HISTORY OF CONSTRUCTION

CFR 257.73(c)(1)

Stingy Run Flyash Pond

Gavin Plant
Cheshire, Ohio

October, 2016

Prepared for: AEP Generation Resources - Gavin Plant

Cheshire, Ohio

Prepared by: American Electric Power Service Corporation

1 Riverside Plaza

Columbus, OH 43215

Document No. GERS-16-015

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1.0 OBJECTIVE

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

2.0 DESCRIPTION OF CFR THE IMPOUNDMENT

The fly ash dam is located approximately 2.5 miles northwest of the plant on Stingy Run which is a tributary to Kyger Creek. Kyger Creek empties into the Ohio River 3.0 miles downstream of the power plant.

The dam was constructed to retain the fly ash produced by the burning of pulverized coal at the General James M. Gavin Power Plant. The fly ash dam is an earthfill zoned embankment 145 feet high. The crest of the dam varies but is a minimum elevation of 735 feet. Reservoir levels are regulated by the 100-foot high intake tower of the principal spillway.

Although originally constructed for settling fly ash, plant operations changed with the installation of scrubbers, so that the plant ceased all fly ash slurry discharges into the reservoir in 1994. Since that time, only direct precipitation, storm water runoff from upstream areas, and acid mine drainage from mined areas enter the reservoir.

AEP has started pond closure project in 2015 and plan to close the pond with a cover system combined with series of channel for water management. At the end of the project the entire flyash pond will be capped and covered with cover system.

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 Gavin Power Plant is located at 7397 N. St Rt. # 7, City of Cheshire, Gallia County, Ohio. It is owned and operated by AEP Generation Resources (GENCO). The facility operates two surface impoundment for storing CCR called the Bottom Ash Complex and Stingy Run Flyash Pond (FAP) or Flyash Reservoir (FAR).

Facility Name	Ohio Department of Natural Resources (ODNR) #
Bottom Ash Complex	8720-003
Stingy Run Flyash Pond	8721-009

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 Stingy Run Flyash Pond (FAP) is a surface impoundment that consists of previously sluiced CCR. The approximately 300-acre FAP is comprised of three valleys, designated as the North Valley, the Middle Valley, and the South Valley. The sluiced ash was treated through settling prior to discharge of the water through a principal spillway structure into Stingy Run downstream of the Flyash Dam (FAD). The FAD discharge structure serves as the sole discharge point for the FAP.

Fly ash disposal was switched to dry disposal into the RWL in conjunction with retrofitting the two generation units with flue gas desulfurization (FGD) technology in 1994. Fly ash is now mixed with the FGD filter cake and disposed at the Residual Waste Landfill (RWL). No more ash is sluiced since 1994

Portions of the FAR were covered in the 1990s with fill generated from the construction of the adjacent RWL. The fill consists primarily of blast rock derived from removal of former bedrock highwalls comprised of sandstone with minor amounts of shale, mine spoil, and residual soils. The fill was placed over approximately 100 acres of the FAR.

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

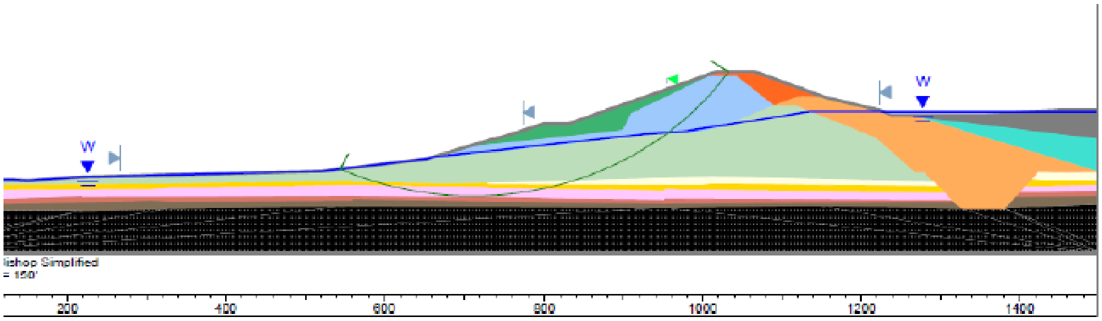
[The name and size in acres of the watershed within which the CCR unit is located.]

The Flyash Pond is located within the Ohio Region watershed (HUC: 05030202) which has a listed acreage of approximately 897,312 acres. The FAR was used primarily for wastewater treatment and disposal of the fly ash and was designed to occupy approximately 300 acres and receives runoff from the surrounding 650 acres drainage basin. The FAR is located in the Kyger Creek watershed.

7.0 DESCRIPTION OF THE FOUNDATION AND ABUTMENT MATERIALS {257.73(c)(1)(V)}

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

The foundation soils under the embankment consist of layers of sand and clay underlain with shale and competent rock. The abutments are formed of the natural hillside sandstone.



Material Name	Color	Unit Weight (lbs/ft3)	Sat. Unit Weight (lbs/ft3)	Cohesion (psf)	Phi (deg)
'88 U/S Clay Core	🟧	126	129	700	16
'88 D/S Random Fill	🟩	126	129	800	16
Bottom Ash zone	🟦	105	115	0	39
'74 Clay Core	🟧	130	133	0	23
'74 D/S Random Fill	🟩	130	136	0	27
'74 U/S Random Fill	🟦	127	130	0	21
Upstream Fly Ash	⬛	100	100	0	30
Upper Clay	⬜	127	127	860	21
Upper Sand	🟨	127	130	0	30
Intermediate Clay	🟪	126	126	0	22
Lower Sand	🟥	127	130	0	30
Lower Clay/Shale	🟫	130	131	0	22
Competant Rock	▩	130	131	5000	0

<u>**SOURCE**</u>: STINGY RUN FLY ASH RESERVOIR, SUPPLEMENTAL INVESTIGATION AND ANALYSIS REPORT, PREPARED BY S&ME ENGINEERING (FORMERLY NAMED BBCM ENGINEERING, INC), 2010.

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.]

The crest of the Stingy Run Flyash Dam is 1500 feet long and 30 feet wide and located above ground surface. The Stingy Run Flyash dam is an earth fill structure. The earthfill embankment consists of a compacted upstream impervious fill zone, a compacted downstream randon fill zone and an uncompacted upstream berm of waste material. A core trench extended the full length of the dam into the rock. 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 riprap on the exposed upstream slope and seeding on the downstream slope with riprap on the lower portion of the slope. Refer to the various layers and engineering properties provided in the previous Section 7.0.

In 1987-88, the fly ash dam was raised from a 104-foot high (elevation ~ 692 feet) to a 145-foot high (elevation ~735 feet) earthfill embankment dam. The fly ash dam raising consisted of an upstream impervious compacted clay core zone, internal fill of compacted bottom ash, downstream compacted random fill, and downstream berm of compacted borrow area bottom ash and mine spoil fill. The internal fill of bottom ash functions as both a light weight fill material and filter/drain. The ash was extended up both abutments to intercept abutment seepage. The bottom ash represents 63 percent of the compacted fill place in the dam raising. The abutments for the dam raising were grouted with a cement/bentonite grout in a program which overlapped the 1973 grouting program. The overburden was removed and the upstream compacted clay core was placed directly on the underlying rock. The crest of the raised dam is at an elevation of at least 735 feet and is 1,800 feet long and 30 feet wide. The crest is 2 feet higher at the centerline of the dam to allow for long-term consolidation.

Construction of the Flyash Dam started sometime in 1972 and was completed by 1974. The raising of the dam was performed during 1987-88. A detailed engineering reports and drawings describing the constructed zone of the structure are included in Attachment B.

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…]

The outlet works for the Flyash Pond consists of a principal spillway, located in the left abutment of the dam is a drop inlet tower with two overflow weir crests, each 4 feet long. The discharge conduit is a 4-foot-diameter steel cylinder reinforced concrete pressure pipe with rubber gasket joints. The pipe is placed on a reinforced concrete cradle. A United States Bureau of

Reclamations (USBR) impact-type stilling basin is provided at the downstream end of the conduit for energy dissipation. The spillway is designed to operate as a decanting tower for the fly ash slurry reservoir. Stop logs were originally provided to raise the elevation of the overflow crest concurrently with the rising buildup of settled fly ash in the reservoir. The initial design decanting elevation was 645.0, over time Gavin Power Plant installed stop logs such that the final decanting actually occurred at Elevation 663 feet. The fly ash entered the reservoir at the discharge points near the upstream end of the reservoir. Floating particles are retained on the pool surface by two concentric skimmers.

The engineering drawings of the structure 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.

Item Description	**Ash Pond**
Normal Pool Elevation	664.0
Maximum Pool Elevation following peak discharge from inflow design flood	670.0
Expected Maximum depth of CCR within impoundment	70

*Lowest Elevation of the Ash Pond = 600 ft.

*Expected Maximum Depth of CCR = 670-600 = 70 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 decant structure and piping to the outfall downstream of the dam. See design drawings in Attachment C for location and details of all appurtenances.

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 flyash dam three observation wells, three weirs, twelve deformation monuments, and two slope inclinometers located within the structure of the dam. These instruments are read on a minimum of every 30 days for the purpose of determining the general condition of the dam. All the instruments location map are provided in Attachment D.

Observation wells are used to monitor the buildup of pore pressure and to evaluate the embankment stability in terms of effective stresses. Settlement monuments and inclinometers are used to monitor horizontal and vertical displacement of the dam. Weirs are installed at the downstream toe ditch of the dam. Weirs are read and the volume of water at various locations are recorded and evaluation for the seepage from the dam.

13.0 AREA – CAPACITY CURVES FOR THE CCR UNIT {257.73 (c)(1)(ix)}

[Area-capacity curves for the CCR unit.]

The area capacity curves for the Stingy Run Flyash Pond is included in the Hydrology and Hydraulic analysis reports and related documents included in Attachment E.

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

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

Since 1994 the ash pond discontinued of receiving ash. The Water from the pond flows into a drop inlet tower with two overflow weir crests, each 4 feet long. The drop inlet structure is connected to a 4-foot diameter concrete pipe located in the northeast corner of the pond, near the upstream slope of the dam. Complete details of each spillway structure are included with the design drawings in Attachment C. Hydrology and Hydraulic Analysis which include calculations for each discharge inlet structure are included in Attachment E.

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

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

The original construction of the dam and Flyash pond was completed in around 1974 and sluicing of ash in the pond was ceased in 1994. A detailed engineering reports and drawings describing the constructed zone of the structure are included in Attachment B.

As required by the CCR rules the Flyash Dam 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.

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.

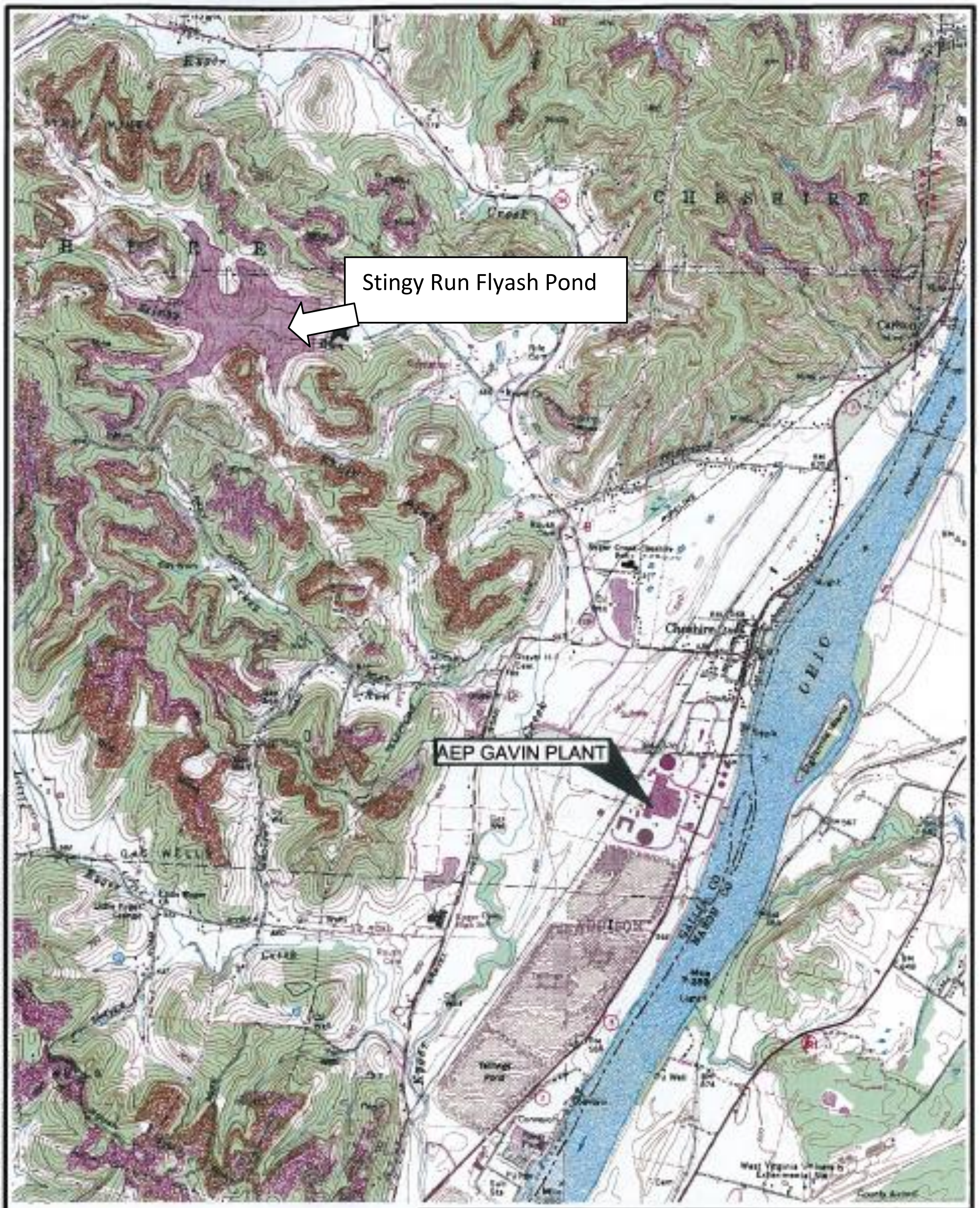
16.0 RECORD OR KNOWLEDGE OF STRUCTURAL INSTABILITY {257.73 (c)(1)(xii)}

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

To date there has been know record of knowledge of the structural instability of the CCR unit.

