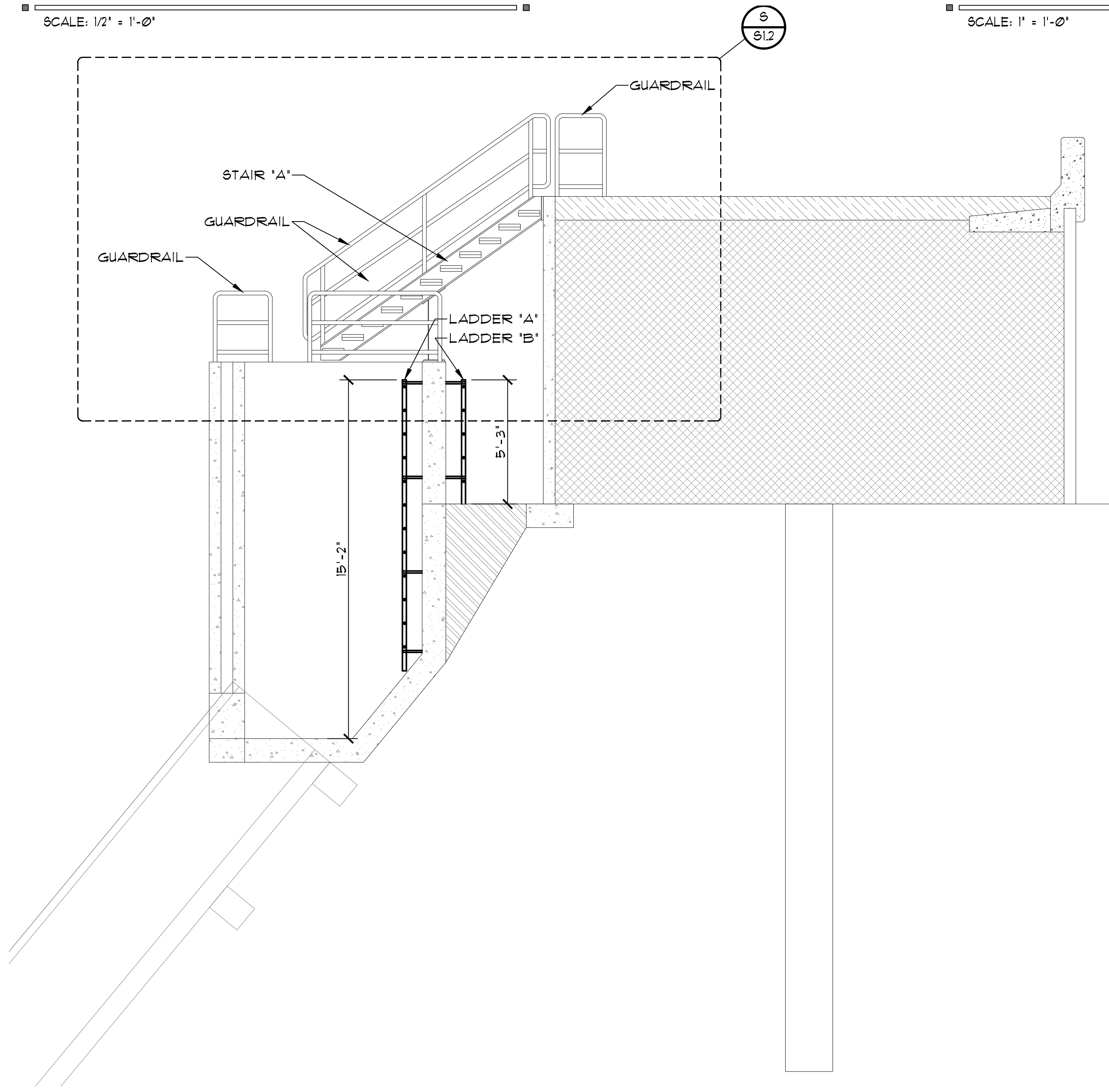
SCALE: 1" = 1'-0"



LADDER SECTION

C
S2.1

SCALE: 1" = 1'-0"



INLET STRUCTURE SECTION FOR REFERENCE ONLY

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

DATE	NO.	DESCRIPTION	APPD.
9/28/12	A	ISSUED FOR PERMIT	RWM

REVISIONS

CARDINAL OPERATING COMPANY

CARDINAL PLANT

BRILLIANT OHIO

DAM RAISING
FLY ASH RETENTION DAM II
S1.1 SERVICE SPILLWAY
INLET STRUCTURE DETAILS

DWG. NO. 13-30093-A

ARCH — ELEC — MECH — STR

SCALE: AS SHOWN
DR:
CH:
SQD SUPV:
DATE:
APPROVED BY
CIVIL ENGINEERING DIVISION

ROBERT W. MILLER
E-58904

PROJECT NUMBER:	011-11497-042	DRAWN BY:	JCW
DRAWING DATE:	9/28/12	ENGINEER:	RWM
LAST UPDATED:	9/28/12	APPROVED BY:	GLM
		SCALE:	AS NOTED

NEW AVENUE architects . engineers

S&ME

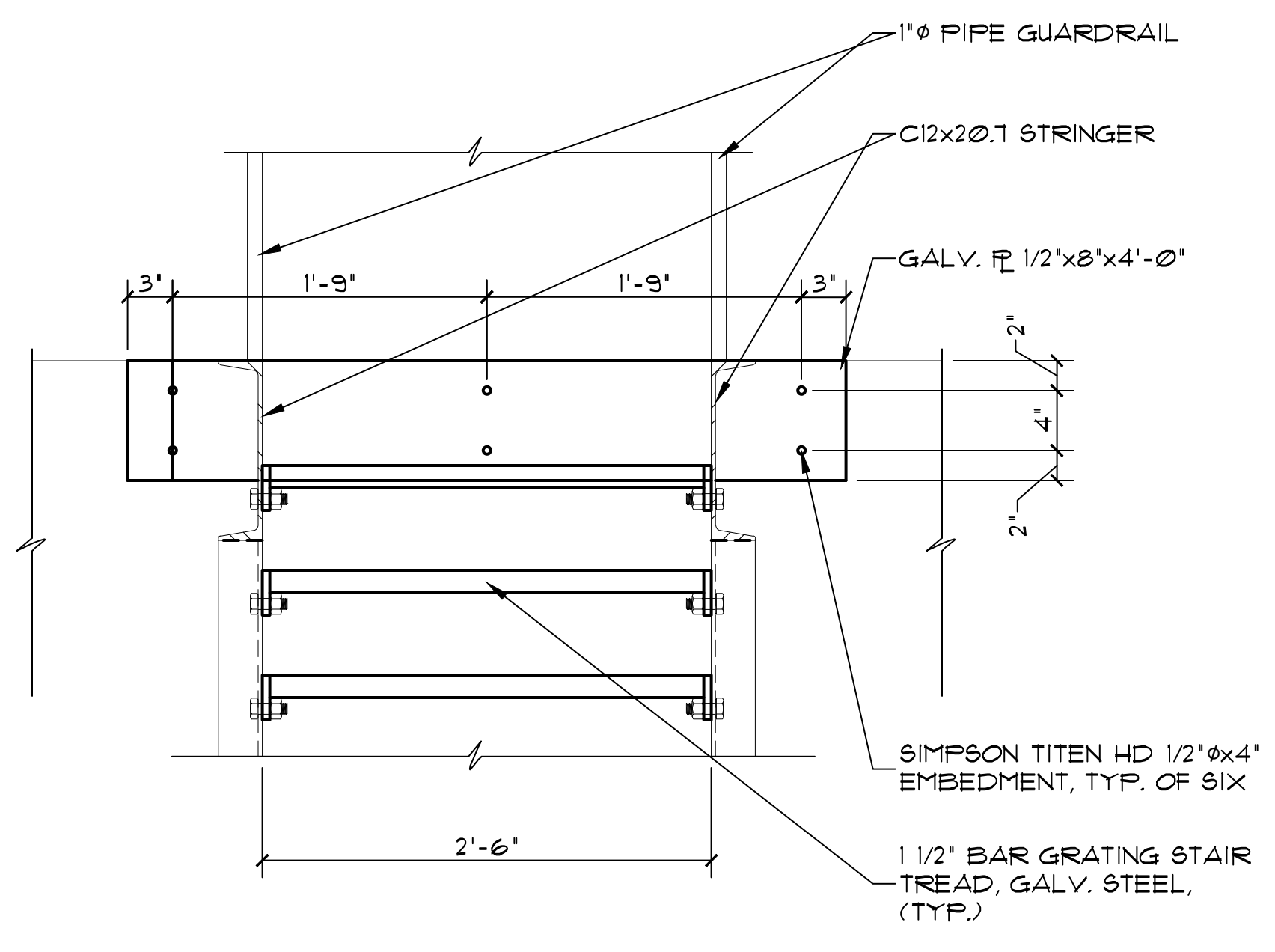
AEP AMERICAN ELECTRIC POWER

1 RIVERSIDE PLAZA
COLUMBUS, OH 43215

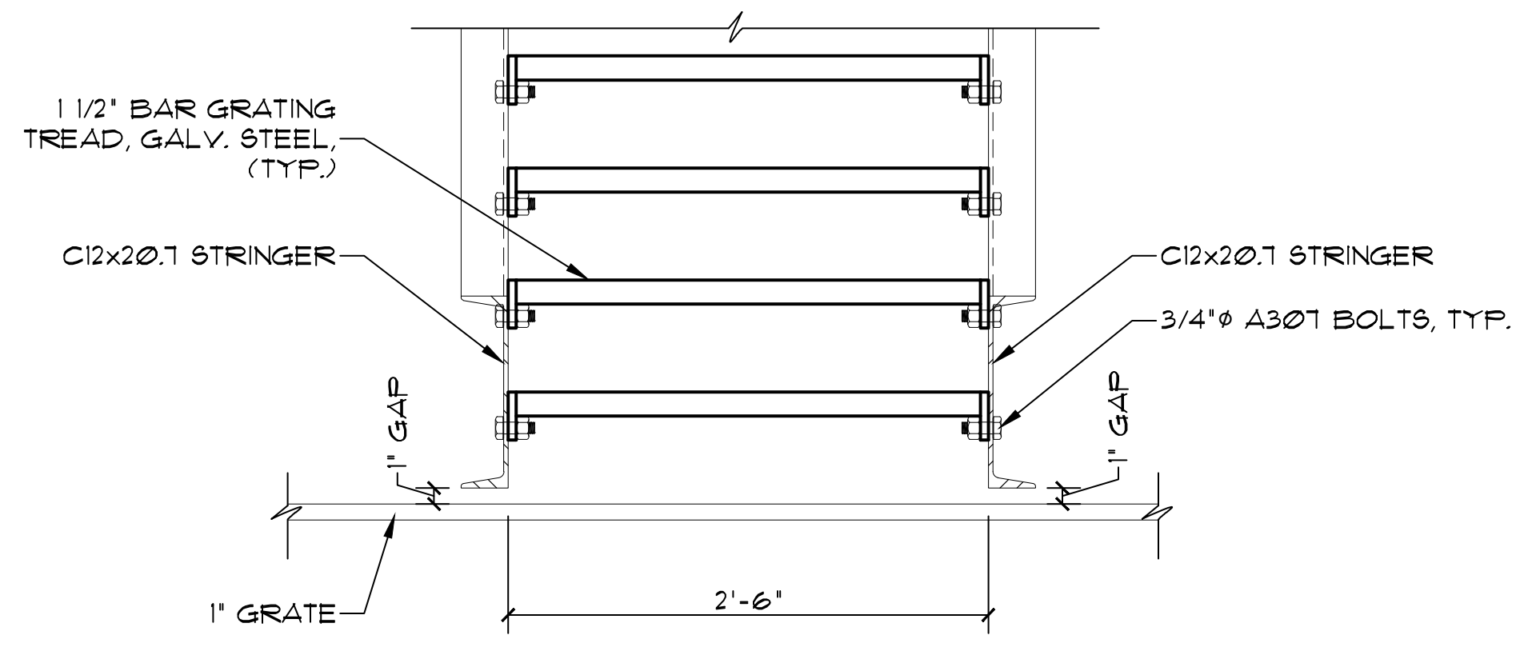
IGS1092

SYSTEM DATE- DD-MMM-YYYY
SYSTEM TIME- HOUR:MINUTE
150≡ FLR.CED

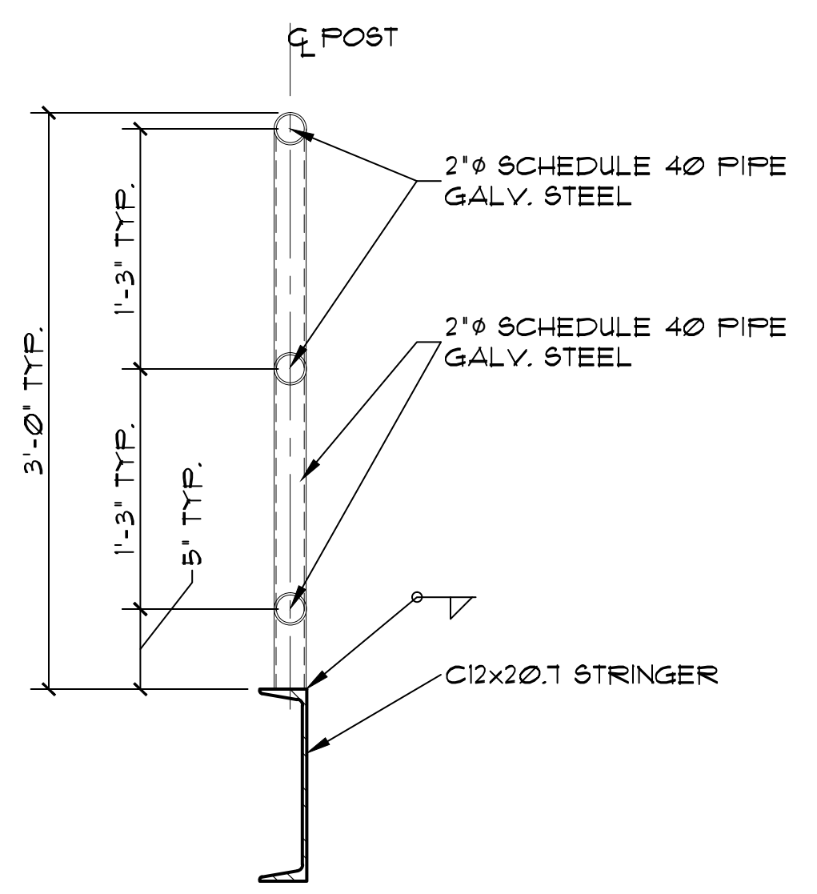
DWG. NO. 13-30094-A



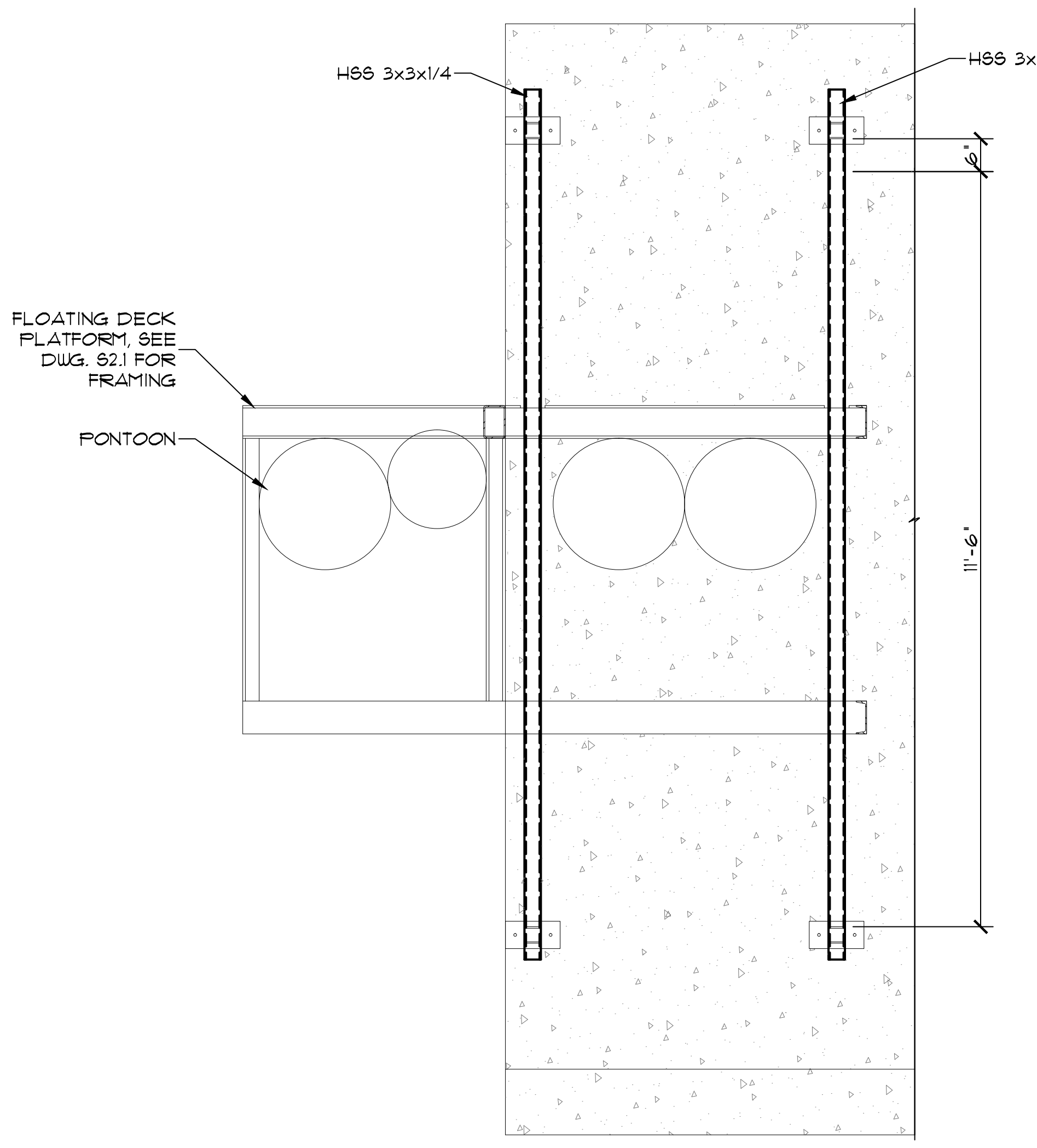
T/S2.2 SECTION
SCALE: 1" = 1'-0"



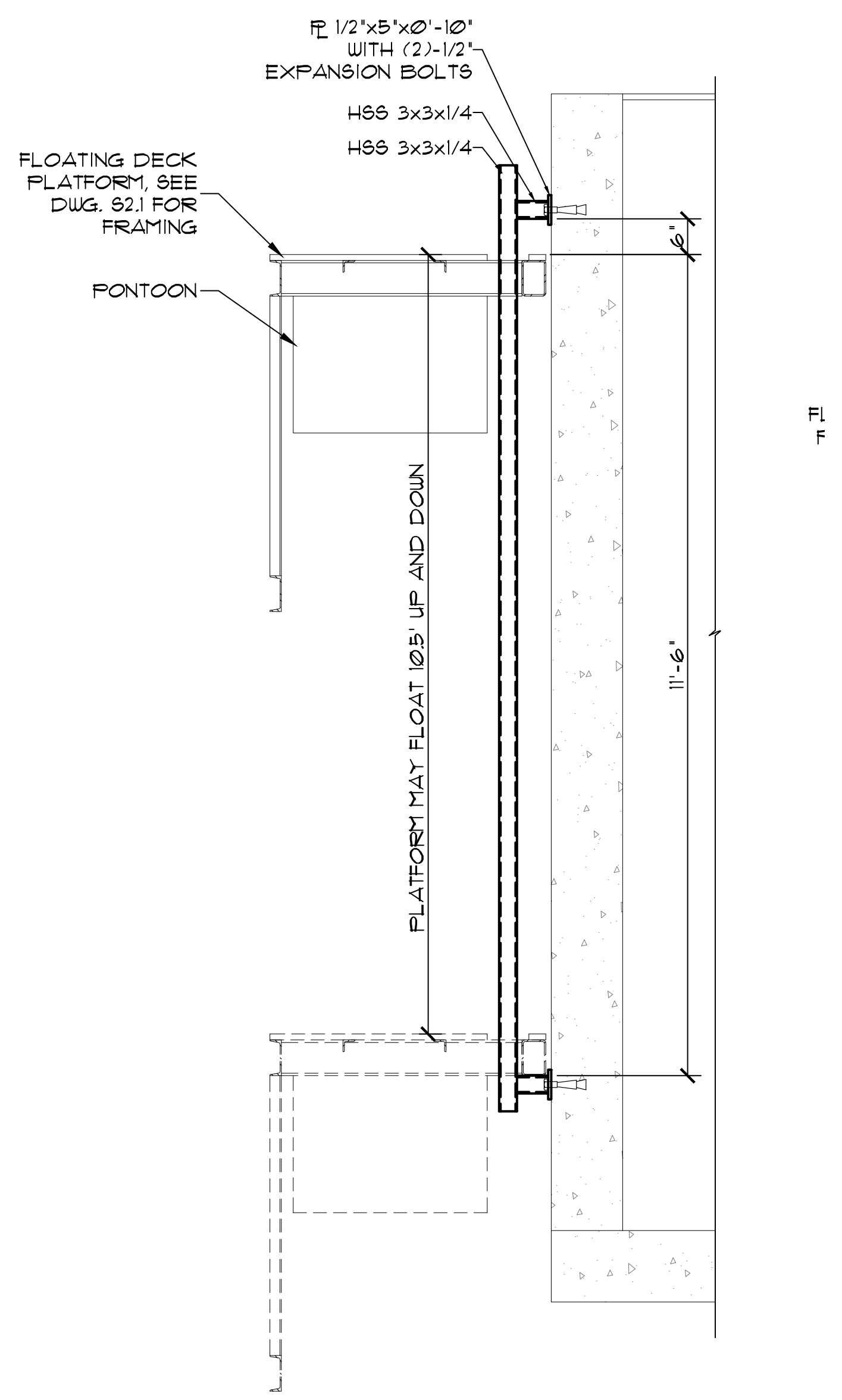
U/S2.2 SECTION
SCALE: 1" = 1'-0"



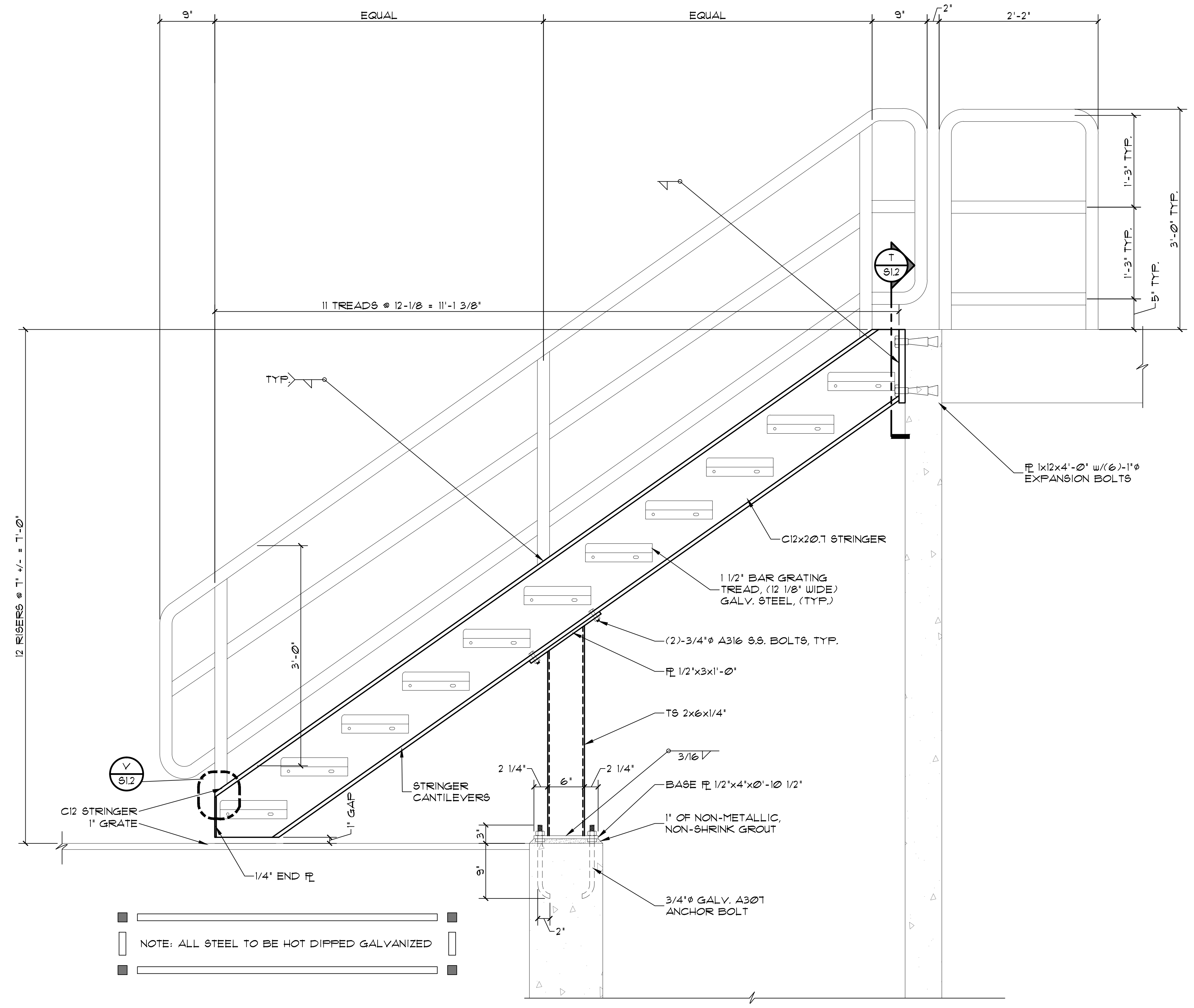
V/S2.2 RAILING DETAIL
SCALE: 1" = 1'-0"



W/S2.2 STAIR "A" SECTION
SCALE: 1/2" = 1'-0"

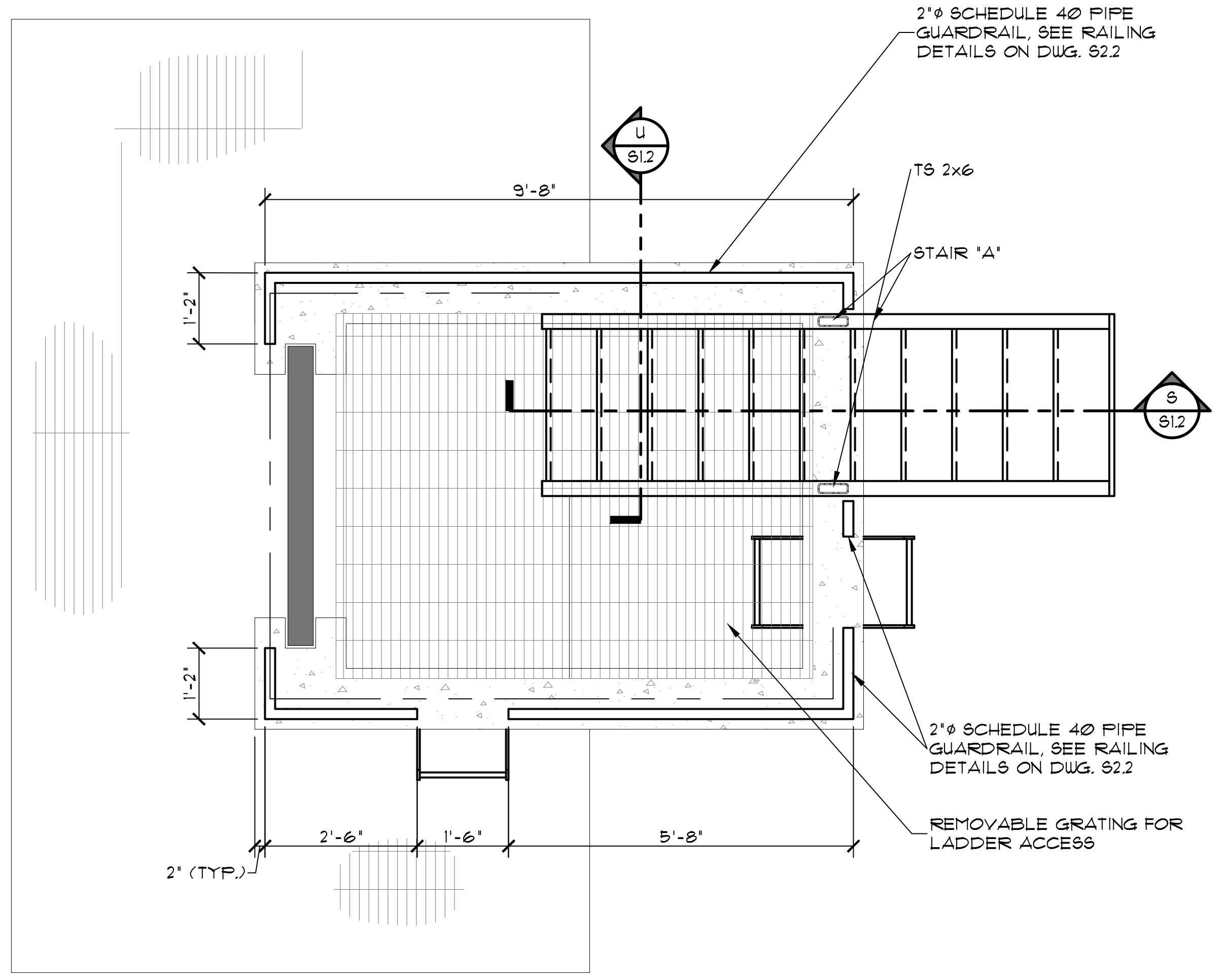


X/S2.2 STAIR "A" SECTION
SCALE: 1/2" = 1'-0"



NOTE: ALL STEEL TO BE HOT DIPPED GALVANIZED

S/S2.2 STAIR "A" SECTION
SCALE: 1" = 1'-0"



(FIELD VERIFY DIMENSIONS FOR GUARDRAIL LENGTHS)

STAIR "A" PLAN
SCALE: 1/2" = 1'-0"

DATE	NO.	DESCRIPTION	APPD.
9/28/12	A	ISSUED FOR PERMIT	RWM

REVISIONS

"THIS DRAWING IS THE PROPERTY OF THE AMERICAN ELECTRIC POWER SERVICE CORP. AND IS LOANED UPON CONDITION THAT IT IS NOT TO BE REPRODUCED OR COPIED, IN WHOLE OR IN PART, OR USED FOR FURNISHING INFORMATION TO ANY PERSON WITHOUT THE WRITTEN CONSENT OF THE AEP SERVICE CORP. , OR FOR ANY PURPOSE DETRIMENTAL TO THEIR INTEREST, AND IS TO BE RETURNED UPON REQUEST"

CARDINAL OPERATING COMPANY
CARDINAL PLANT
BRILLIANT OHIO

DAM RAISING
FLY ASH RETENTION DAM II
S1.2 SERVICE SPILLWAY
INLET STRUCTURE DETAILS

ROBERT W. MILLER
E-58904

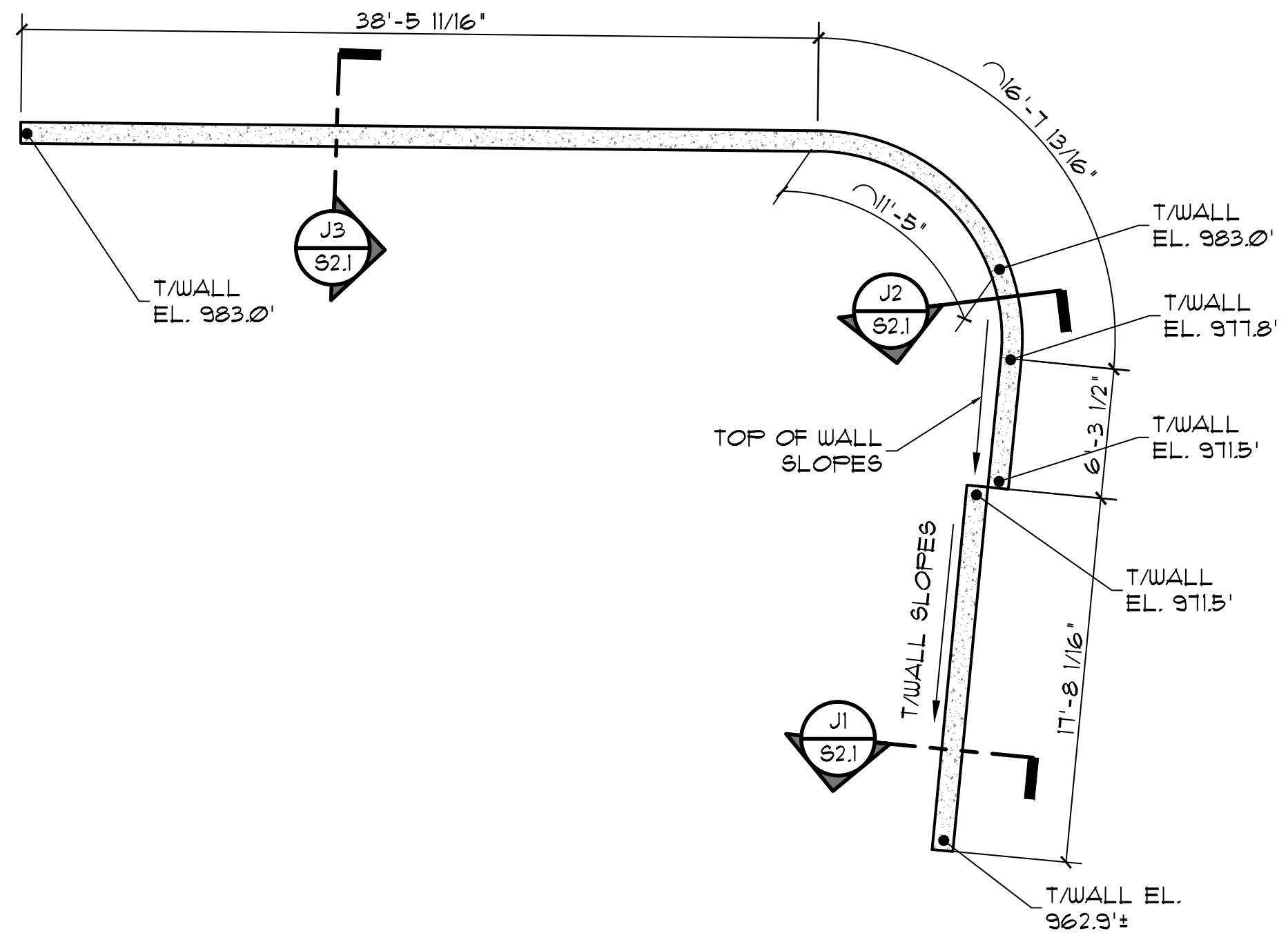
DWG. NO. 13-30094-A

ARCH	——	ELEC	——	MECH	——	STR

SCALE: AS SHOWN
DR:
CH:
SQD SUPV:
DATE:
APPROVED BY
CIVIL ENGINEERING DIVISION

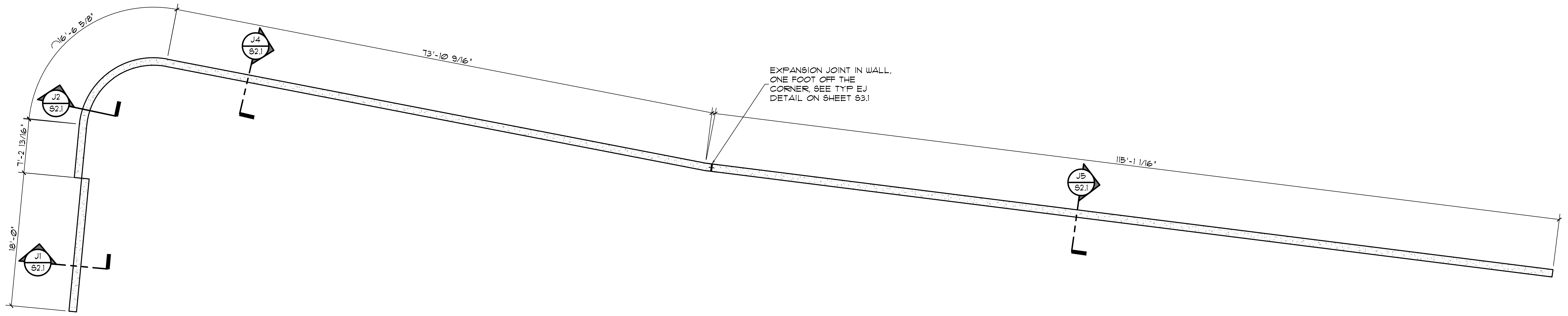
PROJECT NUMBER:	011-11497-042	DRAWN BY:	JCW
DRAWING DATE:	9/28/12	ENGINEER:	RWM
LAST UPDATED:	9/28/12	APPROVED BY:	GLM
		SCALE:	AS NOTED

NEW AVENUE architects . engineers
S&ME
AEP AMERICAN ELECTRIC POWER
1 RIVERSIDE PLAZA
COLUMBUS, OH 43215



RIGHT TRAINING WALL PLAN

SCALE: 1/2" = 1'-0"



LEFT TRAINING WALL PLAN

SCALE: 1/2" = 1'-0"

DATE	NO.	DESCRIPTION	APPD.
9/28/12	A	ISSUED FOR PERMIT	RWM

REVISIONS

CARDINAL OPERATING COMPANY

CARDINAL PLANT

BRILLIANT OHIO

DAM RAISING
FLY ASH RETENTION DAM II
S2.0 EMERGENCY SPILLWAY
RETAINING WALL PLAN

DWG. NO. 13-30095-A

ARCH —— ELEC —— MECH —— STR

SCALE: AS SHOWN
DR:
CH:
SQD SUPV:
DATE:
APPROVED BY
CIVIL ENGINEERING DIVISION

ROBERT W. MILLER
E-58904

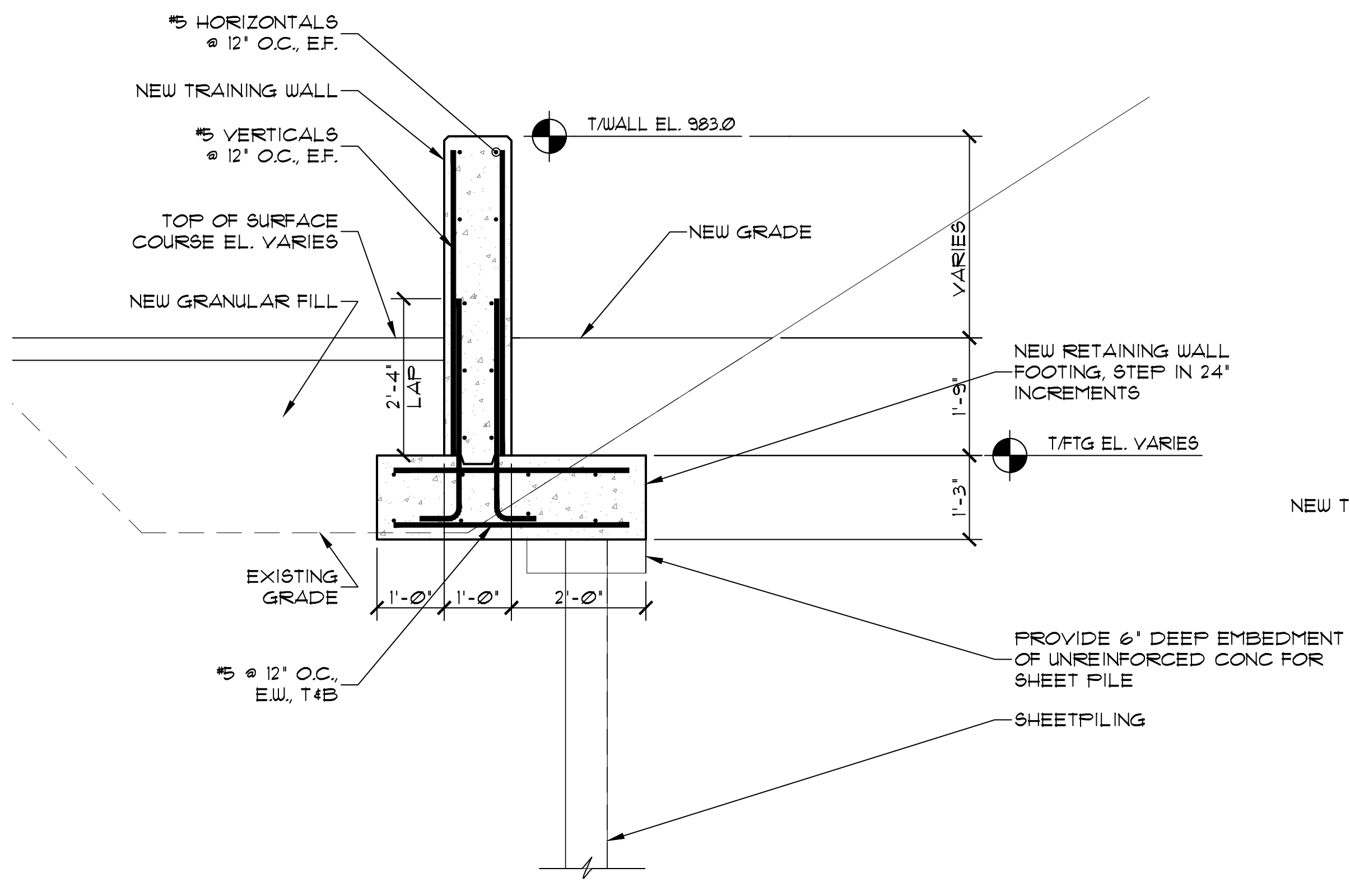
PROJECT NUMBER:	011-11497-042	DRAWN BY:	JCW
DRAWING DATE:	9/28/12	ENGINEER:	RWM
LAST UPDATED:	9/28/12	APPROVED BY:	GLM
		SCALE:	AS NOTED