<u>ATTACHMENT A</u>

LOCATION MAP



USGS Mapping:

CHESHIRE QUADRANGLE
ADDISON QUADRANGLE

VICINITY MAP

AEP Gavin Plant Supplemental Investigation
Stingy Run Fly Ash Reservoir Dam
Cheshire, Ohio

Project: 011-11497-014	Drawn By: MTR
Drawing Date: 6-04-2009	Approved By: MGR
Last Updated: 2-3-2010	Scale: 1" = 3000' 1:1

BBCM
SOLUTIONS TO BUILD ON

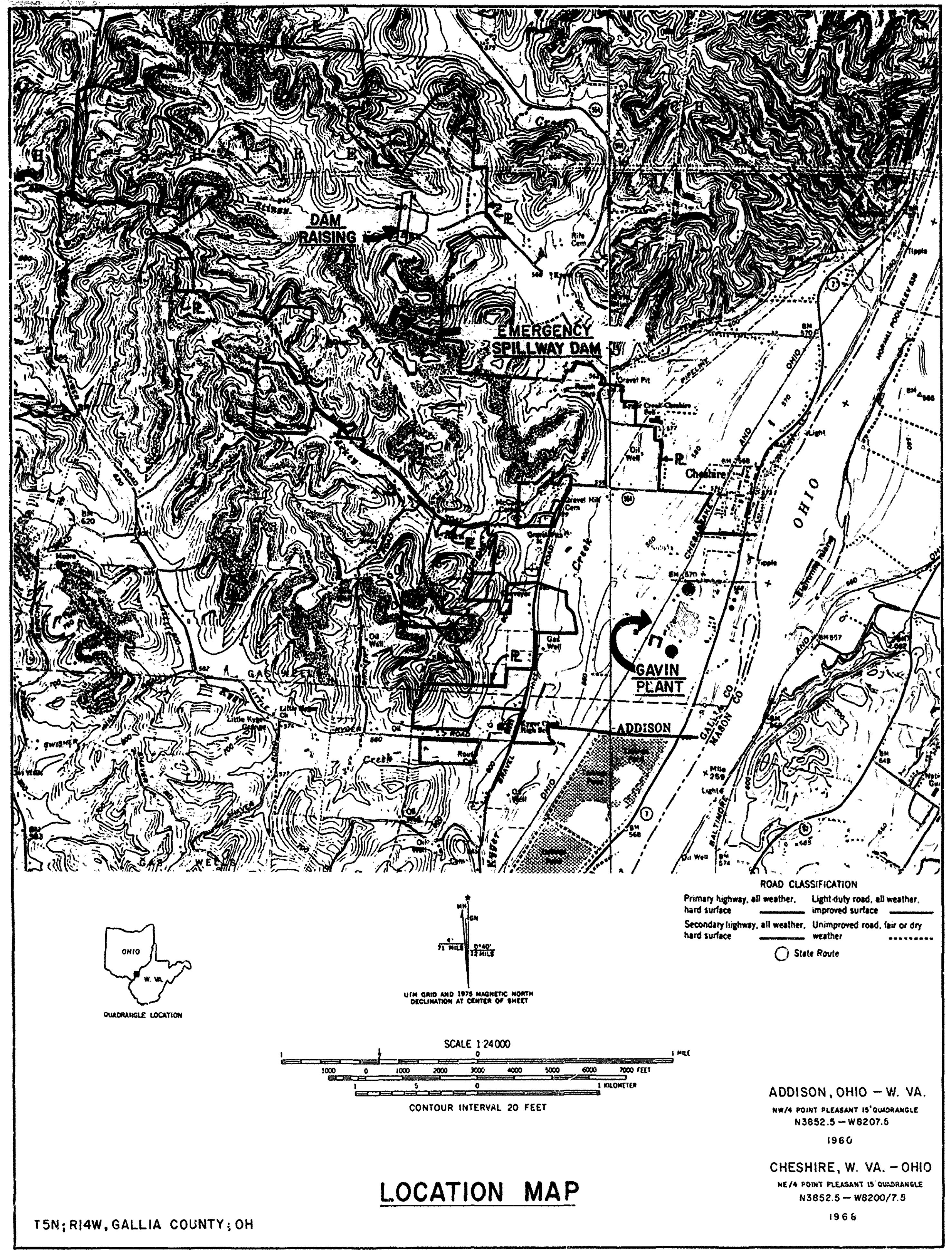
Columbus (614) 793-2226
Cleveland (216) 901-1080
Cincinnati (513) 771-8471
Dayton (937) 424-1811

BBC&M Filename: I:\DEPTS\CADD\Drawings\Projects\011-11497-014\011-11497-014 VMap.dwg Layout: V-Map (2)

ATTACHMENT B

DESIGN REPORTS

Dr. No. 12-3000



LOCATION MAP

DRAWING INDEX

DRAWING NUMBER	TITLE
12-3000	LOCATION MAP AND DRAWING INDEX
12-3000A	GENERAL ARRANGEMENT
12-3000B	BORING AND TEST PIT LOCATION PLAN
12-3000C	INSTRUMENTATION LOCATION PLAN
12-3000D	INSTRUMENTATION PROFILE ALONG CREST OF DAM AND SPILLWAY DAM
12-3000E	INSTRUMENTATION SECTIONS
12-3000F	GEOLOGICAL PROFILE ALONG ℄ OF DAM
12-3000G	GEOLOGICAL PROFILE ALONG CREST OF DAM
12-3000H	EXISTING CORE TRENCH AND EXTENSION PLAN
12-3000I	GROUTING PLAN AND PROFILE NORTH ABUTMENT
12-3000J	GROUTING PLAN AND PROFILE SOUTH ABUTMENT
12-3000K	LAYOUT AND GRADING PLAN
12-3000L	SECTIONS AND DETAILS SHEET 1
12-3000M	SECTIONS AND DETAILS SHEET 2
12-3000N	EMERGENCY SPILLWAY DAM EXCAVATION PLAN
12-3000P	GROUTING PLAN AND PROFILE SPILLWAY DAM
12-3000Q	EMERGENCY SPILLWAY DAM LAYOUT AND GRADING PLAN
12-3000R	EMERGENCY SPILLWAY DAM PROFILE AND SECTIONS
12-3000S	BORROW SITE GRADING PLAN
12-3000T	BORROW SITE SECTIONS
12-3000U	ACCESS STAIRWAY TO SERVICE SPILLWAY STRUCTURE PLAN, SECTIONS, AND DETAILS
12-3000V	ACCESS ROAD SHEET 1
12-3000W	ACCESS ROAD SHEET 2
12-3000X	SPORN-SOUTH POINT TOWER FILL

DESIGN DRAWINGS
DAM RAISING
FOR PHASE II
STINGY RUN
FLY ASH RETENTION POND

GAVIN PLANT
CHESHIRE, OHIO

PREPARED FOR

OHIO POWER COMPANY

PREPARED BY

AMERICAN ELECTRIC POWER
SERVICE CORPORATION
COLUMBUS, OHIO

AS-BUILT DRAWINGS
APRIL, 1989

DATE	NO.	DESCRIPTION	APPD.
4/10/89	2	AS PER KDR NO. 56-043, REV. 1: DRAWING REVIEWED ~ "AS-BUILT" STATUS.	[illegible]
5/[illegible]	1	ADDED DRAWING NO. & TITLE M.W. [illegible]	[illegible]
5/4/87	0	RELEASED FOR CONSTRUCTION	[illegible]

REVISIONS

OHIO POWER COMPANY
GAVIN PLANT
CHESHIRE OHIO

LOCATION MAP
AND
DRAWING INDEX

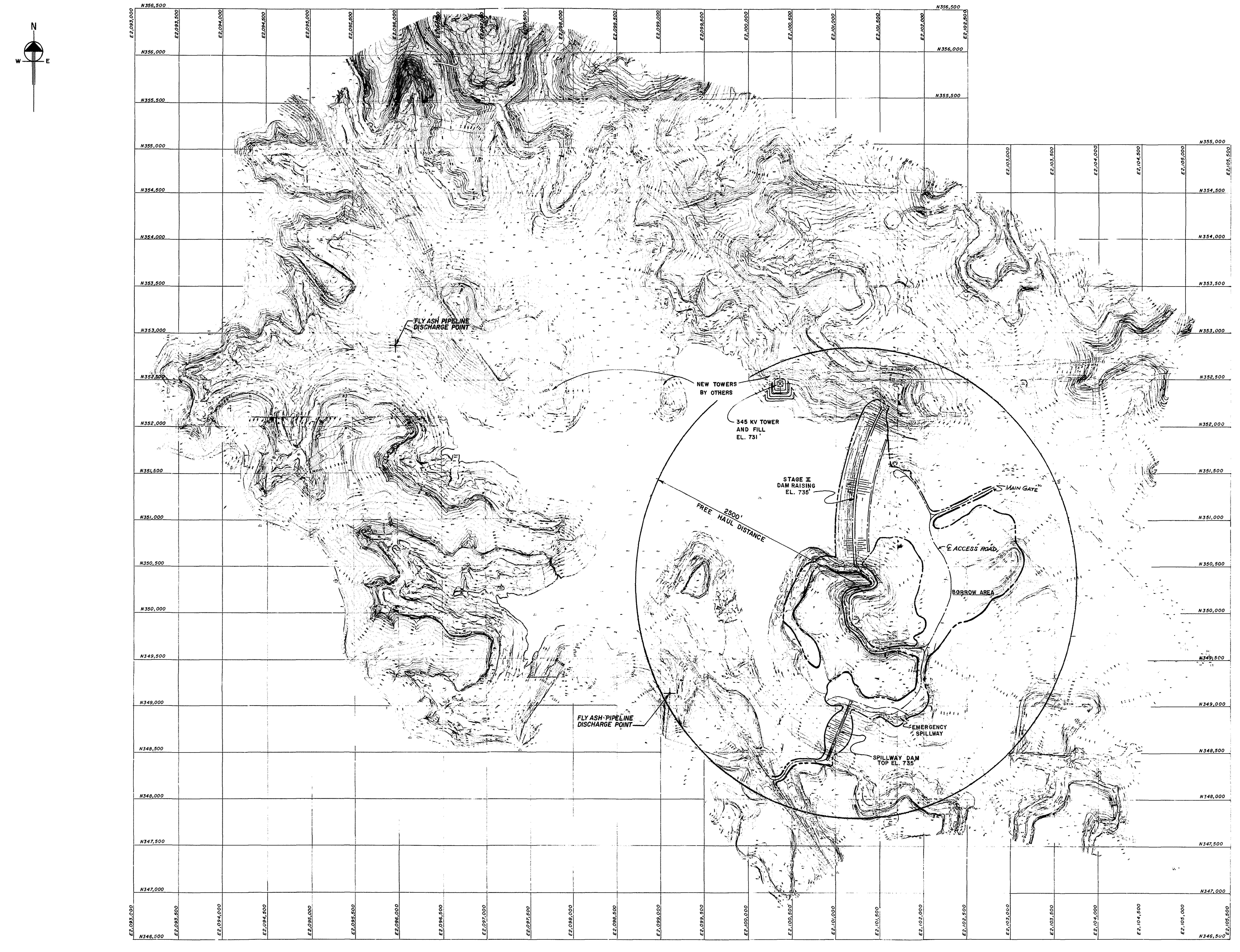
DR. NO. 12-3000-2

ARCH.	-	ELEC	-	MECH.	-	STR.	GS

SCALE: 1"=2000'
DR. S.M.H.
CH. R.T.M.
SQ. LDR R.N.S.
DATE: 12/1/86

APPROVED BY
ENGINEERING DIV R.H. Benett
DESIGN DIV [illegible]

DR. No. 12-3000A



DATE	NO.	DESCRIPTION	APPD.
4/10/89	2	AS PER RDR-56-043, REV. 1 : REVISED GENERAL PLAN, "AS-BUILT" STATUS. KM SRM	WM
5/19/88	1	ADDED 345 KV TOWER AND FILL M.A	QP/JEP
5/4/87	0	RELEASED FOR CONSTRUCTION	JJP

REVISIONS

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OHIO POWER COMPANY

GAVIN PLANT

CHESHIRE OHIO

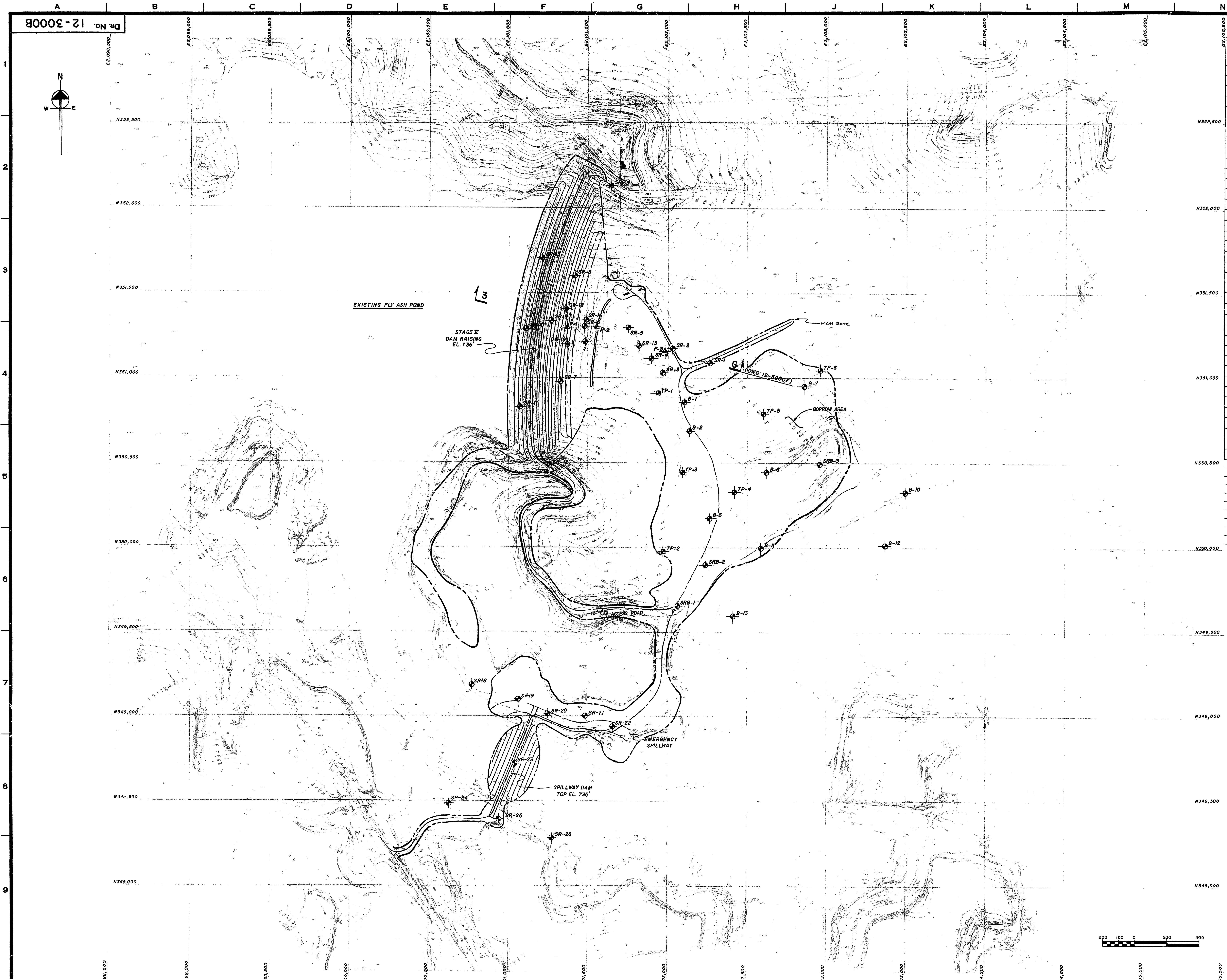
STINGY RUN FLY ASH DAM
STAGE II RAISING
GENERAL ARRANGEMENT

DR. NO. 12-3000A-2

ARCH.	—	ELEC	—	MECH.	—	STR.	95

SCALE: 1"=400'
DR. R.G. DURO
CH. K.M.
SQ LDR RNS
DATE: 2-1-86

APPROVED BY
ENGINEERING DIV B.F. Bennett RHH
DESIGN DIV



BORING NO.	COORDINATES		ELEVATION
	NORTH	EAST	
SR-1	351,088.28	2,102,261.70	582.09
SR-2	351,174.35	2,102,031.21	585.78
SR-3	351,033.74	2,101,965.19	585.85
SR-4	351,115.44	2,101,894.69	587.28
SR-5	351,298.44	2,101,754.80	592.10
SR-6	351,300.66	2,101,476.82	625.43
SR-7	350,970.80	2,101,328.10	634.80
SR-8	351,601.94	2,101,416.62	640.36
SR-9	351,332.72	2,101,266.12	649.49
SR-10	351,287.12	2,101,108.62	698.16
SR-11	350,622.94	2,101,075.47	696.33
SR-12	350,478.39	2,101,258.07	730.52
SR-13	351,706.94	2,101,208.33	696.86
SR-14	352,136.61	2,101,648.51	738.92
SR-15	351,186.4	2,101,820.5	589.5
SR-16	351,327.5	2,101,488.6	624.27
SRB-1	349,653	2,102,061	728.9
SRB-2	349,894	2,102,236	708.2
SRB-3	350,492	2,102,955	778.0
B-1	350,857	2,102,101	595.6
B-2	350,686	2,102,134	610.6
B-5	350,171	2,102,259	666.2
B-6	350,439	2,102,622	730.1
B-7	350,948	2,102,855	626.2
B-10	350,325	2,103,495	650.9
B-11	349,996	2,102,588	687.7
B-12	350,018	2,103,364	605.6
B-13	349,600	2,102,408	649.1
TP-1	350,593	2,101,941	613.9
TP-2	349,973	2,101,961	657.3
TP-3	350,442	2,102,093	616.1
TP-4	350,327	2,102,418	681.1
TP-5	350,791	2,102,603	633.5
TP-6	351,045	2,102,953	616.1
P-1	351,295.7	2,101,367.1	636.8
P-2	351,298	2,101,550.2	617.43
P-3	351,153.3	2,101,983.5	588.38
OW-18	351,402.9	2,101,358.3	643.81
OW-19	351,199.5	2,101,366	635.63
SR-17	351,213.5	2,101,483.0	621.1
SR-18	349,189.1	2,100,774.5	749.5
SR-19	349,098.1	2,101,065.5	751.5
SR-20	349,014.8	2,101,248.5	752.4
SR-21	349,001.2	2,101,490.6	754.0
SR-22	348,938.6	2,101,639.3	750.2
SR-23	348,719.1	2,101,042.1	686.2
SR-24	348,488.6	2,100,629.2	747.2
SR-25	348,398.8	2,100,938.9	743.8
SR-26	348,283.3	2,101,272.5	726.7

LEGEND:

- ⊕ – BORING
- ⊞ – TEST PIT
- △ – PIEZOMETER
- ◉ – OBSERVATION WELL

DATE	NO.	DESCRIPTION	APPD.
4/10/89	1	REVISED PLAN – "AS-BUILT" STATUS. REV SRM	WH
5/4/87	0	RELEASED FOR CONSTRUCTION	2P

REVISIONS

OHIO POWER COMPANY

GAVIN PLANT

CHESHIRE OHIO

STINGY RUN FLY ASH DAM
STAGE II RAISING
BORING & TEST PIT
LOCATION PLAN

DR. NO. 12-3000B-1

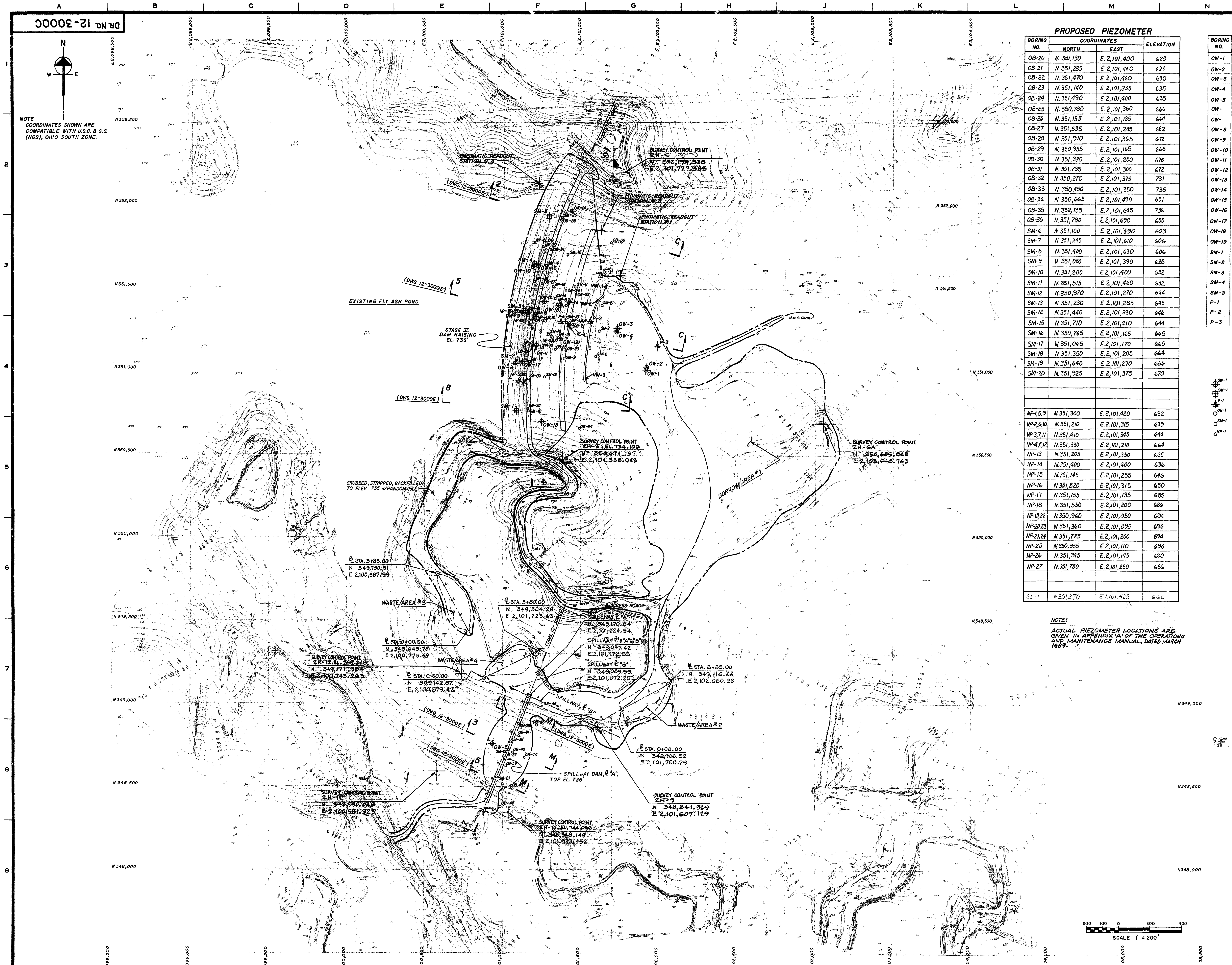
ARCH.	–	ELEC	–	MECH.	–	STR.	95

SCALE: 1"=200'
DR. R.G.DURO
CH. KCM
SQ. LDR RNS
DATE: 12-1-86

APPROVED BY
ENGINEERING DIV B.F. Bennett PHH
DESIGN DIV 2P

DR. NO. 12-3000C

NOTE
COORDINATES SHOWN ARE COMPATIBLE WITH U.S.C. & G.S. (NGS), OHIO SOUTH ZONE.



PROPOSED PIEZOMETER

BORING NO.	COORDINATES NORTH	COORDINATES EAST	ELEVATION
OB-20	N. 351,130	E. 2,101,400	628
OB-21	N. 351,285	E 2,101,440	629
OB-22	N. 351,470	E. 2,101,460	630
OB-23	N. 351,140	E 2,101,335	635
OB-24	N. 351,490	E. 2,101,400	638
OB-25	N. 350,780	E. 2,101,360	666
OB-26	N. 351,155	E. 2,101,185	664
OB-27	N. 351,535	E. 2,101,245	662
OB-28	N. 351,910	E 2,101,365	672
OB-29	N. 350,955	E. 2,101,165	668
OB-30	N 351,335	E. 2,101,200	670
OB-31	N. 351,735	E. 2,101,300	672
OB-32	N. 350,270	E 2,101,375	731
OB-33	N. 350,450	E. 2,101,350	735
OB-34	N. 350,665	E 2,101,490	651
OB-35	N. 352,135	E. 2,101,645	736
OB-36	N. 351,780	E. 2,101,690	650
SM-6	N. 351,100	E 2,101,590	603
SM-7	N 351,245	E. 2,101,610	606
SM-8	N. 351,400	E. 2,101,630	606
SM-9	N. 351,080	E. 2,101,390	628
SM-10	N. 351,300	E. 2,101,400	632
SM-11	N. 351,515	E 2,101,460	632
SM-12	N. 350,970	E. 2,101,270	644
SM-13	N. 351,230	E. 2,101,285	643
SM-14	N. 351,440	E. 2,101,330	646
SM-15	N. 351,710	E. 2,101,410	644
SM-16	N. 350,765	E. 2,101,165	665
SM-17	N. 351,065	E. 2,101,170	665
SM-18	N. 351,350	E. 2,101,205	664
SM-19	N. 351,640	E. 2,101,270	666
SM-20	N. 351,925	E. 2,101,375	670
NP-1,5,9	N. 351,300	E. 2,101,420	632
NP-2,6,10	N. 351,210	E. 2,101,315	639
NP-3,7,11	N. 351,410	E. 2,101,345	644
NP-4,8,12	N. 351,330	E. 2,101,210	664
NP-13	N. 351,205	E. 2,101,350	635
NP-14	N. 351,400	E. 2,101,400	636
NP-15	N. 351,145	E. 2,101,255	646
NP-16	N. 351,520	E. 2,101,315	650
NP-17	N. 351,155	E. 2,101,135	685
NP-18	N. 351,550	E. 2,101,200	686
NP-19,22	N. 350,960	E. 2,101,050	694
NP-20,23	N. 351,360	E. 2,101,095	696
NP-21,24	N. 351,775	E. 2,101,200	694
NP-25	N. 350,955	E. 2,101,110	690
NP-26	N. 351,345	E. 2,101,145	690
NP-27	N. 351,750	E. 2,101,250	686
SI-1	N 351,270	E 1,101,425	660

NOTE:
ACTUAL PIEZOMETER LOCATIONS ARE GIVEN IN APPENDIX 'A' OF THE OPERATIONS AND MAINTENANCE MANUAL, DATED MARCH 1989.