NEW AVENUE architects . engineers

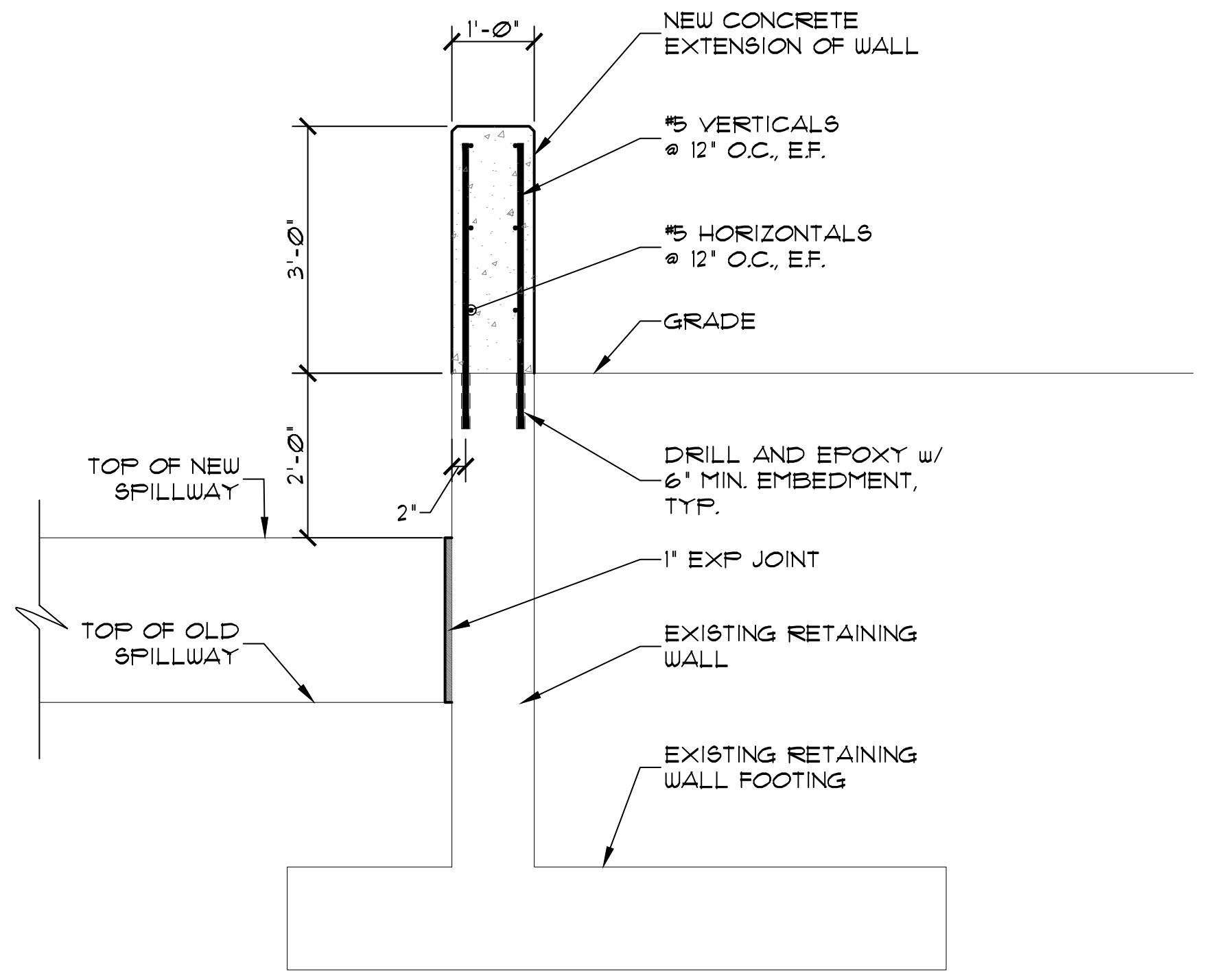
S&ME

AEP AMERICAN ELECTRIC POWER

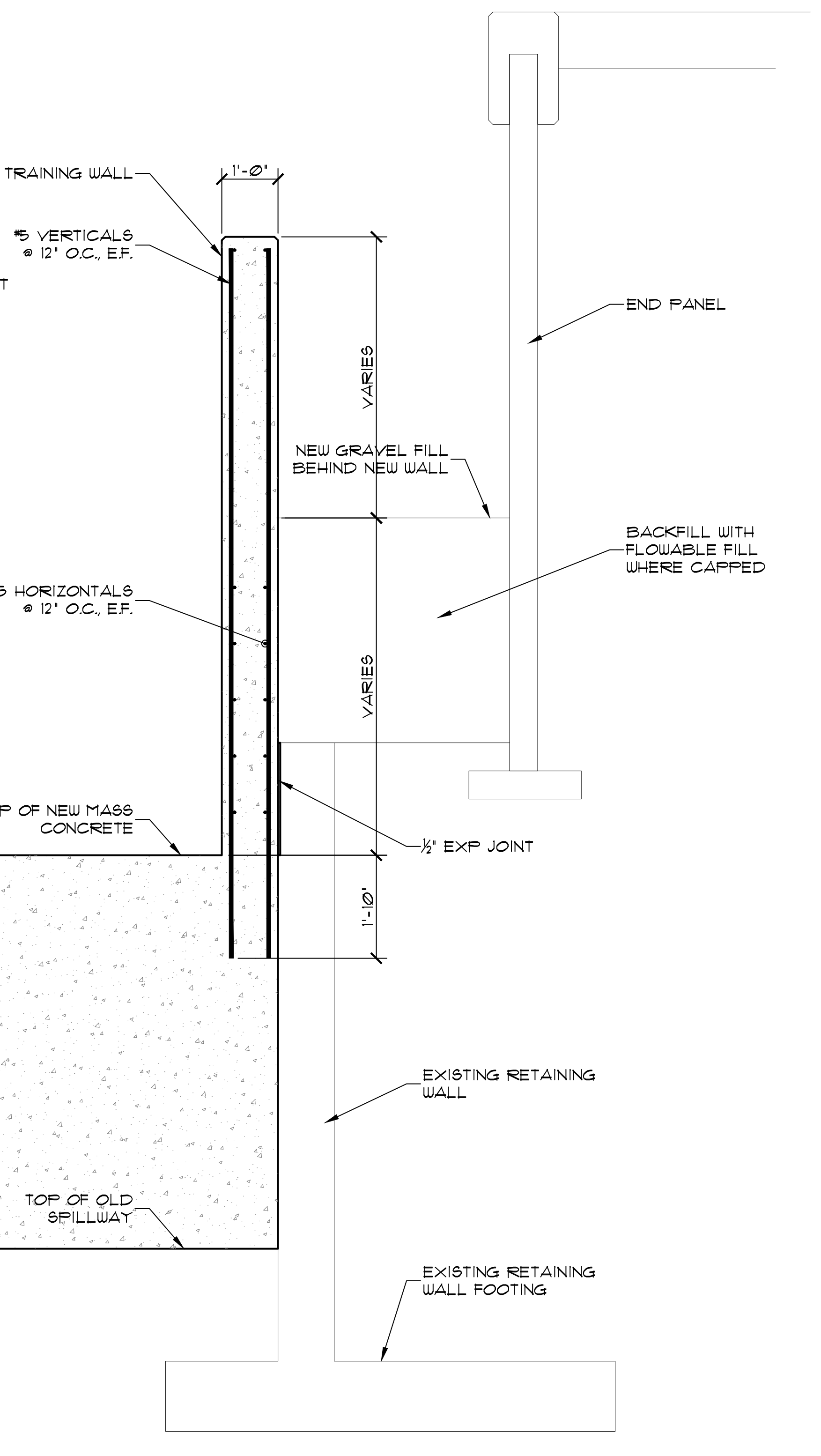
1 RIVERSIDE PLAZA
COLUMBUS, OH 43215



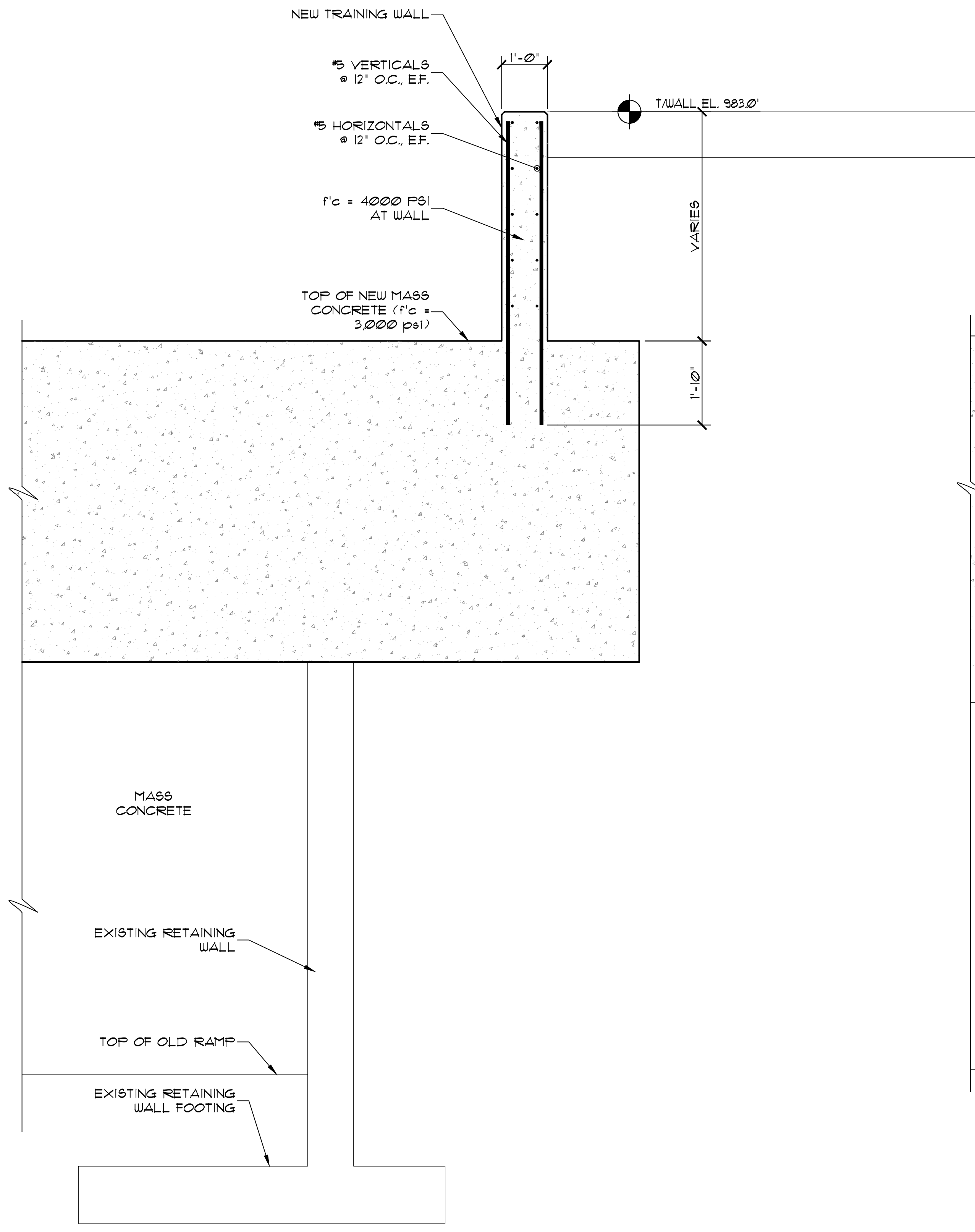
J5 RETAINING WALL SECTION
SCALE: 1/2" = 1'-0"



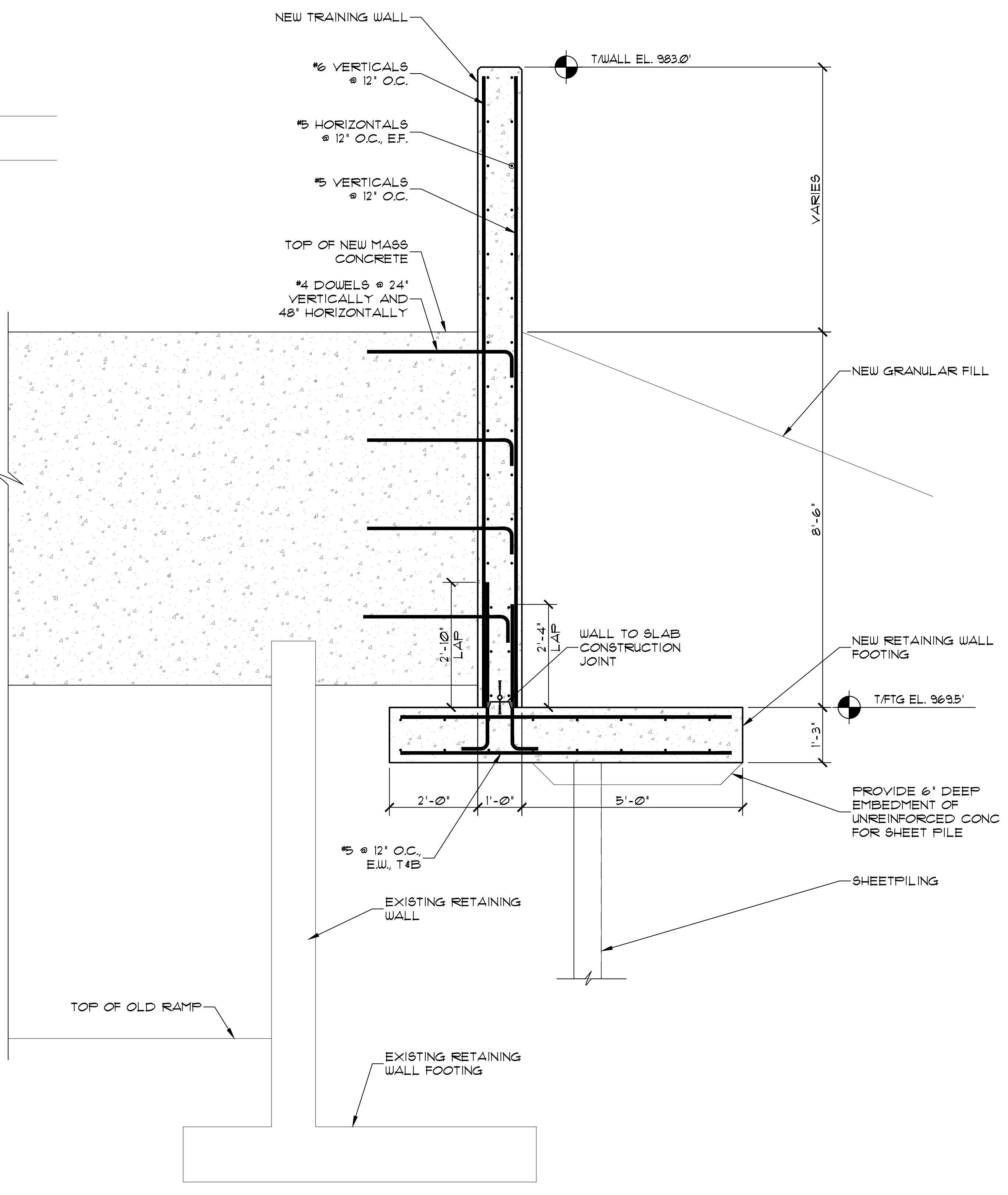
J1 RETAINING WALL SECTION
SCALE: 1/2" = 1'-0"



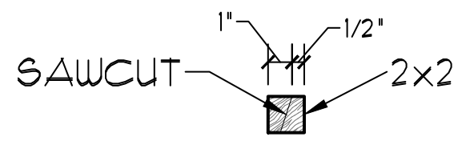
J2 RETAINING WALL SECTION
SCALE: 1/2" = 1'-0"



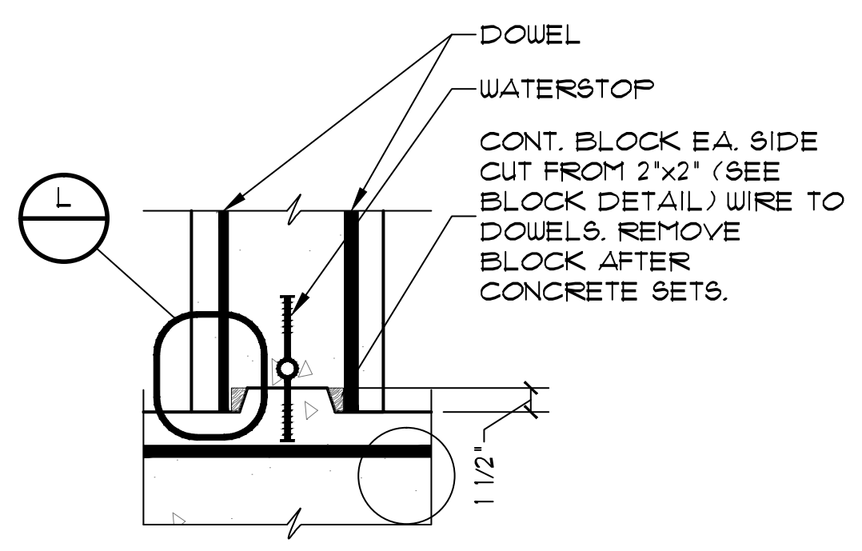
J3 RETAINING WALL SECTION
SCALE: 1/2" = 1'-0"



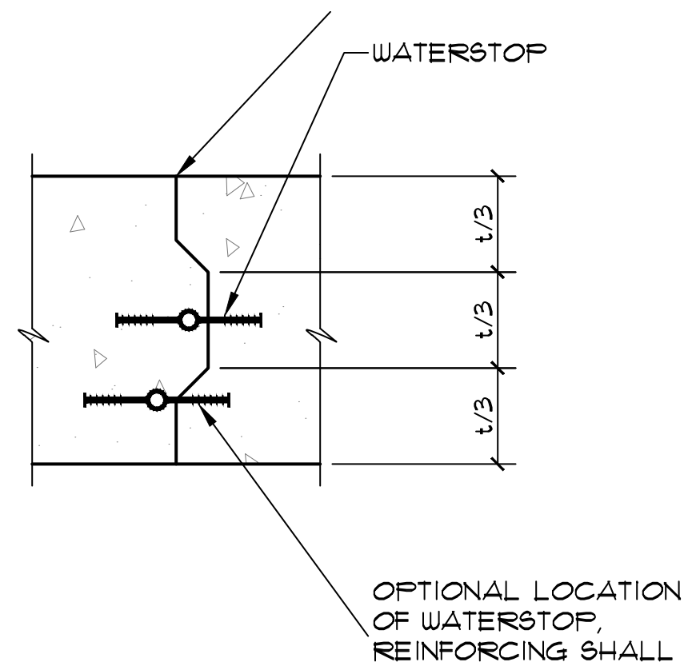
J4 RETAINING WALL SECTION
SCALE: 1/2" = 1'-0"



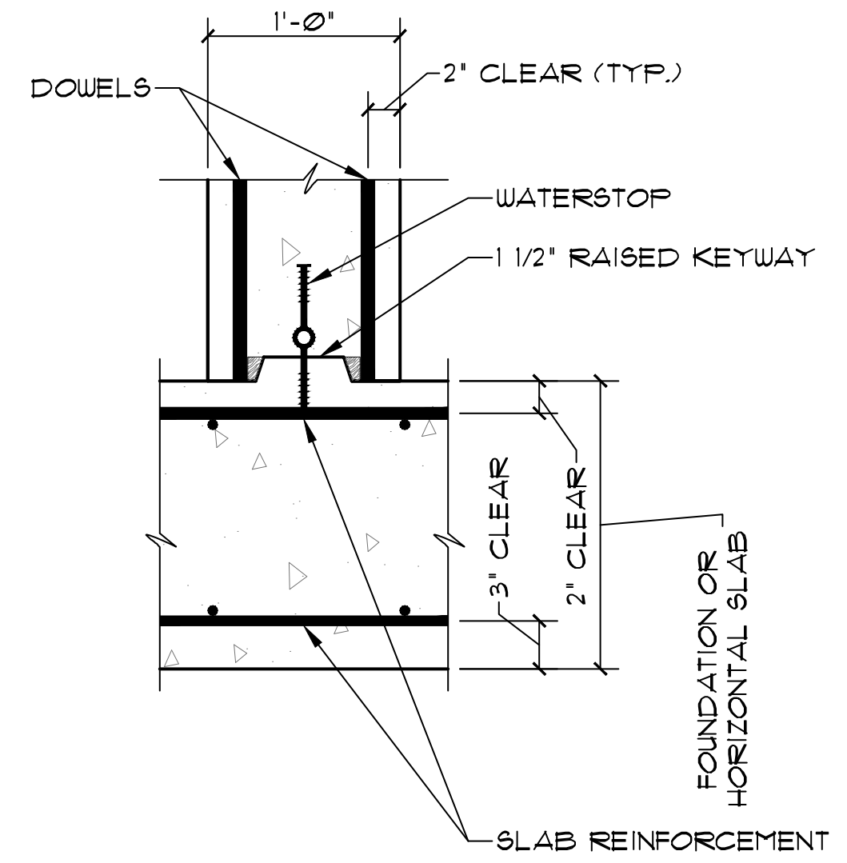
L TYP. KEYWAY BLOCK DETAIL
SCALE: 1 1/2" = 1'-0"



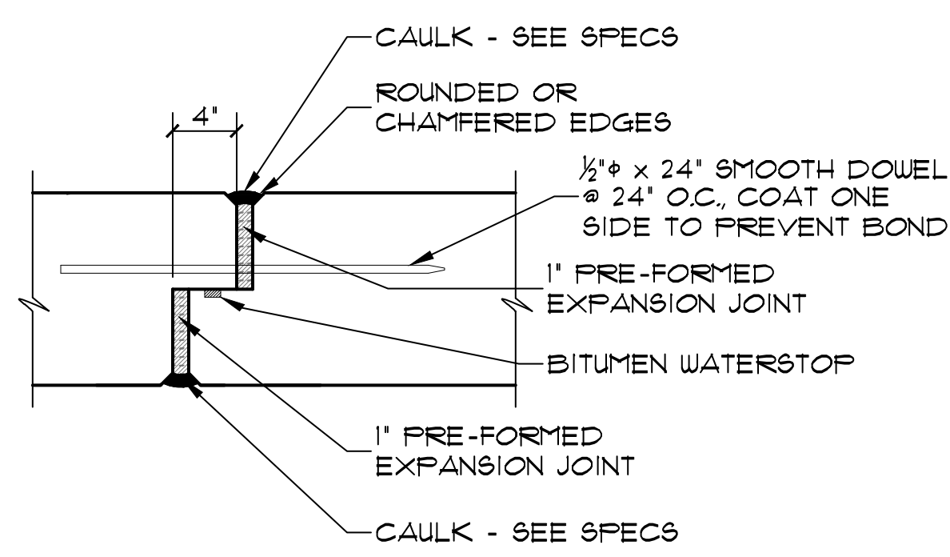
TYP. REVERSE KEYWAY DETAIL
SCALE: 1" = 1'-0"



TYP. WATERSTOP DETAIL
SCALE: 1" = 1'-0"



TYP. WALL TO SLAB C.J.
SCALE: 1" = 1'-0"



NOTE: REINFORCING SHALL NOT EXTEND THRU JOINT CAULKING NOT REQUIRED ON BOTTOM OF FOOTING SLAB.

TYP. WALL EXPANSION JOINT DETAIL
SCALE: 1" = 1'-0"

DATE	NO.	DESCRIPTION	APPD.
9/28/12	A	ISSUED FOR PERMIT	RWM

REVISIONS

CARDINAL OPERATING COMPANY
CARDINAL PLANT
BRILLIANT OHIO

DAM RAISING
FLY ASH RETENTION DAM II
S2.1 EMERGENCY SPILLWAY
RETAINING WALL SECTIONS

DWG. NO. 13-30096-A

ARCH —— ELEC —— MECH —— STR

SCALE: AS SHOWN
DR:
CH:
SQD SUPV:
DATE:
APPROVED BY
CIVIL ENGINEERING DIVISION

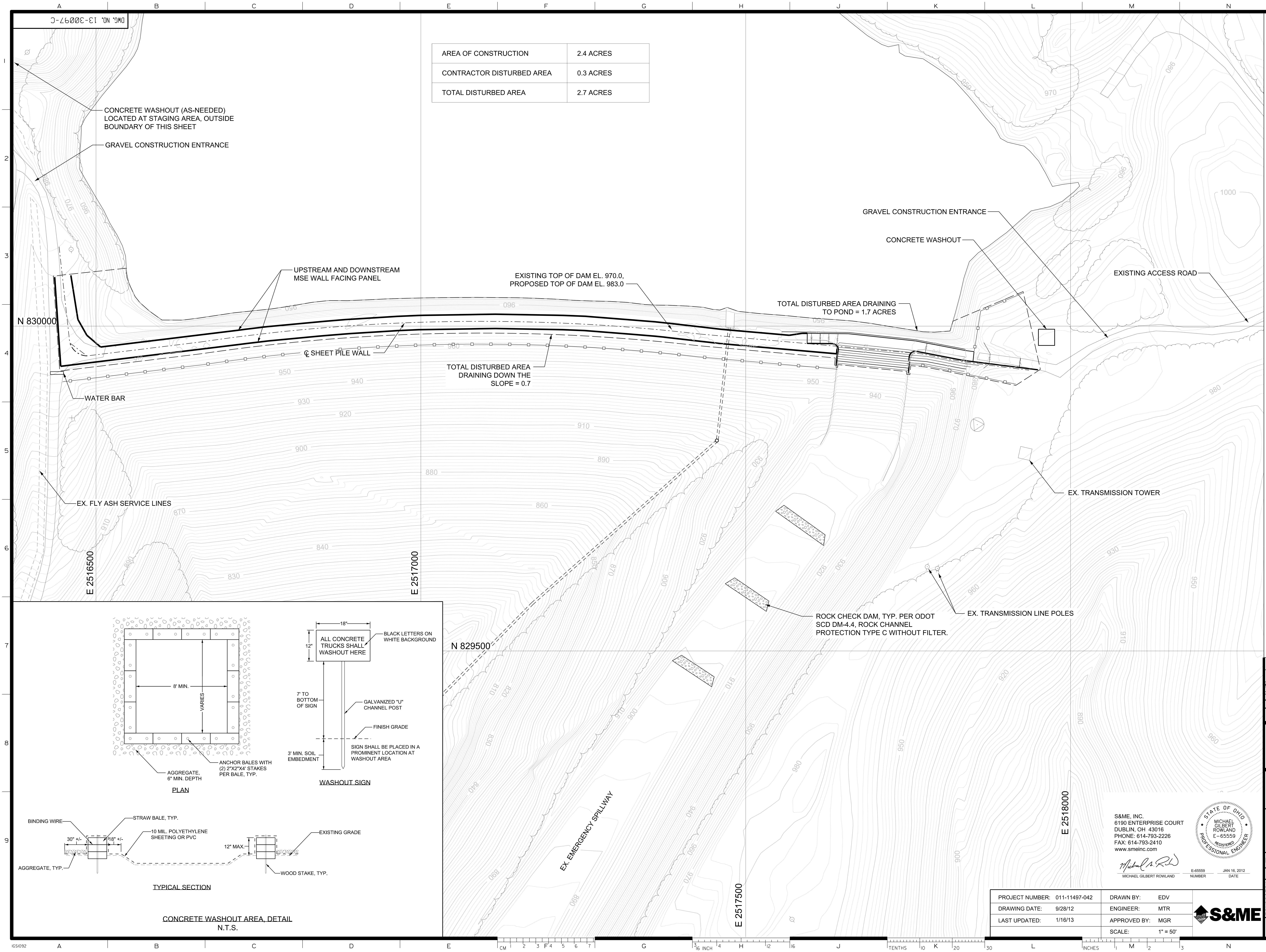
ROBERT W. MILLER
E-58904

PROJECT NUMBER:	011-11497-042	DRAWN BY:	JCW
DRAWING DATE:	9/28/12	ENGINEER:	RWM
LAST UPDATED:	9/28/12	APPROVED BY:	GLM
		SCALE:	AS NOTED

NEW AVENUE architects . engineers
S&ME

AEP AMERICAN ELECTRIC POWER
1 RIVERSIDE PLAZA
COLUMBUS, OH 43215

AREA OF CONSTRUCTION	2.4 ACRES
CONTRACTOR DISTURBED AREA	0.3 ACRES
TOTAL DISTURBED AREA	2.7 ACRES



CONCRETE WASHOUT AREA, DETAIL
N.T.S.

LEGEND

900	EXISTING CONTOUR 10FT INTERVAL
950	PROPOSED GRADE CONTOUR 10FT INTERVAL
	EXISTING ACCESS ROADS
	EXISTING FLY ASH SERVICE LINES
	EXISTING TREELINE
	SILT FENCE
▨	ROCK CHECK DAM
	PROJECT LIMITS
☐	CONCRETE WASHOUT

GENERAL NOTES

1. TOPOGRAPHIC MAPPING DEVELOPED FROM AERIAL SURVEY PERFORMED ON 3/5/2009 AND 1986 AS-BUILT DRAWINGS.

REFERENCE DRAWINGS

ODOT SCD DM-4.4, REV. 4-17-09

DATE	NO.	DESCRIPTION	APPD.
1/16/13	C	REVISED PER ODNR COMMENTS REC'D 12/4/2012	MGR
9/12/12	B	REVISED PER ODNR COMMENTS REC'D 7/18/12	MGR
5/23/12	A	ISSUED FOR PERMIT INITIAL SUMMITTAL	MGR

REVISIONS

CARDINAL OPERATING COMPANY

CARDINAL PLANT

BRILLIANT OHIO

DAM RAISING
FLY ASH RETENTION DAM II
EROSION CONTROL PLAN
SHEET 2 OF 2

DWG. NO. 13-30097-C

ARCH	—	ELEC	—	MECH	—	STR	GS

SCALE: AS SHOWN
DR: EDV
CH: MAA
SQD SUPV: MAA
DATE: 9/09/11
APPROVED BY
CIVIL ENGINEERING DIVISION

AMERICAN ELECTRIC POWER
1 RIVERSIDE PLAZA
COLUMBUS, OH 43215

S&ME, INC.
6190 ENTERPRISE COURT
DUBLIN, OH 43016
PHONE: 614-793-2226
FAX: 614-793-2410
www.smeinc.com

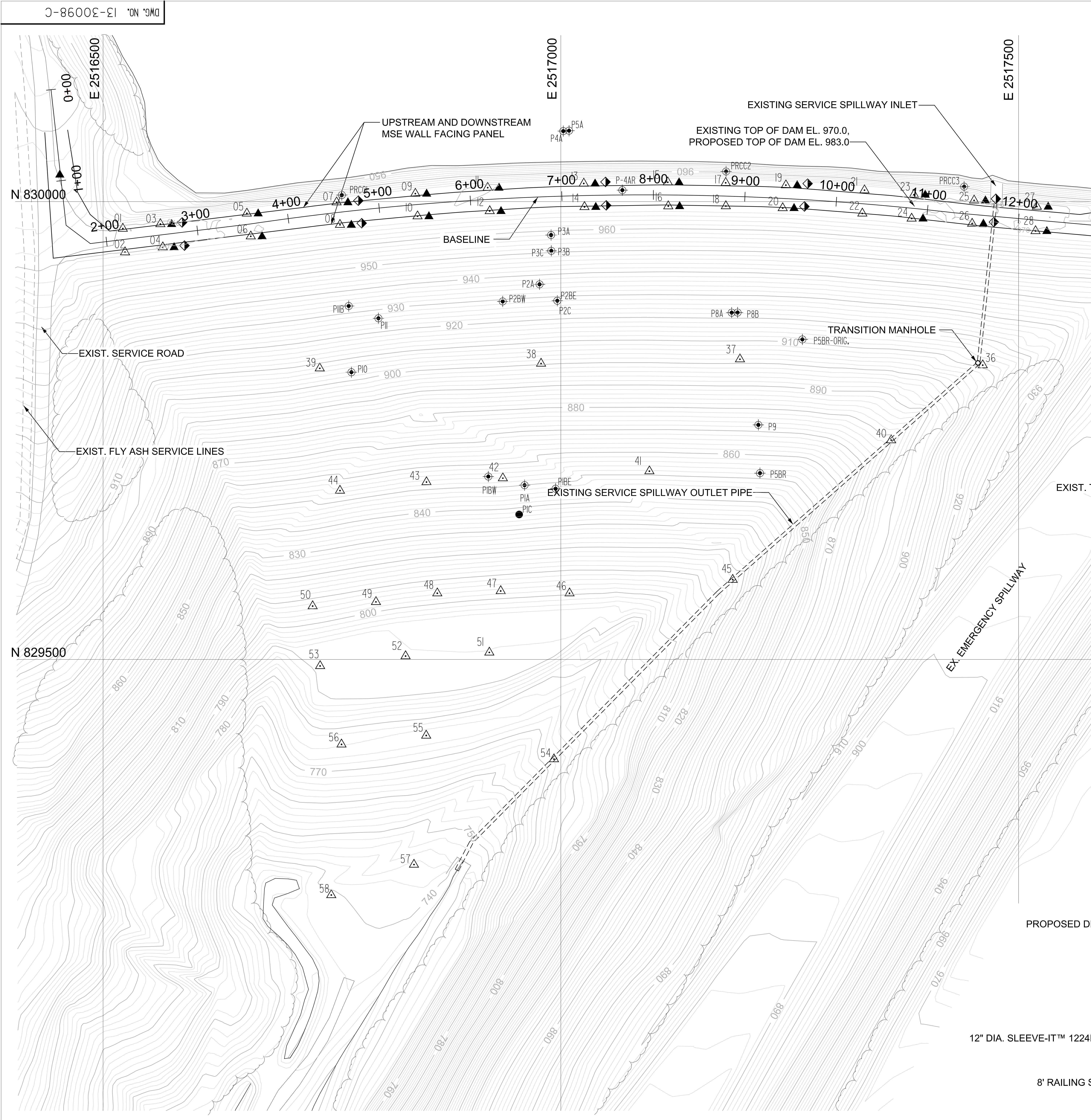
MICHAEL GILBERT ROWLAND E-65559

Michael G. Rowland	E-65559	JAN 16, 2012
MICHAEL GILBERT ROWLAND	NUMBER	DATE

PROJECT NUMBER:	011-11497-042	DRAWN BY:	EDV
DRAWING DATE:	9/28/12	ENGINEER:	MTR
LAST UPDATED:	1/16/13	APPROVED BY:	MGR
		SCALE:	1" = 50'

S&ME

DWG. NO. 13-30098-C



0 50 100
SCALE IN FEET

NOTE: ALL EXISTING DEFORMATION MONUMENTS TO REMAIN ACCESSIBLE EXCEPT #29931 THROUGH #29934.

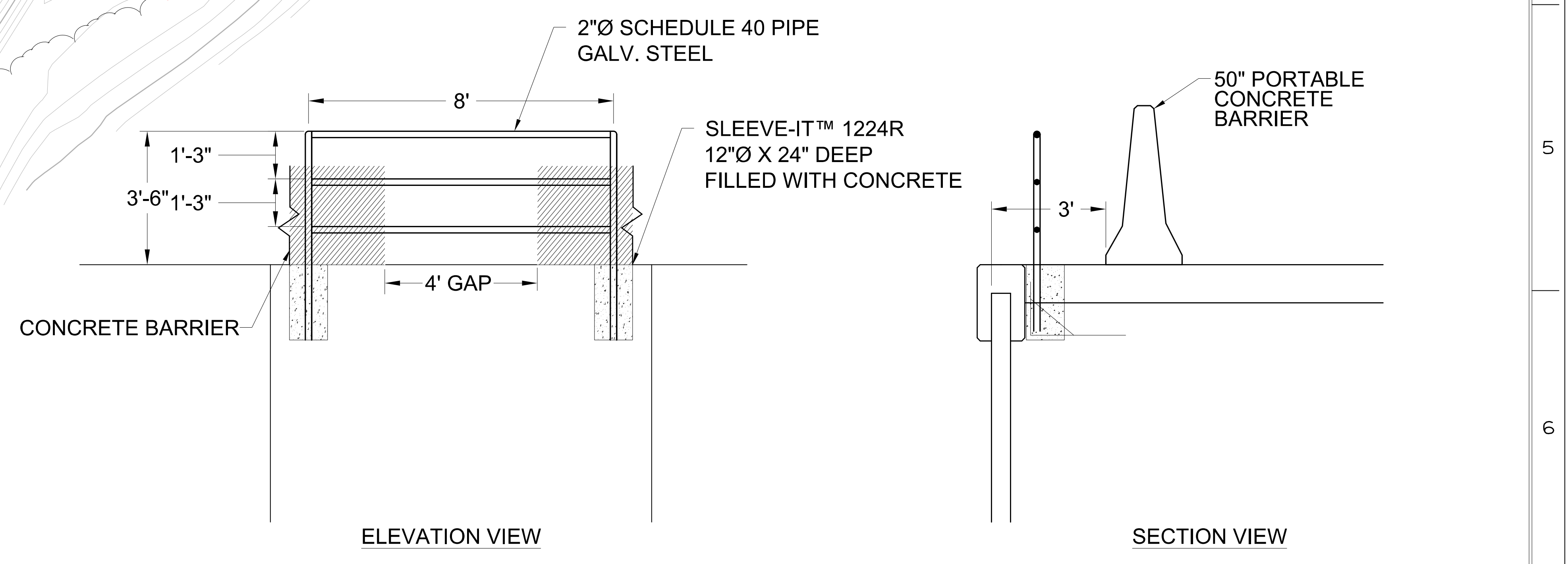
PIEZOMETER #	NORTH	EAST	TIP ELEV.
P8B	829880.385	2517198.101	776.0
P8A	829880.185	2517191.730	802.1
P2BE nested	829893.405	2517001.135	730.0
P2C nested	829893.405	2517001.135	711.0
P3B nested	829965.095	2516994.335	772.3
P3A nested	829965.095	2516994.335	802.3
P3C	829947.865	2516994.545	712.3
P2BW	829892.685	2516941.390	731.1
P2A	829910.895	2516891.870	771.0
P11	829874.275	2516805.610	802.6
P11B	829887.670	2516773.430	789.1
P10	829815.120	2516776.440	769.1
P9	829757.800	2517221.000	771.2
P1BE	829687.855	2516999.390	728.0
P1A	829692.010	2516965.335	752.3
P1C	829692.010	2516965.335	714.4
P5BR-Orig.	829851.135	2517269.030	725.0
P5BR-sl. daylite	829705.160	2517222.820	CONNECTOR EL.807.0
P1BW	829701.320	2516925.760	735.9
PRCC3	830018.040	2517445.700	913.3
PRCC2	830034.680	2517185.460	913.4
PRCC1	830008.870	2516765.950	923.3
P4A	830078.650	2517007.900	801.7
P4AR	830014.140	2517072.240	969.6
P5A	830079.170	2517013.940	774.7

LEGEND

900	EXISTING CONTOUR 10FT INTERVAL
	EXISTING CONTOUR 2FT INTERVAL
	EXISTING ACCESS ROADS
	EXISTING FLY ASH SERVICE LINES
	BENCHMARK
	EXISTING TREELINE
△	EXISTING DEFORMATION MONUMENT NO. 29901-29958 (LAST TWO DIGITS SHOWN ON THE PLAN)
▲	NEW DEFORMATION MONUMENT
◆	NEW TILTMETER
⊕	EXISTING PIEZOMETER
⊕	NEW PIEZOMETER

GENERAL NOTES

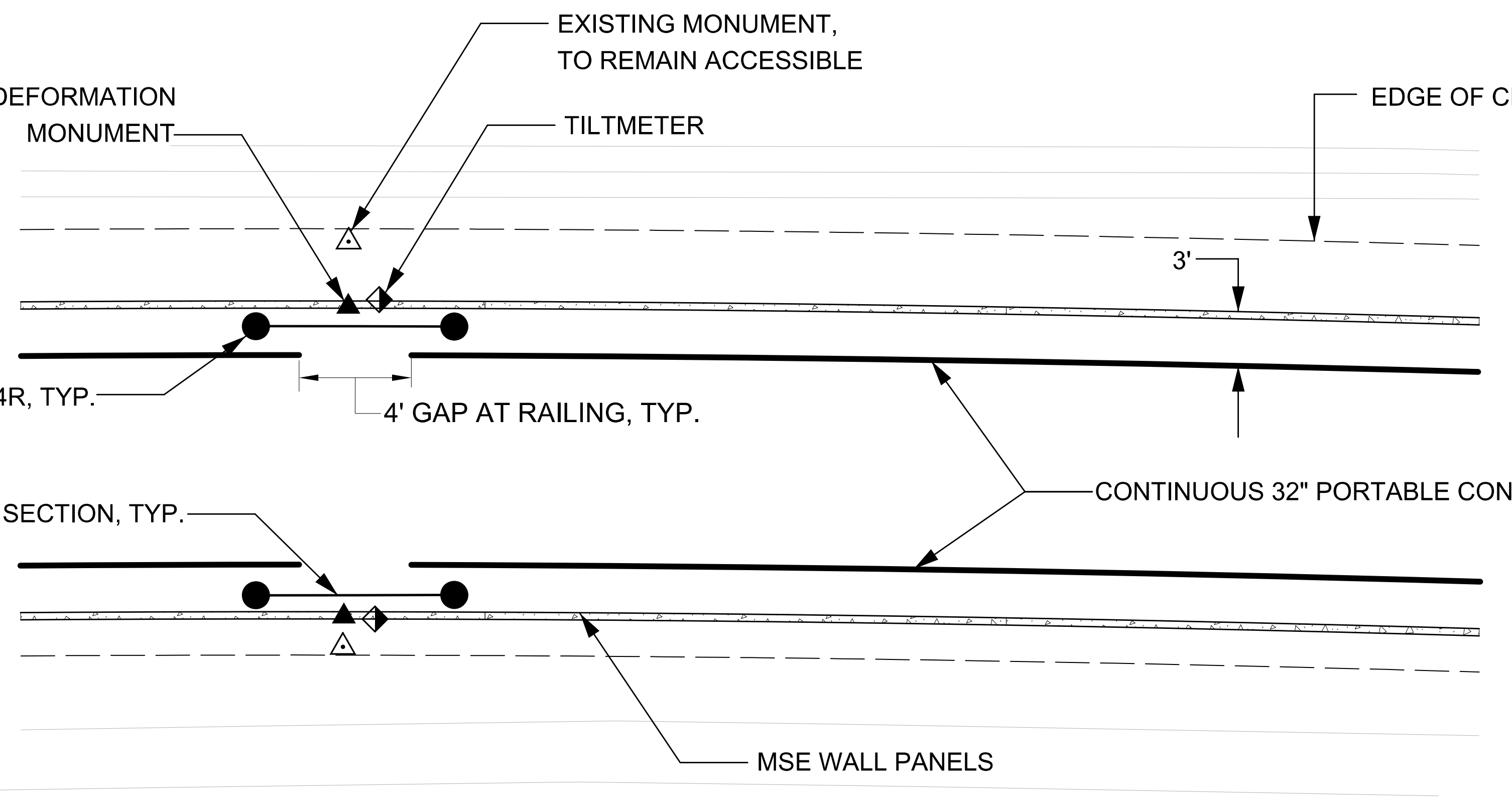
1. TOPOGRAPHIC MAPPING DEVELOPED FROM AERIAL SURVEY PERFORMED ON 3/5/2009 AND 1986 AS-BUILT DATA.



ELEVATION VIEW

SECTION VIEW

RAILING AND BARRIER DETAIL
N.T.S.



RAILING AND BARRIER PLAN
N.T.S.

S&ME, INC.
6190 ENTERPRISE COURT
DUBLIN, OH 43016
PHONE: 614-793-2226
FAX: 614-793-2410
www.smeinc.com

MICHAEL GILBERT ROWLAND
E-65559

Michael A. R

MICHAEL GILBERT ROWLAND	E-65559 NUMBER	JAN 16, 2013 DATE

PROJECT NUMBER:	011-11497-042	DRAWN BY:	EDV
DRAWING DATE:	9/28/12	ENGINEER:	MTR
LAST UPDATED:	1/16/13	APPROVED BY:	MGR
		SCALE:	AS NOTED

S&ME

DATE	NO.	DESCRIPTION	APPD.
1/16/13	C	REVISED PER ODNR COMMENTS REC'D 12/4/2012	MGR
9/12/12	B	REVISED PER ODNR COMMENTS REC'D 7/18/12	MGR
5/23/12	A	ISSUED FOR PERMIT INITIAL SUMMITTAL	MGR

REVISIONS

CARDINAL OPERATING COMPANY
CARDINAL PLANT
BRILLIANT OHIO

DAM RAISING
FLY ASH RETENTION DAM II
INSTRUMENTATION PLAN

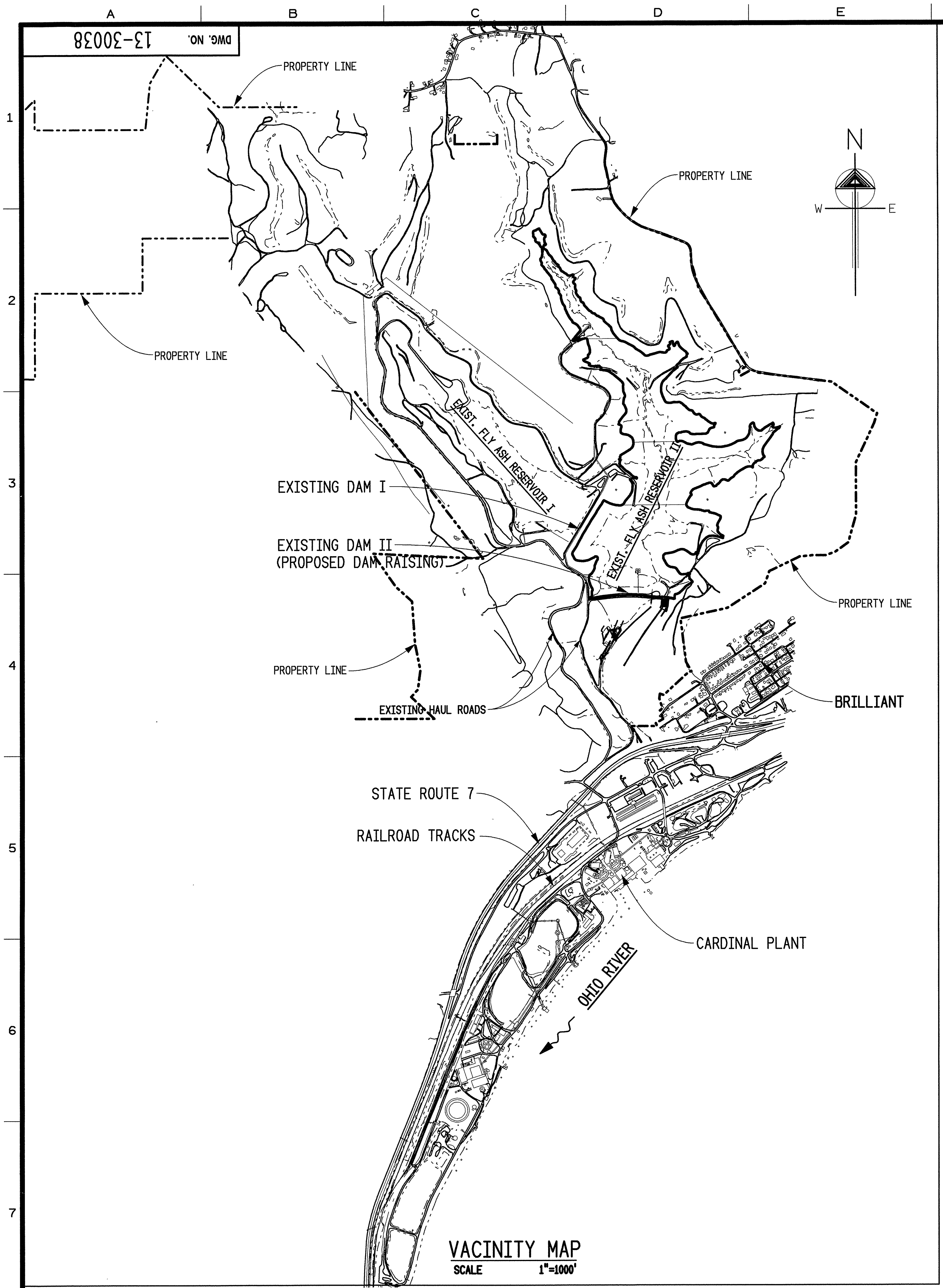
DWG. NO. 13-30098-C

ARCH	—	ELEC	—	MECH	—	STR	GS

SCALE: AS SHOWN
DR:
CH:
SQD SUPV:
DATE:
APPROVED BY
CIVIL ENGINEERING DIVISION

AMERICAN ELECTRIC POWER
1 RIVERSIDE PLAZA
COLUMBUS, OH 43215

DWG. NO. 13-30038



VACINITY MAP
SCALE 1"=1000'

PERMIT APPLICATION
FOR THE
DAM RAISING
OF
FLY ASH RETENTION DAM II
AT THE
CARDINAL PLANT
BRILLIANT, OHIO
SECTION 5, WELLS TOWNSHIP
JEFFERSON COUNTY

PREPARED FOR:
CARDINAL OPERATING COMPANY

PREPARED BY:
AMERICAN ELECTRIC POWER SERVICE CORP.
1 RIVERSIDE PLAZA, COLUMBUS, OHIO 43215
MARCH, 1997

DRAWING INDEX

DAM RAISING DRAWINGS:

DRAWING NUMBER	3	TITLE
13-30038	3	COVER SHEET
13-30039	1	RESERVOIR GENERAL ARRANGEMENT
13-30040	5	GRADING AND DRAINAGE PLAN
13-30041	5	PROFILE AND SECTIONS
13-30042	3	SECTIONS AND DETAILS SHEET 1
13-30043	5	SECTIONS AND DETAILS SHEET 2
13-30044	1	FLY ASH DRAINAGE SHAFT PLAN, SECTIONS & DETAILS.
13-30045	1	DRAINAGE SHAFT ACCESS STAIRS PLAN, SECTIONS & DETAILS.
13-30046	2	SITE CONSTRUCTION PLOT PLAN
13-30047	2	STORMWATER POLLUTION PREVENTION PLAN.
13-30048	3	SLIDE REPAIR SECTIONS & DETAILS.
13-30049	2	SLIDE REPAIR GRADING PLAN.
13-30050	4	SLIDE REPAIR EXCAVATION PLAN.
13-30053	3	GROUT HOLES PLAN & SECTIONS.
13-30054	1	CRACKS / DEFORMATION DATA
13-32000	4	DRAINAGE SHAFT MASONRY & REINFORCING PLAN, SECTIONS & DETAILS
13-32001	4	DRAINAGE SHAFT & MANHOLES PLAN, SECTIONS & DETAILS.
13-32002	2	TRAINING WALLS PLAN, SECTIONS & DETAILS.
13-32003	2	ACCESS STAIRS & SKIMMER PLAN, SECTIONS & DETAILS.
13-32004	1	DRAINAGE SHAFT BULKHEAD, PLAN, SECTIONS & DETAILS.

REFERENCE DRAWINGS: (NOT FOR CONSTRUCTION)

13-3002	1	BORING AND TEST PIT LOCATION PLAN SHEET 1
13-3003	0	BORING AND TEST PIT LOCATION PLAN SHEET 2
13-3004	0	GEOLOGIC PROFILE N-S AXIS
13-3005	1	GEOLOGIC PROFILE E-W AXIS
13-3032	1	EXISTING SERVICE SPILLWAY & COFFERDAM-LAYOUT & GRADING PLAN

BRADLEY M KLUTE
3/31/00

ODNR, DIVISION OF WATER
CLASS 1 DAM

DATE	NO.	DESCRIPTION	APPD.
3/31/00	3	REVISED TO REFLECT AS-BUILT CONDITIONS. FINAL SUBMITTAL TO STATE Amanda Graphics	BMK
5/21/99	2	ADDED DWG. Nos. 13-30054 AG	HJB
6/24/98	1	ADDED DWG. Nos. 13-30047, 13-30048, 13-30049, 13-30050 & 13-30053. RGD	HJB
4/21/97	0	ISSUED FOR CONSTRUCTION.	HJB

REVISIONS

s:/cd/13/geo_site_hydro/30038.dgn

CARDINAL OPERATING COMPANY
CARDINAL PLANT
BRILLIANT OHIO

DAM RAISING
FLY ASH RETENTION DAM II

COVER SHEET

DWG. NO. 13-30038 - 3

SCALE: AS SHOWN
DR: RGD
CH: GFZ
ENGR. HJB
PROJ. ENGR.
DATE: 4/21/97

CIVIL ENGINEERING DIVISION

APPROVED BY H. Joseph Buhac

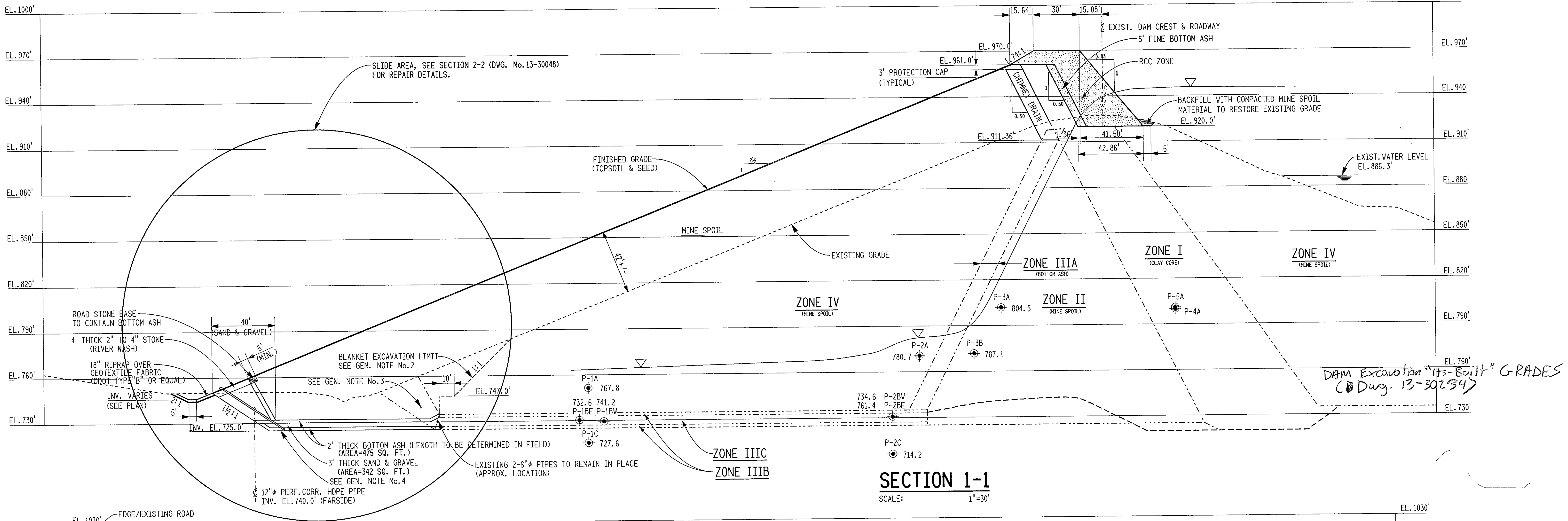
AEP AMERICAN ELECTRIC POWER
1 RIVERSIDE PLAZA
COLUMBUS, OH 43215

IGS1092

SYSTEM DATE-07 APR 2000
SYSTEM TIME-10:51:16
15th FLR. CED

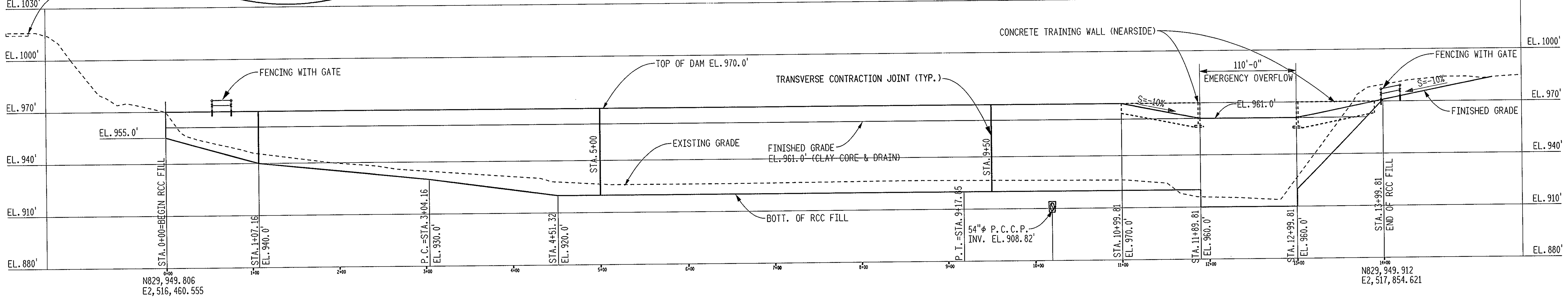
0465

DWG. NO. 13-30041



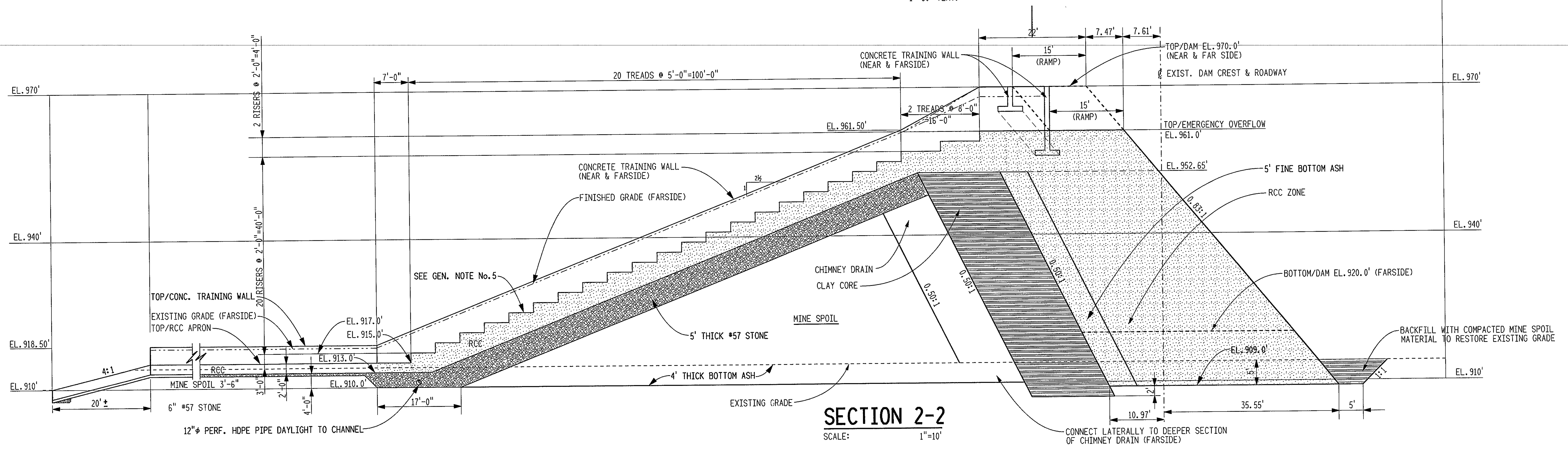
SECTION 1-1

SCALE: 1"=30'



℄ PROFILE - EARTH FILL/RCC DAM

SCALES: 1"=60' HORIZ. 1"=30' VERT.



SECTION 2-2

SCALE: 1"=10'

GENERAL NOTES

1.- FOR SECTIONS LOCATION, SEE DWG. No. 13-30040.

2.- LIMIT WIDTH OF EXCAVATION SECTIONS ALONG THE TOE OF THE DAM TO 20 FEET. PROVIDE SOIL SUPPORT AS REQUIRED.

3.- REMOVE EXISTING 12"⌀ PIPE. STOCKPILE REMOVED SAND & GRAVEL MATERIAL AND RE-USE ONLY A CLEAN PORTION OF MATERIAL TO EXTEND DRAINAGE BLANKET.

4.- REMOVE SOIL OVERBURDEN & CLEAN THE SURFACE OF THE ROCK.

5.- SEAL JOINTS BETWEEN RCC AND TRAINING WALL WITH JOINT FILLER.

REFERENCE DRAWINGS

13-30040 - FLY ASH DAM II RAISING GRADING & DRAINAGE PLAN.

DATE	NO.	DESCRIPTION	APPD.
	6	REVISED TO SHOW INSTALLATION DEPTH OF PNUEMATIC PIEZOMETER. AND DAM "AS-BUILT" EXCAVATION GRADES. Amanda Graphics	
3/31/00	5	REVISED TO REFLECT AS-BUILT CONDITIONS. FINAL SUBMITTAL TO STATE Amanda Graphics	BMK
5/21/99	4	AS-BUILT: REVISED TOE AREA OF SECTION 2-2 AG	HJB
6/22/98	3	EXTENDED CONC. TRAINING WALL REMOVED HIGHER & LOWER RCC STRENGTH FACING & ZONE. ADDED GEN. NOTE No. 5. 2' DIM ON SECT. "2-2" WAS 5'. RGD	HJB
5/20/98	2	REV. SECT. 2-2. INDICATED RCC APRON & BOTTOM ASH BLANKET DRAIN THICKNESS. RGD	HJB
4/23/98	1	REV. SECT. 1-1, SECT. 2-2 & ℄ PROFILE. RGD	HJB
4/21/97	0	ISSUED FOR CONSTRUCTION.	HJB

REVISIONS

s:/cd/13/geo_hydro_site/30041.dgn

CARDINAL OPERATING COMPANY

CARDINAL PLANT

BRILLIANT OHIO

FLY ASH DAM II RAISING
PROFILE & SECTIONS

DWG. NO. 13-30041-6

SCALE: AS NOTED
CIVIL ENGINEERING DIVISION
DR: Amanda Graphics
CH:
ENGR.
PROJ. ENGR.
DATE:
APPROVED BY

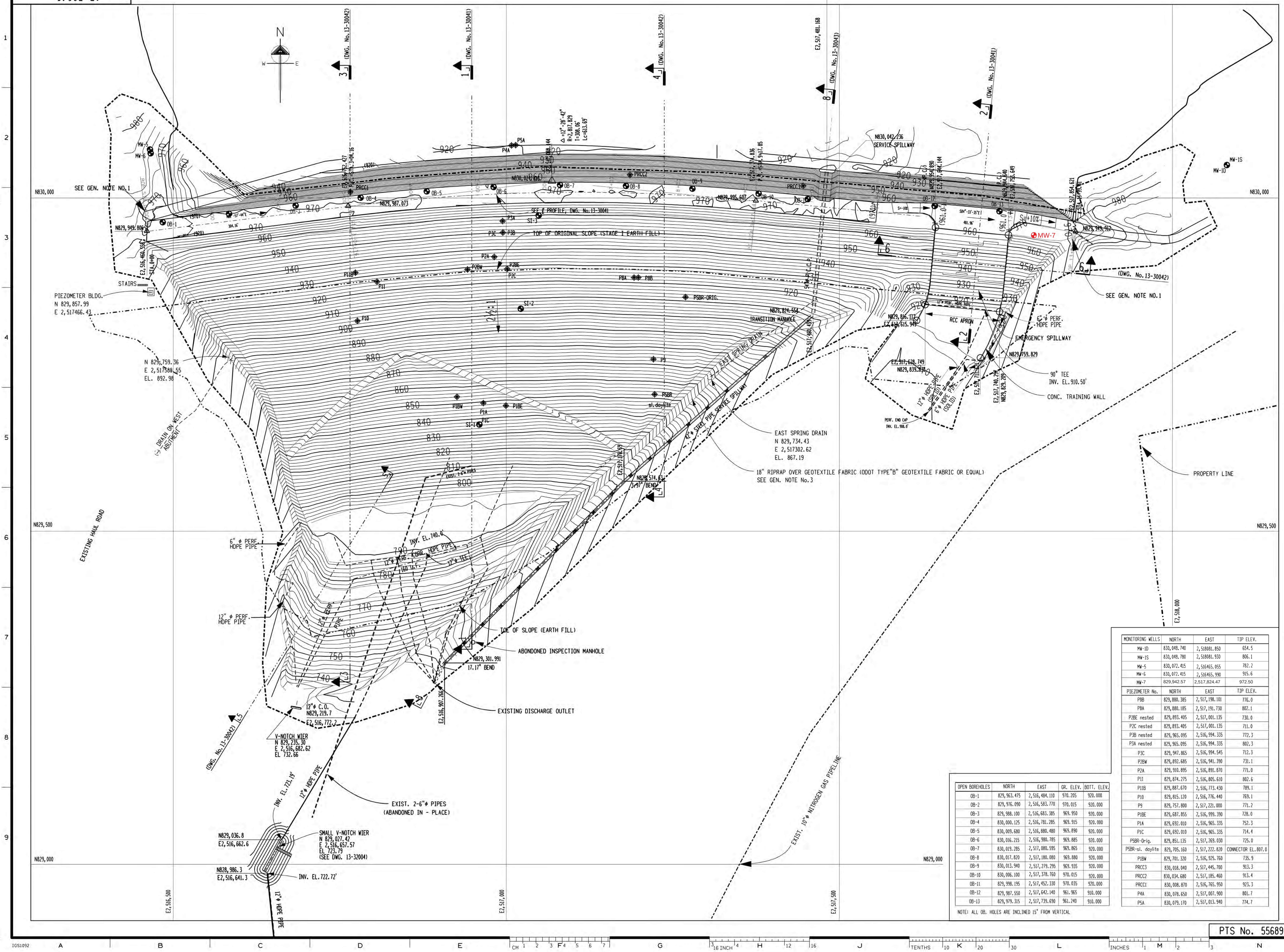
AEP AMERICAN ELECTRIC POWER
1 RIVERSIDE PLAZA
COLUMBUS, OH 43215

PTS No. 55689

SYSTEM DATE-17 MAY 2004
SYSTEM TIME-09:40:55
15th FLR. CED

ATTACHMENT D

INSTRUMENTATION LOCATION MAP



MONITORING WELLS	NORTH	EAST	TIP ELEV.
MW-1D	830,048.740	2,518081.850	654.5
MW-1S	830,048.780	2,518081.930	806.1
MW-5	830,072.415	2,516465.055	782.2
MW-6	830,072.415	2,516465.990	915.6
MW-7	829,942.57	2,517,824.47	972.50
PIEZOMETER No.	NORTH	EAST	TIP ELEV.
P8B	829,880.385	2,517,198.101	776.0
P8A	829,880.185	2,517,191.730	802.1
P2BE nested	829,893.405	2,517,001.135	730.0
P2C nested	829,893.405	2,517,001.135	711.0
P3B nested	829,965.095	2,516,994.335	772.3
P3A nested	829,965.095	2,516,994.335	802.3
P3C	829,947.865	2,516,994.545	712.3
P2BW	829,892.685	2,516,941.390	731.1
P2A	829,910.895	2,516,891.870	771.0
P11	829,874.275	2,516,805.610	802.6
P11B	829,887.670	2,516,773.430	789.1
P10	829,815.120	2,516,776.440	769.1
P9	829,757.800	2,517,221.000	771.2
P1BE	829,687.855	2,516,999.390	728.0
P1A	829,692.010	2,516,965.335	752.3
P1C	829,692.010	2,516,965.335	714.4
P5BR-Orig.	829,851.135	2,517,269.030	725.0
P5BR-sl. daylite	829,705.160	2,517,222.820	CONNECTOR EL.807.0
P1BW	829,701.320	2,516,925.760	735.9
PRCC3	830,018.040	2,517,445.700	913.3
PRCC2	830,034.680	2,517,185.460	913.4
PRCC1	830,008.870	2,516,765.950	923.3
P4A	830,078.650	2,517,007.900	801.7
P5A	830,079.170	2,517,013.940	774.7

OPEN BOREHOLES	NORTH	EAST	GR. ELEV.	BOTT. ELEV.
OB-1	829,963.475	2,516,484.110	970.205	920.000
OB-2	829,976.090	2,516,583.770	970.015	920.000
OB-3	829,988.100	2,516,683.385	969.950	920.000
OB-4	830,000.125	2,516,781.285	969.915	920.000
OB-5	830,009.680	2,516,880.480	969.890	920.000
OB-6	830,016.215	2,516,980.785	969.885	920.000
OB-7	830,019.285	2,517,080.595	969.865	920.000
OB-8	830,017.820	2,517,180.080	969.880	920.000
OB-9	830,013.940	2,517,279.295	969.935	920.000
OB-10	830,006.100	2,517,378.760	970.015	920.000
OB-11	829,998.195	2,517,452.330	970.035	920.000
OB-12	829,987.550	2,517,642.140	961.965	910.000
OB-13	829,979.315	2,517,739.690	961.240	910.000

NOTE: ALL OB. HOLES ARE INCLINED 15° FROM VERTICAL

PTS No. 55689

GENERAL NOTES

1.- EXCAVATE ROCK SURFACE TO ACHIEVE A RIGHT ANGLE CONTACT WITH THE RCC.

2.- THE SOIL OVERBURDEN ON BOTH THE RIGHT & LEFT ABUTMENTS SHALL BE STRIPPED. A 2' BOTTOM ASH DRAINAGE BLANKET SHALL BE PROVIDED OVER THE ENTIRE STRIPPED AREA. ANY SEEPAGE ZONES FOUND DURING STRIPPING SHALL BE DRAINED AS NECESSARY BY A FRENCH DRAIN DAYLIGHTING INTO GROIN DITCH.

3.- ADJUST LOCATION OF GROIN DITCH AS REQUIRED TO CLEAR PIPE SUPPORTS.

LEGEND - EXISTING

×925.5	SPOT ELEVATION
	INTERMEDIATE CONTOUR
----920----	INDEX CONTOUR
	DEPRESSION CONTOUR
	TREES AND TREELINE
	STRUCTURE AND BUILDING
	FENCE
	POLE
	ROADS
	EDGE OF WATER
	MANHOLES / CATCH BASIN
	POWER POLE
	PIPES
	TOWER

LEGEND - PROPOSED

+(970.0')	FIN. GRADE SPOT ELEV.
—920—	FIN. GRADE CONTOUR
— · · ► · · —	DRAINAGE DITCH
	INCLINED BORE HOLES
	VERTICAL BORE HOLES
	PIEZOMETER

REFERENCE DRAWINGS

13-30041 - FLY ASH DAM II RAISING PROFILE & SECTION.

13-30042 - FLY ASH DAM II RAISING SECTIONS & DETAILS SHT.1.

13-30043 - FLY ASH DAM II RAISING SECTIONS & DETAILS SHT.2.

DATE	NO.	DESCRIPTION	APPD.
3/31/00	5	REVISED TO REFLECT AS_BUILT CONDITIONS. FINAL SUBMITTAL TO STATE Amanda Graphics	BMK
5/21/99	4	AS_BUILT: REVISED TOPO, DRAIN PIPES , ADDED TABLES, PIEZOMETERS AND OPEN BORE HOLES. REMOVED MONITORING WELLS 4,3, 2D & 2S AG	HJB
6/22/98	3	REMOVED INTERMEDIATE CONTOURS. INDICATED CONCRETE TRAINING WALL & GEOTEXTILE FABRIC. RGD	HJB
5/20/98	2	DELETED DROP MANHOLE & REV. PIPE ALIGNMENT. RGD	HJB
4/23/98	1	REV. TOE OF DAM TO REFLECT SLIDE REPAIR. RELOCATED DROP MANHOLE & REV. PIPE BEND, 6°30' WAS 6°. ADDED UNDERDRAIN SYSTEM. RGD	HJB
4/21/97	0	ISSUED FOR CONSTRUCTION.	HJB

REVISIONS

s:/cd/13/geo_hydro_site/30040.dgn

CARDINAL OPERATING COMPANY

CARDINAL PLANT

BRILLIANT OHIO

FLY ASH DAM II RAISING GRADING & DRAINAGE PLAN

DWG. NO. 13-30040-5

SCALE: 1"=50'
DR: RGD
CH: GJZ
ENGR. HJB
PROJ. ENGR.
DATE: 4/21/97

CIVIL ENGINEERING DIVISION

APPROVED BY H. Joseph Buhac

AEP AMERICAN ELECTRIC POWER
1 RIVERSIDE PLAZA
COLUMBUS, OH 43215

IGS1092

SYSTEM DATE-26 JAN 2005
SYSTEM TIME-14:08:29
15th FL

ATTACHMENT E

HYDROLOGY AND HYDROLOGIC REPORT

APPENDIX C

HYDROLOGIC AND HYDRAULIC ANALYSIS

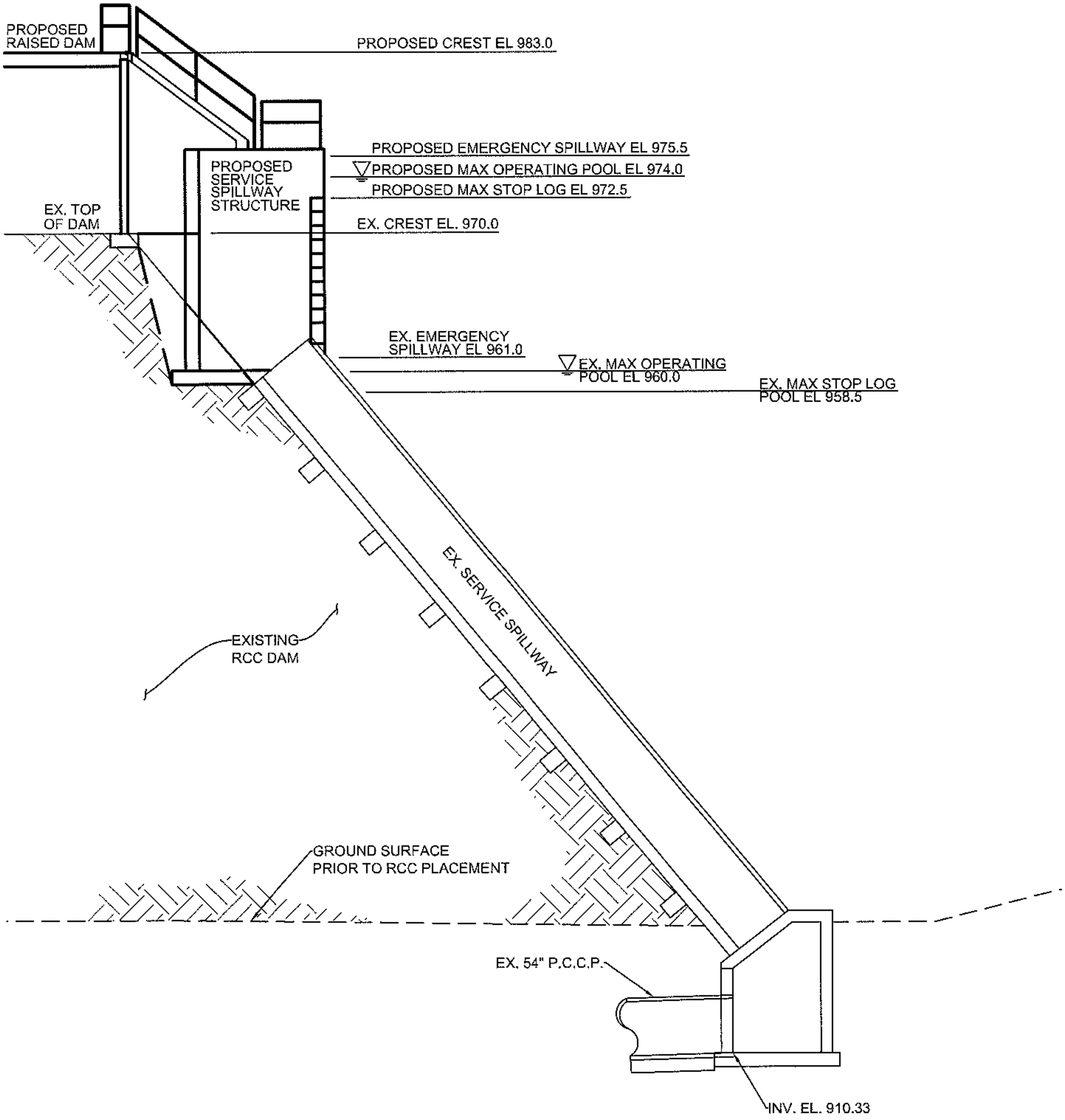
STATE OF OHIO
A.J. Smith
ALLEN J.
SMITH JR
E-71835
REGISTERED
PROFESSIONAL ENGINEER
9-28-12

Appendix C calculations checked and reviewed by: Stephen J Loskota

Stephen J. Loskota, P.E.
S&ME, Inc.

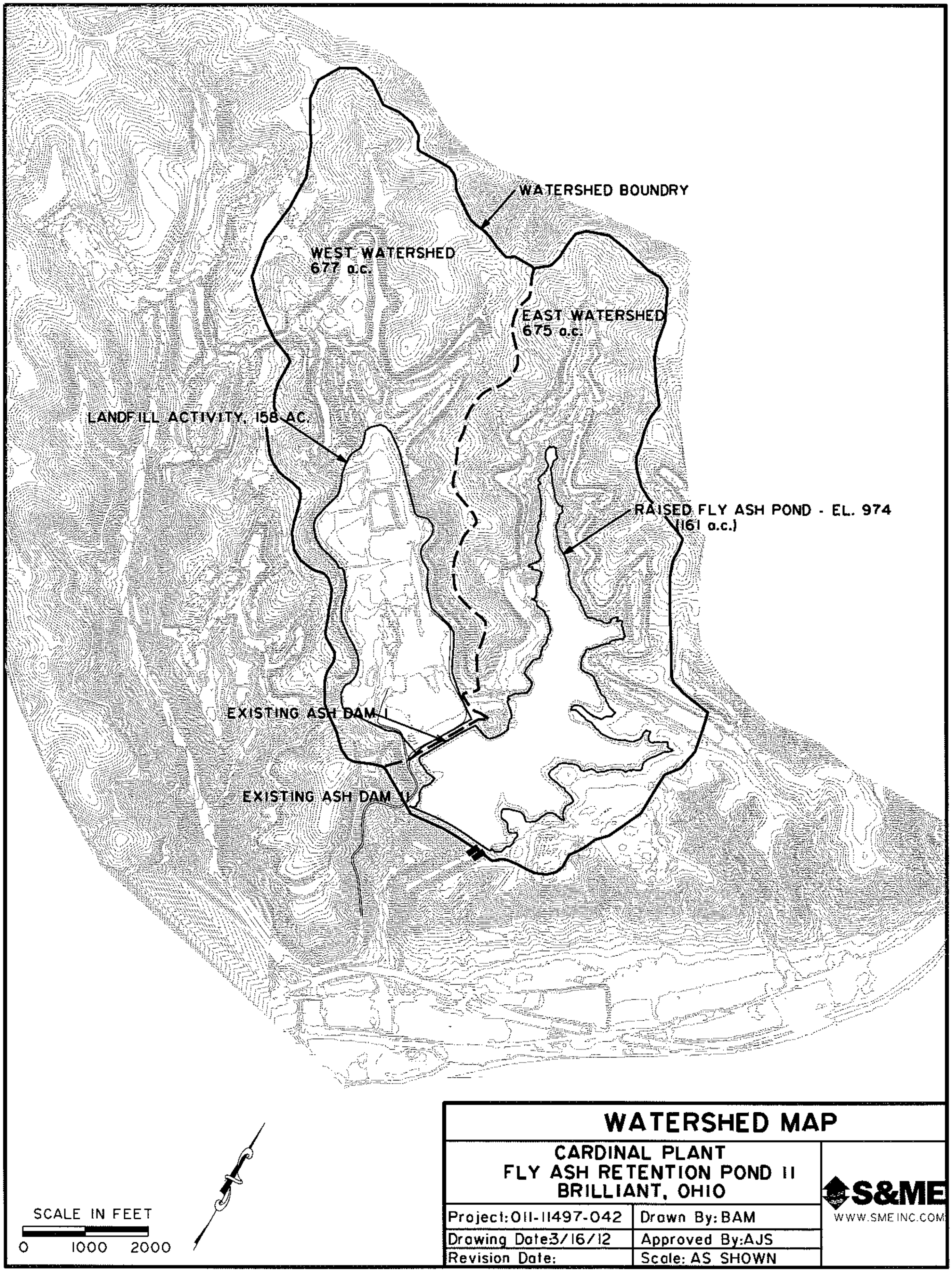
Appendix C calculations prepared by: A.J. Smith

A.J. Smith, P.E.
S&ME, Inc.



SERVICE SPILLWAY SCHEMATIC

NTS



WATERSHED MAP

CARDINAL PLANT
FLY ASH RETENTION POND II
BRILLIANT, OHIO

S&ME
WWW.SMEINC.COM

Project: 011-11497-042
Drawn By: BAM
Drawing Date: 3/16/12
Approved By: AJS
Revision Date:
Scale: AS SHOWN

PLATE 2

Storage Volume Calc
Cardinal Plant - Fly Ash Reservoir No. 2 Raising

Stage-storage for a dam raising design with a proposed crest at El. 983
Area calcuations above El. 970 (present crest) based on ground surface elevation contours
Ground surface elevations taken from 2-foot contour interval base map from aerial photo dated 3-5-2009

	Elev	area (ft^2) (ft^2)	area (ac)	ave area (ft^2)	height (ft)	vol (ft^3) (ft^3)	Vol (ac-ft)	Cum Vol (ac-ft)	*Total Vol (ac-ft)*
ex. pool	960	5,903,719	135.5					0	*9,800*
				5,973,619	2	11,947,238	274		
	962	6,043,519	138.7					274	*10,074*
				6,114,867	2	12,229,735	281		
	964	6,186,216	142.0					555	*10,355*
				6,259,072	2	12,518,144	287		
	966	6,331,928	145.4					842	*10,642*
				6,500,135	4	26,000,539	597		
ex. crest	970	6,668,342	153.1					1,439	*11,239*
				6,850,086	4	27,400,345	629		
prop. pool	974	7,031,831	161.4					2,068	*11,868*
				7,109,159	2	14,218,319	326		
prop. E.S.	975.5	*7,186,488*	165.0					2,395	*12,195*
		(interpolated)		7,212,264	0.5	3,606,132	83		
	976	7,238,040	166.2					2,478	*12,278*
				7,434,667	4	29,738,670	683		
	980	7,631,295	175.2					3,160	*12,960*
				7,820,348	3	23,461,043	539		
prop. crest	983	*8,009,400*	183.9					3,699	*13,499*
		(interpolated)		8,198,450	3	24,595,349	565		
	986	8,387,500	192.6					4,263	*14,063*

Cardinal FAD 2 - Stage-Storage Curves

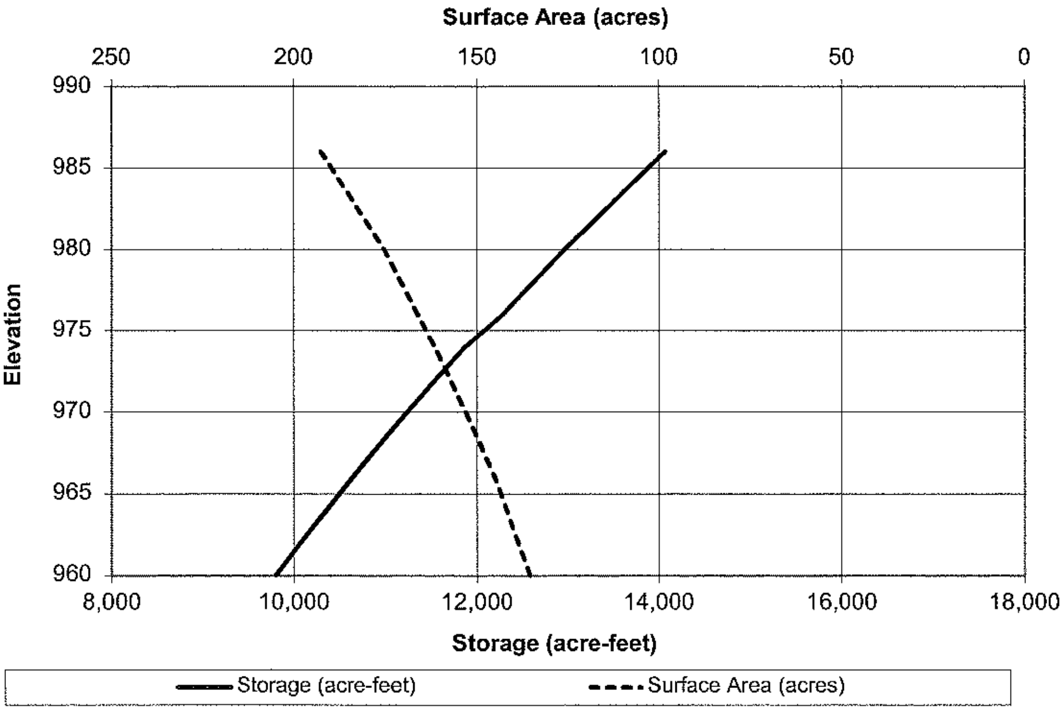


PLATE 3

CARDINAL FAD 2

CALCULATE COMPOSITE CN - WEST WATERSHED

Based off of Worksheet 2 in Appendix D of 210-VI-TR-55, Second Ed., June 1986

Soil Name/ Hydrologic Group	Cover Description	CN	Area (ac)	Product of CN x Area
C	Newly graded areas (no vegetation)	91	158.0	14,378
C	Woods, good	70	519.0	36,330

Totals	677.0	50,708

Composite CN	74.9

Use CN = **75**

Check FAR 1 Landfill Post-Development conditions:

From 2005 FAR 1 PTI by GeoSyntec, Post-Development conditions for the final cover system is a CN of **74**.
(see attached)

Therefore, use current landfill construction condition of CN = **75**.