EXISTING PIEZOMETER

BORING NO.	COORDINATES NORTH	COORDINATES EAST	ELEVATION
OW-1	351,012.0	2,101,909.3	588.17
OW-2	351,030.4	2,101,913.1	588.23
OW-3	351,264.8	2,101,726.3	593.96
OW-4	351,245.1	2,101,725.4	593.34
OW-5	348,756.4	2,100,933.8	689.16
OW-			
OW-			
OW-8	351,055.9	2,101,080.7	700.20
OW-9	351,355.0	2,101,119.1	701.12
OW-10	351.638.2	2,101,186.0	697.09
OW-11	351,156.0	2,101,210.4	656.74
OW-12	351,421.6	2,101,264.4	656.87
OW-13	350,701.9	2,101,246.6	658.22
OW-14	351,966.8	2,101,441.8	673.26
OW-15	351,642.5	2,101,220.1	688.85
OW-16	351,352.9	2,101,159.5	687.33
OW-17	351,060.1	2,101,119.6	688.76
OW-18	351,402.9	2,101,358.3	643.81
OW-19	351,199.5	2,101,366.0	635.63
SM-1	350,767.61	2,101,0[illegible]2.20	695.21
SM-2	351,066.01	2,101,086.29	697.94
SM-3	351,359.98	2,101,126.04	698.60
SM-4	351,646.36	2,101,191.76	697.34
SM-5	351,934.18	2,101,287.62	693.66
P-1	351,295.7	2,101,367.1	636.80
P-2	35[illegible]8.0	2,101,550.2	617.43
P-3	351,153.3	2,101,983.5	588.38

LEGEND:

⊕ OW-1	OBSERVATION WELL (EXISTING)
⊞ SM-1	SETTLEMENT MONUMENT (EXISTING)
▲ P-1	PIEZOMETER (EXISTING)
○ OB-1	OBSERVATION WELL
□ SM-1	SETTLEMENT MONUMENT
△ NP-1	PIEZOMETER

DATE	No	DESCRIPTION	APPD.
4/10/89	1	REVISED PLAN ~ "AS-BUILT" STATUS. SRM	[illegible]
5/4/87	0	RELEASED FOR CONSTRUCTION	[illegible]

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."

OHIO POWER COMPANY

GAVIN PLANT

CHESHIRE OHIO

STINGY RUN FLY ASH DAM
STAGE II RAISING
INSTRUMENTATION
LOCATION PLAN

DR. NO. 12-3000C-1

ARCH. – ELEC – MECH – STR 95

SCALE 1" = 200'
DR. R.M.
CH. R.G. JURO
SQ. LDR. RNS
DATE: 12-1-86

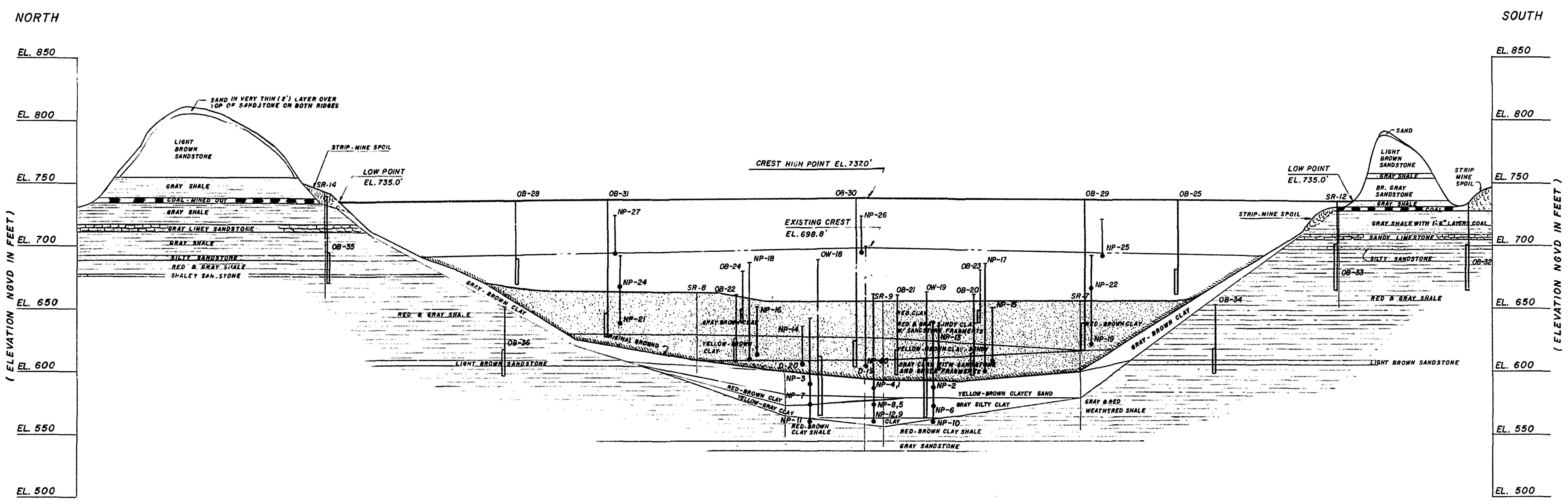
APPROVED BY
ENGINEERING DIV. C. H. Bennett
DESIGN DIV.

AMERICAN ELECTRIC POWER SERVICE CORP.

DR. No. 12-3000D

GENERAL NOTES

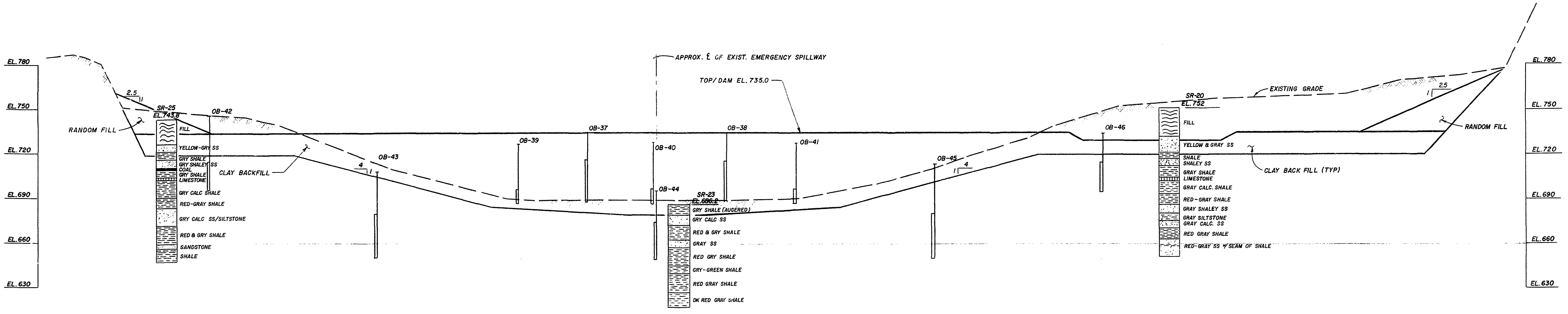
REFERENCE DRAWINGS



PROFILE 4-G
ALONG CREST OF DAM
SCALES: 1" = 100' HOR.
1" = 50' VERT.
(SEE DWG. 12-3000C)

LEGEND

▨ — EXISTING DAM RANDOM FILL



PROFILE 7-F
ALONG CREST OF SPILLWAY DAM
SCALE: 1" = 30'
(SEE DWG. 12-3000C)

DATE	NO.	DESCRIPTION	APPD.
4/10/89	1	AS PER RDR NO. 56-043, REV. 1: DRAWING REVIEWED — "AS-BUILT" STATUS. KCM SRM	
5/4/87	0	RELEASED FOR CONSTRUCTION	

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"

OHIO POWER COMPANY

GAVIN PLANT

CHESHIRE OHIO

STINGY RUN FLY ASH DAM
STAGE II RAISING
INSTRUMENTATION PROFILE
ALONG CREST OF DAM AND
SPILLWAY DAM

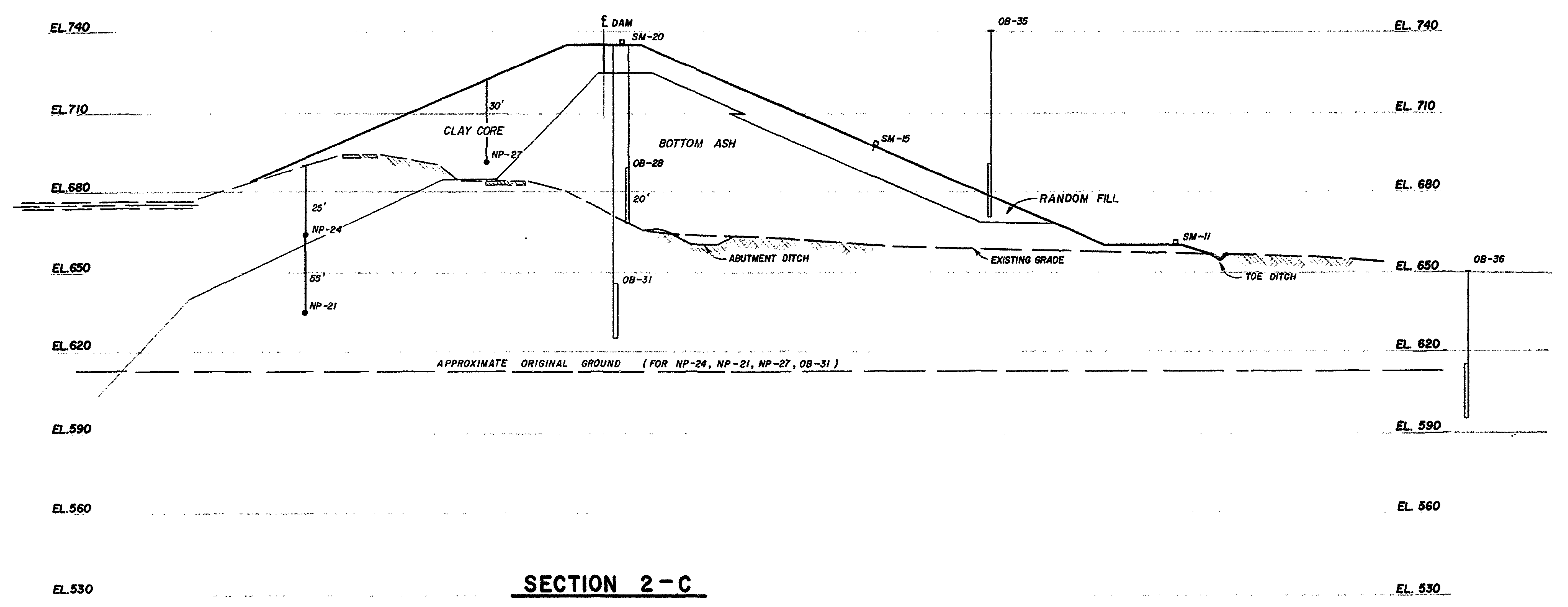
DR. NO. 12-3000D-1

ARCH.	—	ELEC	—	MECH.	—	STR.	95

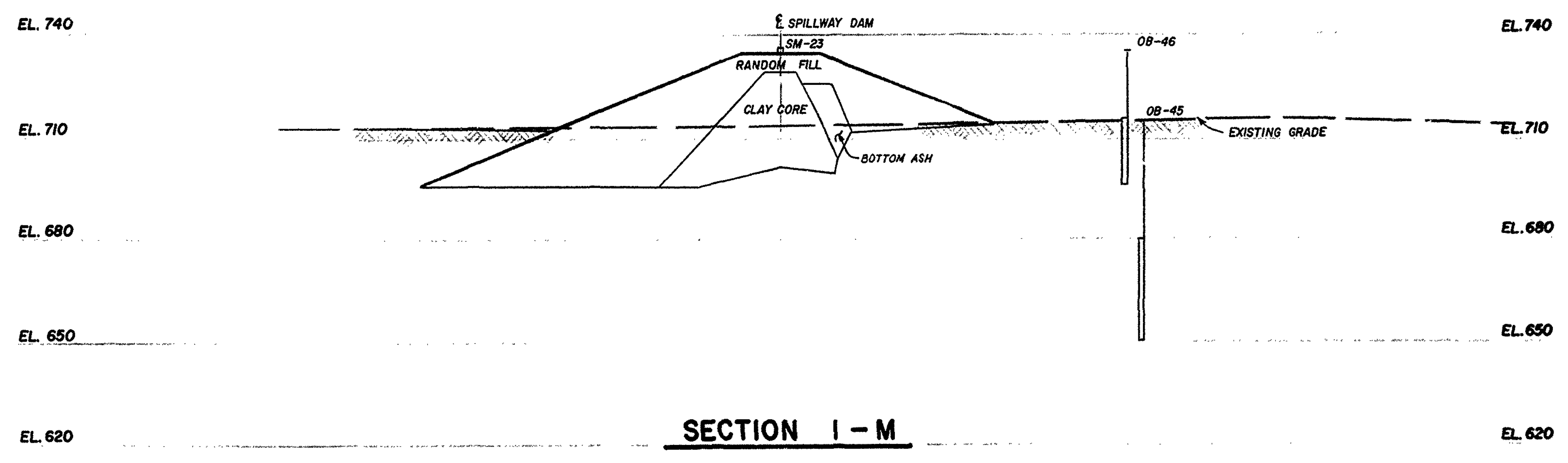
SCALE: AS NOTED
DR. R.M.H.
CH. KCM
SQ. LDR. RNS
DATE: 12-1-86

APPROVED BY
ENGINEERING DIV B.H. Bennett
DESIGN DIV

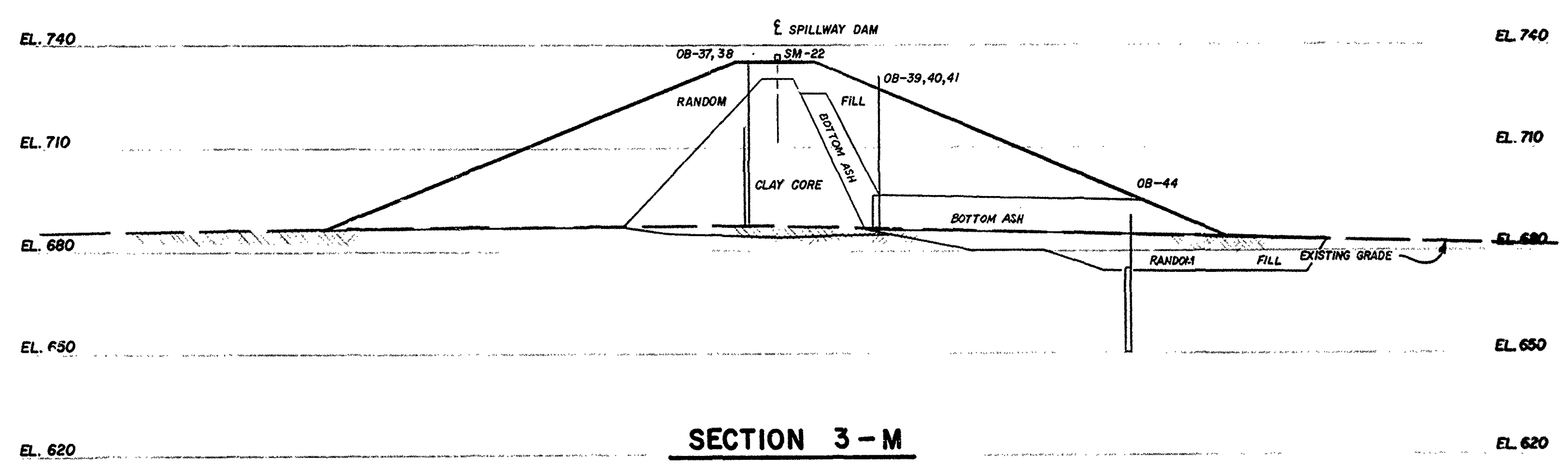
DWG. NO. 12 - 3000E



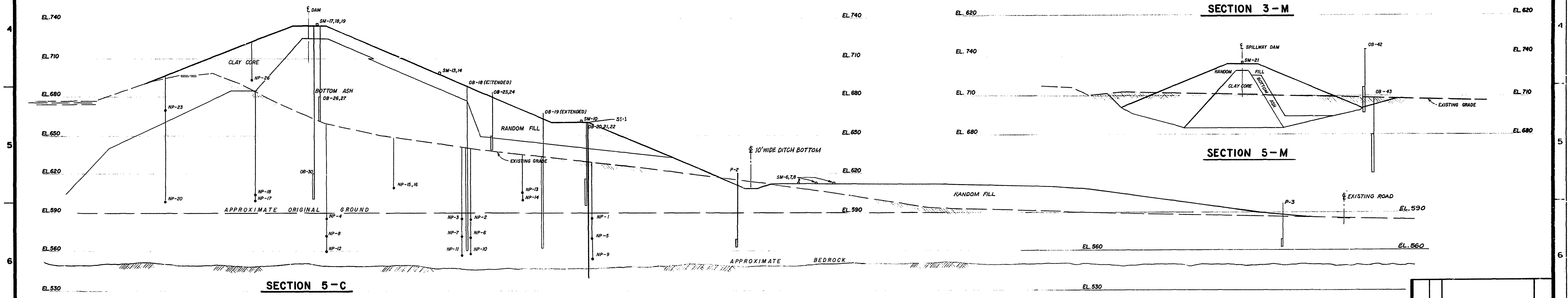
SECTION 2-C



SECTION 1-M

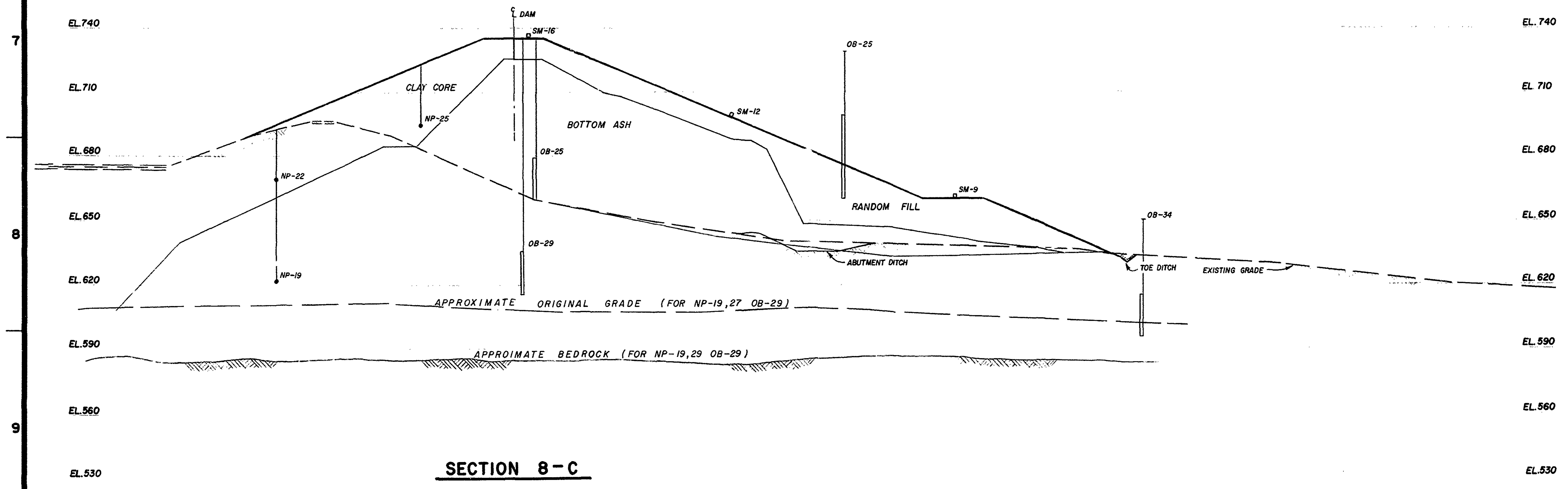


SECTION 3-M



SECTION 5-C

SECTION 5-M



SECTION 8-C

DESCRIPTION	SOURCE	COMPACTION CONTROL	MOISTURE CONTROL	MAX LOOSE LIFT THICKNESS
CLAY CORE	BORROW AREA	$\gamma_D \geq$ 98% (γ_D) MAX. ASTM – D698	-1% TO +2% OF OPTIMUM ASTM – D698	6"
RANDOM FILL	BORROW AREA	$\gamma_D \geq$ 95% (γ_D) MAX. ASTM – D698	-2% TO +2% OF OPTIMUM ASTM – D698	9"
BOTTOM ASH	GAVIN BOTTOM ASH POND	70% OR GREATER RELATIVE DENSITY ASTM D-4253-83 ASTM D-4254-83	SATURATED	9"
SAND & GRAVEL	CONTRACTOR SUPPLIED	70% OR GREATER RELATIVE DENSITY ASTM D-4253-83 ASTM D-4254-83	SATURATED	9"

REFERENCE DRAWINGS

12-3000C INSTRUMENTATION LOCATION PLAN

DATE	NO.	DESCRIPTION	APPD.
4/10/89	1	REVISED SECTIONS – "AS-BUILT" STATUS. KCM SRM	HPm
5/4/87	0	RELEASED FOR CONSTRUCTION	

REVISIONS

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OHIO POWER COMPANY
GAVIN PLANT
CHESHIRE OHIO

STINGY RUN FLY ASH DAM
STAGE II RAISING
INSTRUMENTATION SECTIONS

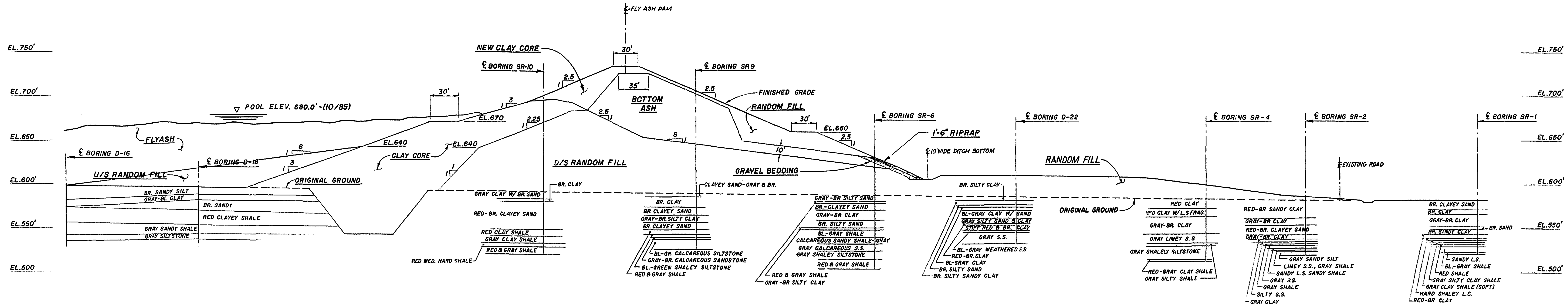
DWG. NO. 12 - 3000E - 1

ARCH. – ELEC. – MECH. – STR. 95

SCALE: 1" = 30'
DR: M. HERSHEY
CH: KCM
SQ. LDR: RNS
DATE: 12-1-86

APPROVED BY
ENGINEERING DIVISION B.H. Bennett
DESIGN DIVISION

DR. No. 12-3000F



PROFILE 3-G

SCALE 1" = 50'
(SEE DWG. 12-3000B)

50 0 50 100

DATE	NO.	DESCRIPTION	APPD.
4/10/89	1	REVISED SECTION "AS-BUILT" STATUS.	
5/4/87	0	RELEASED FOR CONSTRUCTION	

REVISIONS

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OHIO POWER COMPANY

GAVIN PLANT

CHESHIRE OHIO

STINGY RUN FLY ASH DAM
STAGE II RAISING
GEOLOGICAL PROFILE
ALONG ℄ OF DAM

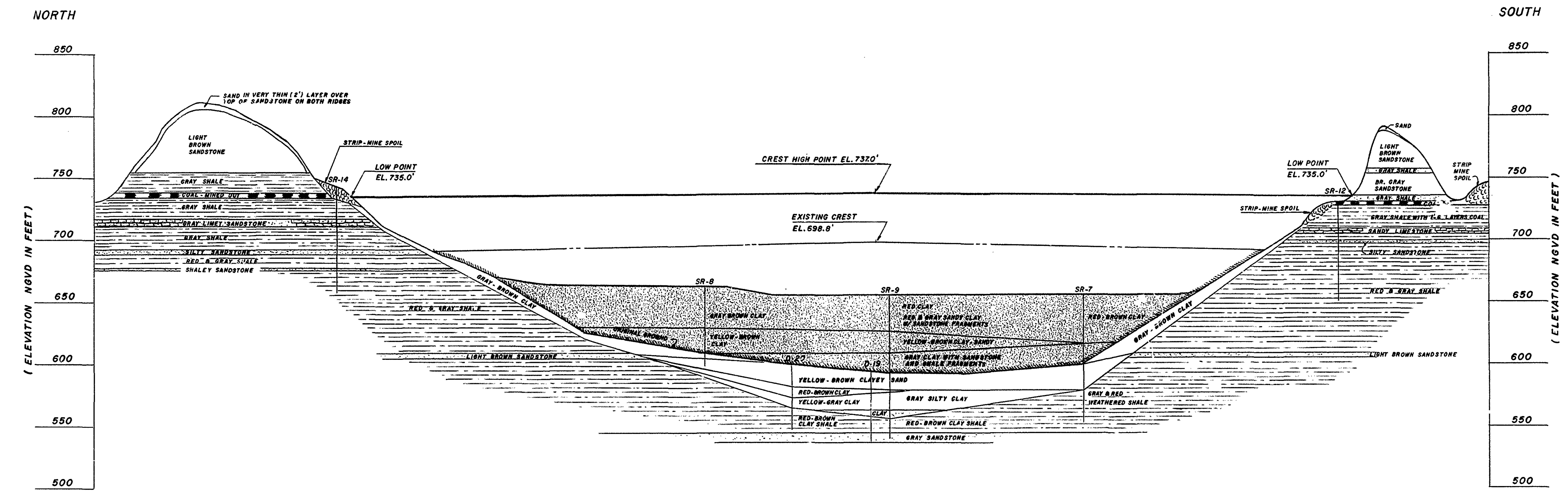
DR. NO. 12-3000F-1

ARCH.	-	ELEC	-	MECH.	-	STR.	GS

SCALE: AS NOTED
DR. R.M.
CH. KCM
SQ. LDR RNS
DATE: 12-1-86

APPROVED BY
ENGINEERING DIV B.H. Bennett
DESIGN DIV

DR. NO. 12 - 3000G



PROFILE 4-G

SCALES: 1" = 100' HOR
1" = 50' VERT.
(SEE DWG. 12-3000K)

LEGEND

▨ EXISTING DAM RANDOM FILL

DATE	NO.	DESCRIPTION	APPD.
4/10/89	1	AS PER RDR No. 56-043, REV. 1: DRAWING REVIEWED — "AS-BUILT" STATUS. KCM SRM	Km
5/4/87	0	RELEASED FOR CONSTRUCTION	

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"