- **Hydrologic Soil Groups:**
  Interim Conditions – Interim site conditions will include exposed temporary waste slopes. FGD waste material is assumed to exhibit similar characteristics to soils of Hydrologic Soils Group C.

  Post-Development - Soil used to construct the final cover system will consist of low permeability material, which will exhibit characteristics of Hydrologic Soils Group C.

- **Curve Number (CN):**
  Interim Conditions – For interim slopes, a CN of 91 is selected, the value recommended by SCS for hydrologic soil group C for "newly graded areas".

  Post-Development - For the final cover system, a curve number (CN) of 74 is used, the value recommended by SCS for hydrologic soil group C for "open spaces in good condition (grass cover > 75%)". A summary of runoff CN values provided by SCS [SCS, 1986] are provided in Table 2.

- **Time of Concentration T_c:** The T_c value represents the total time for stormwater runoff to travel from the hydraulically most distant point of a watershed or drainage area to a point of interest. Factors affecting T_c include surface roughness, channel shape and flow patterns, and slope. For this analysis the calculation of T_c evaluates the impact of three different types of stormwater runoff flow:

  - **sheet flow** – flow over plane surfaces, which is limited to a maximum length of 150 ft.;
  - **shallow concentrated flow** – after about 150 ft., sheet flow will begin to concentrate, but not necessarily defined in a specific channel; and
  - **channel flow** – flow that is confined to a defined channel section.

  The T_c value for a drainage area is the sum of the individual various travel time (T_t) values of the above flow types. The equations for calculating the T_t are presented below

  - **Sheet Flow:** $$T_t = \frac{0.007\ (nL)^{0.8}}{(P_2)^{0.5}\ s^{0.4}}$$

  - **Shallow Concentrated Flow:** $$T_t = \frac{L}{3{,}600\ V}$$

TABLE 2
Summary of Typical Runoff Curve Numbers

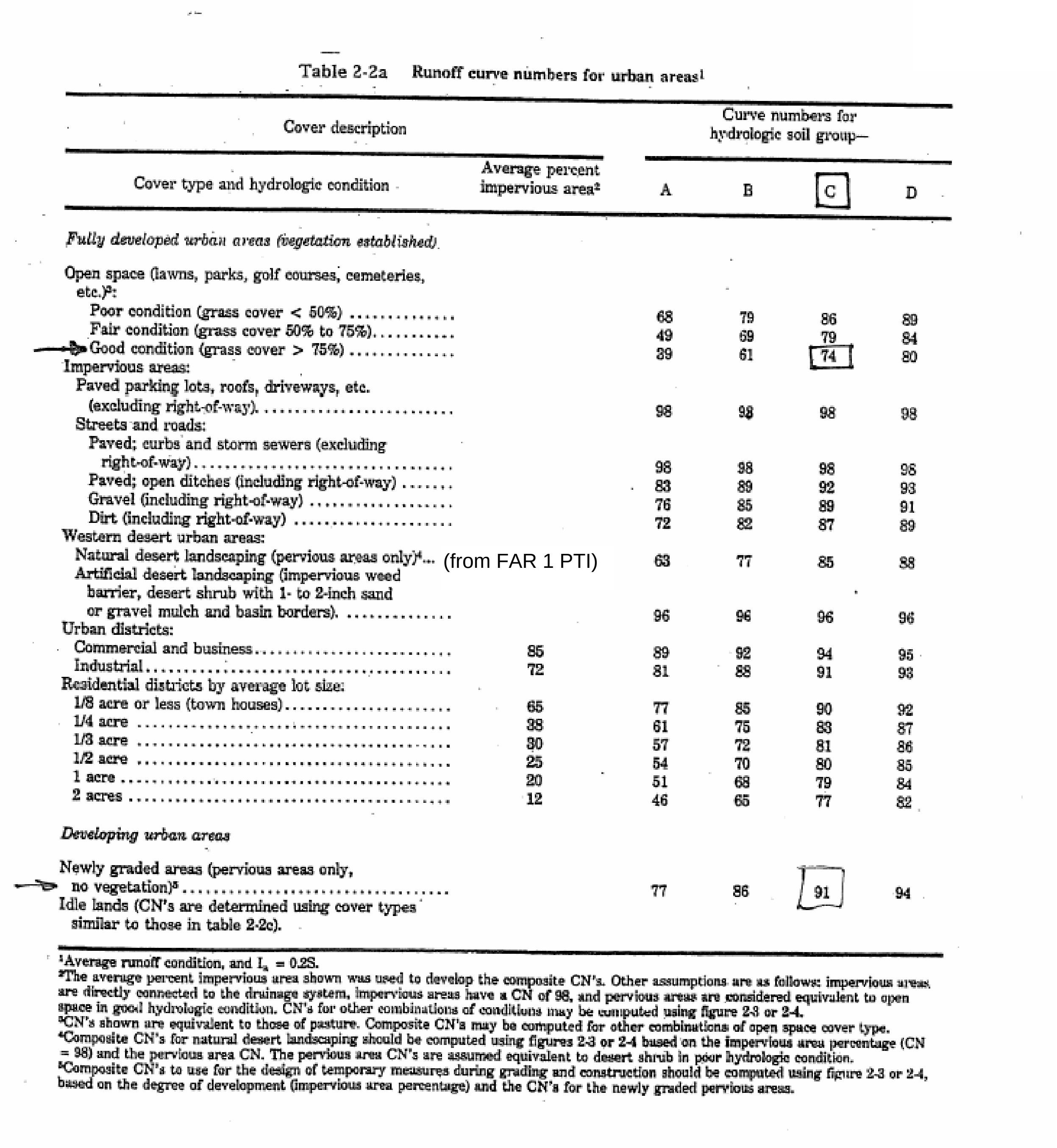


Table 2-2a Runoff curve numbers for urban areas[1]

Cover description		Curve numbers for hydrologic soil group—			
Cover type and hydrologic condition	Average percent impervious area[2]	A	B	C	D
Fully developed urban areas (vegetation established)					
Open space (lawns, parks, golf courses, cemeteries, etc.)[3]:					
Poor condition (grass cover < 50%)		68	79	86	89
Fair condition (grass cover 50% to 75%)		49	69	79	84
Good condition (grass cover > 75%)		39	61	74	80
Impervious areas:					
Paved parking lots, roofs, driveways, etc. (excluding right-of-way)		98	98	98	98
Streets and roads:					
Paved; curbs and storm sewers (excluding right-of-way)		98	98	98	96
Paved; open ditches (including right-of-way)		83	89	92	93
Gravel (including right-of-way)		76	85	89	91
Dirt (including right-of-way)		72	82	87	89
Western desert urban areas:					
Natural desert landscaping (pervious areas only)[4]	(from FAR 1 PTI)	63	77	85	88
Artificial desert landscaping (impervious weed barrier, desert shrub with 1- to 2-inch sand or gravel mulch and basin borders)		96	96	96	96
Urban districts:					
Commercial and business	85	89	92	94	95
Industrial	72	81	88	91	93
Residential districts by average lot size:					
1/8 acre or less (town houses)	65	77	85	90	92
1/4 acre	38	61	75	83	87
1/3 acre	30	57	72	81	86
1/2 acre	25	54	70	80	85
1 acre	20	51	68	79	84
2 acres	12	46	65	77	82
Developing urban areas					
Newly graded areas (pervious areas only, no vegetation)[5]		77	86	91	94
Idle lands (CN's are determined using cover types similar to those in table 2-2c).					

---

[1] Average runoff condition, and $I_a = 0.2S$.

[2] The average percent impervious area shown was used to develop the composite CN's. Other assumptions are as follows: impervious areas are directly connected to the drainage system, impervious areas have a CN of 98, and pervious areas are considered equivalent to open space in good hydrologic condition. CN's for other combinations of conditions may be computed using figure 2-3 or 2-4.

[3] CN's shown are equivalent to those of pasture. Composite CN's may be computed for other combinations of open space cover type.

[4] Composite CN's for natural desert landscaping should be computed using figures 2-3 or 2-4 based on the impervious area percentage (CN = 98) and the pervious area CN. The pervious area CN's are assumed equivalent to desert shrub in poor hydrologic condition.

[5] Composite CN's to use for the design of temporary measures during grading and construction should be computed using figure 2-3 or 2-4, based on the degree of development (impervious area percentage) and the CN's for the newly graded pervious areas.

Table 2-2a Runoff curve numbers for urban areas [1]

Cover description		Curve numbers for hydrologic soil group			
Cover type and hydrologic condition	Average percent impervious area [2]	A	B	C	D
Fully developed urban areas (vegetation established)					
Open space (lawns, parks, golf courses, cemeteries, etc.) [3]:					
Poor condition (grass cover < 50%)		68	79	86	89
Fair condition (grass cover 50% to 75%)		49	69	79	84
Good condition (grass cover > 75%)		39	61	74	80
Impervious areas:					
Paved parking lots, roofs, driveways, etc. (excluding right-of-way)		98	98	98	98
Streets and roads:					
Paved; curbs and storm sewers (excluding right-of-way)		98	98	98	98
Paved; open ditches (including right-of-way)		83	89	92	93
Gravel (including right-of-way)		76	85	89	91
Dirt (including right-of-way)		72	82	87	89
Western desert urban areas:					
Natural desert landscaping (pervious areas only) [4]		63	77	85	88
Artificial desert landscaping (impervious weed barrier, desert shrub with 1- to 2-inch sand or gravel mulch and basin borders)		96	96	96	96
Urban districts:					
Commercial and business	85	89	92	94	95
Industrial	72	81	88	91	93
Residential districts by average lot size:					
1/8 acre or less (town houses)	65	77	85	90	92
1/4 acre	38	61	75	83	87
1/3 acre	30	57	72	81	86
1/2 acre	25	54	70	80	85
1 acre	20	51	68	79	84
2 acres	12	46	65	77	82
Developing urban areas					
Newly graded areas (pervious areas only, no vegetation) [5]		77	86	91	94
Idle lands (CN's are determined using cover types similar to those in table 2-2c).					

---

[1] Average runoff condition, and $I_a = 0.2S$.

[2] The average percent impervious area shown was used to develop the composite CN's. Other assumptions are as follows: impervious areas are directly connected to the drainage system, impervious areas have a CN of 98, and pervious areas are considered equivalent to open space in good hydrologic condition. CN's for other combinations of conditions may be computed using figure 2-3 or 2-4.

[3] CN's shown are equivalent to those of pasture. Composite CN's may be computed for other combinations of open space cover type.

[4] Composite CN's for natural desert landscaping should be computed using figures 2-3 or 2-4 based on the impervious area percentage (CN = 98) and the pervious area CN. The pervious area CN's are assumed equivalent to desert shrub in poor hydrologic condition.

[5] Composite CN's to use for the design of temporary measures during grading and construction should be computed using figure 2-3 or 2-4 based on the degree of development (impervious area percentage) and the CN's for the newly graded pervious areas.

Table 2-2c Runoff curve numbers for other agricultural lands[1]

Cover description		Curve numbers for hydrologic soil group			
Cover type	Hydrologic condition	A	B	C	D
Pasture, grassland, or range—continuous forage for grazing.[2]	Poor	68	79	86	89
	Fair	49	69	79	84
	Good	39	61	74	80
Meadow—continuous grass, protected from grazing and generally mowed for hay.	—	30	58	71	78
Brush—brush-weed-grass mixture with brush the major element.[3]	Poor	48	67	77	83
	Fair	35	56	70	77
	Good	30[4]	48	65	73
Woods—grass combination (orchard or tree farm).[5]	Poor	57	73	82	86
	Fair	43	65	76	82
	Good	32	58	72	79
→ Woods.[6]	Poor	45	66	77	83
	Fair	36	60	73	79
	Good	30[4]	55	70	77
Farmsteads—buildings, lanes, driveways, and surrounding lots.	—	59	74	82	86

[1] Average runoff condition, and $I_a = 0.2S$.

[2] *Poor:* <50%) ground cover or heavily grazed with no mulch.
Fair: 50 to 75% ground cover and not heavily grazed.
Good: > 75% ground cover and lightly or only occasionally grazed.

[3] *Poor:* <50% ground cover.
Fair: 50 to 75% ground cover.
Good: >75% ground cover.

[4] Actual curve number is less than 30; use CN = 30 for runoff computations.

[5] CN's shown were computed for areas with 50% woods and 50% grass (pasture) cover. Other combinations of conditions may be computed from the CN's for woods and pasture.

[6] *Poor:* Forest litter, small trees, and brush are destroyed by heavy grazing or regular burning.
Fair: Woods are grazed but not burned, and some forest litter covers the soil.
Good: Woods are protected from grazing, and litter and brush adequately cover the soil.

NOAA Atlas 14, Volume 2, Version 3
Location name: Mingo Junction, Ohio, US*
Coordinates: 40.2666, -80.6517
Elevation: 1001ft*
* source: Google Maps

POINT PRECIPITATION FREQUENCY ESTIMATES

G.M. Bonnin, D. Martin, B. Lin, T. Parzybok, M.Yekta, and D. Riley

NOAA, National Weather Service, Silver Spring, Maryland

PF_tabular | PF_graphical | Maps_&_aerials

PF tabular

PDS-based point precipitation frequency estimates with 90% confidence intervals (in inches)[1]

Duration	Average recurrence interval(years)									
	1	2	5	10	25	50	100	200	500	1000
5-min	0.320 (0.283–0.363)	0.383 (0.339–0.435)	0.463 (0.409–0.526)	0.524 (0.462–0.594)	0.603 (0.530–0.683)	0.663 (0.581–0.750)	0.720 (0.628–0.814)	0.780 (0.678–0.880)	0.859 (0.742–0.969)	0.916 (0.789–1.03)
10-min	0.497 (0.440–0.565)	0.598 (0.529–0.680)	0.720 (0.636–0.817)	0.809 (0.713–0.916)	0.922 (0.811–1.04)	1.01 (0.881–1.14)	1.09 (0.946–1.23)	1.16 (1.01–1.31)	1.26 (1.09–1.42)	1.33 (1.15–1.50)
15-min	0.610 (0.540–0.692)	0.731 (0.647–0.831)	0.883 (0.781–1.00)	0.995 (0.877–1.13)	1.14 (1.00–1.29)	1.24 (1.09–1.41)	1.35 (1.18–1.52)	1.45 (1.26–1.64)	1.58 (1.36–1.78)	1.67 (1.44–1.88)
30-min	0.807 (0.714–0.916)	0.979 (0.866–1.11)	1.21 (1.07–1.37)	1.38 (1.22–1.57)	1.61 (1.41–1.82)	1.78 (1.56–2.01)	1.95 (1.70–2.20)	2.11 (1.84–2.39)	2.34 (2.02–2.63)	2.50 (2.15–2.82)
60-min	0.985 (0.872–1.12)	1.20 (1.06–1.37)	1.52 (1.34–1.72)	1.76 (1.55–1.99)	2.09 (1.83–2.36)	2.34 (2.05–2.65)	2.60 (2.27–2.94)	2.87 (2.49–3.24)	3.23 (2.79–3.64)	3.51 (3.02–3.96)
2-hr	1.13 (0.992–1.31)	1.37 (1.20–1.58)	1.74 (1.52–2.00)	2.02 (1.76–2.32)	2.41 (2.10–2.76)	2.72 (2.37–3.12)	3.05 (2.63–3.48)	3.39 (2.91–3.86)	3.85 (3.28–4.38)	4.22 (3.58–4.80)
3-hr	1.21 (1.07–1.40)	1.46 (1.29–1.69)	1.84 (1.62–2.14)	2.14 (1.88–2.48)	2.56 (2.24–2.96)	2.90 (2.53–3.34)	3.26 (2.83–3.75)	3.63 (3.13–4.18)	4.15 (3.55–4.77)	4.56 (3.87–5.25)
6-hr	1.44 (1.28–1.65)	1.73 (1.54–1.98)	2.17 (1.93–2.47)	2.52 (2.24–2.87)	3.02 (2.67–3.43)	3.43 (3.01–3.88)	3.86 (3.37–4.37)	4.32 (3.75–4.87)	4.96 (4.26–5.59)	5.49 (4.67–6.17)
12-hr	1.70 (1.53–1.90)	2.04 (1.83–2.28)	2.52 (2.26–2.82)	2.91 (2.61–3.25)	3.49 (3.11–3.88)	3.97 (3.52–4.39)	4.47 (3.94–4.93)	5.01 (4.39–5.51)	5.79 (5.01–6.35)	6.42 (5.51–7.01)
24-hr	2.02 (1.87–2.18)	2.41 (2.24–2.61)	2.94 (2.74–3.18)	3.39 (3.14–3.65)	4.00 (3.70–4.31)	4.51 (4.15–4.85)	5.04 (4.62–5.40)	5.59 (5.10–5.98)	6.37 (5.77–6.81)	6.99 (6.30–7.46)
2-day	2.37 (2.21–2.55)	2.82 (2.63–3.03)	3.42 (3.19–3.67)	3.90 (3.63–4.19)	4.57 (4.24–4.90)	5.11 (4.73–5.47)	5.67 (5.22–6.06)	6.24 (5.73–6.67)	7.04 (6.42–7.51)	7.66 (6.95–8.18)
3-day	2.53 (2.38–2.71)	3.01 (2.83–3.22)	3.63 (3.40–3.88)	4.12 (3.86–4.41)	4.81 (4.49–5.13)	5.36 (4.98–5.71)	5.92 (5.49–6.31)	6.50 (6.00–6.92)	7.29 (6.69–7.76)	7.92 (7.22–8.42)
4-day	2.70 (2.55–2.87)	3.21 (3.02–3.41)	3.84 (3.62–4.08)	4.35 (4.09–4.62)	5.05 (4.74–5.36)	5.61 (5.24–5.95)	6.18 (5.76–6.55)	6.76 (6.27–7.16)	7.55 (6.97–8.01)	8.17 (7.50–8.67)
7-day	3.25 (3.08–3.44)	3.84 (3.63–4.06)	4.55 (4.31–4.82)	5.12 (4.83–5.41)	5.88 (5.54–6.22)	6.48 (6.09–6.85)	7.09 (6.65–7.49)	7.71 (7.20–8.14)	8.52 (7.92–9.00)	9.14 (8.45–9.67)
10-day	3.74 (3.55–3.94)	4.41 (4.19–4.66)	5.18 (4.92–5.47)	5.78 (5.49–6.10)	6.59 (6.24–6.95)	7.22 (6.83–7.61)	7.85 (7.40–8.27)	8.47 (7.96–8.94)	9.29 (8.69–9.80)	9.91 (9.23–10.5)
20-day	5.24 (5.00–5.51)	6.16 (5.87–6.48)	7.13 (6.79–7.50)	7.89 (7.51–8.29)	8.89 (8.45–9.33)	9.64 (9.15–10.1)	10.4 (9.83–10.9)	11.1 (10.5–11.7)	12.0 (11.3–12.7)	12.7 (11.9–13.4)
30-day	6.58 (6.26–6.93)	7.70 (7.33–8.12)	8.83 (8.41–9.30)	9.72 (9.25–10.2)	10.9 (10.3–11.4)	11.7 (11.1–12.4)	12.6 (11.9–13.2)	13.4 (12.7–14.1)	14.4 (13.6–15.2)	15.2 (14.3–16.0)
45-day	8.42 (8.04–8.81)	9.82 (9.38–10.3)	11.1 (10.6–11.7)	12.1 (11.6–12.7)	13.4 (12.8–14.0)	14.4 (13.7–15.0)	15.3 (14.5–16.0)	16.1 (15.3–16.9)	17.2 (16.2–18.0)	17.9 (16.9–18.8)
60-day	10.1 (9.73–10.6)	11.8 (11.3–12.3)	13.3 (12.7–13.8)	14.4 (13.8–15.0)	15.8 (15.1–16.4)	16.8 (16.0–17.5)	17.7 (16.9–18.5)	18.6 (17.7–19.4)	19.6 (18.7–20.5)	20.3 (19.3–21.3)

[1] Precipitation frequency (PF) estimates in this table are based on frequency analysis of partial duration series (PDS).

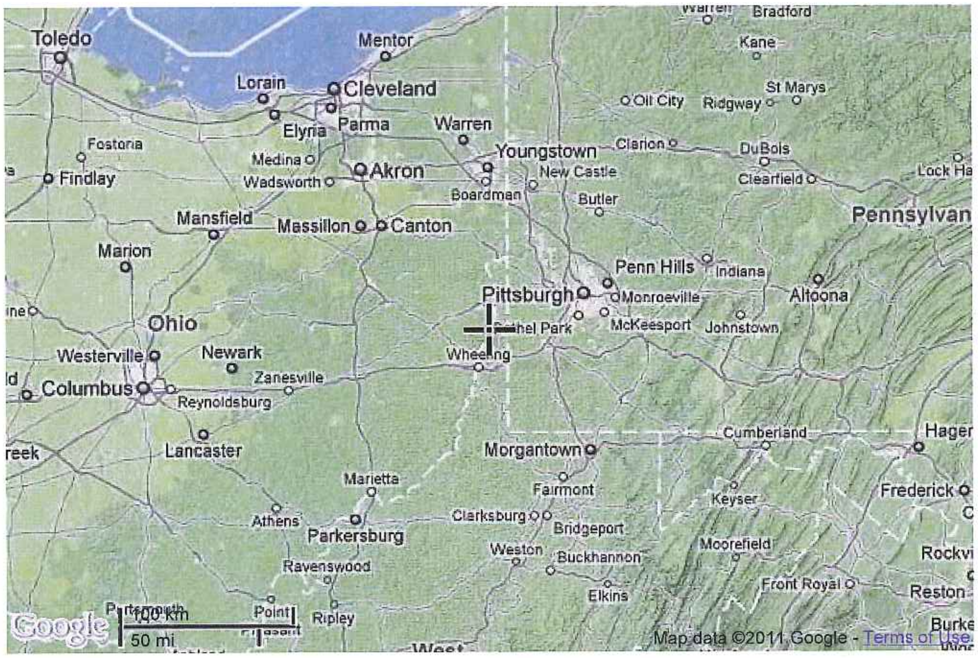
Numbers in parenthesis are PF estimates at lower and upper bounds of the 90% confidence interval. The probability that precipitation frequency estimates (for a given duration and average recurrence interval) will be greater than the upper bound (or less than the lower bound) is 5%. Estimates at upper bounds are not checked against probable maximum precipitation (PMP) estimates and may be higher than currently valid PMP values.

Please refer to NOAA Atlas 14 document for more information.

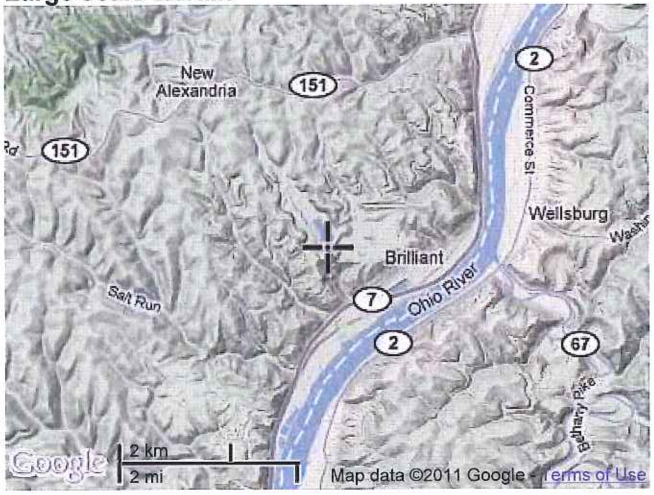
Back to Top

PF graphical

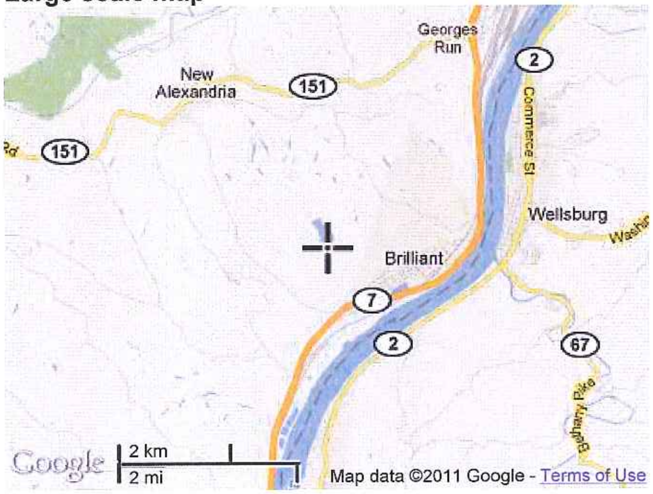
PLATE 9



Large scale terrain



Large scale map



Large scale aerial

PLATE 10

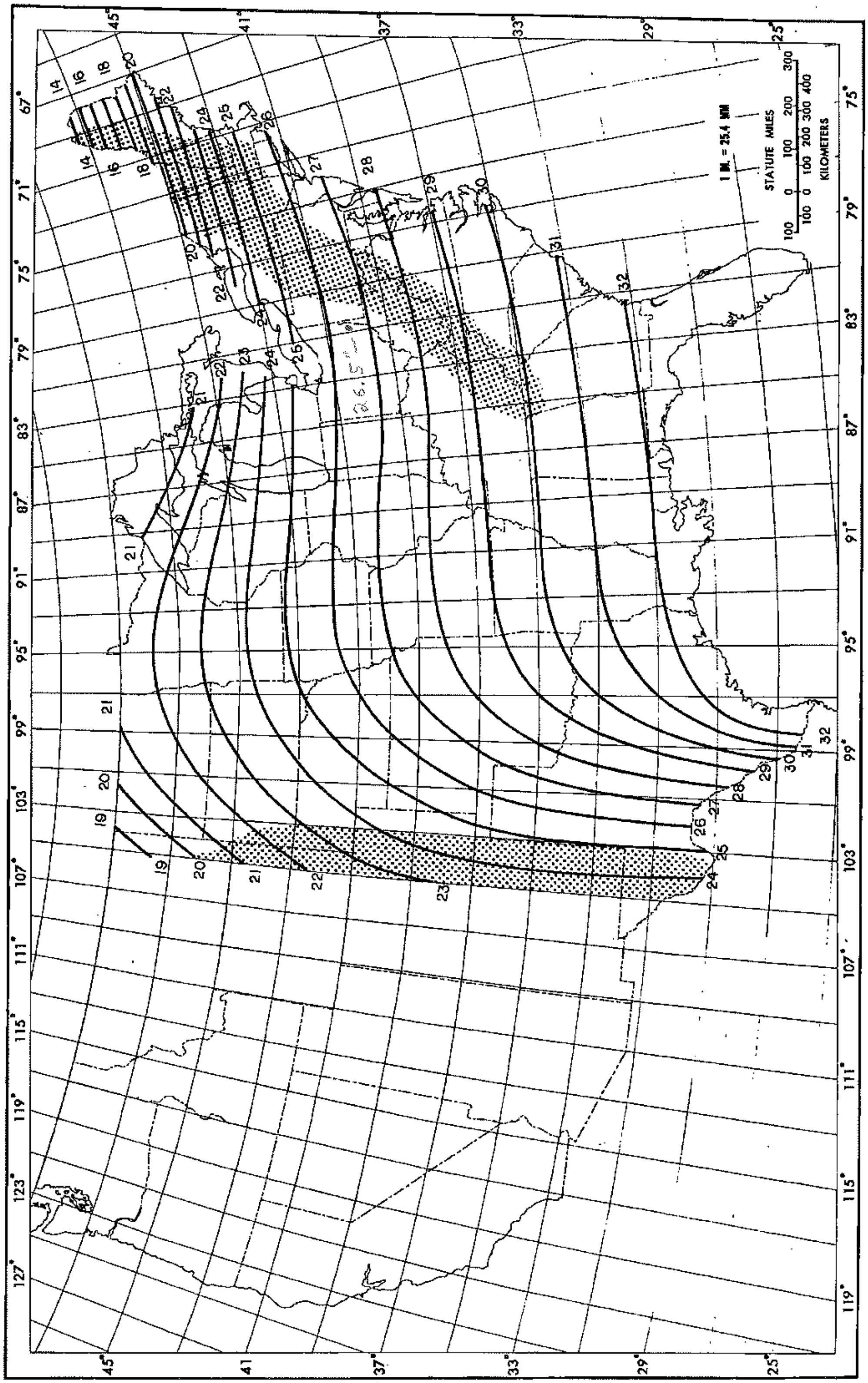


Figure 18.--All-season PMP (in.) for 6 hr 10 mi^2 (26 km^2).

From HMR 51

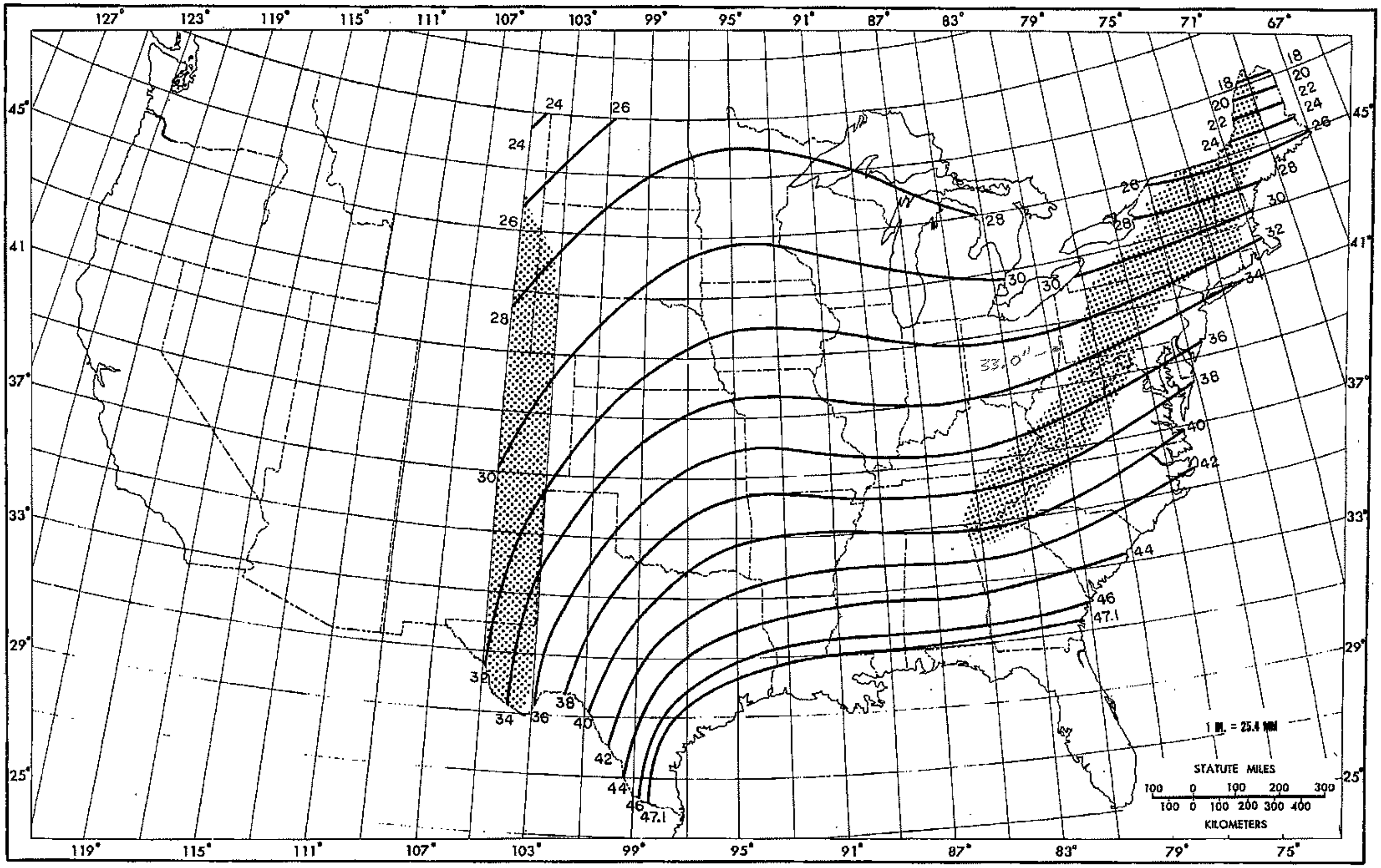


Figure 20.--All-season PMP (in.) for 24 hr 10 mi^2 (26 km^2).

From HMR 51

Cardinal FAD 2 Dam Raising Spillway Capacity - Proposed Top of Dam El. 983, ES width = 105'

	Lake Elevation	Box Struc. Weir Flow	Box Struc. Orifice Flow	Stop Log Weir Flow	**Structure Outflow**	Pipe Inlet Flow	Pressure Pipe Flow	**Control Outflow**	***Control Outflow***	E. Spillway Flow	**Total Outflow**	Control Type
	feet	cfs	cfs	cfs	**cfs**	cfs	cfs	**cfs**	***MGD***	cfs		
Top of Stop Log	972.50	-	-	0.0	**0.0**	-	-	**0.0**	***0.0***	-	**0.0**	-
	973.00	-	-	4.7	**4.7**	615.4	357.8	**4.7**	***3.0***	-	**4.7**	Stop Log Weir Flow
	973.50	-	-	13.3	**13.3**	618.0	358.2	**13.3**	***8.6***	-	**13.3**	Stop Log Weir Flow
Max Operating Pool	974.00	-	-	24.5	**24.5**	620.5	358.6	**24.5**	***15.8***	-	**24.5**	Stop Log Weir Flow
	974.50	-	-	37.7	**37.7**	623.0	358.9	**37.7**	***24.3***	-	**37.7**	Stop Log Weir Flow
	975.00	-	-	52.7	**52.7**	625.6	359.3	**52.7**	***34.0***	-	**52.7**	Stop Log Weir Flow
Emergency Spillway	975.50	-	-	69.2	**69.2**	628.1	359.7	**69.2**	***44.7***	0.0	**69.2**	Stop Log Weir Flow
	976.27	12.3	63.7	87.2	**99.5**	631.9	360.3	**99.5**	***64.3***	200.0	**299.5**	Stop Log Weir Flow
	976.89	73.6	115.7	87.2	**160.8**	635.0	360.8	**160.8**	***103.9***	500.0	**660.8**	Stop Log Weir Flow
	977.70	194.3	159.9	87.2	**247.1**	638.9	361.4	**247.1**	***159.7***	1000.0	**1247.1**	Stop Log Weir Flow
	978.37	319.8	188.8	87.2	**276.0**	642.2	361.9	**276.0**	***178.4***	1500.0	**1776.0**	Stop Log Weir Flow
	978.98	450.9	211.7	87.2	**298.9**	645.2	362.3	**298.9**	***193.2***	2000.0	**2298.9**	Stop Log Weir Flow
	980.06	717.0	247.1	87.2	**334.3**	650.4	363.2	**334.3**	***216.1***	3000.0	**3334.3**	Stop Log Weir Flow
	981.02	985.8	274.8	87.2	**362.0**	655.0	363.9	**362.0**	***233.9***	4000.0	**4362.0**	Stop Log Weir Flow
	981.90	1256.1	297.9	87.2	**385.1**	659.2	364.5	**364.5**	***235.6***	5000.0	**5364.5**	Pressure Pipe Flow
	982.32	1392.5	308.3	87.2	**395.5**	661.2	364.9	**364.9**	***235.8***	5500.0	**5864.9**	Pressure Pipe Flow
	982.73	1530.2	318.1	87.2	**405.4**	663.2	365.2	**365.2**	***236.0***	6000.0	**6365.2**	Pressure Pipe Flow
Top of Dam = 983.0	983.00	1623.2	324.5	87.2	**411.7**	664.4	365.4	**365.4**	***236.1***			

Cardinal FAD 2 Stop Logs Weir Rating
Weir Flow

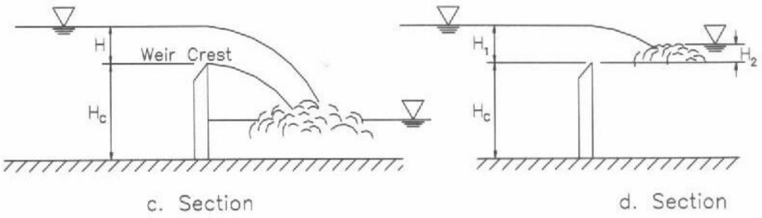
$$Q = C_{SCW} L H^{\frac{3}{2}}$$

$$C_{SCW} = 3.27 + 0.4\left(\frac{H}{H_c}\right)$$

for $H/H_c < 0.3$, C_{SCW} becomes 3.33

***L*= 4.00**
***g*= 32.2**
***Crest Elevation*= 972.5**

Elevation	H	Q
972.50	0.00	0.0
973.00	0.50	4.7
973.50	1.00	13.3
974.00	1.50	24.5
974.50	2.00	37.7
975.00	2.50	52.7
975.50	3.00	69.2
976.00	3.50	87.2



c. Section d. Section

Figure 8-13. Sharp crested weirs.

Reference:
FHWA-SA-96-078
Urban Drainage Design Manual
Hydraulic Engineering Circular 22
November, 1996

Cardinal FAD 2 Existing Spillway Pipe Rating
Pipe Inlet Control

$$Q = CA\sqrt{2gh_1}$$

for C=0.62 orifice equation becomes:

$$Q = 3.91D^2\sqrt{h_1}$$

d= 54.000 INCHES 54" PCCP

Invert Elevation = 910.33

Headwater Elevation (ft.)	Orifice	
	Discharge (cfs)	Velocity (ft/s)
972.50	*612.9*	0.0
973.00	*615.4*	38.7
973.50	*618.0*	38.9
974.00	*620.5*	39.0
974.50	*623.0*	39.2
975.00	*625.6*	39.4
975.50	*628.1*	39.5
976.27	*631.9*	39.8
976.89	*635.0*	39.9
977.70	*638.9*	40.2
978.37	*642.2*	40.4
978.98	*645.2*	40.6
980.06	*650.4*	40.9
981.02	*655.0*	41.2
981.90	*659.2*	41.5
982.32	*661.2*	41.6
982.73	*663.2*	41.7
983.00	*664.4*	41.8

Reference:
FHWA-SA-96-078
Urban Drainage Design Manual
Hydraulic Engineering Circular 22
November, 1996

Cardinal FAD 2 Existing Spillway Pipe Rating
Pressure Pipe Flow Computed with the Energy Equation
(from inlet to outlet)

Manning's n= 0.015
Inlet Invert: 910
Outlet Invert (z_2): 736
Entrance Coefficent K_e= 0.9
Outlet Coefficent K_o= 1.0
MH Coefficent K_{MH}= 0.5
Bends Coefficent K_b= 0.8
Pipe Diameter in inches= 42
Pipe Diameter in feet (D)= *3.50*
Pipe Eq. Length in feet (L)= 852
Darcy-Weisbach *f* = *0.027*

Assuming Free Outlet (TW=El. 739.5):

Headwater Elevation (z_1) (ft)	**Outlet Velocity (ft/s)**	**Outlet Flow Rate (ft^3/s)**
972.50	37.1	357.4
973.00	37.2	357.8
973.50	37.2	358.2
974.00	37.3	358.6
974.50	37.3	358.9
975.00	37.3	359.3
975.50	37.4	359.7
976.27	37.4	360.3
976.89	37.5	360.8
977.70	37.6	361.4
978.37	37.6	361.9
978.98	37.7	362.3
980.06	37.7	363.2
981.02	37.8	363.9
981.90	37.9	364.5
982.32	37.9	364.9
982.73	38.0	365.2
983.00	38.0	365.4

The Darcy-Weisbach friction factor is related to Manning's n through the following equation:

$$f = \frac{185 \quad n^{2}}{D^{\frac{1}{3}}}$$

The Energy Equation is:

$$\frac{p_1}{\gamma}+\frac{v_1^2}{2g}+z_1=\frac{p_2}{\gamma}+\frac{v_2^2}{2g}+z_2+\sum h_L$$

Where:

$$\sum h_L=\frac{v^2}{2g}\left(f\frac{L}{D}+K_e+K_o+K_b\right)$$

Because p_1, v_1 and p_2 all are equal to 0 the energy equation becomes:

$$z_1-z_2=\frac{v^2}{2g}+\frac{v^2}{2g}\left(f\frac{L}{D}+K_e+K_o+K_b\right)$$

Solving for v gives:

$$v=\sqrt{\frac{2g(z_1-z_2)}{\left(1+\left(f\frac{L}{D}+K_e+K_o+K_b\right)\right)}}$$

Determine flow rate Q by:

$$Q = VA$$

Cardinal FAD 2 Vertical Box Structure Overflow Rating
Weir Flow

$$Q = C_{SCW} L H^{\frac{3}{2}}$$

$$C_{SCW} = 3.27 + 0.4\left(\frac{H}{H_c}\right)$$

for $H/H_c < 0.3$, C_{SCW} becomes 3.33

***Size*= 5'-8" x 7'-6 " inside dimensions**

***L*= 26.3**

***g*= 32.2**

***Crest Elevation*= 976.0**

Elevation	H	Q
976.00	0.00	0.0
976.27	0.27	12.3
976.89	0.89	73.6
977.70	1.70	194.3
978.37	2.37	319.8
978.98	2.98	450.9
980.06	4.06	717.0
981.02	5.02	985.8
981.90	5.90	1256.1
982.32	6.32	1392.5
982.73	6.73	1530.2
983.00	7.00	1623.2

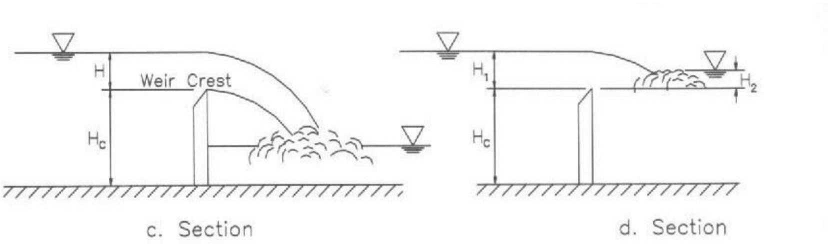


Figure 8-13. Sharp crested weirs.

Reference:
FHWA-SA-96-078
Urban Drainage Design Manual
Hydraulic Engineering Circular 22
November, 1996

Cardinal FAD 2 Vertical Box Structure Overflow Rating
Orifice Flow

$$Q = CA\sqrt{2gh_1}$$

Size= **5'-8" x 7'-6 " inside dimensions**
A= **42.5** **S.F.**
Grating % Open Area **60** **%**
Orifice Centroid Elevation = **976.0**

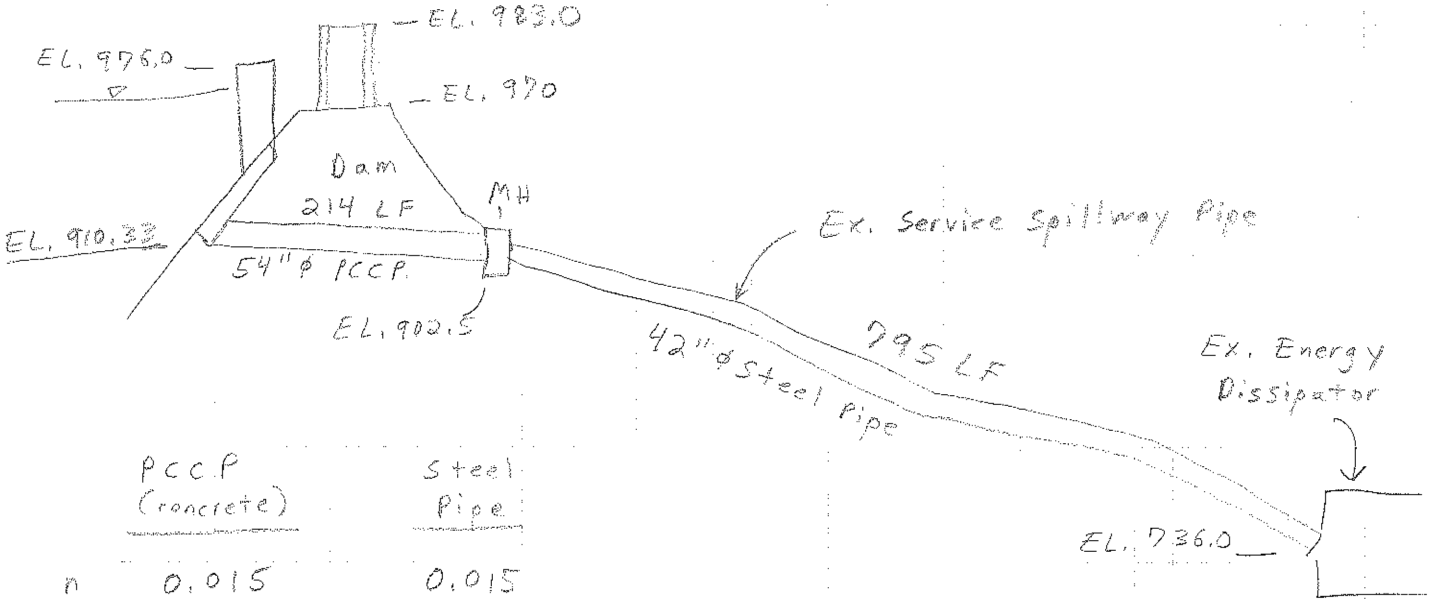
Headwater Elevation (ft.)	Orifice	
	Discharge (cfs)	Velocity (ft/s)
976.00	*0.0*	
976.27	*63.7*	1.5
976.89	*115.7*	2.7
977.70	*159.9*	3.8
978.37	*188.8*	4.4
978.98	*211.7*	5.0
980.06	*247.1*	5.8
981.02	*274.8*	6.5
981.90	*297.9*	7.0
982.32	*308.3*	7.3
982.73	*318.1*	7.5
983.00	*324.5*	7.6

Reference:
FHWA-SA-96-078
Urban Drainage Design Manual
Hydraulic Engineering Circular 22
November, 1996

S&ME
Cincinnati (513) 771-8471
Cleveland (216) 901-1000
Columbus (614) 793-2226

Project/Proposal No. ______ Calculated By AJS Date ______
Project/Proposal Name Cardinal Dam Checked By ______ Date ______
Subject Pressure Pipe Flow Sheet ______ of ______

Cardinal Dam - Extended Service Spillway



	PCCP (concrete)	Steel Pipe
n	0.015	0.015
D	54"	42"
f	0.025	0.027
L	214'	795'
L_e	57'	

$$f = \frac{185 n^2}{D^{1/3}}$$

Equivalent Length of 54" pipe to 42":

$$L_e = L\left(\frac{f}{f_e}\right)\left({}^{D_e}/_{D}\right)^5 \quad , \quad \frac{f}{f_e} = 0.93$$

$$= (214\ ft)(0.93)\left({}^{3.5}/_{4.5}\right)^5 = 57'$$

Outlet Control - Full Pipe Flow

$$\text{Headwater} = \text{Tailwater} + \frac{V^2}{2g}\left(1 + K_e + K_{MH} + K_{bends} + K_f + K_o\right)$$

$K_e = 0.9$
$K_o = 1.0$
$K_f = f\left(\frac{L}{D}\right) = 0.027\left(\frac{795+57}{3.5}\right) = 6.6$

$$HW = TW + \frac{V^2}{2g}\left(1 + 0.9 + 0.5 + 4(0.2) + 6.6 + 1.0\right)$$

$$HW - TW = \frac{V^2}{2g}(10.8)$$

Let TW = Crown of pipe ≈ 739.5

$$\therefore \left(\frac{(HW - 739.5)\,2g}{10.8}\right)^{1/2} = V \ ; \quad V = 2.44\,(HW - 739.5)^{1/2}$$

$A = 9.621\ ft^2$
$Q = VA$

- See spreadsheet for HW vs. Q, stop log weir flow [illegible] until ~ EL. 982

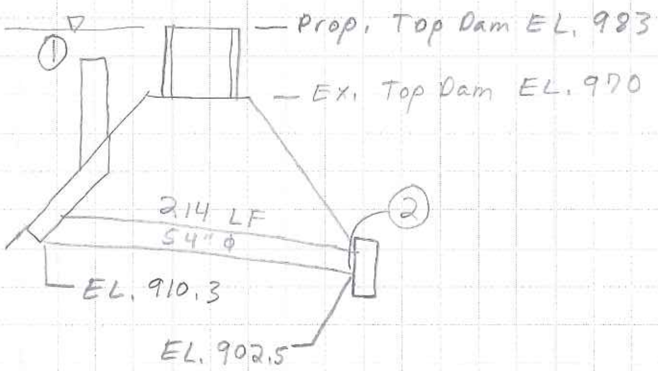
Project/Proposal No. 011-11497-042 Calculated By AJS Date 9-24-12
Project/Proposal Name Cardinal Checked By SJC Date 9/27/12
Subject Spillway Pipe Internal Pressure Sheet ______ of ______

Reference 2000 As-Buit Dwg. No. 13-30043-5 General Note 2

Check pressure in 54" P.C.C.P. Pipe is rated for 35 psi.

Weir inlet should control except for higher headwaters.

Check pressure with headwater at Top of Dam (EL. 983.0)



$Z_1 = 983.0$
$Z_2 = 902.5$
$80.5'$ ✓

Energy Equation:

$$\frac{p_1}{\gamma} + \frac{V_1^2}{2g} + Z_1 = \frac{p_2}{\gamma} + \frac{V_2^2}{2g} + Z_2 + \Sigma h_L \quad ; \quad p_1, V_1 = 0$$

$$\frac{p_2}{\gamma} = (Z_1 - Z_2) - \frac{V_2^2}{2g} - \Sigma h_L$$

$$\frac{p_2}{62.4\ lb/ft^3} = (983.0 - 902.5) - \frac{V_2^2}{64.4} - \Sigma h_L$$

$$p_2\ [lb/ft^2] = 80.5(62.4) - \frac{V_2^2}{1.03} - \Sigma h_L (62.4)$$

$$p_2\ [lb/in^2] = 34.9 - \frac{V_2^2}{148} - \frac{\Sigma h_L}{0.4}$$

$80.5' = 34.9$ psi ✓

$\therefore p_2 < 34.9$ psi with flow

34.9 psi < 35 psi ✓

S&ME

Cincinnati (513) 771-8471
Cleveland (216) 901-1000
Columbus (614) 793-2226

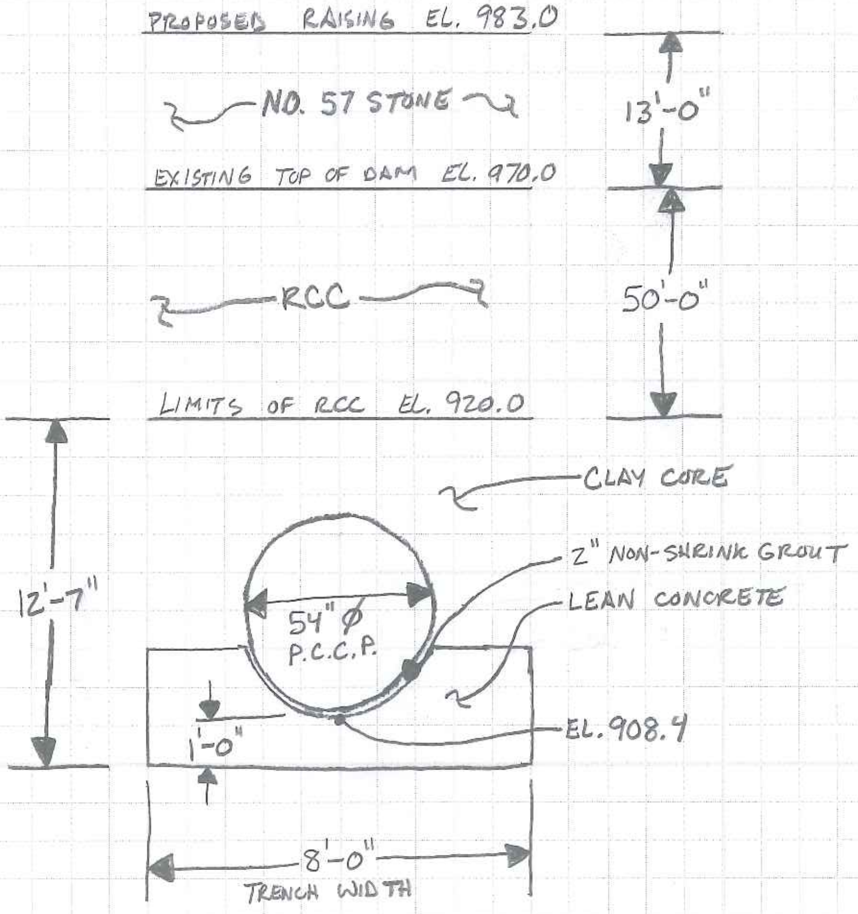
Project/Proposal No. 1176-12-004A Calculated By MRM Date 9/26/12
Project/Proposal Name CARDINAL FAD II RAISING Checked By SJL Date 9/27/12
Subject PIPE STRENGTH CALCULATIONS Sheet ______ of ______

TASK: DETERMINE SUITABILITY OF EXISTING 54"⌀ P.C.C.P SPILLWAY PIPE UNDER ADDITIONAL PROPOSED LOADING CONDITIONS.

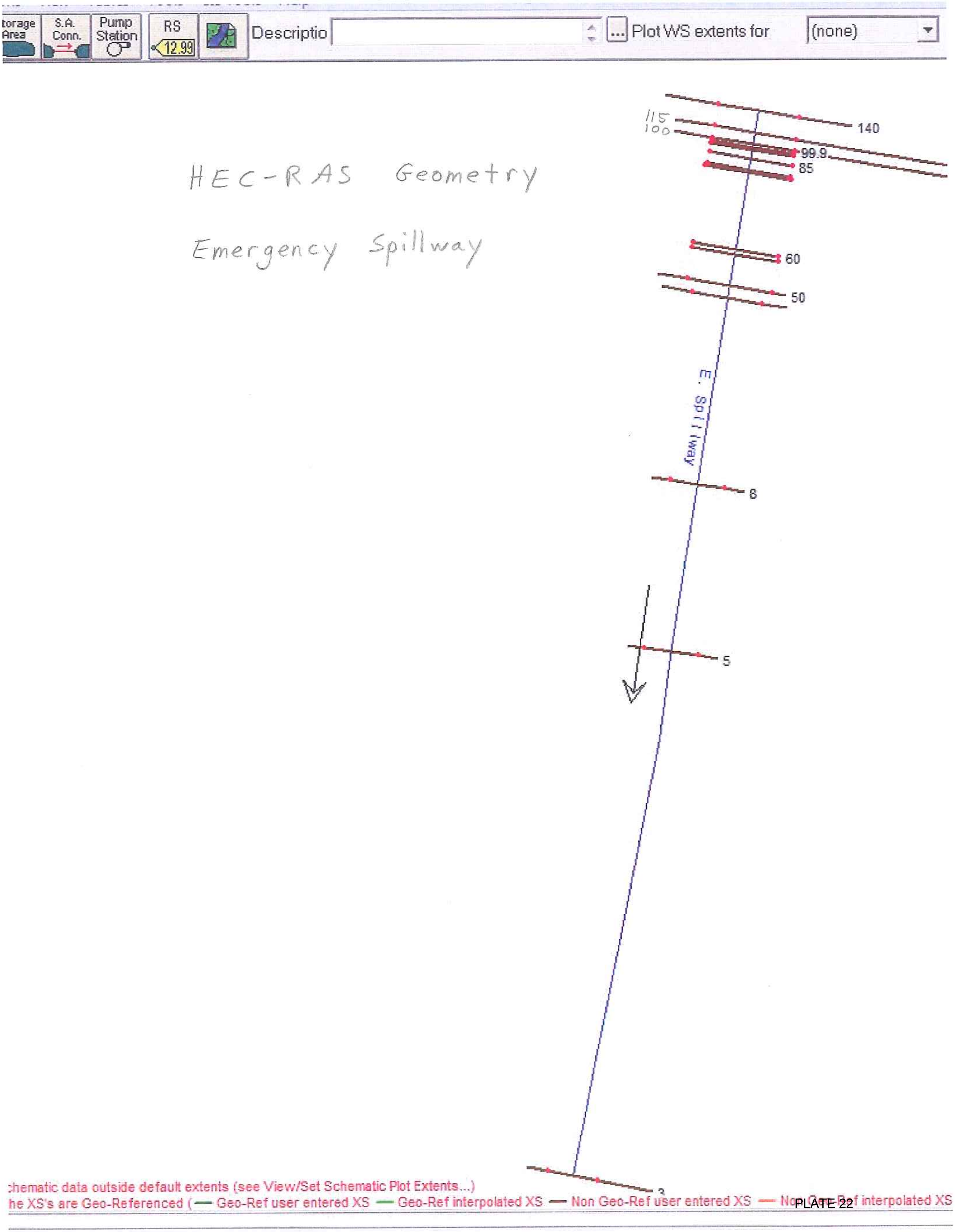
RESULTS:

COVER OVER SPILLWAY PIPE INCLUDING ADDITIONAL COVER = 74.6 FT. BASED ON DRAWING 13-30043-5, THE 54"⌀ SPILLWAY PIPE WAS DESIGNED TO HANDLE 80 FT SO OUR ADDITIONAL COVER STILL FALLS WITHIN THE PIPE STRENGTH CAPACITY.

SKETCH OF EXISTING VS PROPOSED (NOT TO SCALE)



EL. 983.0 (PROP. TOD)
− EL. 908.4 (INV. PIPE)
74.6 FT COVER



HEC-RAS Geometry

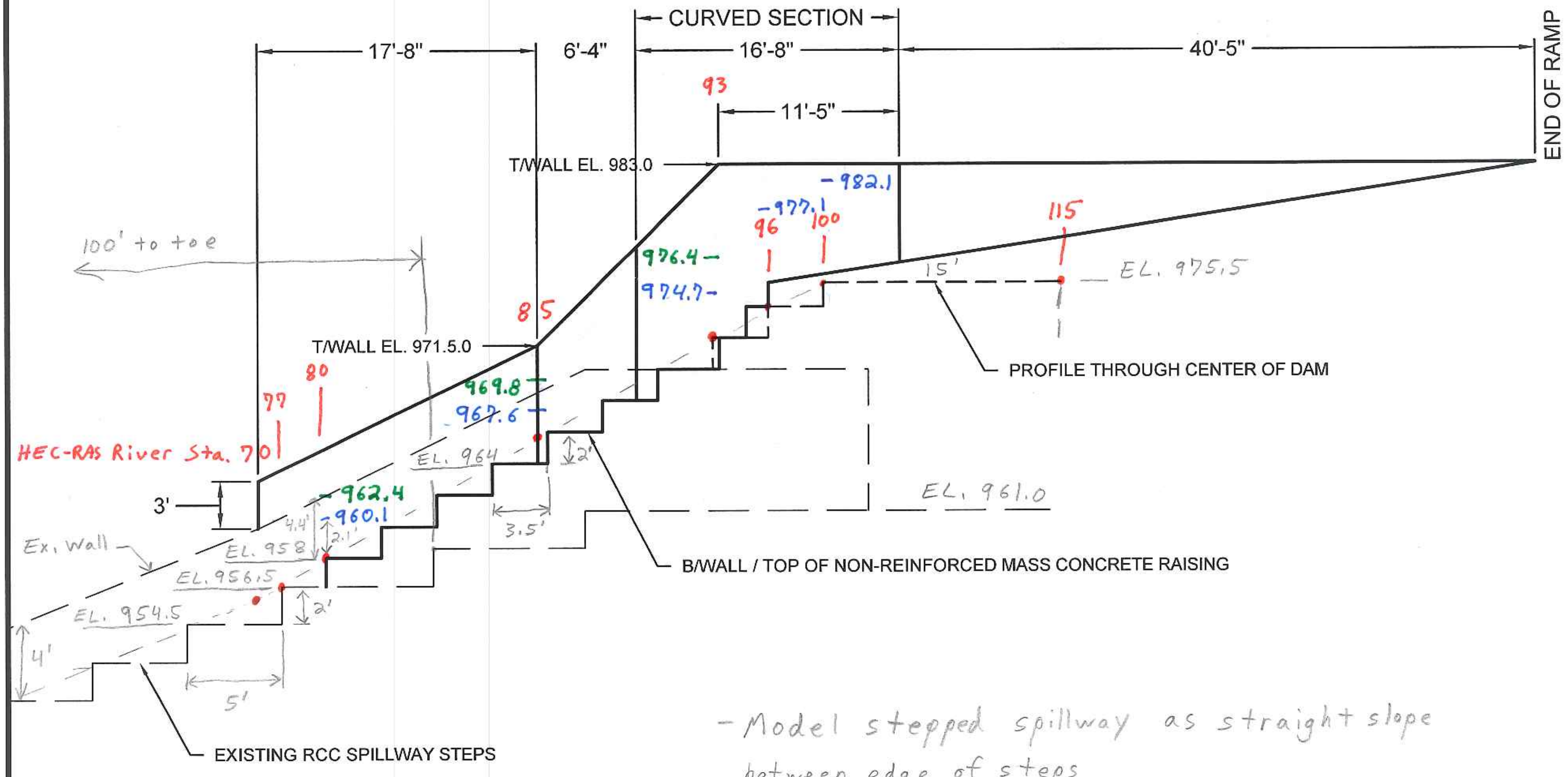
Emergency Spillway

chematic data outside default extents (see View/Set Schematic Plot Extents...)

he XS's are Geo-Referenced (— Geo-Ref user entered XS — Geo-Ref interpolated XS — Non Geo-Ref user entered XS — Non Geo-Ref interpolated XS

PLATE 22

HEC-RAS Cross-Section Stations on Proposed Emerg. Spillway



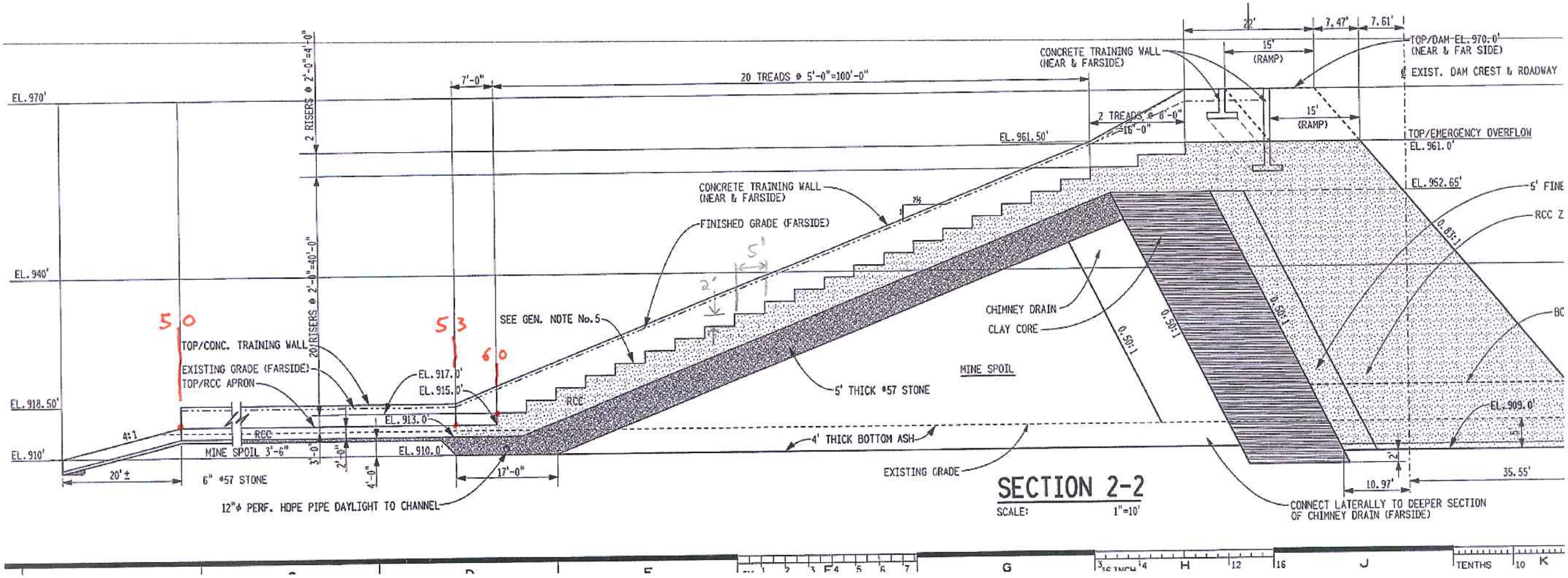
- Critical W.S. (~4.5' deep)
- Resultant W.S. (2.0 to 2.5' deep)

- Model stepped spillway as straight slope between edge of steps
- Use Manning's n = 0.07

RIGHT TRAINING WALL PROFILE

From 2000 As-Built
Dwg. No. 13-30041-6

HEC-RAS Cross-Section Stations on Existing Emerg. Spillway



HEC-RAS Plan: Plan 22   River: E. Spillway   Reach: 1

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
1	140	PF 1	200.00	940.00	976.27	940.28	976.27	0.000000	0.03	8703.65	240.00	0.00
1	140	PF 2	500.00	940.00	976.89	940.51	976.89	0.000000	0.07	8853.88	240.00	0.00
1	140	PF 3	1000.00	940.00	977.70	940.81	977.70	0.000000	0.13	9047.40	240.00	0.00
1	140	PF 4	1500.00	940.00	978.37	941.07	978.37	0.000000	0.19	9209.44	240.00	0.01
1	140	PF 5	2000.00	940.00	978.98	941.29	978.98	0.000000	0.25	9354.95	240.00	0.01
1	140	PF 6	3000.00	940.00	980.06	941.69	980.06	0.000001	0.37	9613.49	240.00	0.01
1	140	PF 7	4000.00	940.00	981.02	942.04	981.02	0.000001	0.48	9844.20	240.00	0.01
1	140	PF 8	5000.00	940.00	981.90	942.37	981.90	0.000002	0.59	10056.02	240.00	0.02
1	140	PF 9	5500.00	940.00	982.32	942.54	982.32	0.000002	0.64	10156.63	240.00	0.02
1	140	PF 10	6000.00	940.00	982.73	942.69	982.73	0.000002	0.69	10254.30	240.00	0.02
1	115.1	PF 1	200.00	975.50	976.16		976.26	0.008703	2.61	81.00	142.14	0.57
1	115.1	PF 2	500.00	975.50	976.74		976.88	0.005540	3.18	173.06	175.04	0.50
1	115.1	PF 3	1000.00	975.50	977.52		977.68	0.003505	3.51	327.48	219.50	0.44
1	115.1	PF 4	1500.00	975.50	978.19		978.36	0.002549	3.63	488.12	255.18	0.39
1	115.1	PF 5	2000.00	975.50	978.80		978.96	0.001970	3.65	649.07	274.43	0.35
1	115.1	PF 6	3000.00	975.50	979.87		980.04	0.001413	3.73	962.46	308.48	0.31
1	115.1	PF 7	4000.00	975.50	980.83		981.00	0.001091	3.75	1267.33	323.63	0.29
1	115.1	PF 8	5000.00	975.50	981.71		981.89	0.000908	3.78	1557.18	335.36	0.27
1	115.1	PF 9	5500.00	975.50	982.13		982.31	0.000843	3.81	1698.33	340.92	0.26
1	115.1	PF 10	6000.00	975.50	982.54		982.71	0.000790	3.83	1837.51	346.32	0.25
1	115	PF 1	200.00	975.50	976.16		976.26	0.001226	2.62	80.95	142.13	0.57
1	115	PF 2	500.00	975.50	976.74		976.88	0.000780	3.19	173.02	175.02	0.50
1	115	PF 3	1000.00	975.50	977.52		977.68	0.000493	3.51	327.44	219.49	0.44
1	115	PF 4	1500.00	975.50	978.19		978.36	0.000359	3.63	488.11	255.18	0.39
1	115	PF 5	2000.00	975.50	978.80		978.96	0.000277	3.65	649.04	274.43	0.35
1	115	PF 6	3000.00	975.50	979.87		980.04	0.000199	3.73	962.44	308.48	0.31
1	115	PF 7	4000.00	975.50	980.83		981.00	0.000153	3.75	1267.31	323.63	0.29
1	115	PF 8	5000.00	975.50	981.71		981.89	0.000128	3.78	1557.18	335.36	0.27
1	115	PF 9	5500.00	975.50	982.13		982.31	0.000119	3.81	1698.33	340.92	0.26
1	115	PF 10	6000.00	975.50	982.54		982.71	0.000111	3.83	1837.51	346.32	0.25
1	100	PF 1	200.00	975.50	976.12		976.24	0.001460	2.76	76.48	140.33	0.62
1	100	PF 2	500.00	975.50	976.72		976.86	0.000820	3.24	170.09	174.07	0.52
1	100	PF 3	1000.00	975.50	977.51		977.67	0.000502	3.53	325.32	218.89	0.44
1	100	PF 4	1500.00	975.50	978.18		978.35	0.000362	3.64	486.46	254.98	0.39
1	100	PF 5	2000.00	975.50	978.79		978.96	0.000279	3.66	647.63	274.26	0.36
1	100	PF 6	3000.00	975.50	979.87		980.04	0.000199	3.74	961.40	308.37	0.32

HEC-RAS Plan: Plan 22   River: E. Spillway   Reach: 1 (Continued)

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
1	100	PF 7	4000.00	975.50	980.83		981.00	0.000154	3.75	1266.50	323.59	0.29
1	100	PF 8	5000.00	975.50	981.71		981.88	0.000128	3.79	1556.51	335.33	0.27
1	100	PF 9	5500.00	975.50	982.13		982.30	0.000119	3.81	1697.67	340.90	0.26
1	100	PF 10	6000.00	975.50	982.53		982.71	0.000111	3.84	1836.83	346.30	0.25
1	99.9	PF 1	200.00	975.50	975.98	975.98	976.22	0.091785	3.95	50.60	105.01	1.00
1	99.9	PF 2	500.00	975.50	976.39	976.39	976.83	0.075231	5.37	93.08	105.02	1.01
1	99.9	PF 3	1000.00	975.50	976.91	976.91	977.62	0.064567	6.77	147.71	105.04	1.01
1	99.9	PF 4	1500.00	975.50	977.34	977.34	978.28	0.058886	7.75	193.68	105.05	1.01
1	99.9	PF 5	2000.00	975.50	977.73	977.73	978.86	0.055267	8.53	234.60	105.06	1.01
1	99.9	PF 6	3000.00	975.50	978.43	978.43	979.91	0.050473	9.76	307.49	105.08	1.01
1	99.9	PF 7	4000.00	975.50	979.05	979.05	980.84	0.047379	10.74	372.45	105.09	1.01
1	99.9	PF 8	5000.00	975.50	979.62	979.62	981.69	0.044943	11.56	432.64	105.11	1.00
1	99.9	PF 9	5500.00	975.50	979.89	979.89	982.10	0.044050	11.94	460.88	105.12	1.00
1	99.9	PF 10	6000.00	975.50	980.15	980.15	982.49	0.043279	12.30	488.18	105.12	1.01
1	96	PF 1	200.00	974.00	974.32	974.48	974.88	0.376398	6.04	33.13	105.01	1.89
1	96	PF 2	500.00	974.00	974.53	974.89	975.78	0.417760	8.99	55.65	105.01	2.18
1	96	PF 3	1000.00	974.00	974.86	975.41	976.78	0.338287	11.13	89.86	105.02	2.12
1	96	PF 4	1500.00	974.00	975.16	975.84	977.53	0.280174	12.37	121.29	105.03	2.03
1	96	PF 5	2000.00	974.00	975.43	976.23	978.18	0.242904	13.30	150.45	105.03	1.96
1	96	PF 6	3000.00	974.00	975.95	976.93	979.29	0.196469	14.67	204.51	105.04	1.85
1	96	PF 7	4000.00	974.00	976.43	977.55	980.26	0.167924	15.70	254.77	105.05	1.78
1	96	PF 8	5000.00	974.00	976.88	978.12	981.13	0.148546	16.55	302.19	105.06	1.72
1	96	PF 9	5500.00	974.00	977.09	978.39	981.55	0.141529	16.95	324.66	105.07	1.70
1	96	PF 10	6000.00	974.00	977.30	978.65	981.95	0.134987	17.30	346.96	105.07	1.68
1	93	PF 1	200.00	972.00	972.28	972.48	973.00	0.556401	6.79	29.47	105.01	2.26
1	93	PF 2	500.00	972.00	972.48	972.89	973.98	0.561951	9.82	50.91	105.01	2.49
1	93	PF 3	1000.00	972.00	972.76	973.41	975.21	0.508273	12.57	79.53	105.01	2.55
1	93	PF 4	1500.00	972.00	973.00	973.84	976.16	0.450602	14.26	105.17	105.02	2.51
1	93	PF 5	2000.00	972.00	973.23	974.23	976.95	0.402379	15.47	129.30	105.02	2.46
1	93	PF 6	3000.00	972.00	973.66	974.93	978.24	0.331961	17.17	174.72	105.03	2.35
1	93	PF 7	4000.00	972.00	974.07	975.55	979.32	0.283724	18.38	217.66	105.04	2.25
1	93	PF 8	5000.00	972.00	974.46	976.12	980.27	0.249809	19.34	258.54	105.04	2.17
1	93	PF 9	5500.00	972.00	974.65	976.39	980.72	0.237044	19.78	278.10	105.05	2.14
1	93	PF 10	6000.00	972.00	974.83	976.65	981.15	0.225141	20.17	297.57	105.05	2.11
1	85	PF 1	200.00	965.40	965.68	965.88	966.41	0.574097	6.85	29.19	105.01	2.29

HEC-RAS Plan: Plan 22 River: E. Spillway Reach: 1 (Continued)

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
1	85	PF 2	500.00	965.40	965.88	966.29	967.40	0.574378	9.89	50.58	105.02	2.51
1	85	PF 3	1000.00	965.40	966.13	966.81	968.77	0.573735	13.04	76.70	105.02	2.69
1	85	PF 4	1500.00	965.40	966.33	967.24	969.96	0.567474	15.29	98.14	105.03	2.79
1	85	PF 5	2000.00	965.40	966.52	967.63	971.03	0.556871	17.05	117.30	105.04	2.84
1	85	PF 6	3000.00	965.40	966.85	968.33	972.88	0.525993	19.72	152.20	105.05	2.89
1	85	PF 7	4000.00	965.40	967.16	968.95	974.45	0.491903	21.68	184.55	105.06	2.88
1	85	PF 8	5000.00	965.40	967.45	969.52	975.80	0.457272	23.19	215.67	105.07	2.85
1	85	PF 9	5500.00	965.40	967.60	969.79	976.42	0.441252	23.83	230.82	105.07	2.83
1	85	PF 10	6000.00	965.40	967.74	970.05	976.99	0.425444	24.41	245.88	105.08	2.81
1	80	PF 1	200.00	958.00	958.29	958.48	958.96	0.503104	6.59	30.37	105.01	2.16
1	80	PF 2	500.00	958.00	958.50	958.89	959.91	0.507624	9.53	52.49	105.02	2.37
1	80	PF 3	1000.00	958.00	958.76	959.41	961.21	0.507442	12.57	79.57	105.02	2.54
1	80	PF 4	1500.00	958.00	958.97	959.84	962.36	0.508227	14.79	101.45	105.03	2.65
1	80	PF 5	2000.00	958.00	959.15	960.23	963.42	0.508657	16.60	120.53	105.04	2.73
1	80	PF 6	3000.00	958.00	959.46	960.93	965.38	0.509361	19.53	153.67	105.05	2.84
1	80	PF 7	4000.00	958.00	959.74	961.55	967.15	0.504588	21.85	183.14	105.06	2.92
1	80	PF 8	5000.00	958.00	960.01	962.12	968.76	0.494769	23.74	210.63	105.06	2.95
1	80	PF 9	5500.00	958.00	960.13	962.39	969.50	0.488171	24.57	223.93	105.07	2.97
1	80	PF 10	6000.00	958.00	960.26	962.65	970.22	0.481204	25.33	236.95	105.07	2.97
1	77	PF 1	200.00	956.50	956.75	956.98	957.66	0.830089	7.65	26.14	105.01	2.70
1	77	PF 2	500.00	956.50	956.95	957.39	958.68	0.715384	10.56	47.36	105.01	2.77
1	77	PF 3	1000.00	956.50	957.20	957.91	960.04	0.646310	13.51	74.00	105.02	2.84
1	77	PF 4	1500.00	956.50	957.41	958.34	961.20	0.609956	15.62	96.04	105.03	2.88
1	77	PF 5	2000.00	956.50	957.60	958.73	962.29	0.594015	17.39	115.05	105.03	2.93
1	77	PF 6	3000.00	956.50	957.91	959.43	964.26	0.571945	20.22	148.42	105.04	3.00
1	77	PF 7	4000.00	956.50	958.19	960.05	966.05	0.555768	22.49	177.91	105.05	3.04
1	77	PF 8	5000.00	956.50	958.45	960.62	967.68	0.539953	24.38	205.17	105.06	3.07
1	77	PF 9	5500.00	956.50	958.58	960.89	968.44	0.531104	25.20	218.33	105.06	3.08
1	77	PF 10	6000.00	956.50	958.70	961.15	969.16	0.522282	25.96	231.19	105.06	3.08
1	70	PF 1	200.00	956.10	956.41	956.58	957.00	0.399350	6.14	32.55	105.01	1.94
1	70	PF 2	500.00	956.10	956.60	956.99	958.00	0.497833	9.47	52.80	105.01	2.35
1	70	PF 3	1000.00	956.10	956.85	957.51	959.36	0.527568	12.72	78.65	105.02	2.59
1	70	PF 4	1500.00	956.10	957.05	957.94	960.54	0.531031	14.98	100.12	105.03	2.70
1	70	PF 5	2000.00	956.10	957.23	958.33	961.64	0.535132	16.85	118.71	105.03	2.79
1	70	PF 6	3000.00	956.10	957.54	959.03	963.63	0.533230	19.80	151.57	105.04	2.90
1	70	PF 7	4000.00	956.10	957.82	959.65	965.43	0.527691	22.14	180.70	105.05	2.97

HEC-RAS Plan: Plan 22 River: E. Spillway Reach: 1 (Continued)

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
1	70	PF 8	5000.00	956.10	958.08	960.22	967.08	0.518801	24.08	207.65	105.06	3.02
1	70	PF 9	5500.00	956.10	958.20	960.49	967.84	0.511571	24.92	220.80	105.06	3.03
1	70	PF 10	6000.00	956.10	958.32	960.75	968.58	0.505503	25.71	233.47	105.06	3.04
1	69.9	PF 1	200.00	956.10	956.39	956.57	956.99	0.438962	6.20	32.24	110.01	2.02
1	69.9	PF 2	500.00	956.10	956.58	956.96	957.99	0.541413	9.53	52.45	110.02	2.43
1	69.9	PF 3	1000.00	956.10	956.81	957.47	959.35	0.571684	12.79	78.22	110.04	2.67
1	69.9	PF 4	1500.00	956.10	957.01	957.89	960.53	0.574944	15.06	99.60	110.05	2.79
1	69.9	PF 5	2000.00	956.10	957.17	958.27	961.63	0.578905	16.93	118.13	110.05	2.88
1	69.9	PF 6	3000.00	956.10	957.47	958.94	963.61	0.576374	19.89	150.87	110.07	2.99
1	69.9	PF 7	4000.00	956.10	957.73	959.54	965.42	0.570016	22.24	179.90	110.08	3.07
1	69.9	PF 8	5000.00	956.10	957.98	960.09	967.07	0.560262	24.19	206.75	110.09	3.11
1	69.9	PF 9	5500.00	956.10	958.10	960.37	967.82	0.552450	25.03	219.84	110.10	3.12
1	69.9	PF 10	6000.00	956.10	958.21	960.62	968.56	0.545879	25.82	232.47	110.11	3.13
1	60	PF 1	200.00	917.00	917.30	917.47	917.86	0.386204	5.97	33.50	110.02	1.91
1	60	PF 2	500.00	917.00	917.53	917.86	918.69	0.392246	8.65	57.78	110.03	2.10
1	60	PF 3	1000.00	917.00	917.79	918.37	919.83	0.395936	11.45	87.34	110.05	2.27
1	60	PF 4	1500.00	917.00	918.01	918.79	920.83	0.396575	13.47	111.35	110.06	2.36
1	60	PF 5	2000.00	917.00	918.20	919.16	921.75	0.396544	15.12	132.33	110.07	2.43
1	60	PF 6	3000.00	917.00	918.53	919.84	923.44	0.395788	17.77	168.89	110.09	2.53
1	60	PF 7	4000.00	917.00	918.82	920.44	925.00	0.396406	19.94	200.62	110.10	2.60
1	60	PF 8	5000.00	917.00	919.09	920.99	926.46	0.395771	21.80	229.49	110.12	2.66
1	60	PF 9	5500.00	917.00	919.21	921.27	927.16	0.395239	22.63	243.10	110.13	2.68
1	60	PF 10	6000.00	917.00	919.33	921.52	927.86	0.395732	23.44	256.04	110.13	2.71
1	53	PF 1	200.00	915.00	915.34	915.47	915.79	0.280219	5.42	36.88	110.02	1.65
1	53	PF 2	500.00	915.00	915.58	915.86	916.54	0.286142	7.87	63.51	110.03	1.83
1	53	PF 3	1000.00	915.00	915.88	916.37	917.55	0.285147	10.38	96.38	110.04	1.95
1	53	PF 4	1500.00	915.00	916.11	916.79	918.45	0.289592	12.26	122.36	110.06	2.05
1	53	PF 5	2000.00	915.00	916.31	917.17	919.29	0.295808	13.84	144.49	110.07	2.13
1	53	PF 6	3000.00	915.00	916.66	917.84	920.86	0.306301	16.45	182.38	110.08	2.25
1	53	PF 7	4000.00	915.00	916.95	918.44	922.34	0.315517	18.62	214.84	110.10	2.35
1	53	PF 8	5000.00	915.00	917.22	918.99	923.73	0.321483	20.48	244.25	110.11	2.42
1	53	PF 9	5500.00	915.00	917.34	919.27	924.41	0.324559	21.33	257.90	110.12	2.46
1	53	PF 10	6000.00	915.00	917.46	919.52	925.08	0.327665	22.15	270.95	110.12	2.49
1	52.9	PF 1	200.00	915.00	915.35	915.47	915.77	0.101776	5.22	38.34	110.02	1.56
1	52.9	PF 2	500.00	915.00	915.58	915.86	916.53	0.114142	7.79	64.19	110.03	1.80

HEC-RAS Plan: Plan 22   River: E. Spillway   Reach: 1 (Continued)

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
1	52.9	PF 3	1000.00	915.00	915.88	916.37	917.53	0.115522	10.32	96.95	110.04	1.94
1	52.9	PF 4	1500.00	915.00	916.12	916.79	918.43	0.117970	12.21	122.89	110.06	2.04
1	52.9	PF 5	2000.00	915.00	916.32	917.17	919.27	0.120881	13.80	144.98	110.07	2.12
1	52.9	PF 6	3000.00	915.00	916.66	917.84	920.84	0.125556	16.41	182.83	110.08	2.24
1	52.9	PF 7	4000.00	915.00	916.96	918.44	922.32	0.129536	18.59	215.27	110.10	2.34
1	52.9	PF 8	5000.00	915.00	917.22	918.99	923.71	0.132129	20.44	244.66	110.11	2.42
1	52.9	PF 9	5500.00	915.00	917.35	919.27	924.39	0.133445	21.30	258.30	110.12	2.45
1	52.9	PF 10	6000.00	915.00	917.47	919.53	925.06	0.134768	22.12	271.35	110.12	2.48
1	50	PF 1	200.00	914.20	914.91		915.01	0.009510	2.56	78.07	110.03	0.54
1	50	PF 2	500.00	914.20	915.46		915.66	0.008811	3.61	138.44	110.05	0.57
1	50	PF 3	1000.00	914.20	916.17		916.50	0.007907	4.61	216.78	110.08	0.58
1	50	PF 4	1500.00	914.20	916.77		917.20	0.007375	5.31	282.36	110.11	0.58
1	50	PF 5	2000.00	914.20	917.28		917.82	0.007096	5.89	339.49	110.13	0.59
1	50	PF 6	3000.00	914.20	918.19		918.92	0.006743	6.83	439.76	110.17	0.60
1	50	PF 7	4000.00	914.20	918.99		919.89	0.006531	7.59	527.71	110.20	0.61
1	50	PF 8	5000.00	914.20	919.72		920.77	0.006356	8.22	609.31	113.67	0.62
1	50	PF 9	5500.00	914.20	920.06		921.18	0.006299	8.52	647.93	115.29	0.62
1	50	PF 10	6000.00	914.20	920.39		921.58	0.006233	8.79	685.98	116.86	0.62
1	49.9	PF 1	200.00	914.20	914.91		915.01	0.009218	2.52	78.78	112.38	0.53
1	49.9	PF 2	500.00	914.20	915.46		915.66	0.008274	3.50	141.01	114.22	0.55
1	49.9	PF 3	1000.00	914.20	916.17		916.50	0.007140	4.39	223.38	116.61	0.55
1	49.9	PF 4	1500.00	914.20	916.77		917.20	0.006450	4.98	293.73	118.62	0.55
1	49.9	PF 5	2000.00	914.20	917.29		917.82	0.006035	5.44	356.12	120.37	0.55
1	49.9	PF 6	3000.00	914.20	918.21		918.91	0.005460	6.16	468.14	123.45	0.54
1	49.9	PF 7	4000.00	914.20	919.02		919.87	0.005051	6.70	569.64	126.18	0.54
1	49.9	PF 8	5000.00	914.20	919.77		920.75	0.004734	7.14	664.25	128.67	0.53
1	49.9	PF 9	5500.00	914.20	920.11		921.16	0.004621	7.34	708.55	129.82	0.53
1	49.9	PF 10	6000.00	914.20	920.44		921.56	0.004516	7.52	751.83	130.93	0.53
1	30	PF 1	200.00	914.00	914.53	914.53	914.80	0.036552	4.15	48.50	92.33	1.00
1	30	PF 2	500.00	914.00	914.98	914.98	915.46	0.029619	5.60	90.14	94.28	1.00
1	30	PF 3	1000.00	914.00	915.55	915.55	916.30	0.025256	7.02	144.55	96.77	0.99
1	30	PF 4	1500.00	914.00	916.02	916.02	917.00	0.023257	8.04	190.31	98.82	1.00
1	30	PF 5	2000.00	914.00	916.44	916.44	917.62	0.021709	8.81	232.32	100.66	0.99
1	30	PF 6	3000.00	914.00	917.18	917.18	918.70	0.019651	10.02	308.60	103.92	0.99
1	30	PF 7	4000.00	914.00	917.83	917.83	919.64	0.018488	10.99	376.97	106.76	0.99
1	30	PF 8	5000.00	914.00	918.43	918.43	920.50	0.017510	11.79	441.74	109.39	0.99