OHIO POWER COMPANY

GAVIN PLANT

CHESHIRE OHIO

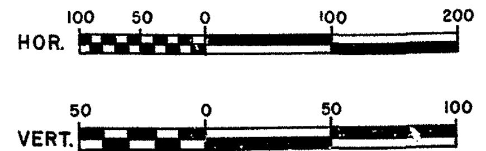
STINGY RUN FLY ASH DAM
STAGE II RAISING
GEOLOGICAL PROFILE
ALONG CREST OF DAM

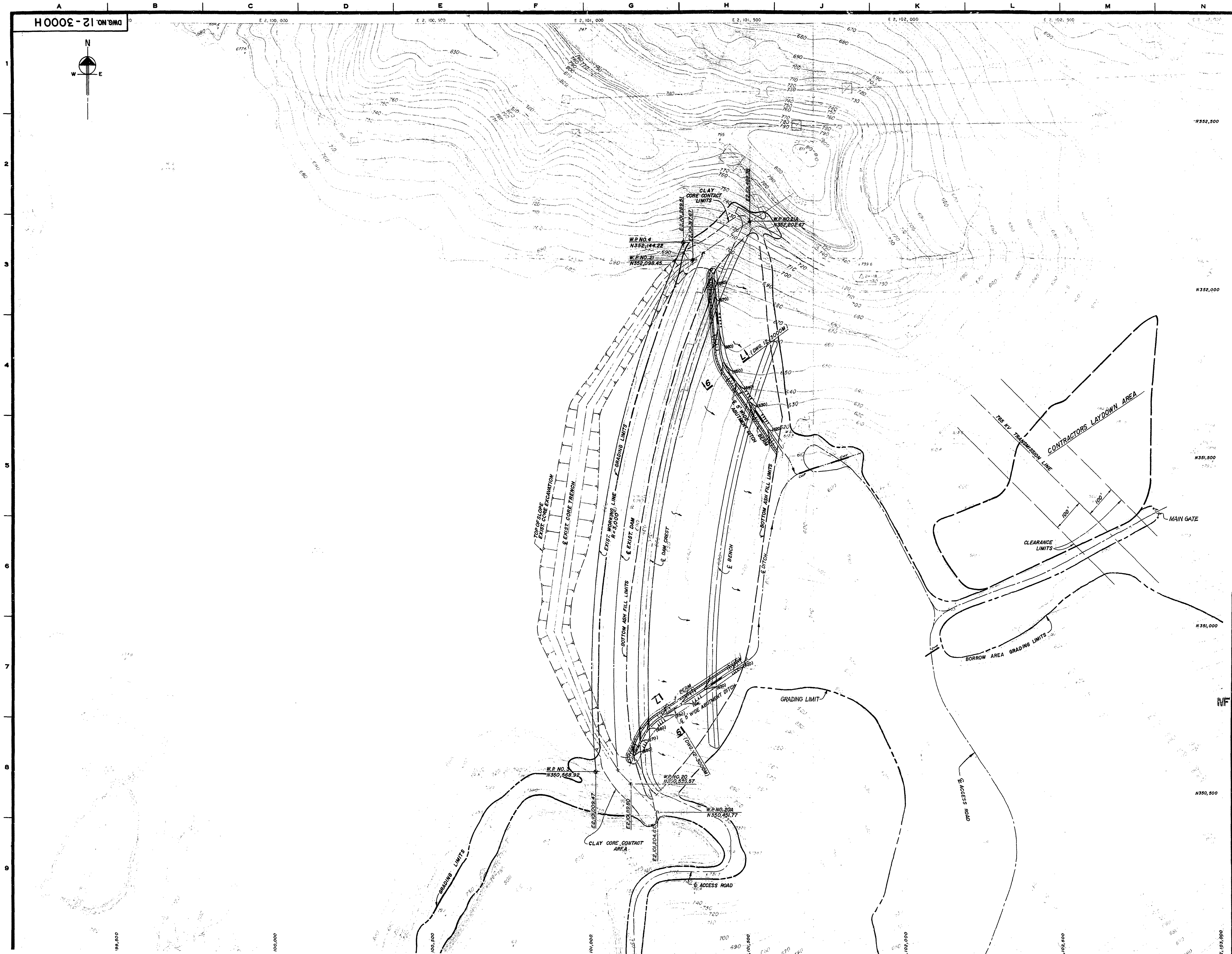
DR. NO. 12 - 3000G - 1

ARCH.	-	ELEC	-	MECH.	-	STR.	GS

SCALE: AS NOTED
DR. R.G.DURO
CH. KCM
SQ. LDR RNS
DATE: 12-1-86

APPROVED BY
ENGINEERING DIV B.H. Barnett AHA
DESIGN DIV





GENERAL NOTES

1. ALL AREAS WITHIN THE GRADING LIMITS OF THE DAM RAISING AND BORROW AREAS ARE TO BE CLEARED AND GRUBBED AND STRIPPED.

REFERENCE DRAWINGS

12-3000I - GROUTING PLAN & PROFILE NORTH ABUTMENT.
12-3000J - GROUTING PLAN & PROFILE SOUTH ABUTMENT.
12-3000K - LAYOUT & GRADING PLAN.

FOREIGN DWG. - HARZA ENGINEERING CO.
DWG. NO. 670C181 - EXCAVATION & FOUNDATION GROUTING.
12-3000V - ACCESS ROAD SHEET 1
12-3000W - ACCESS ROAD SHEET 2

NF

DATE	NO.	DESCRIPTION	APPD.
4/10/89	1	REVISED PLAN ~ "AS-BUILT" STATUS. KCM SRM	
5/4/87	0	RELEASED FOR CONSTRUCTION	

REVISIONS

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OHIO POWER COMPANY

GAVIN PLANT

CHESHIRE OHIO

STINGY RUN FLY ASH DAM
STAGE II RAISING.
EXISTING CORE TRENCH & PROPOSED EXTENSION PLAN

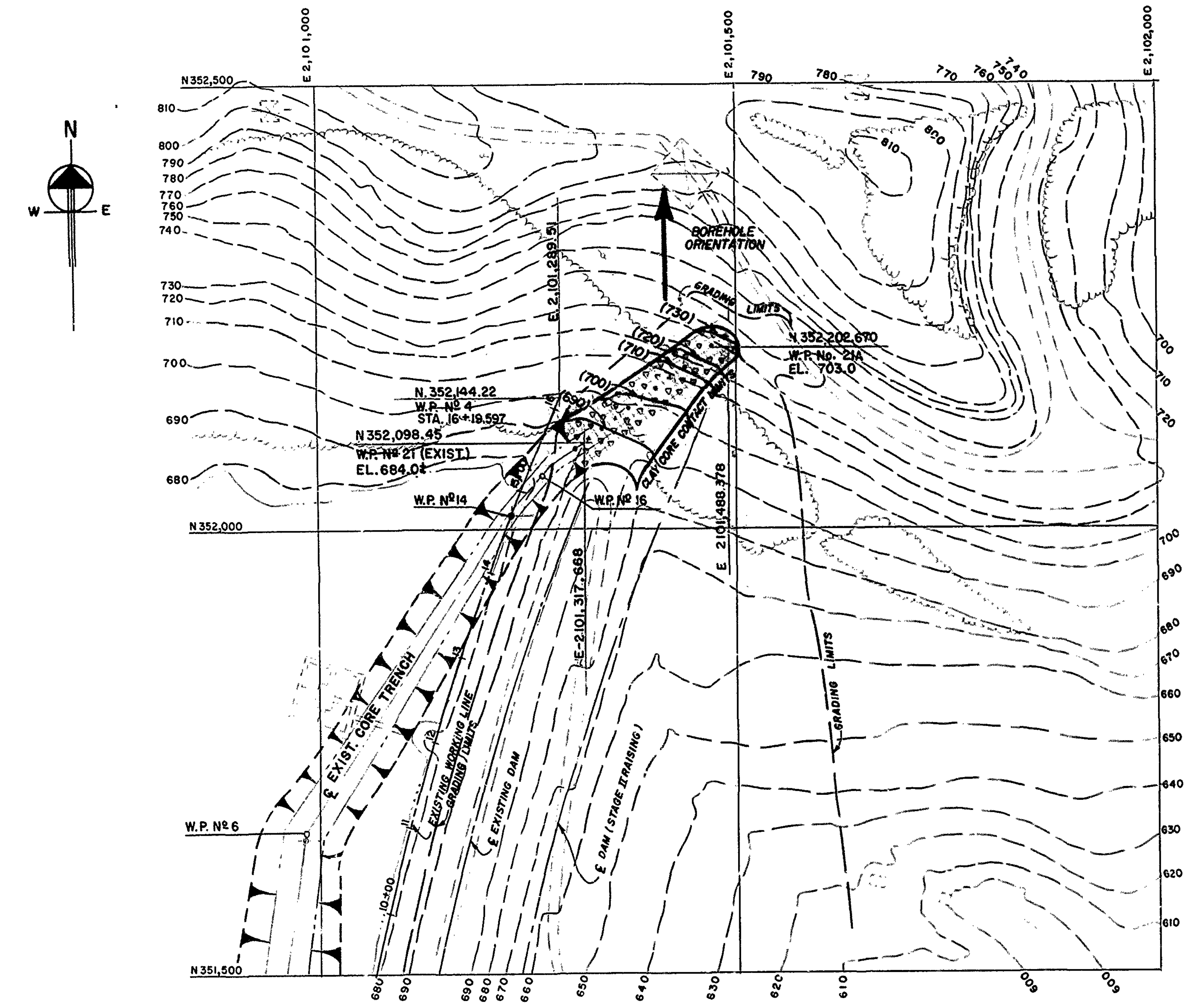
DR. NO. 12-3000H-1

ARCH.	-	ELEC.	-	MECH.	-	STR.	GS

SCALE: 1" = 100'
DR. R. G. DURO
CH. KCM
SQ. LDR. RNS
DATE: 12-1-86

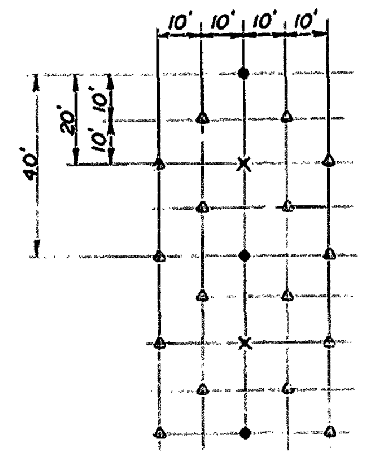
APPROVED BY
ENGINEERING DIV. B. H. Bennett PHS
DESIGN DIV.

DR. No. 12-3000I



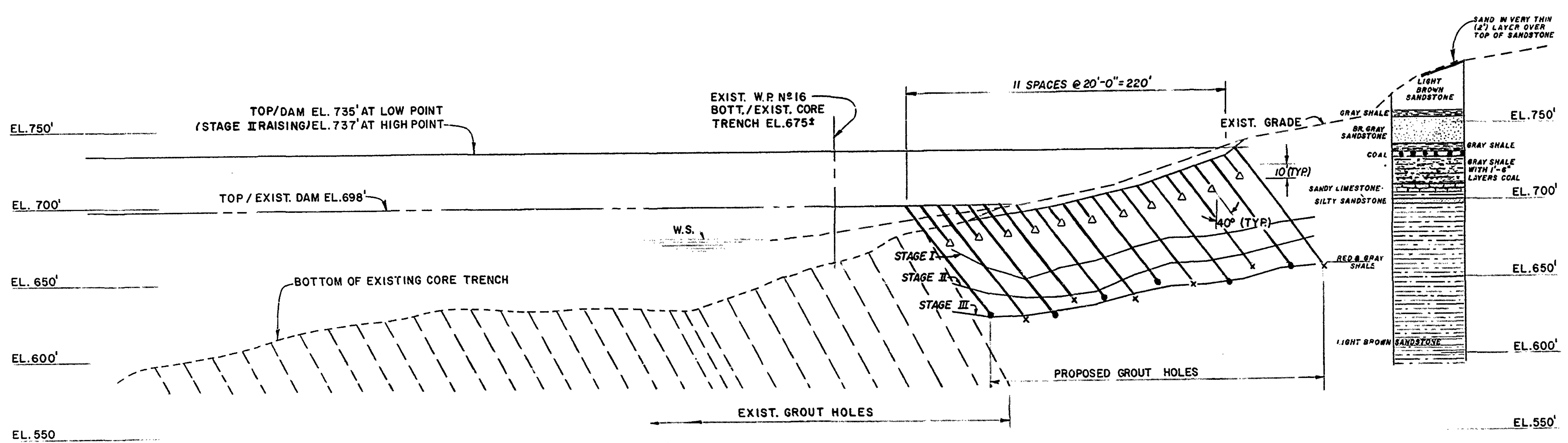
GROUTING PLAN

SCALE: 1" = 100'



GROUT PATTERN

- ● PRIMARY GROUT HOLES
- × SECONDARY GROUT HOLES
- △ GROUT CAP HOLES



℄ PROFILE OF CORE TRENCH

SCALE: 1" = 50'
(FROM W.P. Nº6 TOWARDS NORTH)

NOTE: DASHED LINES DENOTE EXISTING GROUT HOLES.

"AS-BUILT" NOTES:

1) GROUTING HOLE ALIGNMENTS AS PER DRAWING.

2) GROUTING SUMMARY REPORT FOUND IN:
MONTHLY CONSTRUCTION REPORT TO O.D.N.R.;
NO. 7 DATED 12/14/87

3) GROUTING RAW DATA FOUND IN:
MONTHLY CONSTRUCTION REPORTS TO O.D.N.R.;
NO. 6 DATED 11/9/87
NO. 7 DATED 12/14/87

GENERAL NOTES

REFERENCE DRAWINGS

12-3000K - LAYOUT & GRADING PLAN
12-3000H - EXISTING CORE TRENCH & PROPOSED EXTENSION PLAN.