HEC-RAS Plan: Plan 22   River: E. Spillway   Reach: 1 (Continued)

Reach	River Sta	Profile	Q Total (cfs)	Min Ch El (ft)	W.S. Elev (ft)	Crit W.S. (ft)	E.G. Elev (ft)	E.G. Slope (ft/ft)	Vel Chnl (ft/s)	Flow Area (sq ft)	Top Width (ft)	Froude # Chl
1	30	PF 9	5500.00	914.00	918.73	918.73	920.91	0.016968	12.11	474.09	110.67	0.98
1	30	PF 10	6000.00	914.00	919.00	919.00	921.30	0.016592	12.44	504.77	111.88	0.98
1	8	PF 1	200.00	904.00	904.67	904.63	904.94	0.027464	4.20	48.00	72.75	0.90
1	8	PF 2	500.00	904.00	905.04	905.15	905.74	0.040065	6.77	74.76	74.23	1.17
1	8	PF 3	1000.00	904.00	905.57	905.82	906.77	0.039315	8.85	115.07	76.42	1.24
1	8	PF 4	1500.00	904.00	906.00	906.37	907.63	0.039161	10.37	148.05	78.16	1.29
1	8	PF 5	2000.00	904.00	906.39	906.86	908.38	0.037780	11.48	179.08	79.77	1.31
1	8	PF 6	3000.00	904.00	907.04	907.72	909.74	0.037500	13.41	231.48	82.40	1.36
1	8	PF 7	4000.00	904.00	908.40	908.49	910.56	0.018523	12.07	347.73	87.98	1.01
1	8	PF 8	5000.00	904.00	909.09	909.19	911.54	0.017473	12.91	409.07	90.78	1.01
1	8	PF 9	5500.00	904.00	909.39	909.50	912.00	0.017277	13.34	436.71	92.01	1.01
1	8	PF 10	6000.00	904.00	909.71	909.82	912.44	0.016859	13.69	465.80	93.30	1.01
1	5	PF 1	200.00	898.00	898.66	898.63	898.94	0.029687	4.30	46.83	72.47	0.93
1	5	PF 2	500.00	898.00	899.06	899.15	899.73	0.037008	6.61	76.49	73.98	1.13
1	5	PF 3	1000.00	898.00	899.67	899.82	900.73	0.032456	8.36	121.81	76.25	1.14
1	5	PF 4	1500.00	898.00	900.34	900.38	901.52	0.022966	8.83	174.27	78.78	1.02
1	5	PF 5	2000.00	898.00	900.84	900.87	902.24	0.021127	9.63	214.07	80.66	1.01
1	5	PF 6	3000.00	898.00	901.26	901.73	903.61	0.029568	12.50	248.50	82.24	1.22
1	5	PF 7	4000.00	898.00	902.35	902.50	904.60	0.019600	12.31	339.75	86.30	1.04
1	5	PF 8	5000.00	898.00	903.03	903.21	905.59	0.018457	13.17	399.59	88.86	1.03
1	5	PF 9	5500.00	898.00	903.33	903.52	906.06	0.018223	13.61	426.65	90.00	1.04
1	5	PF 10	6000.00	898.00	903.65	903.84	906.51	0.017721	13.95	455.43	91.19	1.03
1	3	PF 1	200.00	878.00	878.68	878.66	878.98	0.030014	4.41	46.07	70.91	0.94
1	3	PF 2	500.00	878.00	879.12	879.19	879.77	0.034177	6.58	78.12	74.75	1.10
1	3	PF 3	1000.00	878.00	879.73	879.87	880.76	0.030618	8.32	125.22	80.06	1.12
1	3	PF 4	1500.00	878.00	880.34	880.42	881.53	0.023880	8.99	175.92	85.40	1.04
1	3	PF 5	2000.00	878.00	880.77	880.91	882.21	0.023299	9.95	213.82	89.19	1.05
1	3	PF 6	3000.00	878.00	881.56	881.75	883.38	0.021500	11.29	286.81	96.06	1.05
1	3	PF 7	4000.00	878.00	882.38	882.48	884.37	0.018037	11.88	368.74	103.24	1.00
1	3	PF 8	5000.00	878.00	883.14	883.15	885.25	0.015782	12.35	448.84	109.79	0.96
1	3	PF 9	5500.00	878.00	883.17	883.44	885.68	0.018586	13.47	453.01	110.12	1.04
1	3	PF 10	6000.00	878.00	883.60	883.73	886.05	0.016496	13.38	500.82	113.85	1.00

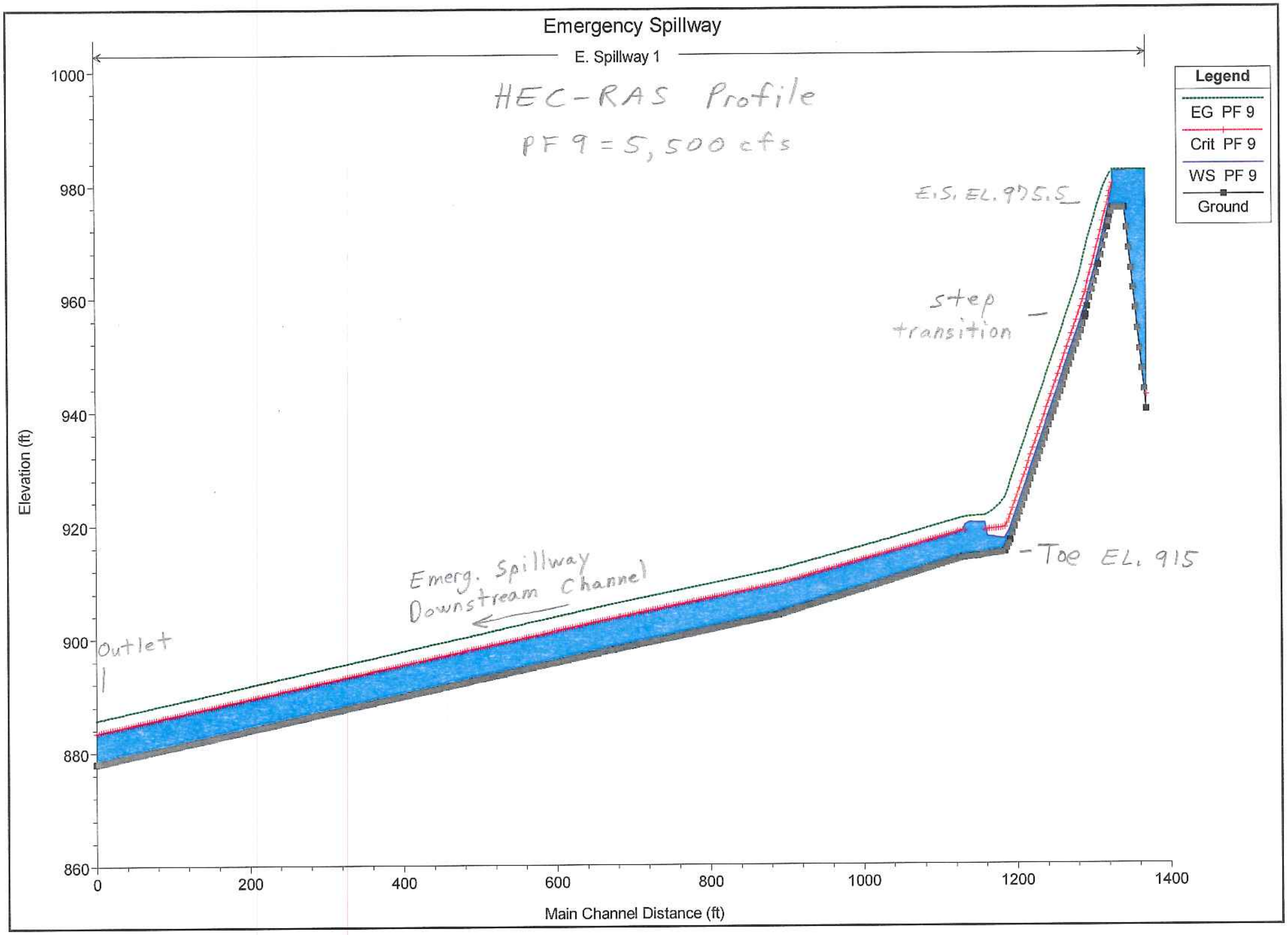
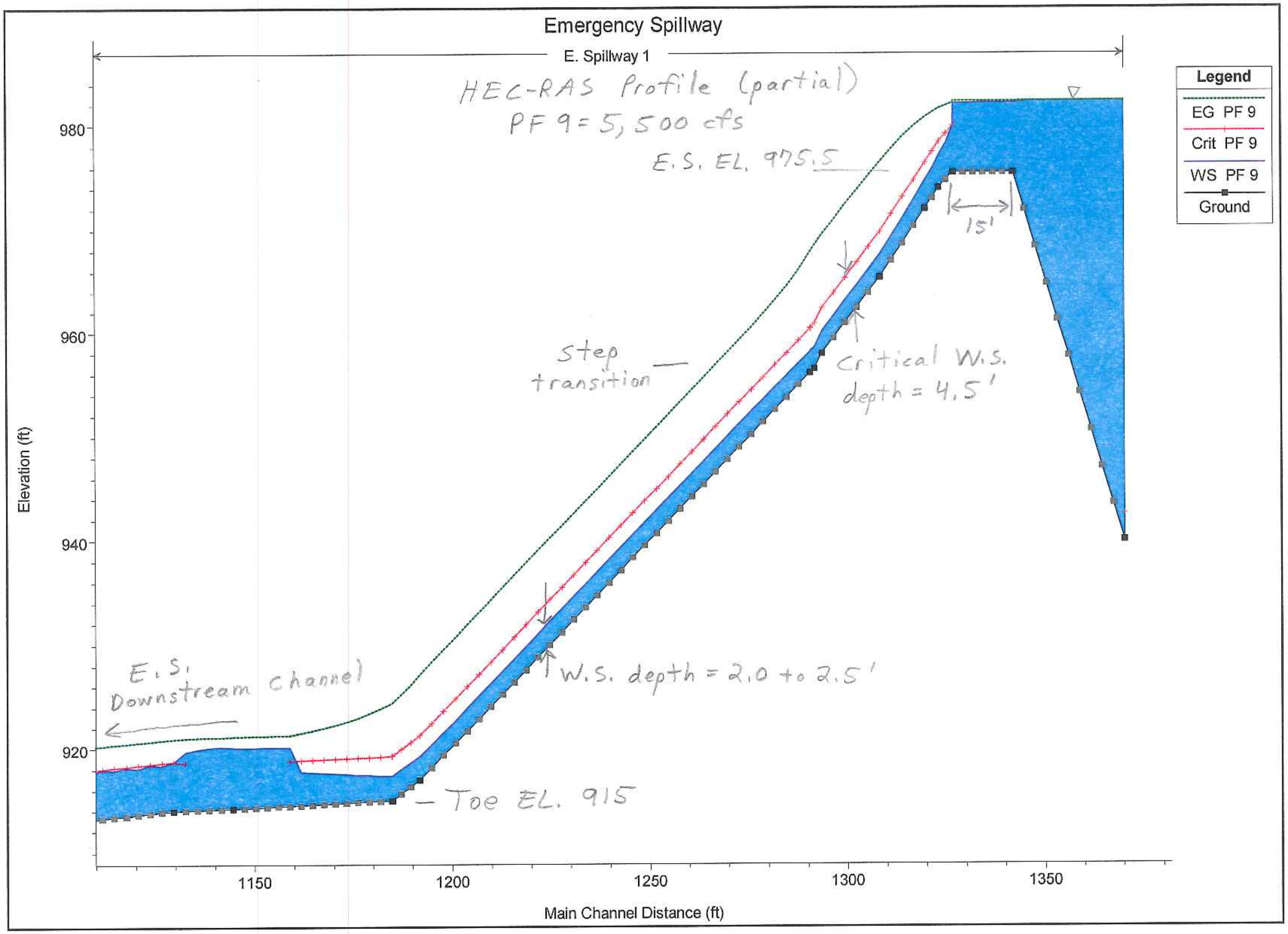
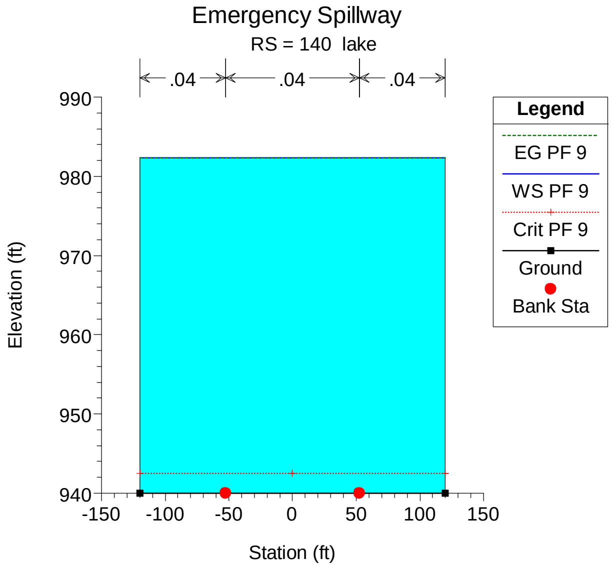
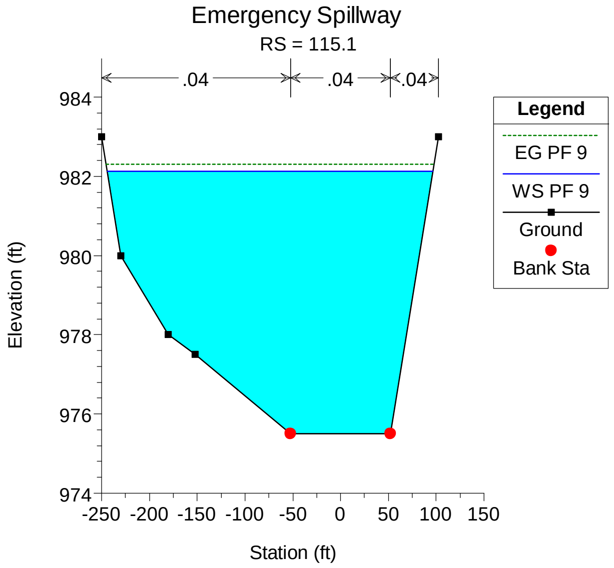

Emergency Spillway
E. Spillway 1
HEC-RAS Profile
PF 9 = 5,500 cfs
Legend
EG PF 9
Crit PF 9
WS PF 9
Ground
E.S. EL. 975.5
step transition
-Toe EL. 915
Emerg. Spillway Downstream Channel
Outlet
Elevation (ft)
1000
980
960
940
920
900
880
860
Main Channel Distance (ft)
0
200
400
600
800
1000
1200
1400

PLATE 31

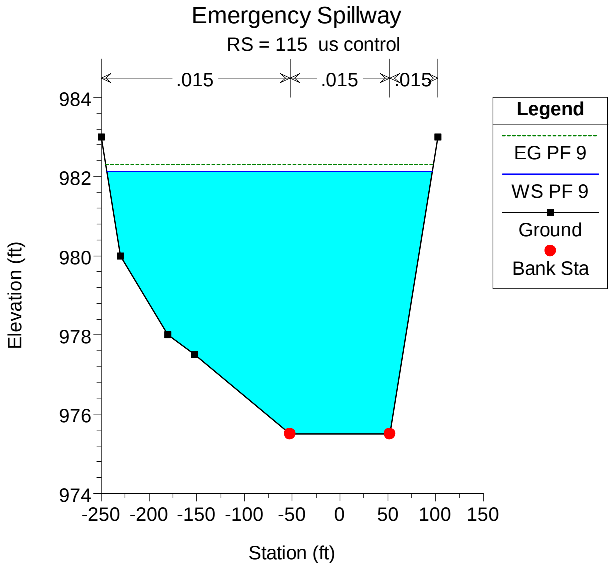

Emergency Spillway
E. Spillway 1
HEC-RAS Profile (partial)
PF 9 = 5,500 cfs
E.S. EL. 975.5
15'
Step transition
Critical W.S. depth = 4.5'
W.S. depth = 2.0 to 2.5'
E.S. Downstream channel
Toe EL. 915
Legend
EG PF 9
Crit PF 9
WS PF 9
Ground
Elevation (ft)
980
960
940
920
1150
1200
1250
1300
1350
Main Channel Distance (ft)


Emergency Spillway

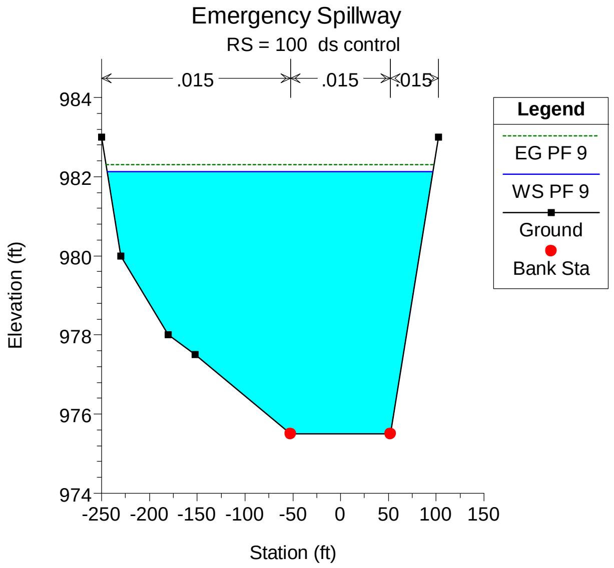
RS = 140 lake


Emergency Spillway

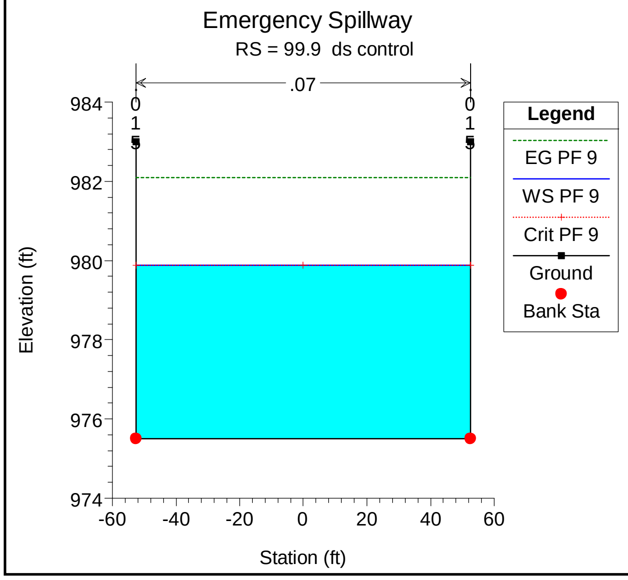
RS = 115.1


Emergency Spillway

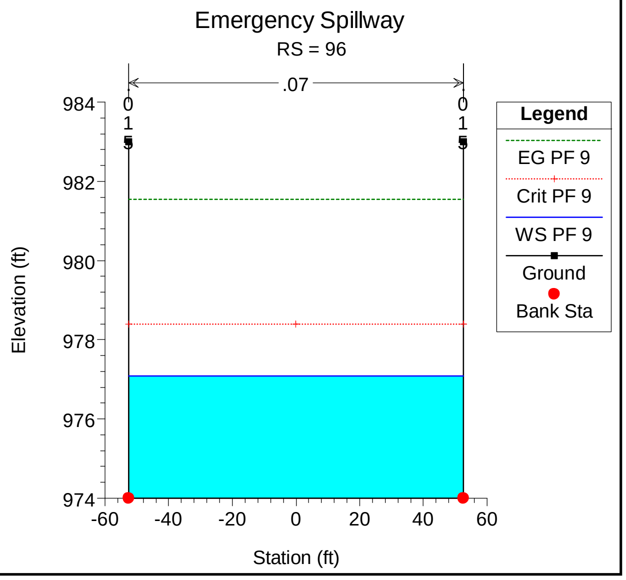
RS = 115 us control


Emergency Spillway

RS = 100 ds control


Emergency Spillway

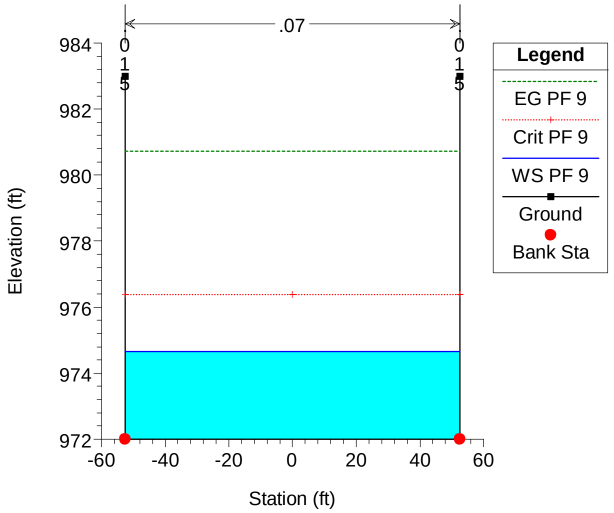
RS = 99.9 ds control


Emergency Spillway

RS = 96

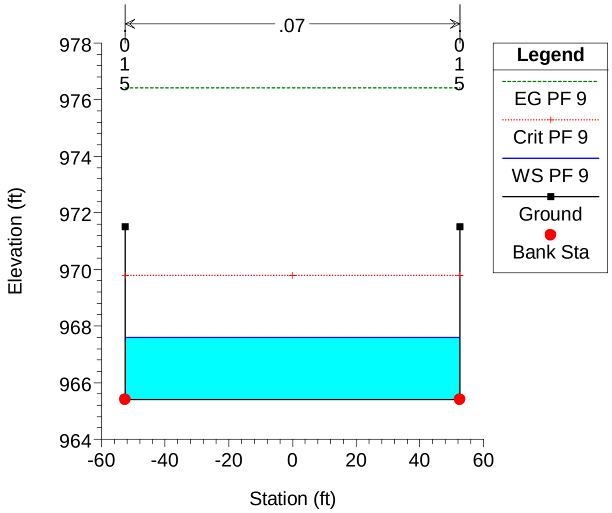
Emergency Spillway

RS = 93 Wall vertical bend



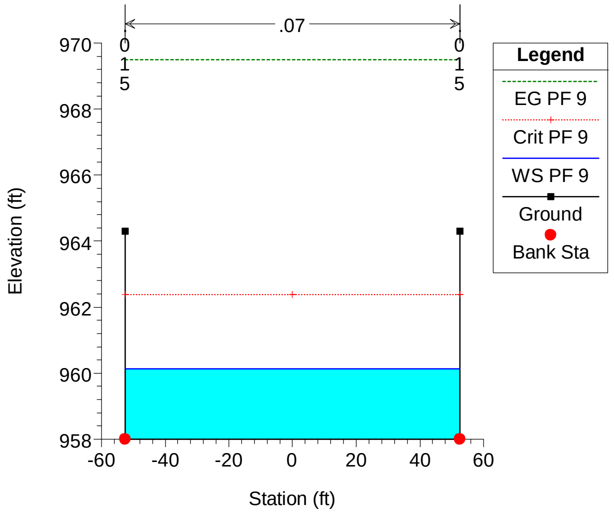
Emergency Spillway

RS = 85 Wall vertical bend



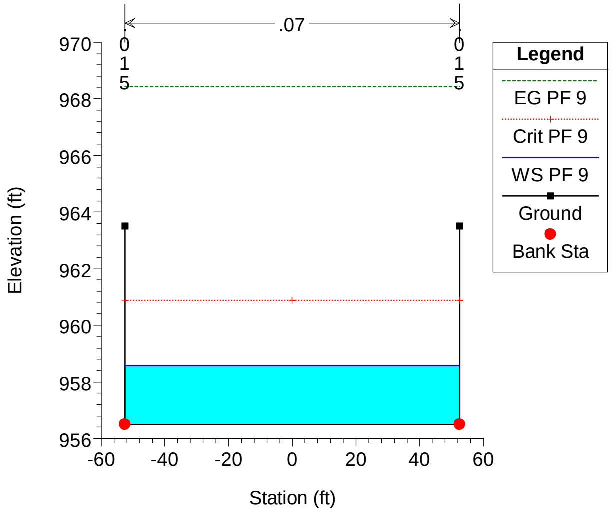
Emergency Spillway

RS = 80 transition



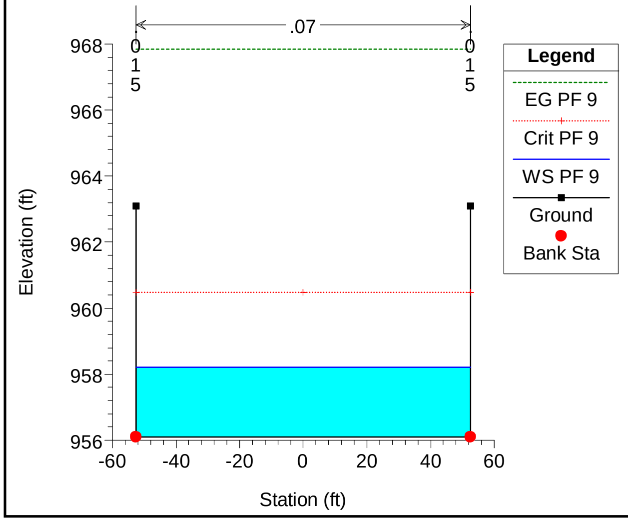
Emergency Spillway

RS = 77 ex. spillway steps



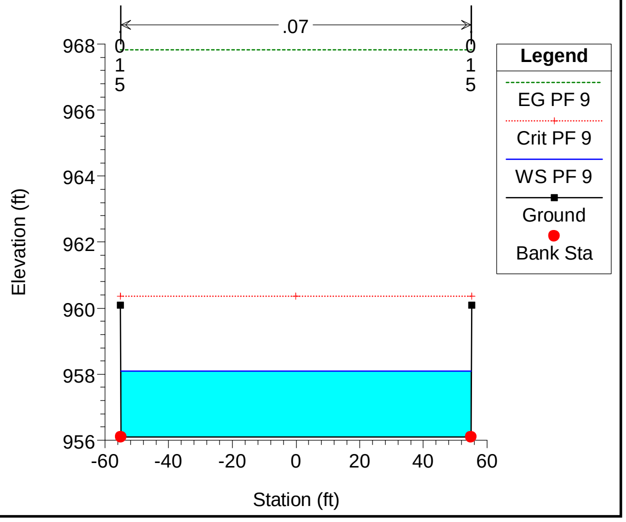
Emergency Spillway

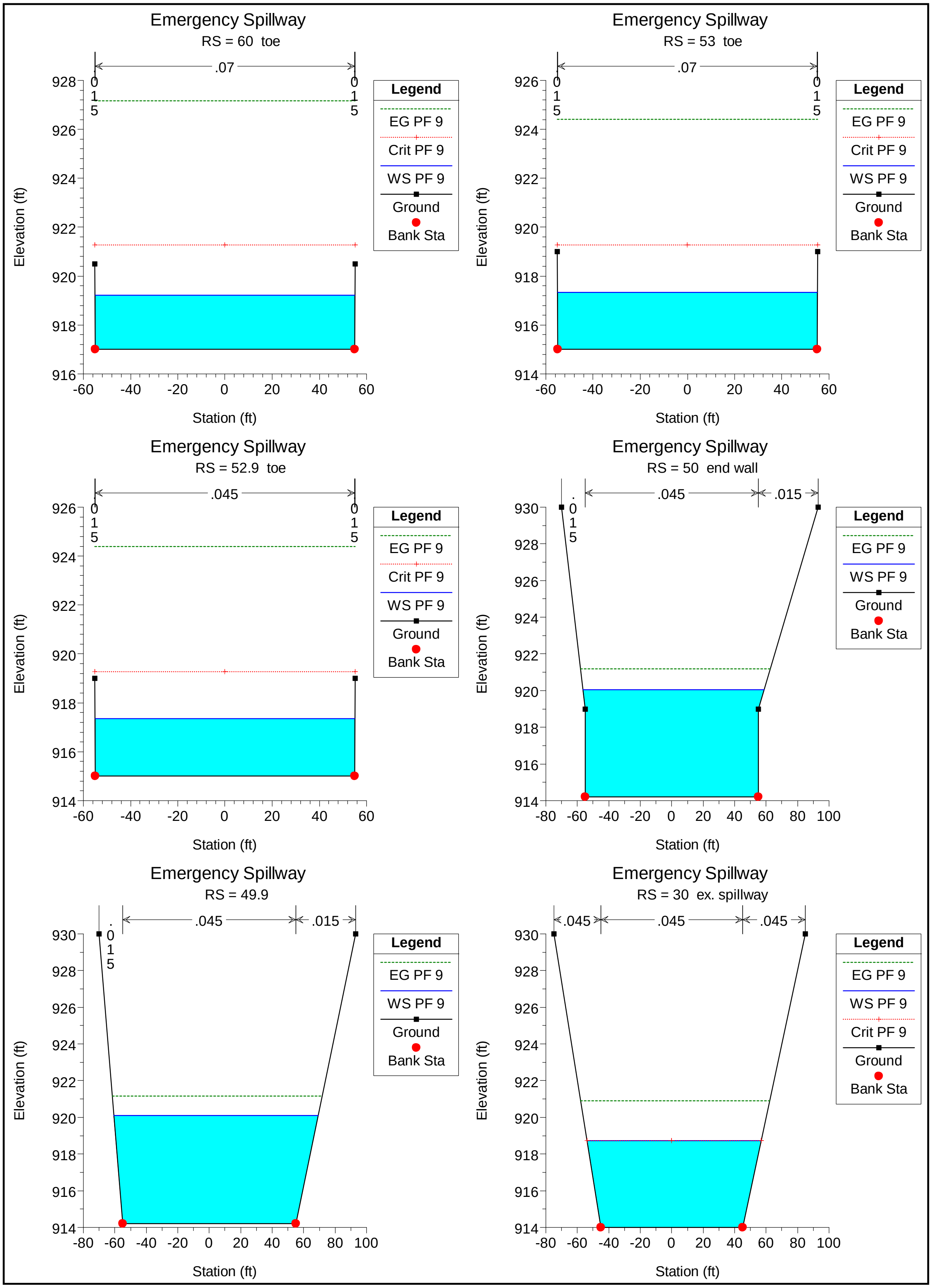
RS = 70 end wall

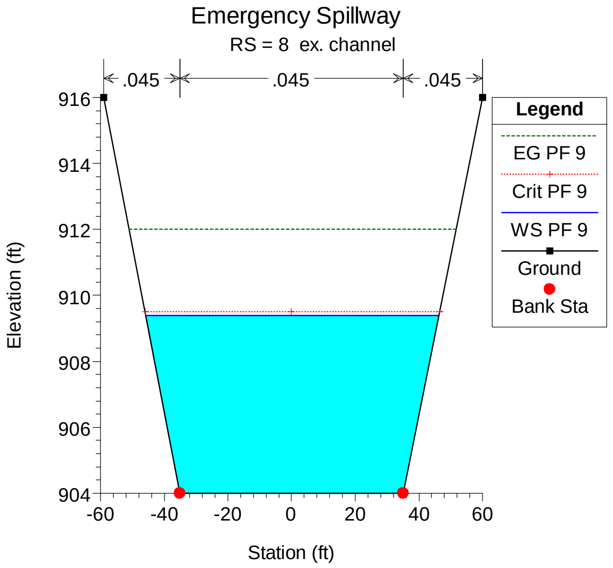


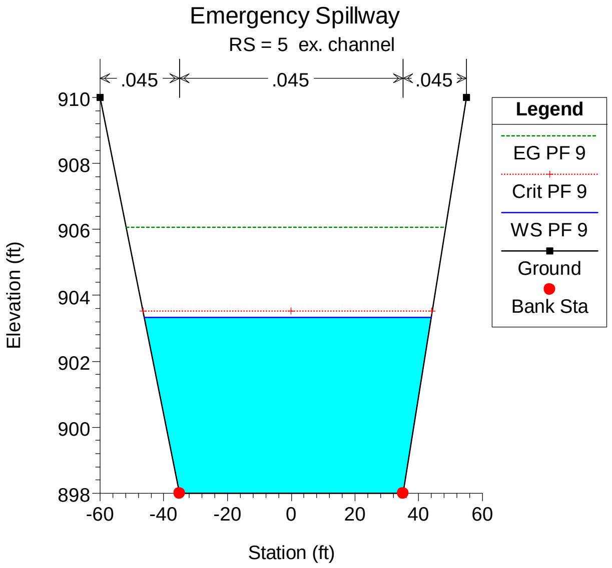
Emergency Spillway

RS = 69.9 ex. wall




Emergency Spillway
RS = 60 toe
.07
0 1 5
928
926
924
922
920
918
916
-60
-40
-20
0
20
40
60
Elevation (ft)
Station (ft)
Legend
EG PF 9
Crit PF 9
WS PF 9
Ground
Bank Sta
Emergency Spillway
RS = 53 toe
.07
0 1 5
926
924
922
920
918
916
914
Emergency Spillway
RS = 52.9 toe
.045
0 1 5
926
924
922
920
918
916
914
Emergency Spillway
RS = 50 end wall
.045
.015
. 0 1 5
930
928
926
924
922
920
918
916
914
-80
-60
-40
-20
0
20
40
60
80
100
Emergency Spillway
RS = 49.9
.045
.015
. 0 1 5
Emergency Spillway
RS = 30 ex. spillway
.045
.045
.045


Emergency Spillway
RS = 8 ex. channel
.045
.045
.045
Elevation (ft)
916
914
912
910
908
906
904
-60
-40
-20
0
20
40
60
Station (ft)
Legend
EG PF 9
Crit PF 9
WS PF 9
Ground
Bank Sta


Emergency Spillway
RS = 5 ex. channel
.045
.045
.045
Elevation (ft)
910
908
906
904
902
900
898
-60
-40
-20
0
20
40
60
Station (ft)
Legend
EG PF 9
Crit PF 9
WS PF 9
Ground
Bank Sta

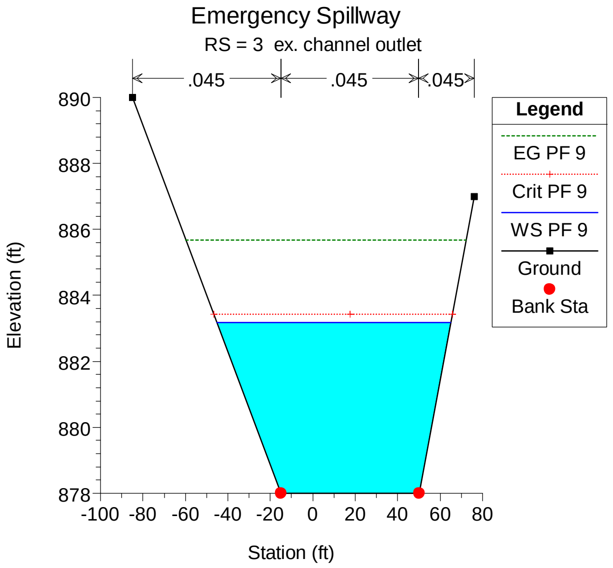

Emergency Spillway
RS = 3 ex. channel outlet
.045
.045
.045
Elevation (ft)
890
888
886
884
882
880
878
-100
-80
-60
-40
-20
0
20
40
60
80
Station (ft)
Legend
EG PF 9
Crit PF 9
WS PF 9
Ground
Bank Sta

Table 3-1 (Continued) Manning's 'n' Values

Type of Channel and Description	**Minimum**	**Normal**	**Maximum**
C. *Excavated or Dredged Channels*			
1. **Earth, straight and uniform**			
a. Clean, recently completed	0.016	0.018	0.020
b. Clean, after weathering	0.018	0.022	0.025
c. Gravel, uniform section, clean	0.022	0.025	0.030
d. With short grass, few weeds	0.022	0.027	0.033
2. **Earth, winding and sluggish**			
a. No vegetation	0.023	0.025	0.030
b. Grass, some weeds	0.025	0.030	0.033
c. Dense weeds or aquatic plants in deep channels	0.030	0.035	0.040
d. Earth bottom and rubble side	0.028	0.030	0.035
e. Stony bottom and weedy banks	0.025	0.035	0.040
f. Cobble bottom and clean sides	0.030	0.040	0.050
3. **Dragline-excavated or dredged**			
a. No vegetation	0.025	0.028	0.033
b. Light brush on banks	0.035	0.050	0.060
4. **Rock cuts**			
a. Smooth and uniform	0.025	0.035	0.040
b. Jagged and irregular	0.035	0.040	0.050
5. **Channels not maintained, weeds and brush**			
a. Clean bottom, brush on sides	0.040	0.050	0.080
b. Same as above, highest stage of flow	0.045	0.070	0.110
c. Dense weeds, high as flow depth	0.050	0.080	0.120
d. Dense brush, high stage	0.080	0.100	0.140

Other sources that include pictures of selected streams as a guide to n value determination are available (Fasken, 1963; Barnes, 1967; and Hicks and Mason, 1991). In general, these references provide color photos with tables of calibrated n values for a range of flows.

Although there are many factors that affect the selection of the n value for the channel, some of the most important factors are the type and size of materials that compose the bed and banks of a channel, and the shape of the channel. Cowan (1956) developed a procedure for estimating the effects of these factors to determine the value of Manning's n of a channel. In Cowan's procedure, the value of n is computed by the following equation:

Table 3-1 (Continued) Manning's 'n' Values

Type of Channel and Description	Minimum	Normal	Maximum
B. *Lined or Built-Up Channels*			
1. **Concrete**			
a. Trowel finish	0.011	0.013	0.015
b. Float Finish	0.013	0.015	0.016
c. Finished, with gravel bottom	0.015	0.017	0.020
d. Unfinished	0.014	0.017	0.020
e. Gunite, good section	0.016	0.019	0.023
f. Gunite, wavy section	0.018	0.022	0.025
g. On good excavated rock	0.017	0.020	
h. On irregular excavated rock	0.022	0.027	
2. **Concrete bottom float finished with sides of:**			
a. Dressed stone in mortar	0.015	0.017	0.020
b. Random stone in mortar	0.017	0.020	0.024
c. Cement rubble masonry, plastered	0.016	0.020	0.024
d. Cement rubble masonry	0.020	0.025	0.030
e. Dry rubble on riprap	0.020	0.030	0.035
3. **Gravel bottom with sides of:**			
a. Formed concrete	0.017	0.020	0.025
b. Random stone in mortar	0.020	0.023	0.026
c. Dry rubble or riprap	0.023	0.033	0.036
4. **Brick**			
a. Glazed	0.011	0.013	0.015
b. In cement mortar	0.012	0.015	0.018
5. **Metal**			
a. Smooth steel surfaces	0.011	0.012	0.014
b. Corrugated metal	0.021	0.025	0.030
6. **Asphalt**			
a. Smooth	0.013	0.013	
b. Rough	0.016	0.016	
7. **Vegetal lining**	0.030		0.500

S&ME
Cincinnati (513) 771-8471
Cleveland (216) 901-1000
Columbus (614) 793-2226

Project/Proposal No. 11497-042 Calculated By AJS Date 9-14-12
Project/Proposal Name Cardinal Checked By SJC Date 9/27/12
Subject Stepped Spillway n Sheet ______ of ______

Determine Manning's "n" for stepped spillway model in HEC-RAS

References: Boes, R.M., and Hager, W.H. (2003). "Hydraulic Design of Stepped Spillways." J. Hydraul. Eng. 129(9).

Ghare, A.D., Porey P.D., and Ingle, R.N. (2005). "Discussion of "Hydraulic Design of stepped Spillways" by Boes + Hager". J. Hydraul. Eng. 131. p. 524.

Fig. 1 of Discussion plots Manning's n vs. $\frac{Y_c}{H}$ for different H^*

$Y_c = \frac{h_c}{h_u}$ normalized critical depth

h_c = critical depth

h_u = uniform flow depth

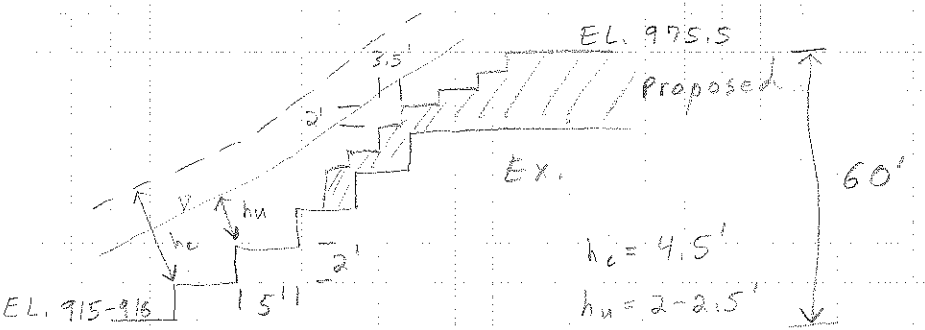
H = spillway height = 60'

$H^* = \frac{\text{spillway height}}{\text{step height}} = \frac{60'}{2'} = 30$

from HEC-RAS, $\frac{h_c}{h_u} = \frac{4.5'}{2'} = 2.25 = Y_c$ (for n=0.07)

$\frac{Y_c}{H} = \frac{2.25}{60} = 0.0375$

From Fig. 1, $\underline{n = 0.07}$ for $\frac{Y_c}{H} = 0.0375$, $H^* = 30$



flow and main stream skimming flows: An experimental study." *Can. J. Civ. Eng.* 31(1), 33-44.

Henderson, F. M. (1966). *Open channel flow*, MacMillan, New York.