FOREIGN DWG.-HARZA ENGINEERING CO.
DWG. Nº670C181A-DRILLING & GROUTING
NORTH ABUTMENT

DATE	NO.	DESCRIPTION	APPD.
4/10/89	1	AS PER RDR NO. 56-043, REV. 1: REVISED PROFILE & ADDED NOTES, "AS-BUILT" STATUS. KM SRM	Wm
5/4/87	0	RELEASED FOR CONSTRUCTION	

REVISIONS

OHIO POWER COMPANY

GAVIN PLANT

CHESHIRE OHIO

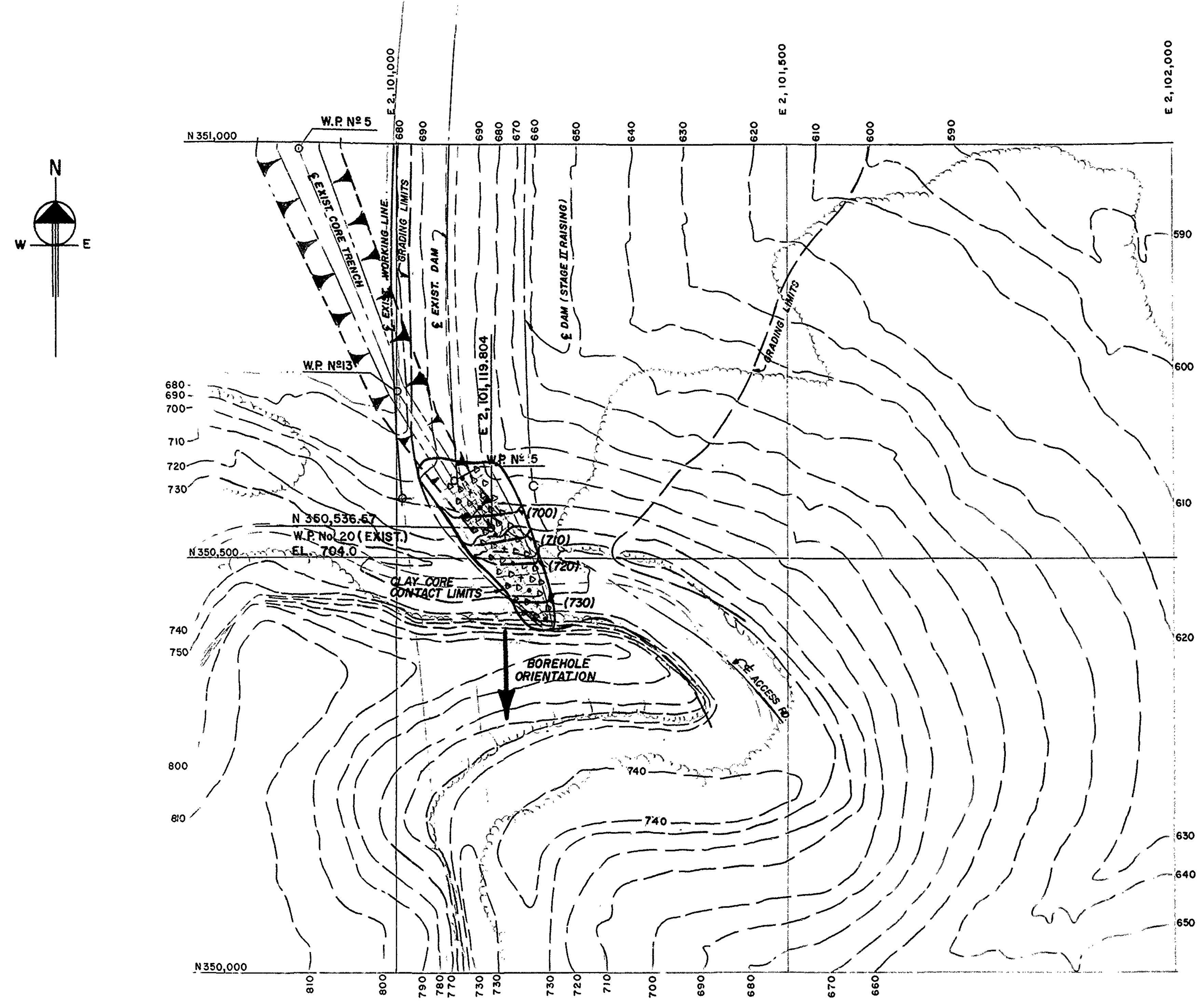
STINGY RUN FLY ASH DAM
STAGE II RAISING.
GROUTING PLAN & PROFILE
NORTH ABUTMENT

DR. NO. 12-3000I - 1

ARCH.	-	ELEC	-	MECH.	-	STR.	95

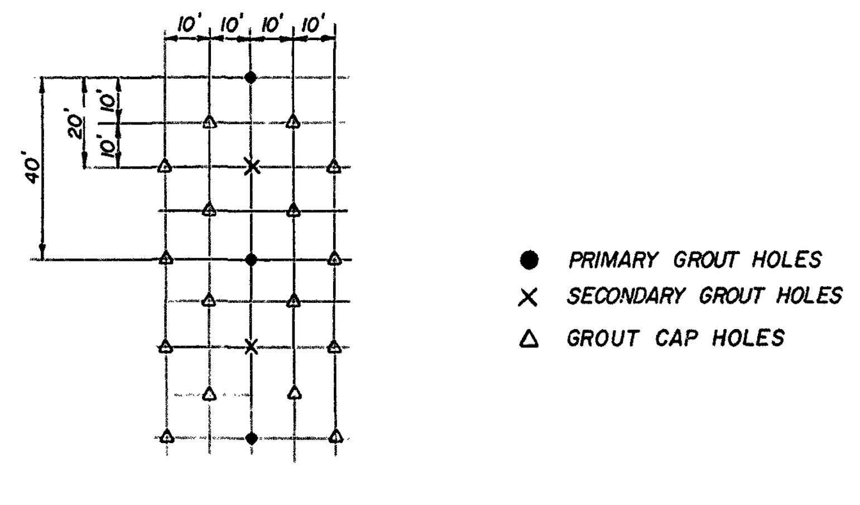
SCALE: AS NOTED
DR. R. M.
CH. R.G. DURO
SQ. LDR BVS
DATE: 12-1-86
APPROVED BY
ENGINEERING DIV B.H. Barnett
DESIGN DIV

DR. No. 12-3000J



GROUTING PLAN

SCALE: 1" = 100'

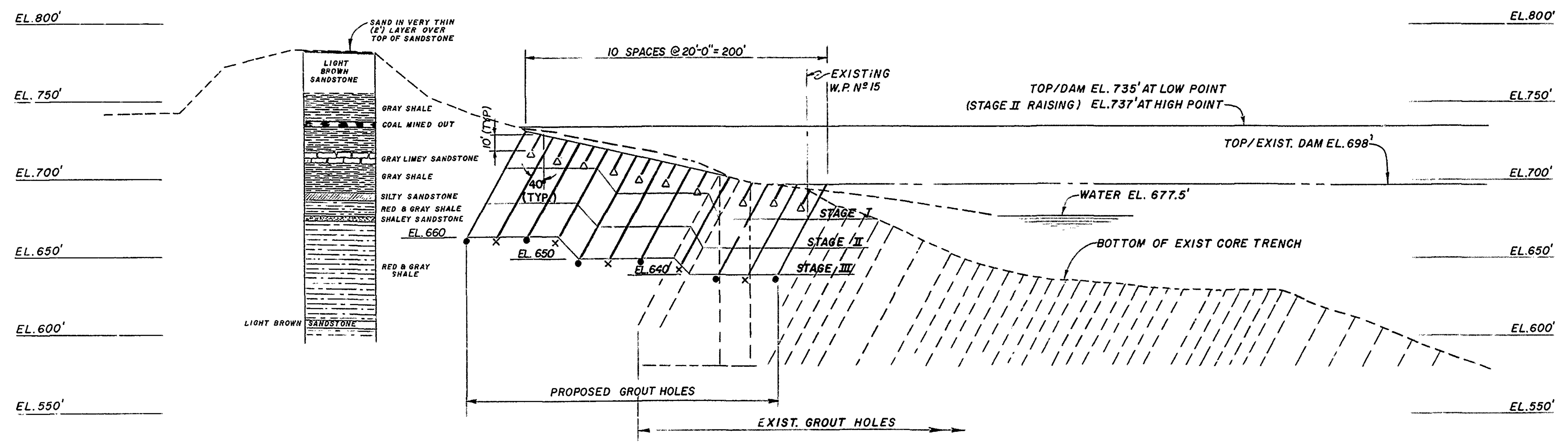


● PRIMARY GROUT HOLES

× SECONDARY GROUT HOLES

△ GROUT CAP HOLES

GROUT PATTERN



℄ PROFILE OF CORE TRENCH

SCALE: 1" = 50'

(FROM W.P. Nº 5 TOWARDS SOUTH)

NOTE DASHED LINES DENOTE EXISTING GROUT HOLES.

"AS-BUILT" NOTES:

1) GROUTING HOLE ALIGNMENTS AS PER DRAWING.

2) GROUTING SUMMARY REPORT FOUND IN:
MONTHLY CONSTRUCTION REPORT TO O.D.N.R.;
NO. 7 DATED 12/14/87

3) GROUTING RAW DATA FOUND IN:
MONTHLY CONSTRUCTION REPORTS TO O.D.N.R.;
NO. 6 DATED 11/9/87;
NO. 7 DATED 12/14/87

GENERAL NOTES

REFERENCE DRAWINGS

12-3000K - LAYOUT & GRADING PLAN

FOREIGN DWG. - HARZA ENGINEERING CO.
DWG. Nº 6700C1818 - DRILLING & GROUTING
SOUTH ABUTMENT

DATE	NO.	DESCRIPTION	APPD.
4/10/89	1	AS PER RDR NO. 56-043, REV. 1: ADDED NOTES ~ "AS-BUILT" STATUS. KLM SRM	
5/4/87	0	RELEASED FOR CONSTRUCTION	

REVISIONS

OHIO POWER COMPANY

GAVIN PLANT

CHESHIRE OHIO

STINGY RUN FLY ASH DAM
STAGE II RAISING
GROUTING PLAN & PROFILE
SOUTH ABUTMENT

DR. NO. 12-3000J-1

ARCH.	–	ELEC	–	MECH.	–	STR.	95

SCALE: AS NOTED

DR. R.M

CH. R.G DURO

SQ. LDR KNS

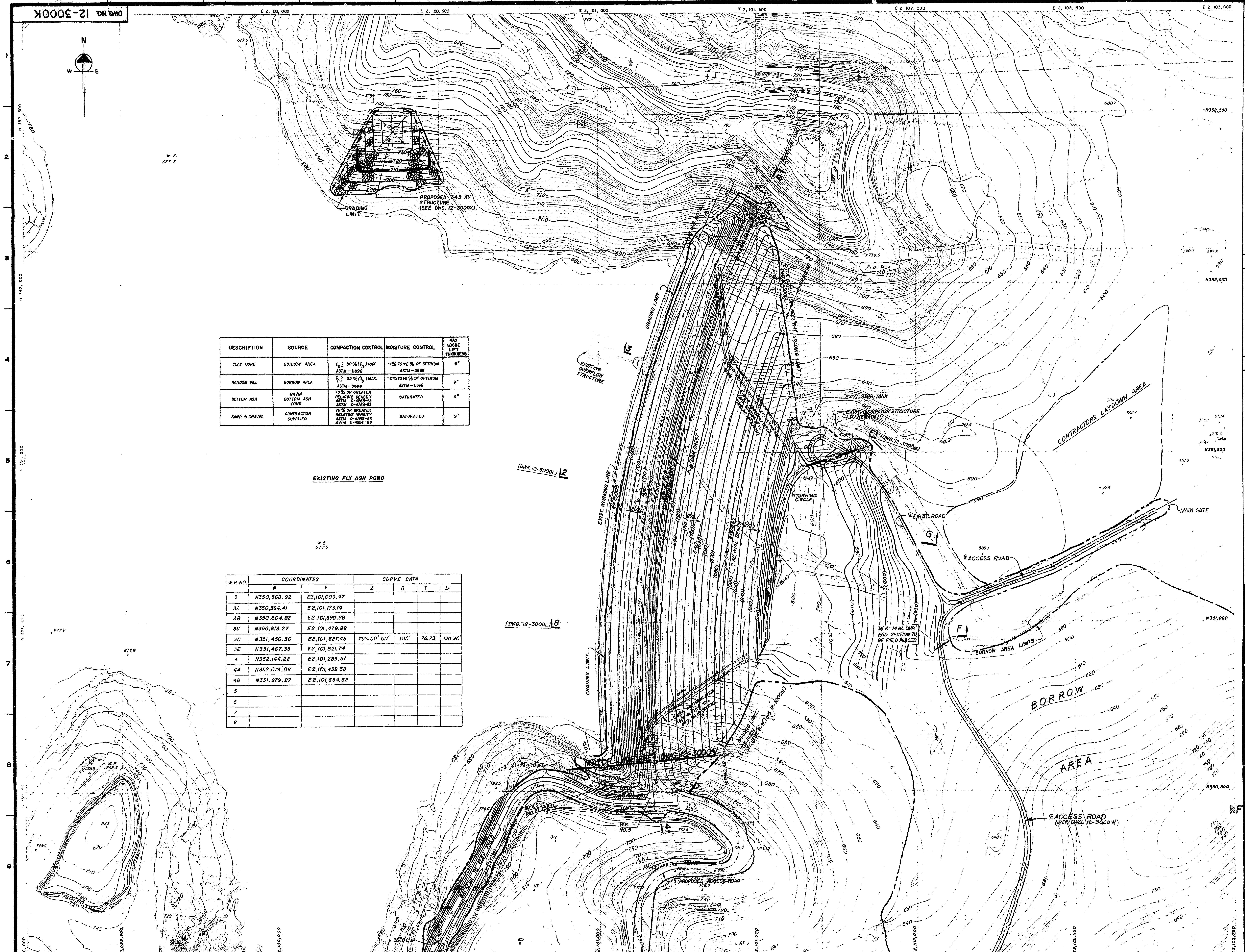
DATE: 12-1-86

APPROVED BY

ENGINEERING DIV B.H. Barnett

DESIGN DIV

AMERICAN ELECTRIC POWER SERVICE CORP.