Laali, A. R., and Michel, J. M. (1984). "Air entrainment in ventilated cavities: Case of the fully developed 'half cavity'." *J. Fluids Eng., Trans ASME*, Sept., 106, 327–335.

Matos, J. (2000). "Hydraulic design of stepped spillways over RCC dams." *Intl Workshop on Hydraulics of Stepped Spillways*, Zürich, Switzerland, H. E. Minor and W. H. Hager, eds., Balkema, Rotterdam, The Netherlands, 187–194.

Michel, J. M. (1984). "Some features of water flows with ventilated cavities." *J. Fluids Eng., Trans ASME*, Sept., 106, 319–326.

Ohtsu, I, Yasuda, Y., and Takahashi, M. (2000). "Characteristics of skimming flow over stepped spillways." *J. Hydraul. Eng.*, 126(11), 869–871.

Shvajnshtejn, A. M. (1999). "Stepped spillways and energy dissipation." *Gidrotekh. Stroit.*, 5, 15–21 (in Russian).

Silberman, E., and Song, C. S. (1961). "Instability of ventilated cavities." *J. Ship Res.*, 5(1), 13–33.

Toombes, L., and Chanson, H. (2000). "Air-water flow and gas transfer at aeration cascades: A comparative study of smooth and stepped chutes." *Int. Workshop on Hydraulics of Stepped Spillways*, Zürich, Switzerland, Balkema, Rotterdam, The Netherlands, 77–84.

Verron, J., and Michel, J. M. (1984). "Base-vented hydrofoils of finite span under a free surface: An experimental investigation." *J. Ship Res.*, 28(2), 90–106.

Yasuda, Y., and Chanson, H. (2003). "Micro- and macroscopic study of two-phase flow on a stepped chute." *Proc., 30th IAHR Biennial Congress*, Thessaloniki, Greece, J. Ganoulis and P. Prinos, eds., vol. D, 695–702.

Yasuda, Y., and Ohtsu, I. (1999). "Flow resistance of skimming flow in stepped channels." *Proc., 28th IAHR Congress*, Graz, Austria, session B14, (CD-ROM).

Discussion of "Hydraulic Design of Stepped Spillways" by Robert M. Boes and Willi H. Hager

September 2003, Vol. 129, No. 9, pp. 671–679.
DOI: 10.1061/(ASCE)0733-9429(2003)129:9(671)

A. D. Ghare[1]; P. D. Porey[2]; and R. N. Ingle[3]

[1] Sr. Lecturer, Civil Engineering Dept., D. C. V. Raman Institute of Technology, Nagpur, India.

[2] Professor, Civil Engineering Dept., Visvesvaraya National Institute of Technology, Nagpur, India.

[3] Emeritus Fellow, Civil Engineering Dept., Visvesvaraya National Institute of Technology, Nagpur, India.

The authors are to be complimented for presenting extensive experimental data on characteristics of aerated skimming flow over stepped spillways along with hydraulic design aspects of stepped spillways. The authors have focused their attention on various aspects, including onset of skimming flow, aeration characteristics, residual energy, and training wall design.

Considering the applicability of the design guidelines, the discussers would like to know the height of stepped spillway in the experimental setup for all 3 cases. Further, the authors may clarify regarding the limiting height of prototype stepped spillways up to which the design guidelines presented in this paper could be applied.

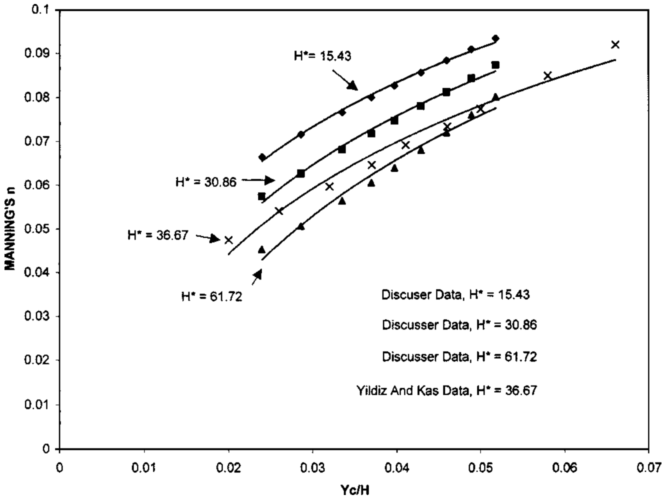


Fig. 1. Variation of Manning's n for different $H^\star$ values

The discussers would also like to know the number of steps provided in each case and the location of first step along the spillway profile. Can the authors suggest any readily usable explicit guidelines from hydraulic considerations for deciding on the step height, apart from the given RCC lift thickness? Some other investigators, including Rice and Kadavy (1996), Yildiz and Kas (1998), Chamani and Rajaratnam (1999) have indicated that the step height s affects the energy dissipation over stepped spillway.

Eq. (24) includes K, the roughness height perpendicular to the pseudobottom, which can be considered to be a representative term for step height s. In the last paragraph on energy dissipation, it is mentioned that Fig. 12 gives an idea of main parameters involved in the expression of relative residual energy. However, Fig. 12 does not indicate effect of any step height parameter on relative residual energy head ratio $[H_{\text{res}}/H_{\max}]$. Fig. (1) shows a plot compiled by discussers based on experimental data obtained by Ghare (2003) and Yildiz and Kas (1998), which show the effect of step height on Manning's equivalent n for a stepped spillway. In this plot H^* is considered a ratio of spillway height to step height. Can authors provide any other dimensionless plot that covers all the main parameters including step height s affecting the performance of the stepped spillway under skimming flow regime?

Proposed Eq. (24) is based on the results obtained from Eqs. (20) and (21). Hence the use of Eq. (24) appears to be a tedious process. As indicated by the authors in Fig. (12), the variation in relative residual energy head ratio for $\Phi = 40°$ and 50° is not appreciable; hence a simpler relationship for relative residual energy can be presented eliminating Φ as a variable. The resulting relationship would be applicable for Φ greater than 40°. Without a properly designed energy dissipation system on the downstream side, the hydraulic design of a stepped spillway system would be incomplete. The discussers would like to know the opinion of the authors regarding the applicability of the conventional conjugate depth relationship for stilling basin design in case of a stepped spillway where highly aerated flow near the toe of the spillway is encountered.

References

Chamani, M. R., and Rajaratnam, N. (1999). "Characteristics of skimming flows over stepped spillways." *J. Hydr. Engrg.* 125(4), 361–367.

J. Hydraul. Eng. 2005.131:525-527.

$=(h_{w,e1}+h_{w,e2})/2=0.87$ m in the continuity equation yields a terminal velocity of $v_{w,e}=q_d/h_{w,e}=20/0.87\approx 23$ m/s.

If the chute was long enough for the attainment of uniform flow, i.e., $H_{\mathrm{dam}}=H_{\mathrm{dam},u}\approx 70$ m, the normalized residual head would read $H_{\mathrm{res}}/H_{\max}=0.36$ according to Eq. (24*b*), with $f_b=0.067$ from Eq. (21), $D_{h,w,u}\approx 4h_{w,u}=4\cdot 0.80=3.20$ m and $0.1<K/D_{h,w,u}=0.23<1.0$. In this case, 64% of the flow energy of $H_{\max}\approx 75.2$ m would be dissipated on the spillway, and the terminal velocity would amount to $v_{w,e}\approx 20/0.80=25$ m/s.

Training Wall Design

With $\eta=1.2$ for concrete dams, the required sidewall height from Eq. (25) is $h_d=2.09$ m, with $h_{90,u}=1.74$ m from Eq. (5). A sidewall height of 2.1 m is proposed. If the downstream dam face were prone to erosion, and if it were essential to avoid overtopping of the training walls, distinction should be made about whether the crest profile above the point of tangency is smooth or stepped. In the latter case, the required wall height should be at least $h_d=1.5h_{90,u}=2.61$ m, whereas for a smooth crest profile, the wall height should be $h_d=h_{spray}=4s=4\times 1.2=4.8$ m over about $L=25s=25\times 1.2=30$ m from the crest to allow for the spray resulting from nappe impact on the first steps below the smooth crest (Boes and Minor 2002).

Conclusions

The following findings of the present experimental study apply:

1. The onset of skimming flow is expressed by the ratio of critical depth to step height and follows a linear function as expressed in Eq. (1).
2. The uniform equivalent clear water depth $h_{w,u}$ on stepped spillways depends on the chute angle and unit discharge only, as given in Eq. (4).
3. The characteristic uniform mixture depth $h_{90,u}$ according to Eq. (5) is a function of step height, unit discharge and chute angle.
4. The drawdown length to the approximate location of uniform flow attainment as given in Eq. (13) depends on chute angle and unit discharge only.

5 The bottom roughness friction factor is approximated for a wide range of spillway angles and relative roughness by Eq. (20) or (21).

6. The significant effect of aeration on the reduction of friction factors is illustrated by the ratio f_w/f_m as function of the mean air concentration, Eq. (22), where f_w and f_m are friction factors with and without consideration of flow aeration, respectively.
7. A general expression of residual energy head along stepped chutes is given in Eq. (24), with distinction between developing and uniform flow regions.
8. Stepped spillway training walls should be designed according to Eq. (25), taking into account the erosion potential of the downstream dam face.

These conclusions in conjunction with the results of Boes and Hager (2003) allow for the hydraulic design of stepped spillways for a wide range of boundary conditions including typical applications both for embankment and gravity dams.

Acknowledgment

The present project was financed by the Swiss National Science Foundation, Grant No. 21-45424.95. The assistance of Professor Y. Yasuda, Nihon University, Tokyo, in providing experimental data is also gratefully acknowledged.

Notation

The following symbols are used in this paper:

b = spillway or river width;
$\overline{C}$ = depth-averaged air concentration;
C_i = depth-averaged air concentration at inception point;
$\overline{C_u}$ = uniform depth-averaged air concentration;
$C(y)$ = local air concentration;
$D_{h,w}$ = $4R_{h,w}$ hydraulic diameter;
$D_{h,\mathrm{eff}}$ = $wD_{h,w}$ effective hydraulic diameter;
F = $u/(gh)^{1/2}$ local Froude number;
F_0 = $q_w/(gh_0^3)^{1/2}$ approach Froude number at jetbox;
F_* = $q_w/(g\sin\phi s^3)^{1/2}$ roughness Froude number;
f = Darcy–Weisbach friction factor of unaerated flow;
f_b = friction factor of bottom roughness;
f_m = Darcy–Weisbach friction factor in two-phase flow without consideration of aeration;
f_s = skin friction factor of sidewall roughness;
f_w = Darcy–Weisbach friction factor in two-phase flow with consideration of aeration;
g = gravitational acceleration;
H_{dam} = vertical spillway or dam height;
$H_{\mathrm{dam},u}$ = vertical distance from spillway crest to close uniform equivalent clear water flow;
$H_{\max}$ = maximum reservoir energy head;
H_{res} = residual energy head;
h = local flow depth;
h_c = critical depth;
h_d = training wall design height;
h_m = mixture depth;
$h_{m,i}$ = mixture depth at inception point;
h_{spray} = spray height resulting from nappe impact on steps;
h_u = uniform flow depth;
h_w = $(1-\overline{C})h_{90}$ equivalent clear water depth;
$h_{w,e}$ = clear water depth at chute end;
$h_{w,i}$ = clear water depth at inception point;
$h_{w,u}$ = uniform equivalent clear water depth;
h_{90} = $h(C=0.90)$ characteristic mixture depth with local air concentration of $C=0.90$;
h_0 = approach flow depth at jetbox;
$h_{90,u}$ = uniform characteristic mixture depth;
K = $s\cdot\cos\phi$ roughness height perpendicular to pseudobottom;
L_i = black water length from spillway crest to inception point;
L_s = $s/\sin\phi=K/(\sin\phi\cos\phi)=2K/\sin(2\phi)$ distance between step edges, roughness spacing;
Q_d = design discharge;
Q_w = water discharge;
q_d = design discharge per unit width;
q_w = water discharge per unit width;
R = $uD_{h,w}/\nu$ Reynolds number;
$R_{h,w}$ = hydraulic radius;
S_f = friction slope;
s = step height;
u = flow velocity in x direction;
$v_{m,i}$ = mixture velocity at inception point;
$v_{w,e}$ = clear water velocity at chute end;

$v_{w,i}$ = clear water velocity at inception point;
w = shape correction coefficient;
x = streamwise coordinate originating at spillway crest;
x_s = $h_c^3/(h_{w,u}^2 \sin \phi)$ scaling length;
x_u = drawdown length from spillway crest to close uniform equivalent clear water flow;
Y = h/h_u normalized local flow depth;
Y_c = h_c/h_u normalized critical depth;
y = transverse coordinate originating at pseudobottom;
z_i = vertical black water length from spillway crest to inception point;
α = energy correction coefficient;
η = safety factor;
ν = kinematic viscosity of water
Π_1 = $0.5-0.42\sin(2\phi)$ function taking into account roughness spacing;
Π_2 = $(K/D_{h,w})^{0.2}$ function taking into account relative chute roughness;
σ = factor originating from Gauckler–Manning–Strickler formula;
ϕ = chute angle from horizontal; and
χ = x/x_s normalized streamwise coordinate.

References

Boes, R. M. (2000). ‘‘Zweiphasenströmung und Energieumsetzung auf Grosskaskaden.’’ PhD thesis, VAW, ETH Zurich, Switzerland (in German).

Boes, R. M., and Hager, W. H. (1998). ‘‘Fiber-optical experimentation in two-phase cascade flow.’’ *Proc., Int. RCC Dams Seminar*, K. Hansen, ed., Denver.

Boes, R. M., and Hager, W. H. (2003). ‘‘Two-phase flow characteristics of stepped spillways.’’ *J. Hydraul. Eng.*, 129(9), 661–570.

Boes, R. M., and Minor, H.-E. (2000). ‘‘Guidelines for the hydraulic design of stepped spillways.’’ *Proc., Int. Workshop on Hydraulics of Stepped Spillways*, VAW, ETH Zurich, H.-E. Minor and W. H. Hager, eds., Balkema, Rotterdam, The Netherlands, 163–170.

Boes, R., and Minor, H.-E. (2002). ‘‘Hydraulic design of stepped spillways for RCC dams.’’ *Hydropower Dams*, 9(3), 87–91.

Chamani, M. R., and Rajaratnam, N. (1999a). ‘‘Onset of skimming flow on stepped spillways.’’ *J. Hydraul. Eng.*, 125(9), 969–971.

Chamani, M. R., and Rajaratnam, N. (1999b). ‘‘Characteristics of skimming flow over stepped spillways.’’ *J. Hydraul. Eng.*, 125(4), 361–368.

Chanson, H. (1994). *Hydraulic design of stepped cascades, channels, weirs and spillways*, Pergamon, Oxford, U.K.

Chanson, H. (1996). ‘‘Prediction of the transition nappe/skimming flow on a stepped channel.’’ *J. Hydraul. Res.*, 34(3), 421–429.

Chanson, H. (2000). ‘‘Experience, operation and accidents with stepped cascades.’’ *Presented at the Int. Workshop on Hydraulics of Stepped Spillways*, VAW, ETH, Zurich.

Chanson, H., Yasuda, Y., and Ohtsu, I. (2000). ‘‘Flow resistance in skimming flow: A critical review.’’ *Proc., Int. Workshop on Hydraulics of Stepped Spillways*, VAW, ETH Zurich, H.-E. Minor and W. H. Hager, eds., Balkema, Rotterdam, The Netherlands, 95–102.

Chow, V. T. (1959). *Open channel hydraulics*, McGraw-Hill, New York.

Frizell, K. H., Smith, D. H., and Ruff, J. F. (1994). ‘‘Stepped overlays proven for use in protecting overtopped embankment dams.’’ *Proc., ASDSO Annual Conf.*, Boston.

Hager, W. H. (1991). ‘‘Uniform aerated chute flow.’’ *J. Hydraul. Eng.*, 117(4), 528–533.

Hager, W. H., and Blaser, F. (1998). ‘‘Drawdown curve and incipient aeration for chute flow.’’ *Can. J. Civ. Eng.*, 25(3), 467–473.

Hager, W. H., and Boes, R. M. (2000). ‘‘Backwater and drawdown curves in stepped spillway flow.’’ *Proc., Int. Workshop on Hydraulics of Stepped Spillways*, VAW, ETH Zurich, H.-E. Minor and W. H. Hager, eds., Balkema, Rotterdam, The Netherlands, 129–136.

Homann, C., Schramm, J., Demny, G., and Köngeter, J. (2000). ‘‘Wellenerscheinungen in der Hochwasserentlastungskaskade der Sorpetalsperre.’’ *Proc., 2. Ju Wi-Treffen*, VAW, ETH Zurich, Switzerland, 30–32 (in German).

James, C. S. (2001). ‘‘Discussion of ‘Onset of skimming flow on stepped spillways’ by M. R. Chamani and N. Rajaratnam.’’ *J. Hydraul. Eng.*, 127(6), 519.

Marchi, E. (1961). ‘‘Il moto uniforme delle correnti liquide nei condotti chiusi e aperti.’’ *Energ. Elet.*, 38(4), 289–301, (5), 393–413 (in Italian).

Matos, J. (2000a). ‘‘Discussion of ‘Hydraulics of skimming flow on modeled stepped spillways’ by G. G. S. Pegram, A. K. Officer, and S. R. Mottram.’’ *J. Hydraul. Eng.*, 126(12), 948–950.

Matos, J. (2000b). ‘‘Discussion of ‘Characteristics of skimming flow over stepped spillways’ by M. R. Chamani and N. Rajaratnam.’’ *J. Hydraul. Eng.*, 126(11), 865–869.

Matos, J. (2001). ‘‘Discussion of ‘Onset of skimming flow on stepped spillways’ by M. R. Chamani and N. Rajaratnam.’’ *J. Hydraul. Eng.*, 127(6), 519–521.

Matos, J., and Quintela, A. (1995). ‘‘Guidelines for the hydraulic design of stepped spillways for concrete dams.’’ *ICOLD Energy Dissipation Bull.*.

Matos, J., Sanchez, M., Quintela, A., and Dolz, J. (1999). ‘‘Characteristic depth and pressure profiles in skimming flow over stepped spillways.’’ *Proc., 28th IAHR Congress*, (CD-ROM) H. Bergmann, R. Krainer, and H. Breinhälter, eds., Graz, Austria B14.

Ohtsu, I., and Yasuda, Y. (1997). ‘‘Characteristics of flow conditions on stepped channels.’’ *Proc., 27th IAHR Congress*, F. M. Holly and A. Alsaffar, eds., San Francisco, 583–588.

Ohtsu, I., Yasuda, Y., and Takahashi, M. (2000). ‘‘Discussion of ‘Hydraulics of skimming flow on modeled stepped spillways’ by G. G. S. Pegram, A. K. Officer, and S. R. Mottram.’’ *J. Hydraul. Eng.*, 126(12), 950–951.

Ohtsu, I., Yasuda, Y., and Takahashi, M. (2001). ‘‘Discussion of ‘Onset of skimming flow on stepped spillways’ by M. R. Chamani and N. Rajaratnam.’’ *J. Hydraul. Eng.*, 127(6), 522–524.

Rajaratnam, N. (1990). ‘‘Skimming flow in stepped spillways.’’ *J. Hydraul. Eng.*, 116(4), 587–591.

Schläpfer, D. (2000). ‘‘Treppenschussrinnen.’’ MSc thesis, VAW, ETH Zurich, Switzerland (in German).

Schröder, R. C. M. (1990). ‘‘Hydraulische Methoden zur Erfassung von Rauheiten.’’ *DVWK Bull. 92*, Paul Parey, Germany (in German).

Stephenson, D. (1991). ‘‘Energy dissipation down stepped spillways.’’ *Int. Water Power Dam Constr.*, 43(9), 27–30.

Tatewar, S. P., Ingle, R. N., and Porey, P. D. (2001). ‘‘Discussion of ‘Onset of skimming flow on stepped spillways,’ by M. R. Chamani and N. Rajaratnam.’’ *J. Hydraul. Eng.*, 127(6), 524.

Tozzi, M. J. (1992). ‘‘Caracterização/comportamento de escoamentos em vertedouros com paramento em degraus.’’ PhD thesis. Univ. of São Paulo, Sao Paulo, Brazil (in Portuguese).

Wahrheit-Lensing, A. (1996). ‘‘Selbstbelüftung und Energieumwandlung beim Abfluss über treppenförmige Entlastungsanlagen.’’ PhD thesis, Univ. of Karlsruhe, Karlsruhe, Germany (in German).

Yasuda, Y., and Ohtsu, I. (1999). ‘‘Flow resistance of skimming flows in stepped channels.’’ *Proc., 28th IAHR Congress*, H. Bergmann, R. Krainer, and H. Breinhälter, eds. (CD-ROM), Graz, Austria, B14.

Yildiz, D., and Kas, I. (1998). ‘‘Hydraulic performance of stepped chute spillways.’’ *Hydropower Dams*, 5(4), 64–70.

J. Hydraul. Eng. 2003.129:671-679.

Cardinal Dam Raising - Top of Dam El. 983.0 - PMF Event
Flow vs. Stage Hydrograph

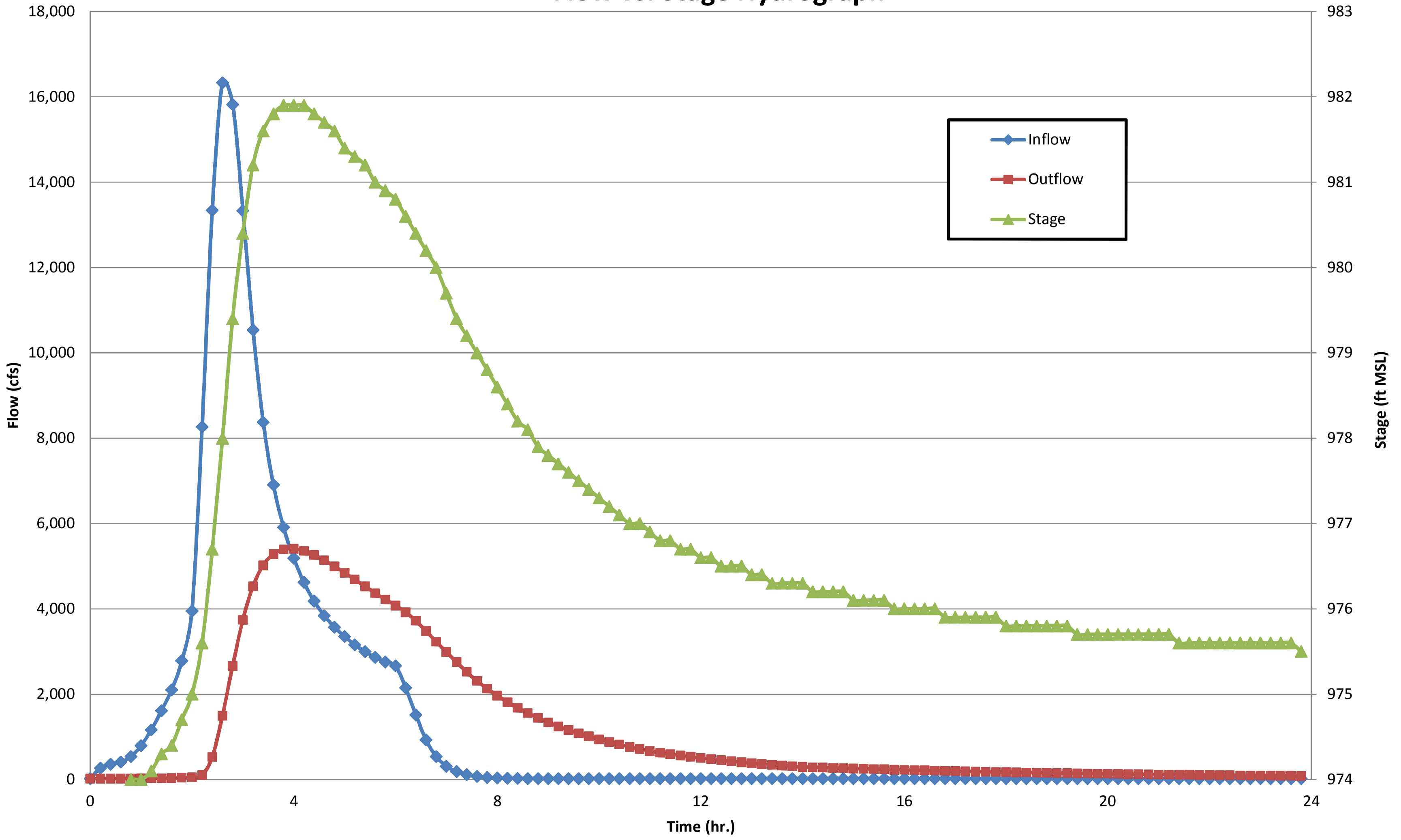


PLATE 43

ATTACHMENT F

MAINTENANCE PLAN

Operation, Maintenance, and Inspection Manual for Fly Ash Dam II & Bottom Ash Ponds Complex Dikes

American Electric Power

Cardinal Operating Company
306 County Road 7E Brilliant,
Ohio 43913

Plant Ash Dam: 0105-004
Fly Ash No. 1 Dam: 0205-009
Fly Ash No. 2 Dam: 0205-010

March 2015

RECEIVED
ODNR
MAR 1 9 2015
Soil & Water Resources
Dam Safety Program

STATE OF OHIO
Soil and Water Resources
DAM SAFETY REPAIR PROGRAM
APR 06 2015
APPROVED

TABLE OF CONTENTS

LIST OF EXHIBITS

APPENDICES

Appendix	Description
A	Dam Maintenance Record
B	Dam Inspection Instructions and Dam Inspection Checklists
C	Reference Drawings and Photos
D	ODNR Fact Sheets
E	Dam Inspection Guidelines

1.0 INTRODUCTION

This Operation, Maintenance, and Inspection (OM&I) Manual was prepared in accordance with Section 1501:21-15-06 of the Ohio Laws and Administrative Rules for Issuing Construction Permits for and Making Periodic Inspections of Dams, Dikes, and Levees. It is intended to assist the owner in regular operation, maintenance, and inspection activities. This manual was prepared for Cardinal Plant's Fly Ash Dam II (FAD II) and the Bottom Ash Ponds (BAP) complex conveying coal ash slurry. Exhibit 1 shows the location of the dams.

The Cardinal FAD II coal ash dam and the BAP complex dikes have been conservatively designed and carefully constructed; however, small problems can develop over time. Experience has shown that some of these small problems can become major problems if corrective measures are not promptly taken. The main intent of this manual, therefore, is to provide the guidelines for a regular operation, maintenance, and inspection program that will detect problems at an early stage so that they can then be corrected. This manual presents the procedures for the operation, maintenance and inspection of the FAD II and the BAP complex dikes.

Much of the information in this manual has been based on the requirements of publications issued by the Ohio Department of Natural Resources (ODNR), Division of Water, Dam Inspection Section. The publications are a series of Fact Sheets; copies of pertinent Fact Sheets are contained in Appendix D. In addition to providing basic recommendations for operation, maintenance, and inspection procedures, the Fact Sheets give a great deal of background information, including causes of dam failures, common problems and solutions, and reference to organizations and bureaus which can provide information and advice. The Fact Sheets are valuable publications to have as an adjunct to this manual.

This OM&I Manual supersedes any and all previous OM&I Manuals that have been used at the facility.

2.0 PROJECT DESCRIPTION

2.1 General

FAD I, FAD II, and the BAP complex are owned by AEP and Buckeye Power and operated by Cardinal Operating Company. They are located near the Cardinal Power Plant in Wells Township, Jefferson County, near Brilliant, Ohio. The Cardinal FAD I and FAD II are located approximately 1 mile northwest of the Cardinal Power Plant. The BAP complex is located at the southern part of the Cardinal power plant. The ponds were constructed for the settling/sedimentation and collection/storage of coal combustion byproducts. Exhibit 1 shows the FAD II and BAP complex in relation to the Cardinal Plant.

2.2 Fly Ash Dam I

Cardinal Fly Ash Dam I (FAD I) is the plant's original fly ash retention dam constructed in the early 1970s. The dam is an earth and rockfill dam having a final design crest elevation of 1001.5 feet. The dam has upstream (u/s) and downstream (d/s) slopes of approximately 2.5 Horizontal to 1 Vertical (2.5H:1V). As ash placement behind FAD I reached its maximum allowed level, Cardinal FAD II was constructed and began operation in the late 1980s. Fly Ash Dam I reservoir is closed, no longer receives fly ash slurry, and has no permanent pool. This area has been remitted by the Ohio Environmental Protection Agency (EPA) as a solid waste landfill (Permit to Install [PTI] Permit No. 06-07993, dated May 11, 2007) for the disposal of synthetic gypsum generated by the air pollution control equipment constructed at the Cardinal plant that captures sulfur dioxide emissions. Flow through FAR I is conveyed to FAR II via the FAD I emergency spillway.

2.3 Fly Ash Dam II

FAD II is located on Blockhouse Run, which flows directly into the Ohio River. Blockhouse Run splits into two branches, designated as the East Branch and the West Branch. The split in Blockhouse Run is approximately one mile upstream of the Ohio River. Runoff from both the east and west branch watersheds drains into the reservoir.

Fly Ash Reservoir II (FAR II), created by FAD II, is utilized for the storage of fly ash, which is discharged as slurry from six (6) 10” discharge pipes located at the upstream (north) end of the reservoir as shown on Exhibit 2. The fly ash settles out within the reservoir as the water flows toward the dam where the effluent overflows through the service spillway (overflow structure). Stop logs are placed in the discharge shaft of the overflow structure as necessary to maintain settling action or to limit discharge. The reservoir will cover approximately 168 acres at Elevation 974, the maximum operating pool elevation.

The FAD II dam consists of a 250-foot high arched embankment with a 13 ft high MSE Wall on top of the roller compacted concrete (RCC) cap on the upper 50 feet of the upstream face and an emergency spillway on the left abutment that is an open channel cut through rock. The dam has a crest elevation of 983 feet. The dam crest has a width of 22 feet and a length of 1,645 feet. The dam is designed for a storage capacity of 11,868 acre-feet with stop logs at elevation 972.5 feet and with a corresponding maximum operating pond elevation of 974 feet. Table 1 summarizes pertinent information for FAD II.

Table 1 FAD II and BAP Complex Data

Parameter	FAD II	BAP Complex
Embankment Crest Elevation (feet)	983	670
Emergency Spillway Crest Elevation (feet)	975.5	665.5
Maximum Operating Pool Level (feet)	974.0	665
Operating Pool Freeboard (feet)	9	665
Maximum Stop Log Elevation (feet)	972.5	665.5
Surface Area (acres) at Pool Level	161	29

Table 2 includes a list of inlet and outlet structures in addition to an inventory of the works and other significant components existing at the FAD II and their location and characteristics. In addition, Appendix C includes reference information in form of water cycle Diagram, Drawings, and photos of the components.

Table 2

Features and appurtenances	Description
Embankments	Approximately 1645 ft at crest elevation of 983.
Inflow pipes	Six 10” diameter fly ash sluicing steel pipes, 12.87 MGD (EL. 962)
Spillways	Sizes 48” wide, Max elevation: 972.5, adjusted with 6” high stop logs (concrete).
Emergency spillway/overflow	Size: 110.5'x 7.5' elevation: 975.5 (Concrete)
Embankment drainage systems	Exhibit 2 and Appendix C
Monitoring weirs, flumes	Exhibit 2 and Appendix C
Piezometers and monitoring wells	Appendix C
Inclinometers	Annual monitoring, See Appendix C for location
Staff gauge & signage	Exhibit 2 and Appendix C
Settlement monuments	Annual monitoring, See Appendix C for location
Abandoned structures	Grouted in place, Exhibit 2

2.3.1 Fly Ash Dam II Service Spillway (Over Flow Structure)

The service spillway is extended with a new vertical concrete shaft structure with one side opening on top of a sloping concrete shaft structure with one side opening, four feet wide, connecting into a 54 inch diameter pre-stressed concrete cylinder pipe (PCCP).

The bottom of the sloping concrete shaft and the entire 54-inch concrete pipe were constructed within bedrock as part of the 1997 FAD II rising. Stop logs are utilized to promote settling action and control the operating pool level.

Stop logs will be incorporated into the new vertical section to continue to allow for the incremental raising of the operating pool.

2.3.2 Fly Ash Dam II Emergency Spillway

The principle spillway (or overflow structure) is located on the left abutment and is an open channel cut through rock. The flow capacity of the emergency spillway is designed to pass the Probable Maximum Flood when the reservoir reaches its maximum pond elevation, without overtopping the dam. At intermediate pool levels, floods of lesser magnitude will be discharged through the service spillway.

The fly ash dam is normally unattended and the service spillway structure has no remote controlled system to regulate the flow. Because of the nature of the pond and the design of the dam and service spillway structure, there exists sufficient freeboard to mitigate concerns of overtopping during a rainfall event.

2.3.3 Downstream Effects

There are no dams or residences located above the dam or in the east or west watershed boundaries. There are no dams located downstream that could be operated during an emergency to store flood flows. The Ohio River, Cardinal Plant, State Route 7 and the Tidddale subdivision of Brilliant, Ohio, all lie directly downstream of the proposed dam. Therefore, a sudden failure of the dam will likely result in loss of human life and damage to homes, high value utility installation and both a railroad and a public road.

2.4 Bottom Ash Pond Complex

The BAP Complex at the Cardinal Plant consists of a BAP (approximately 20 acres) and a Recirculation Pond (RCP) (approximately 9 acres). Flow from the BAP is discharged to the RCP. The exterior dike crest elevation varies and an overflow conduit with an inlet elevation of approximately 665.5 feet controls the maximum Recirculation Pond water level. In 2008, plastic sheet piling was driven across the recirculation pond to modify its flow pattern in preparation of allowing the present overflow structure to discharge from the basin. The arrangement of the BAP Complex is shown in Exhibit 3 and Table 1 summarizes pertinent information for BAP Complex.

The bottom ash pond complex is located along the west bank of the river just to the south of the main plant area. The bottom ash pond complex consists of two components: the bottom ash pond and the recirculation pond (RCP). The bottom ash pond complex is utilized for the storage and collection of bottom ash, Bottom ash-laden water and other storm water is discharged via thirteen (13) pipes into the northwest corner of the bottom ash pond, the coarse bottom ash settles out closer to the discharge lines while the finer bottom ash settles out at farther locations within the pond. Near the southeast side of the bottom ash pond, Overflow Discharge structure (a drop outlet and a 36”-pipe) controls flow from the bottom ash pond into the recirculation pond. The water in the RCP is used to sluice the fly ash form the plant to FAD II via the pump station

Table 3 includes a list of inlet and outlet structures in addition to an inventory of the works existing at the BAP complex and other significant components and their location and characteristics. In addition, Appendix C includes such information in form of water cycle Diagrams, Drawings, and photos of the referenced components.

Table 3

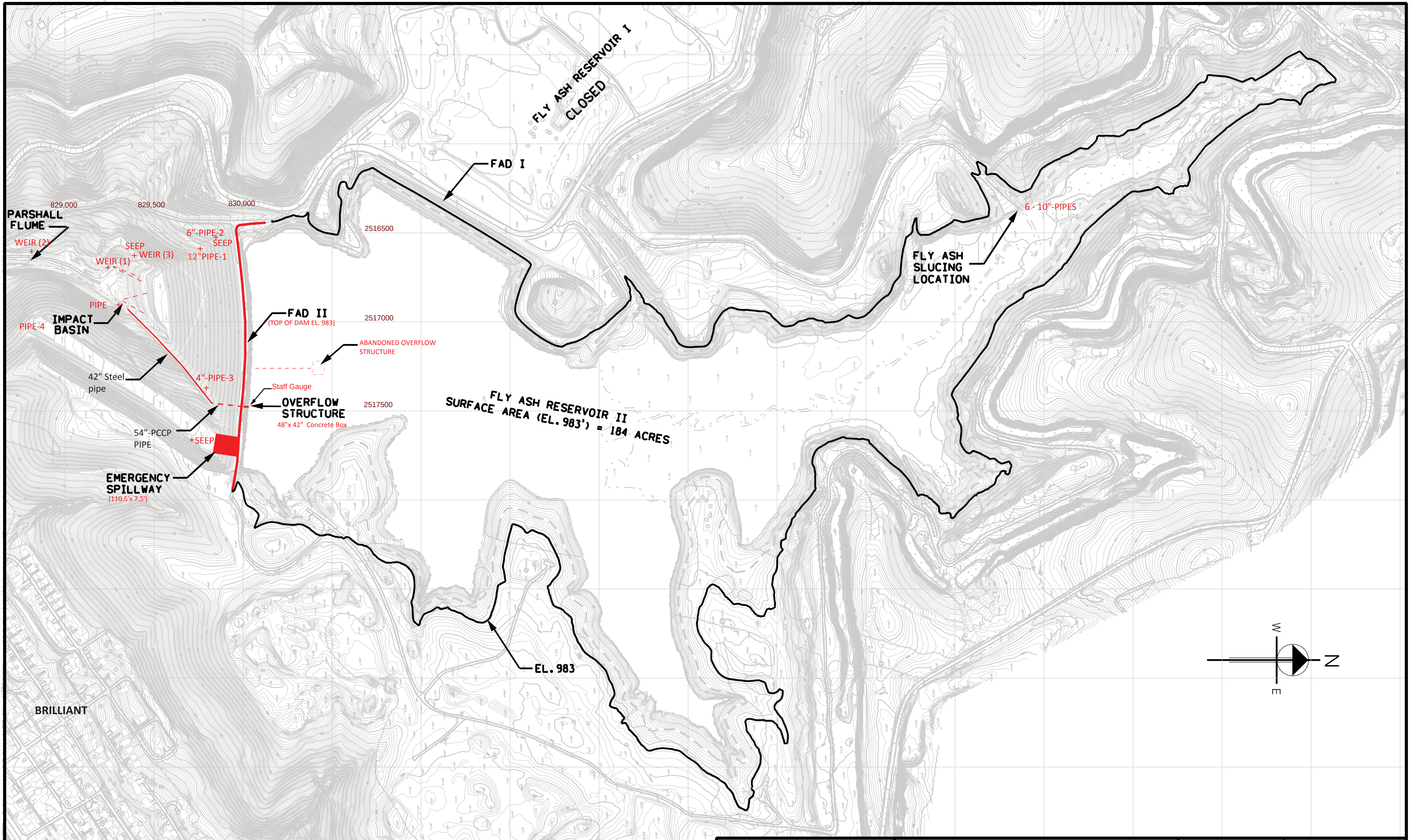
Features and appurtenances	Description
Embankments	Approximately 4700 ft at crest elevation of 670.
Inflow pipes	13 10” diameter fly ash sluicing pipes
Outflow pipes	36” diameter steel pipe into to 36” diameter PVC pipe Exhibit 3 and Appendix C Pumphouse intake pipes: Two 21” diameter for ash sluicing (El 660).
Spillways	Drop inlet with stoplogs and 36” pipe;
Monitoring weirs, flumes	Exhibit 3 and Appendix C
Piezometers and monitoring wells	Annual monitoring, See Exhibit 3 for location
Staff gauge & signage	Exhibit 3
Emergency spillway/overflow	Sharp –crested 3 ft wide 10” weir at EL 665.5
Pump house	Intakes elevation 660, capacity:16.9 MGD

The BAP is located north of the RCP and they are separated by an earthen embankment. Perimeter dikes surround the bottom ash pond complex and are referred to as the BAP complex dike. The crest elevation of the embankments varies with a minimum elevation of 670 feet MSL. An overflow conduit with a variable inlet elevation and a pipe between the

BAP and the RCP controls the maximum BAP water level. The total length of the Interior embankment is approximately 2,500 feet and the total length of the exterior embankment along the Ohio River is approximately 2,000 feet. For comparison, the normal pool for this stretch of the Ohio River is El. 644. Both ponds are isolated from exterior surface water inflow. An overflow conduit with an inlet elevation of approximately 665.5 feet controls the maximum recirculation pond water level. In 2008, plastic sheet piling was driven across the recirculation pond to modify its flow pattern in preparation of allowing the present overflow structure to discharge from the basin. In 2010, the top of the BAP complex exterior dikes were re-graded to insure that the minimum elevation of 670 is applicable all over the dike. The arrangement of bottom ash complex is shown in Exhibit 3.

2.4.2 Downstream Effects

FAD II located upstream of the BAP complex dikes. The Ohio River located downstream of the BAP complex dikes. Therefore, sudden failures of the dikes will not likely result in loss of human life or damage to homes.



DRN BY: Amanda Graphics

DATE: 1-16-2014

SCALE: NTS

AMERICAN ELECTRIC POWER

BRILLANT CARDINAL PLANT OHIO

FLY ASH DAMS I AND II

AEP AMERICAN ELECTRIC POWER

AEP SERVICE CORP.
1 RIVERSIDE PLAZA
COLUMBUS, OH 43215

EXHIBIT 4

3.0 OPERATION OF THE RESERVOIRS

3.1 Mechanical Equipment

The mechanical equipment associated with the FAD II includes three aerators a pump station. The pump station is use to provide water for Ohio American Energy Inc's (OAEI) coal prep plant and is operated by OAEI. The aerators operated by AEP (Please see table 4 below for contact info). The aerators are necessary to mix the pond waters and maintain oxygenated conditions to promote algae bloom to consume phosphate carryover from the synthetic gypsum pollution control equipment. Therefore, the aerators should be inspected periodically to assure proper operating conditions.

The mechanical equipment associated with the BAP Complex includes the pumps located at the Pumphouse in the RCP area. Plant control room coordinator is responsible for monitoring and adjusting the pumping rates for the recirculation water. Typical and maximum flow rates are included in the Plant water cycle included in appendix C.

Table 4. Contacts List for Operating, Maintenance, and Inspecting the dams.

Name	Address	Phone	Responsibility
Eric (Randy) Sims	306 County Road 7 East Brilliant, OH 43913	(740) 314-9982	Dam safety Officer
Unit 3 Team Leader	306 County Road 7 East Brilliant, OH 43913	(740) 598-6530	Management of flow rates in and from impoundments

3.2 Outflow Measurements

Flow measurements from FAD II are measured utilizing a Parshal flume at the outlet of the impact basin immediately downstream from the dam as shown on Exhibit 2.

3.3 Drawdown Plan

There is no drain for the fly ash reservoir II due to its purpose of sedimentation. The only procedure that exists for lowering the pool elevations is the removal of the grouted stop logs in the drop inlet structures. If necessary, use alternate means to drain the pond, such as siphons or pumps. It may be necessary to excavate a hole in accumulated fly ash to enhance removal of water. All drawdown activities are to be coordinated with AEP Civil Engineering.

3.4 Safe Rate of Reservoir Drawdown

Deliberate drawdown beyond normal operational requirements shall typically not exceed 1 foot per week, except for emergency situations. Faster drawdown rates may be required under emergency conditions with the approval of the AEP Geotechnical Engineering.

3.5 Safe Dredging and temporary Stockpiling

BAP is the only pond among Cardinal Plant ponds that currently involves dredging and temporary stockpiling material above the top of dike elevation. Dredging and temporary stockpiling activities take place on regular bases to allow for the use of the bottom ash pond for settling of bottom ash. The dredged material is being beneficially used in the construction activities at the plant. Coarse bottom ash excavated closer to the sluicing point and stockpiled temporarily to allow for water draining. The finer bottom ash is usually dredged into dredging cell that exists within the BAP complex. The dredging unit is not allowed to operate next to the toe of the dam due not only to water depth requirements but also for dam safety. Once dewatered, the stockpiles are excavated and materials transported off-site for beneficial use in landfill construction.

3.6 Vandalism

"No Trespassing" signs shall be posted where appropriate. Railings or fences and warning signs shall be erected around dangerous areas.

3.7 Emergency Conditions

If any of the following conditions occur or appear imminent, the Emergency Action Plan (EAP) (separate document) shall be implemented immediately:

1. Overtopping or nearly overtopping of the embankment.
2. Piping through the embankment, spillway, or foundation.
3. A large slide in the embankment.

3.8 Records

Accurate records shall be kept of the following items:

1. Maintenance and major repairs. Appendix A contains a sample maintenance/repair log; an alternate log system may be used following plant record keeping procedures.

2. Specific observations and changes recorded and photographs taken during normal inspection periods (see Appendix B).
3. Date, hour, and maximum elevation of extreme high-water occurrences and the associated rainfall.
4. Amount, rate, and reasons for drawdown.
5. Readings made of water levels in piezometers in and near the embankment.
6. Complete and up-to-date set of as-built plans and specifications which show all changes made since the completion of the dam.
7. Visual observation of the horizontal and vertical alignment on an annual basis. If needed, the alignments should be surveyed to verify any changes.
8. Seepage location, quantity and content of flow, and size of wet area for later comparison. V-notch weirs can be used to collect and measure flow rates.
9. Erosion location and extent of erosion for later comparison.

4.0 MAINTENANCE PLAN

This section describes general maintenance procedures to be implemented at Cardinal FAD II and the BAP complex. In addition to the information provided in the following paragraphs, the ODNR has prepared a series of Fact Sheets for guidance on operation and maintenance at dams; several pertinent fact sheets are included in Appendix D for quick reference by AEP. Maintenance work to control seepage; repair cracks, slides, sloughing, damaged or deteriorated riprap; fill settled or low areas in the embankment; and repair concrete appurtenances should be performed based on the recommendations of AEP Civil Engineering.

4.1 Vegetation

1. Grassed areas shall be mown at least twice per year.
2. Paths created by pedestrian, vehicular, or animal traffic shall be minimized, and any barren areas which develop should be seeded.
3. Any cracks and/or erosion gullies which develop shall be completely filled with thoroughly compacted soil. The area shall be resodded if less than 100 square feet (sf), and reseeded if larger than 100 sf.
4. Trees and brush shall not be permitted to grow on the embankment. Tree and brush growth in the creek channel downstream of the FAD II impact basin shall be minimized. Remove any trees or brushes from the embankment and within 25 ft of the groins before they become established. The roots of any tree that is cut down should be pulled out. The resulting hole should be backfilled with tamped topsoil and reseeded. Replace areas of sparse or displaced riprap on the upstream slopes. This should be budgeted and performed annually to assure no growth of trees and brush on the embankment. ODNR Fact Sheet 94-28, Trees and Brush, in Appendix D, outlines the importance of properly maintained embankment vegetation.

4.2 Erosion

1. Promptly repair any eroded areas on the embankment to prevent more serious damage to the embankment (see Section 4.1 Vegetation). Repair erosion gullies to provide an even slope surface. Minor rills and gullies shall be filled with compacted cohesive soil, and then top soiled and seeded.
2. Erosion in large gullies can be slowed by stacking and securing bales of hay across the gully until permanent repairs can be made.
3. Causes of erosion shall be eliminated. Surface drainage should be spread out in thin layers as sheet flow.

4.3 Seepage

1. Any areas of seepage shall be noted and observed for evidence of piping erosion. Seepage containing soil is a sign of potential serious damage to the dam which may lead to failure of the dam and should be promptly addressed. Professional engineering assistance for control of any seepage problems shall be obtained.
2. Maintain written records of seepage (see Section 3.7 Records).

4.4 Cracks, Slides, Sloughing, and Settlement

1. Cracks, slides, sloughing, and settlement are signs of embankment distress and indicate that maintenance or remedial work is necessary.
2. A Professional Engineer shall determine the cause of stress before any repairs are made. Maintain written records of problems found and repairs completed (see Section 3.7 Records).

4.5 Rodent Control

1. Activities of rodents, such as groundhogs, muskrats, and beavers can endanger the structural integrity and proper performance of an embankment. Groundhogs and muskrats burrow into an embankment, thereby weakening it and creating seepage paths. Rodent control is therefore essential for a well-maintained dam. Refer to ODNR Fact Sheet 94-27, Rodent Control, in Appendix D, for further information.
2. Repair rodent burrows and implement rodent control procedures as follows:

i. Rodents may be controlled by fumigants. More detailed information on rodent control is contained in ODNR Fact Sheet 94-27, Rodent Control, in Appendix D. Fumigate rodent burrows with ignitable gas cartridges. To fumigate a burrow, light and drop an ignitable gas cartridge as deep into the burrow as possible. The burrow entrances should then be plugged with compacted soil. The procedure should be repeated at all burrow holes. The gas in the cartridge is non-poisonous. However, one should avoid inhaling the gas. Gas cartridges can be purchased at any local farm supply store.

ii. Backfill burrows by following the mud-packing method. First, place one to two lengths of metal stove or vent pipe in a vertical position over the entrance of the burrow. Mud-packing slurry should be made by adding water to a 90 percent bottom ash and 10 percent cement mixture. The slurry should then be poured into the burrow through the vertical pipe. Fly ash or bentonite may be added, as needed, to increase the flowability of slurry. After the burrow is filled, the pipe should be removed. Dry earth should be tamped into the burrow entrance and reseeded. A method for backfilling by mud packing is described in ODNR Fact Sheet 94-27, Rodent Control, in Appendix D.

4.6 Debris

Debris shall be removed from the outlet structures and their discharge pipes to allow free discharge. Caution should be used during high pond levels.

4.7 Concrete Structures

1. All deteriorated concrete surfaces (i.e., spalling, cracking, pitting, etc.) shall be repaired.
2. If sealant is observed to be missing from construction/expansion joints on the concrete outlet structures, monitor the condition and replace the sealant if necessary.

4.8 Toe Drain

1. The toe drain outlets should be inspected and observations recorded on a semiannual basis. Space to record these observations is provided in the Inspection Record form in Appendix B.

2. Areas of known seepage should be monitored for evidence of piping erosion. Seepage containing soil is a sign of potential serious damage to the dam which may lead to failure of the dam and should be promptly addressed. Professional engineering assistance for control of any seepage problems should be obtained.
3. In addition to quarterly monitoring, the toe drain outlet should be monitored during and after periods of high reservoir levels (greater than 2 foot of water over the principal spillway). If flow significantly increases at any time, contact a Professional Engineer for evaluation of the recorded data.

5.0 INSPECTION PROGRAM

5.1 Purpose

The purpose of this inspection program is to detect and document any changes in condition of the dam. AEP has an established Dam Inspection and Maintenance Program (DIMP) applicable throughout the service life of the facility. When a change in condition is detected, AEP-Civil Engineering staff and/or a Professional Engineer shall be contacted to identify any necessary remedial repair or maintenance work. The DIMP also provides a mechanism by which to activate the EAP which is made part of this Operations, Maintenance and Inspection Manual. The program consists of the following steps:

1. Conduct scheduled and unscheduled field inspections to check for signs of malfunction and to read the geotechnical instrumentation.
2. Graphically plot and interpret field measurements.
3. Investigate problems as they develop.
4. Design and implement preventive and remedial measures as required.
5. Perform regularly scheduled and routine maintenance work on the dam and its appurtenances.
6. Activate the EAP in the event that an unsafe condition is detected.

The description of the field instrumentation and the details of the DIMP are presented in the following sections.

For clear identification, a pictorial representation of potential problems and resolutions has been excerpted from Federal Emergency Management Agency (FEMA) 145, Dam Safety: An Owner's Guidance Manual, August 1987, and is contained in Appendix E for reference.

5.2 Personnel

Inspections shall be performed by a responsible person familiar with this Operation, Maintenance, and Inspection Manual. The same personnel shall perform all regular dam inspections to maintain consistency in reporting as well as familiarity with the structure. A checklist outlining the major inspection items for the dam and appurtenances is provided in Appendix B. Plant personnel should use this checklist to inspect the dam and report the findings. Currently, Mr. Randy Sims is the plant personnel responsible for performing Dam Inspections. Copies of the inspection findings should be sent to AEP Civil Engineering for evaluation.

5.3 Periodic Inspections

a. Periodic inspection of the dams is extremely important. AEP has regularly inspected the dams on a quarterly basis. AEP shall continue quarterly inspections.

   i. Three of the quarterly inspections can be completed by Cardinal Plant personnel.

   ii. The fourth quarterly inspections shall be completed by an engineer knowledgeable in dam safety. This inspector may be either a qualified AEP engineer or an independent consulting engineer. This inspection shall be a comprehensive review of field conditions and instrumentation readings.

b. Inspection instructions and an inspection checklist to be used to record observations are found in Appendix B.

c. The inspection procedures and findings must be documented in writing. The quarterly inspection reports shall be maintained for a minimum of 10 years.

d. If problems are found during an inspection that may affect the integrity of the dam, the EAP for the dam shall be followed for the appropriate emergency condition (A, B, or C) and the identified problems shall be placed under increased surveillance and scheduled for repair as appropriate. See also Appendix E for additional guidance.

e. Problems found during an inspection which do not immediately affect the integrity of the dam shall be noted and scheduled for follow-up monitoring and repair as appropriate.

5.4 Event Inspections

A brief inspection shall be made within 24 hours of unusual event such as seismic activity or significant precipitation event (e.g., greater than 3 inch of rain in 24 hours or 6 inches of rain in seven days) or within 24 to 48 hours after placing three or more stoplogs in the drop-inlet structures to ensure that the outlet structures and their discharge pipes are unobstructed, no earth slide has occurred, no significant erosion gullies have formed, and no seepage is present. Concentrate inspections at known problem areas; pool level; debris at outlet structure; new or increased seepage. These Inspections shall be recorded on the dam inspection checklist. Instrumentation should be recorded if new or increased seepage is detected during this inspection.

5.5 Informal Inspections

Informal inspections include both daily and weekly surveillance by Plant personnel looking for changes in conditions (slips along dam face, erosion gullies, excessive settlement, malfunctioning drains, new seepage areas, etc).

Informal inspections shall be made after every significant precipitation event (e.g., greater than 1/2 inch of rain or 3 inches of snow in 24 hours) to ensure that the outlet structures and their discharge pipes are unobstructed, no earth slide has occurred, no significant erosion gullies have formed, and no seepage is present.

These inspections shall be documented either on the checklist form or on an inspection log by indicating the date and time of the inspection, the inspector name(s), the weather conditions, any observed deficiencies or unusual change in the operating or physical conditions, and the overall physical condition of the dam or dike.

5.6 Instrumentation —Fly Ash Dam II

The following instrumentation has been installed to monitor key aspects of the dam's performance:

5.6.1 Seepage Collection/Measurement

Since the 1997 raising, seepage has been identified at three primary locations, specifically:

1. Along the right abutment of FAD II from a spring.
2. Along the left channel slope of the emergency spillway channel.
3. Above the discharge channel along the left side emerging from the bedrock
4. Additionally, a new seep was identified in June of 2013 along the right downstream abutment/dam groin. In October of 2013, an inverted filter and drain was installed. The pipe exiting the drain has been monitored at regular intervals since this time

and the seepage rate has been found to be approximately 0.25 gallons/minute and seepage itself free of fines. One last reading should be obtained within the week prior to stop log placement.

5. Any additional seeps discovered after the pool level has been raised will be added to the inspection list and monitored. If possible, collect seepage and monitor the flow through the use of a V-notch weir or a pipe.
6. Attention should be given to the area at the right groin downstream of the installed PVC sheet pile #79 to be able to trigger any seepage occurring in that area.

If seepage increases by more than 25% at any location, AEP Civil Engineering will immediately be contacted for evaluation.

AEP maintains a Drain and Seepage Zone Spreadsheet detailing drain number and location. This worksheet is included in Appendix B, Section 6 – Pipe Drains as part of the inspection checklist.

5.6.2 Piezometers/Observation Wells

1. Water levels in the piezometers shall be determined and recorded on a quarterly basis to monitor changes in the pore pressures within the dam. Water levels shall be measured to the nearest tenth of a foot. A form for recording the piezometer readings is provided in Appendix B.
2. In addition to quarterly monitoring, the piezometers shall be monitored during and after periods of high pool levels (pool level rise greater than 2 feet from a precipitation event). If piezometer water levels within the dam rise more than 2 feet during a flood event, contact AEP-Civil Engineering staff and/or a Professional Engineer for evaluation of the recorded data.
3. All piezometer monitoring must be done with regard to the safety of the personnel performing the monitoring. Personnel shall cease monitoring activities if weather conditions become hazardous (i.e., lightning), if failure of the dam is imminent, or if safe exit from the embankment will be cut off by flood flows.

5.6.3 Surface Monuments

More than 60 survey monuments have been installed on FAD II to monitor horizontal and vertical movements (See Appendix C). A monitoring plan illustration can be found in

Appendix B. Annual surveys are performed by AEP Civil Laboratory. Copies of the surveys should be sent to:

1. Cardinal Plant Manager
2. AEP Civil Engineering.

5.6.4 Slope Inclinometers

Five slope inclinometers have been installed on FAD II to monitor horizontal movements with depth along the central section of the dam (See Appendix C). Annual reading of the slope inclinometers are performed by AEP Civil Engineering Laboratory. Copies of the readings should be sent to:

1. Cardinal Plant Manager
2. AEP Civil Engineering.

6.0 EMERGENCY ACTION PLAN

The EAP for FAD II is made part of this O&M Manual but is provided as a separate document. The EAP includes the notification flowcharts of individuals/agencies that will be contacted in the event of unsafe conditions detected at any of the three dams.

6.1 Unsafe — Emergency

Each of the malfunctions listed under the UNSAFE — EMERGENCY performance corresponds to a rapid/instantaneous failure condition. Therefore, in the event that one or more of these malfunctions are detected, there may not be enough time for a thorough evaluation of the situation. Accordingly, the first action to be taken by field personnel is notifying the Team Leader who in turn should activate the EAP.

6.2 Unsafe — Non Emergency

Malfunctions under the category of UNSAFE — NON EMERGENCY corresponds to potentially hazardous conditions. These types of malfunctions should allow sufficient time for an expedient evaluation of the situation and for the implementation of remedial measures. Accordingly, the recommended immediate response in the event that one or more of these malfunctions is detected is to use an ALERT as dictated by the EAP and to upgrade the inspection and monitoring program.

6.3 Marginal Deficiency

The malfunctions in the Marginal Deficiency category do not pose a serious threat to the safety of the dam: Therefore, the appropriate field response is to alert the AEP Civil Engineering of the situation and follow up with the inspection checklist report.

6.4 Minor Deficiency

The remaining malfunctions correspond to maintenance rather than immediate safety related problems. These conditions, if detected, will not require any special immediate response other than the normal reporting required under the Dam Inspection and Maintenance Program. If appropriate, an order for maintenance work should be written and implemented by plant personnel.

INSPECTION RESPONSE TABLE

Performance Level of the Dam	Malfunctions or Undesirable Features	Actions to be Taken By Field Personnel (In Order Indicated)
UNSAFE Emergency	• Overtopping or activation of emergency spillway • Breach or slide below the waterline, which reaches the dam crest and/or seeps water. • Springs on abutment or downstream slope with muddy water and progressively increasing flow rate.	1. Notify Team Leader who in turn should issue a Notification. (See EAP) 2. Continue 24-hr. surveillance program, if possible. 3. Read all field instrumentation daily, if possible.
UNSAFE Non-emergency	• Springs on abutments or downstream face with muddy water but stable flow rate. • Pipes, cavities, or holes, which could be attributed to internal erosion, even without evidence of seepage. • Clogged drains. • Slide with no seepage and that does not reach the dam crest. • Noticeable increase in amount of foundation or abutment seepage or piezometer level.	1. Notify Team Leader who in turn should issue an Alert (see EAP). 2. Initiate a daily surveillance program. 3. Read all field instrumentation daily, if possible. 4. Report on Inspection Checklist.
MARGINAL Deficiency	• Cracks parallel or transverse to the dam. • Soft zones in downstream face or toe. • Previously undetected springs with clear water and stable flow rate on face of dam or abutments. • Excessive settlement of crest.	1. Contact AEP Civil Engineering. 2. Report on Inspection Checklist.
MINOR Deficiency	• Damaged instrumentation. • Sloughing. • Rodent burrows. • Surface or riprap erosion. • Trees and tall vegetation on embankments or spillway channel. • Poor vegetal cover.	1. Report on inspection Checklist. 2. Write repair order, if appropriate.

7.0 OWNER'S REVIEW

This Operation, Maintenance, and Inspection Manual was prepared for AEP's Cardinal facility fly ash dam II and bottom ash pond complex and supersedes all previous versions. I have read the Manual on behalf of AEP and understand the actions that will be required of AEP, and acknowledge that the information contained herein is, to the best of my knowledge, accurate as of the date of my signature.

Charles W George
(Signature)

2-27-15
Date

Charles W George
Plant Manager

APPENDIX A
DAM MAINTENANCE RECORD

CARDINAL FAD II
DAM MAINTENANCE RECORD

FOR YEAR _____________

Maintenance	Date	Initials	Comments [a]
1. Cut/mow grass and clear brush			
2. Cut/mow grass and clear brush			
3. Cut/mow grass and clear brush			
4. Cut/mow grass and clear brush			
5. Remove debris from outlet structures			
6. Repair eroded areas			
7. Concrete repair (describe)			
8. Repair rodent damage			
9. Piezometers Maintenance (if required)			
10. Other (specify)			
11. Other (specify)			

[a] Use additional sheets if necessary.

____

Signature

APPENDIX B
DAM INSPECTION INSTRUCTIONS
AND
DAM INSPECTION CHECKLIST

A. Dam Inspection Instructions

1. Dike Inspection Checklist

a. Inspectors and others should include names and affiliations.

b. Weather and site conditions should include weather conditions and the condition of the ground surface (i.e., wet, snow covered, dry, etc.), at the time of the inspection. Note, if the inspection is occurring immediately after a heavy precipitation (e.g., greater than 0.5 inch rainfall or 3 inches of snow in the preceding 24 hours)

c. Fill in the information requested. Obvious problems will require maintenance. Monitoring will be recommended if there is potential for a problem to occur in the future.

2. Comments

a. A brief description of any noted irregularities, needed maintenance, or problems for each item checked should be made. Abbreviations and short descriptions are recommended.

3. Sketches and Field Measurements

a. Explanatory sketches, measurements of cracks, settlement, and additional explanation of observations should be placed on these pages. A copy of the Cardinal Plant Dam Inspection Location Plan should be used to indicate the locations of any concerns identified during an inspection.

b. Definitions:

CW	Clear Water
BA	Bottom Ash
GPM	Gallons Per Minute
MGD	Million Gallons per Day

CARDINAL PLANT
FLY ASH DAM II
INSPECTION CHECKLIST

CARDINAL PLANT
FLY ASH DAM II
INSPECTION CHECKLIST

1. GENERAL INFORMATION

Date of Inspection	
Inspected by	
Reason for Inspection	
Weather	
Temperature	
Rainfall During Previous 7 Days	
Reservoir Elevation: Fly Ash Dam II	
Available Spillway Freeboard (974.0 - Reservoir Elevation)	
Available Dam Crest Freeboard (983.0 - Reservoir Elevation)	

2. EMBANKMENT CONDITION

Note the conditions of the overflow structures and, to the extent practicable, the discharge pipes. Signify good conditions with a checkmark, problem areas with an X in the appropriate spaces below. The FAD II Inspection Location Page shall be used to indicate malfunction locations. Place a number or letter (location code) on the plan at each problem area. Place the same letter(s) or number(s) next to appropriate malfunction. Place sketches, notes, and comments.

Malfunction	**"✓"or "X"**	**Location Code**	**Descriptive Features**
Bulges			Areal extent and elevation
Cavities or Holes			General shape, size, and elevation
Cracks			Length, width, depth and elevation
Surficial Erosion, Gullies			Length, width, depth, areal extent
Sloughing/Slides			Areal extent, vertical drop
Soft Soil			Areal extent and vegetation
Springs/Seepage/ Wetness			Flow rate, muddy or clear water, areal extent, and elevation
Rodent Burrows			Size, areal extent if clustered
Poor Vegetal Cover			Areal extent

Malfunction	" ✓ "or "X"	Location Code	Descriptive Features
Trees or Tall Vegetation			Areal extent, height, trunk size
Excessive Crest Settlement			Settlement/affected crest length
Defects in Crest Road			Size, areal extent
Clogged Drains			Color and origin of deposit/size of color
Deteriorated Rip Rap Outlet Channel			Areal extent
Other (Please specify and describe)			

Note: All malfunctions which occur within the same general area should be shown in the same descriptive sketch or narrative for that particular problem area.

3. OVERFLOW STRUCTURE

Inspect the below listed structures. Place a "✓"in the space if the condition is good; place an "X" in the space if a problem is found and describe the problem below. If necessary, continue description of problem on Page 12, NOTES AND COMMENTS.

Description	**"✓" or "X"**	**Location Code**	**Descriptive Features**
Does discharge flow appear normal?			
Condition of concrete at spillway shaft			
Are extra stop logs available?			
Have stop logs been added? If yes, note number, date, and new top elevation			
Obstruction: note location(s)			
Have obstructions been removed?			
Are access stairs OK?			
Are the any rusted areas in the skimmer?			
Other (please specify)			

4. OUTLET WORKS

Please note the conditions with regard to the following items. If a problem is observed, please describe it.

Does the discharge flow appear normal at the energy dissipater?	
Is the condition of concrete at energy dissipater and Parshall flume OK?	
Is the condition of the Parshall flume OK?	
Is flow through the Parshall flume without turbulence?	
Is there any erosion or riprap problem at the outlet channel?	
Is rubble from the hillside obstructing or threatening to obstruct the outlet channel?	
Other comments.	

5. EMERGENCY SPILLWAY

Please note the conditions with regard to the following items. If a problem is observed, please describe it.

Are there any trees or obstructions in the channel?	
Is there evidence of instability on the side slopes?	
Are there erosion gullies or problems with the vegetal cover in the channel?	
Other comments.	

6. PIPE DRAINS

- Using a stopwatch, determine the time in seconds it takes each of the drainage blanket pipes to fill a 1- or 5-gallon bucket.
- Calculate the pipes discharge in gallons per minute (gpm).

Discharge = 60/time in seconds or 300/time in seconds.

- Record the measurements and describe the turbidity of the discharge in the table below.
- Note: The 12″ diameter spring flow (north of the large weir) can be calculated from the large weir flow minus the sum of all other incoming flows.

Pipe	Time (Sec)	Discharge (gpm)	Description
12″ Dia. Solid E. Underdrain El. 735 (North of Large Weir)			
12″ Dia. Perf. W. Underdrain El. 734 (North of Large Weir)			
12″ Dia. Solid Spring Outlet El. 738 (North of Large Weir)	See Note Above		
4″ Dia. Solid Spring Outlet El. 867 (East Abutment Ditch)			
12″ Dia. Solid Spring Outlet El. 893 (West Abutment Ditch)			
6″ Dia. Solid E. Sprg. Outlet El. 739 (@ Energy Dissipater)			
4″ Dia. Solid W. Sprg. Outlet El. 739 (@ Energy Dissipater)			
6″ Dia. Solid E. Groin Drain El. 907 (In Emerg. Spillway)			
12″ Dia. Solid RCC Drain El. 908 (In Emerg. Spillway)			
6″ Dia. Solid Right Groin Channel. Outlet El. 943			
Other			

7. V-NOTCH WEIRS

7.1 The large 12-inch weir measures the total surface flows, spring flows, and the underdrain flows from the riprap slide repair area.

- Read the head of water acting on the large weir from the staff gauge which is attached to a lumber post located approximately 5 feet upstream of the weir.

- With this reading and the rating curve for a 90° V-notch weir shown Page 14, determine the discharge over the weir in gpm. Record the water head and discharge as follows:

Head, inches	
Discharge, gpm	
Has a significant snowmelt occurred during the last 2 days?	
Additional comments about condition of the _______	

7.2 The small 6-inch weir (located south of the large weir in a small basin) measures all of the dam internal drainage blanket flows.

- Read the head of water acting on the weir from the floor of the weir and subtract 6 inches to obtain the correct reading.

- With this reading and the rating curve for a 90° V-notch weir shown on Page 14, determine the discharge over the weir in gpm.

- Record the water head and discharge as follows:

Head, inches	
Discharge, gpm	
Has a significant snowmelt occurred during the last 2 days?	
Additional comments about condition of the ___________________________	

8. PNEUMATIC PIEZOMETERS

8.1 Obtain water level readings at the piezometers that follow:

- Use the portable indicator to read the pressure, in psi, at each pneumatic piezometer following the procedure outlined in the Instruction Manual for Pneumatic-Pressure Transducer Model 51421102.
- Determine the pressure head in feet of water by multiplying the pressure by 2.308.
- Determine the water elevation or total head by adding the pressure head, in feet of water, to the corresponding elevation of the transducer tip (elevation head).
- Record the pressure and total head calculations in the table below.

Note: The piezometers with an asterisk (*) in front of their identification number should be read on the same schedule as the field inspections. All other piezometers should be read every three months.

PIEZOMETER RECORD

Piezometer No.	Pressure (psi)	Pressure Head (ft)	Elevation Head (ft)	Total Head (ft)	Comments
EXAMPLE	10.5	24.2	730.4	754.5	
P-1A			752.30		
P-2A			771.00		
P-3A			801.30		
P-3B			772.30		
*P-1BE			728.00		
*P-1BW			735.90		
*P-2BE			730.00		
*P-2BW			731.10		
*P-1C			714.40		
*P-2C			711.00		
*P-3C			712.30		
*P-4A			798.90		
P-5A			774.70		
P-5BR			725.30		
P-8A			802.10		
*P-8B			776.00		
*P-9			771.20		
*P-10			769.10		
*P-11			802.60		
P-11B			789.10		
P-RCC1			923.30		
P-RCC2			913.40		
P-RCC3			913.30		

Additional comments regarding piezometer readings and the condition of the terminal panel and housing structure.

9. HYDRAULIC (STANDPIPE) PIEZOMETERS

- Use a water level indicator to measure the depth to water in each hydraulic piezometer. Determine the water elevation (i.e., total head) by subtracting the depth of water from the elevation of the top of riser for the corresponding piezometers.
- Record the readings and calculations on the table below.
- The schedule for reading the hydraulic piezometers should be the same as for conducting the field inspections.

Piezometer

Piezometer No.	Elevation of Top of Riser	Depth to Water	Water Elevation	Comments
MW-1D	968.630			
MW-1S	968.630			
MW-5	980.205			
MW-6	980.555			
MW-7	972.500			
Additional comments regarding condition of the piezometer riser, protection casing, vented cap, etc.				

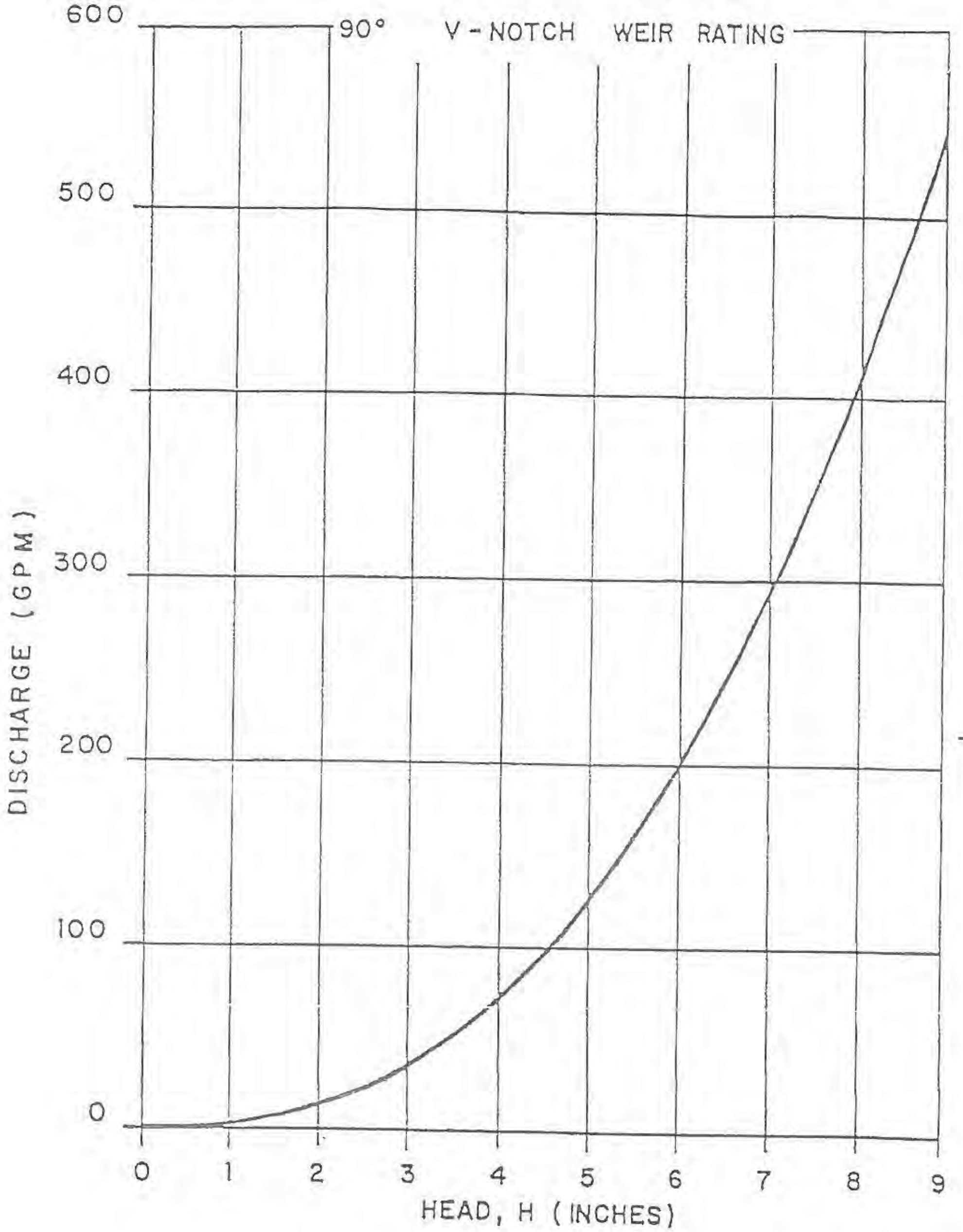
Open Bore Hole (RCC Zone)

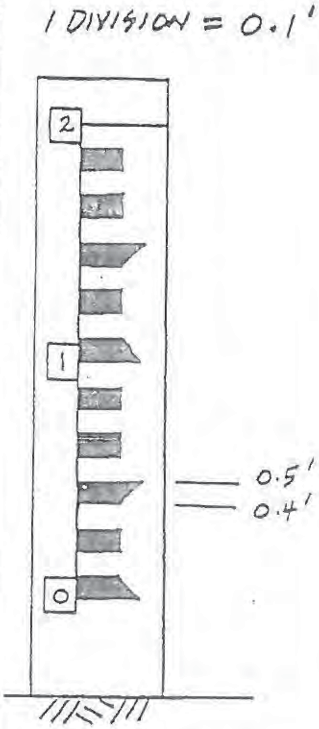
Bore Hole No.	Elevation of Top of RCC	Depth to Water	Water Elevation	Comments
OB-1	970.205			
OB-2	970.015			
OB-3	969.950			
OB-4	696.915			
OB-5	969.890			
OB-6	696.885			
OB-7	969.865			
OB-8	969.880			
OB-9	969.935			
OB-10	970.015			
OB-11	970.035			
OB-12	961.965			
OB-13	961.240			

10. NOTES AND COMMENTS

11. REPAIR ORDERS WRITTEN AND REPAIRS DONE SINCE PREVIOUS INSPECTION

90° V-NOTCH WEIR RATING

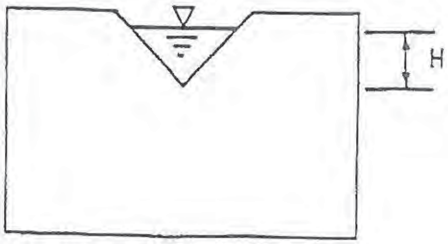




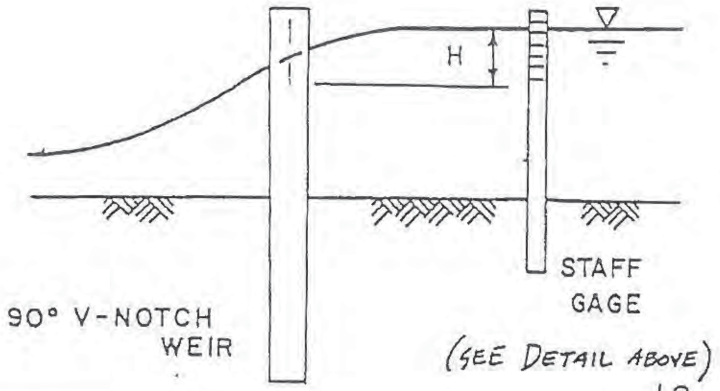
STAFF GAGE DETAIL

*Subtract 0.36 ft from the reading for Weir #1

FRONT VIEW



SIDE VIEW



CARDINAL PLANT

FLY ASH RETENTION DAM II

SEEPAGE MEASURING WEIR

FIGURE 2

CARDINAL PLANT
BOTTOM ASH/RECLAIM DIKE
INSPECTION CHECKLIST

CARDINAL PLANT
BOTTOM ASH/RECLAIM POND AREAS
INSPECTION CHECKLIST

1. GENERAL INFORMATION

Date of Inspection	
Inspected by	
Weather	
Temperature	
Bottom Ash Pond Elevation	
Recirculation Pond Elevation	

2. EMBANKMENT CONDITION

Please refer to the Cardinal Ash Storage Areas Inspection Location Plan. Place a number or letter (Location Code) on the location plan at each problem area and place the same number(s) or letter(s) next to the appropriate malfunction below. For each problem area, provide a sketch or narrative describing the pertinent features of the malfunction(s) under NOTES and COMMENTS section.

Malfunction	**" ✓ "or "X"**	**Location Code**	**Descriptive Features**
Bulges			Areal extent and elevation
Cavities or Holes			General shape, size, and elevation
Cracks			Length, width, depth and elevation
Excessive Crest Settlement			Settlement/affected crest length
Rodent Burrows			Size, areal extent if clustered
Slides			Length, width, vertical drop & elevation
Sloughing			Areal extent and elevation
Springs/Seepage/ Wetness			Flow rate, muddy or clear water, areal extent, and elevation
Soft Soil			Areal extent and vegetation
Surficial Erosion			Length, width, depth, areal extent

Malfunction	"✔" or "X"	Location Code	Descriptive Features
Trees or Tall Vegetation			Areal extent, height, trunk size
Deteriorated Rip Rap			Areal extent
Poor Vegetal Cover			Areal extent
Other (Please specify and describe)			

Note: All malfunctions which occur within the same general area should be shown in the same descriptive sketch or narrative for that particular problem area.

3. OVERFLOW STRUCTURE

Please mark the appropriate spaces below with a checkmark if condition is good or briefly note observed problems; if necessary, continue description of problem under NOTES and COMMENTS.

Description	"✔" or "X"	Location Code	Descriptive Features
Does bottom ash discharge flow appear normal?			
Condition of bottom ash spillway tower.			
Condition of bottom ash skimmer.			
Are they any rusted areas in the skimmer?			
Obstructions: note location.			

Description	"✓" or "X"	Location Code	Descriptive Features
Have obstructions been removed?			
Are access stairs and walkway OK?			
Condition of recirculation structure.			
Does the recirculation overflow pipe have flow coming from it?			
Condition of concrete apron.			
Other (please specify)			

4. OUTLET WORKS

Please note the conditions with regard to the following items. If a problem is observed, please describe it.

Does the discharge flow appear normal at the recirculation pond?	
Other comments.	

5. EMERGENCY SPILLWAY

Both emergency spillways were removed from service in 1988 by backfilling with clay and bottom ash. The elevations are the same as the existing embankment crest. Please note the conditions with regard to the following items. If a problem is observed, please describe it.

Other comments.

6. HYDRAULIC (STANDPIPE) PIEZOMETERS

Use a water level indicator to measure the depth to water in each hydraulic piezometer. Determine the water elevation (i.e., total head) by subtracting the depth of water from the elevation of the top of riser for the corresponding piezometer. Record the readings and calculations on the table below. The schedule for reading the hydraulic piezometers should be the same as for conducting the field inspections.

Piezometer No.	Elevation of Top of Riser	Depth to Water	Water Elevation	Comments
1	671.56			Destroyed
2	672.47			
3	671.54			
B-0902	670.60			
B-0904	671.08			
B-0905	652.57			

7. NOTES AND COMMENTS

8. <u>REPAIR ORDERS WRITTEN AND REPAIRS DONE SINCE PREVIOUS INSPECTION</u>

Bottom Ash Complex Exhibit

APPENDIX C
REFERENCE DRAWINGS AND PHOTOS

Notes:

A) All flows mweasured in Million Gallons per Day
Average Flow = MGD
Maximum Flow = (MGD)

B) Max. Flows and Runoff based on 10-yr/24-hr Storm

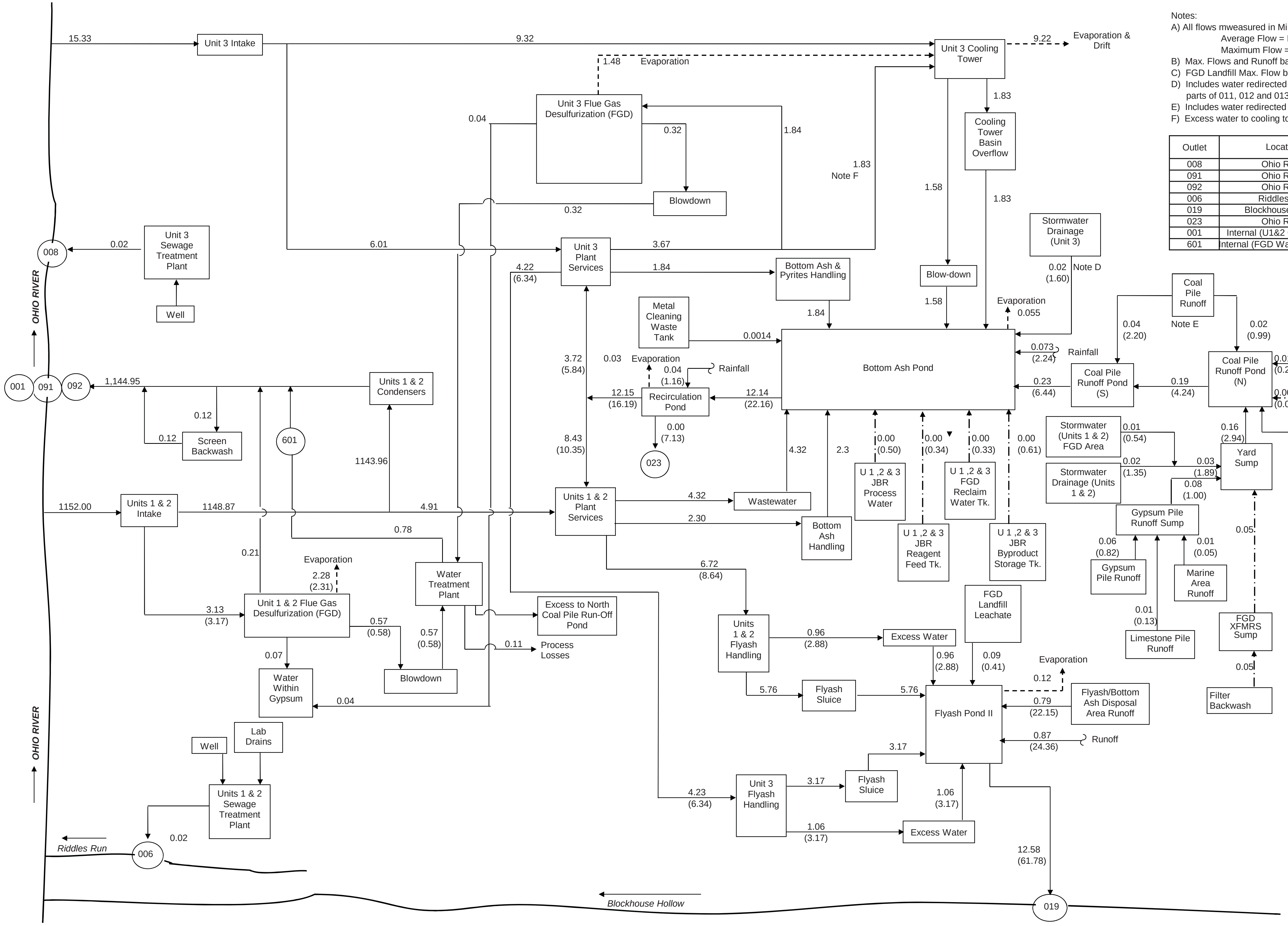
C) FGD Landfill Max. Flow basedon 25-yr/24-hr Storm

D) Includes water redirected from Outfalls 016, 017, and parts of 011, 012 and 013.

E) Includes water redirected from Outfall 010

F) Excess water to cooling tower.

Outlet	Location	Average Flow (MGD)
008	Ohio River	0.02
091	Ohio River	1,144.95
092	Ohio River	1,144.74
006	Riddles Run	0.02
019	Blockhouse Hollow	12.58
023	Ohio River	0.00
001	Internal (U1&2 Condensers)	1,143.96
601	Internal (FGD Water Treatment)	0.78



Process water or Storm Water
Evaporation
Intermittent Flow
XXX - Outlet

12/10/2011

Cardinal Operating Company

Cardinal Plant

Average Flow - MGD
Maximum Flow - (MGD)

Water Balance Diagram

Water & Ecological Resource Services

AEP