DESCRIPTION	SOURCE	COMPACTION CONTROL	MOISTURE CONTROL	MAX LOOSE LIFT THICKNESS
CLAY CORE	BORROW AREA	$\gamma_D \geq 98\% (\gamma_D)$ MAX ASTM – D698	-1% TO +2 % OF OPTIMUM ASTM – D698	6"
RANDOM FILL	BORROW AREA	$\gamma_D \geq 95\% (\gamma_D)$ MAX. ASTM – D698	-2% TO +2 % OF OPTIMUM ASTM – D698	9"
BOTTOM ASH	GAVIN BOTTOM ASH POND	70% OR GREATER RELATIVE DENSITY ASTM D-4253-83 ASTM D-4254-83	SATURATED	9"
SAND & GRAVEL	CONTRACTOR SUPPLIED	70% OR GREATER RELATIVE DENSITY ASTM D-4253-83 ASTM D-4254-83	SATURATED	9"

W.P. NO.	COORDINATES		CURVE DATA			
	N	E	Δ	R	T	Lc
3	N350,568.92	E2,101,009.47				
3A	N350,584.41	E2,101,173.74				
3B	N350,604.82	E2,101,390.28				
3C	N350,613.27	E2,101,479.88				
3D	N351,450.36	E2,101,627.48	75°-00'-00"	100'	76.73'	130.90'
3E	N351,467.35	E2,101,821.74				
4	N352,144.22	E2,101,289.51				
4A	N352,073.06	E2,101,438 38				
4B	N351,979.27	E2,101,634.62				
5						
6						
7						
8						

GENERAL NOTES

MATERIALS

36"Ø-14 GA. C.M.P. ———— 100 L.F.
36"Ø-14 GA. C.M.P. END SECTION — 1 REQ'D.
BY:

REFERENCE DRAWINGS

12-3000L -	SECTIONS & DETAILS SHEET 1.
12-3000M -	SECTIONS & DETAILS SHEET 2.
12-3000G -	GEOLOGICAL PROFILE ALONG CREST OF DAM.
12-3000V	ACCESS ROAD SHEET 1.
12-3000W	ACCESS ROAD SHEET 2.
12-3000X	SPORN-SOUTH POINT TOWER FILL

DATE	NO.	DESCRIPTION	
4/10/89	2	REVISED PLAN — "AS-BUILT" STATUS. SRM KCM	
5/9/88	1	ADD FILL CONFIGURATION FOR 345 KV STRUCTURE 7-DE M.L./KCM	
5/4/87	0	RELEASED FOR CONSTRUCTION	

REVISIONS

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OHIO POWER COMPANY

GAVIN PLANT

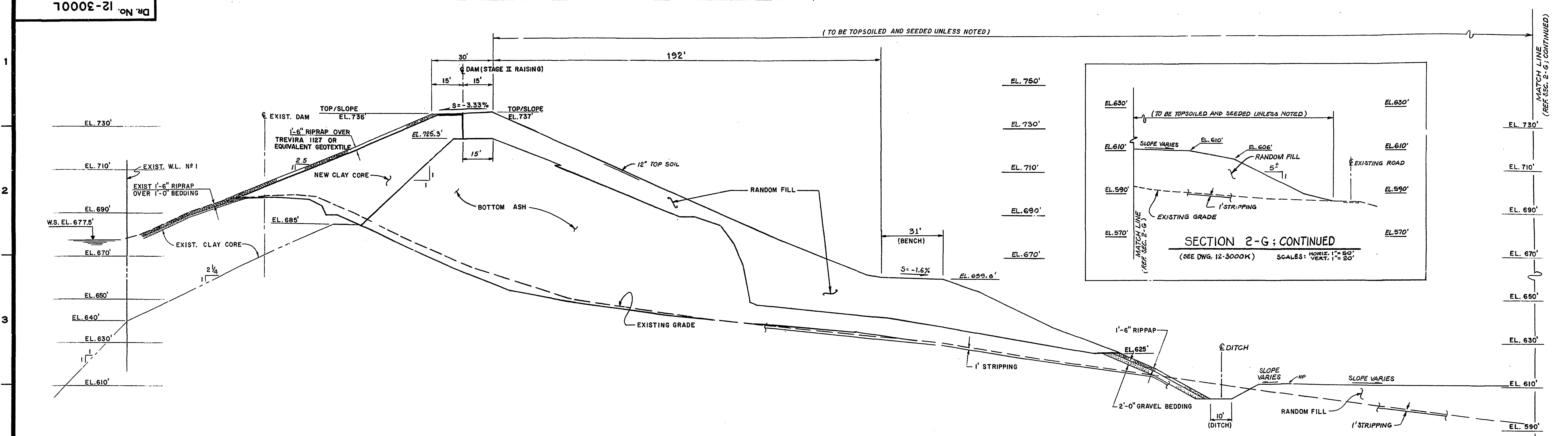
CHESHIRE OHIO

STINGY RUN FLY ASH DAM
STAGE II RAISING
LAYOUT & GRADING PLAN

DR. NO. 12-3000K-2

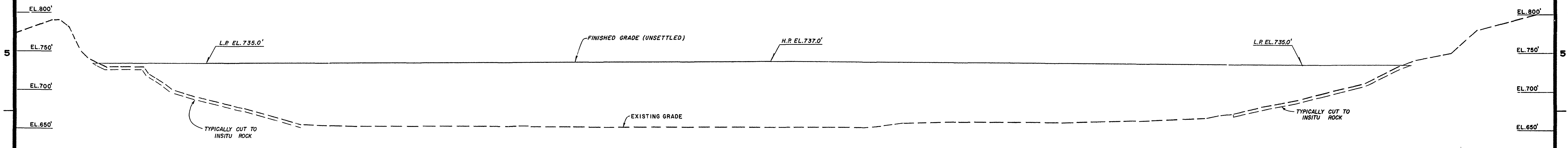
SCALE: 1" = 100'
DR. R.G. DURO
KCM
BNS
12-1-86
APPROVED
ENGINEER DIV. B.H. Bennett
DESIGN DIV.

DR. No. 12-3000L



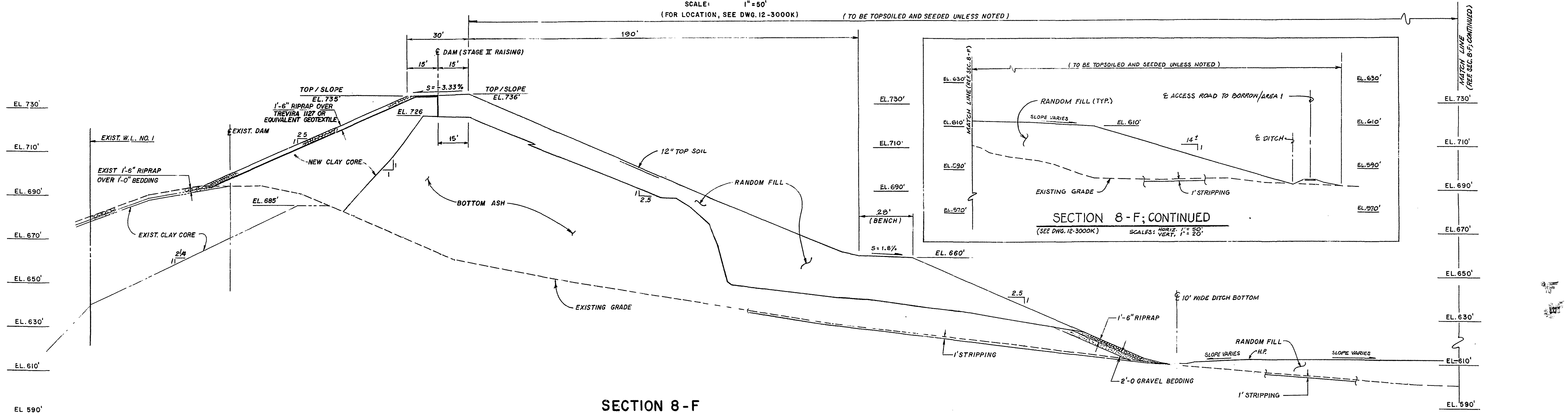
SECTION 2-G

SCALE: 1"= 20'
(FOR LOCATION, SEE DWG. 12-3000K)



SECTION 4-G

TOP OF DOWNSTREAM SIDE
SCALE: 1" = 50'
(FOR LOCATION, SEE DWG. 12-3000K)



SECTION 8-F

SCALE: 1" = 20'
(FOR LOCATION, SEE DWG. 12-3000K)

DATE	NO.	DESCRIPTION	APPD.
4/10/89	2	REVISED SECTIONS — "AS-BUILT" STATUS. KCM BRM	
4/12/88	1	MODIFIED THE INTERNAL CONFIGURATION AS PER RDR-GA-DO-3386 M. A. KCM	
5/4/87	0	RELEASED FOR CONSTRUCTION	

REVISIONS

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OHIO POWER COMPANY

GAVIN PLANT

CHESHIRE OHIO

STINGY RUN FLY ASH DAM
STAGE II RAISING
SECTIONS & DETAILS
SHEET 1

DR. NO. 12-3000L - 2

ARCH. – ELEC – MECH. – STR. GS

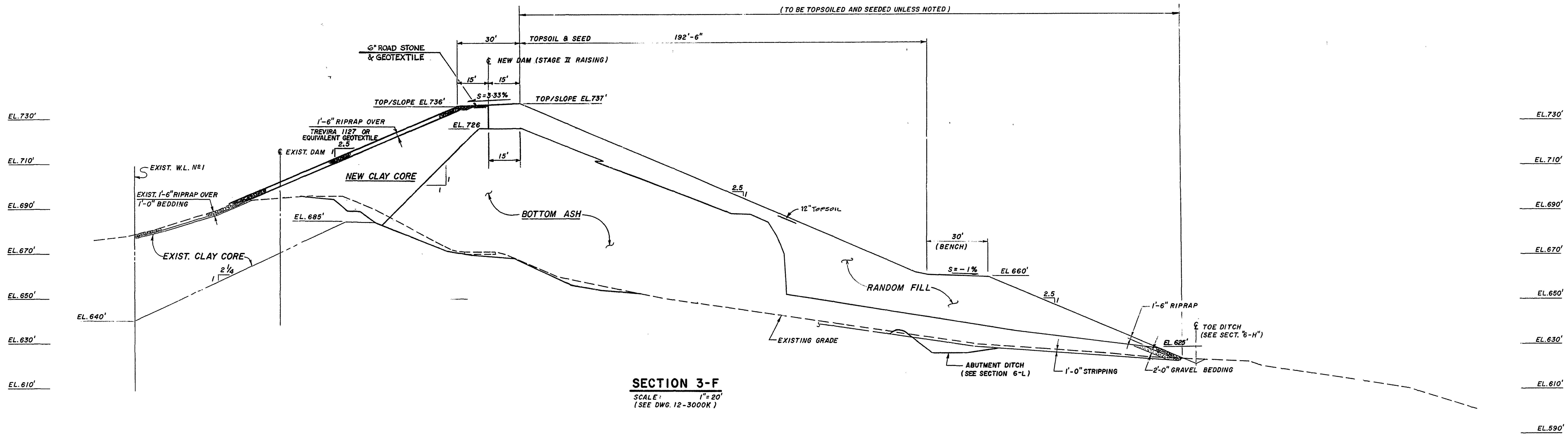
SCALE: AS NOTED
DR. R.M./R.G.D.
CH. H.G. DURO
SQ. LDR RNS
DATE: 12-1-86

APPROVED BY
ENGINEERING DIV
B.H. Bennett
DESIGN DIV

AMERICAN ELECTRIC POWER SERVICE CORP.

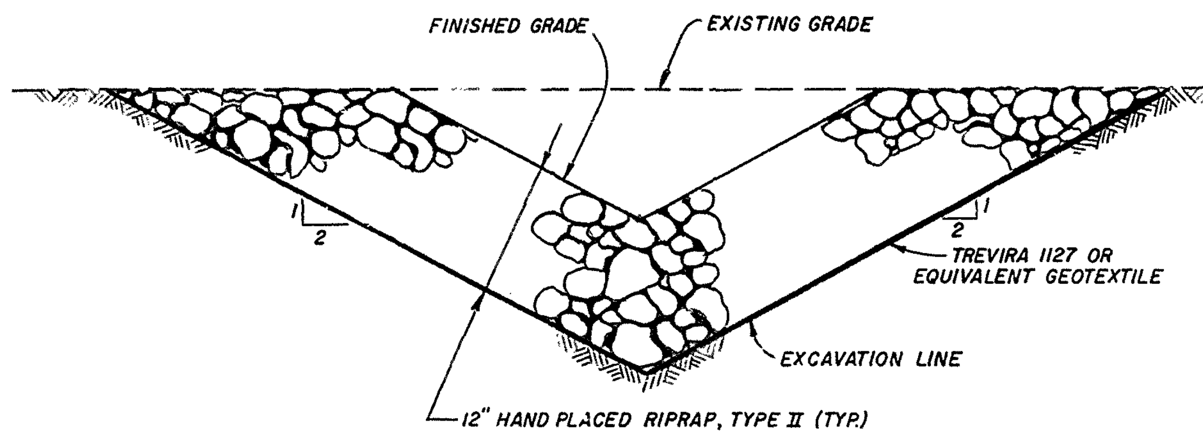
DR. No. 12-3000M

GENERAL NOTES



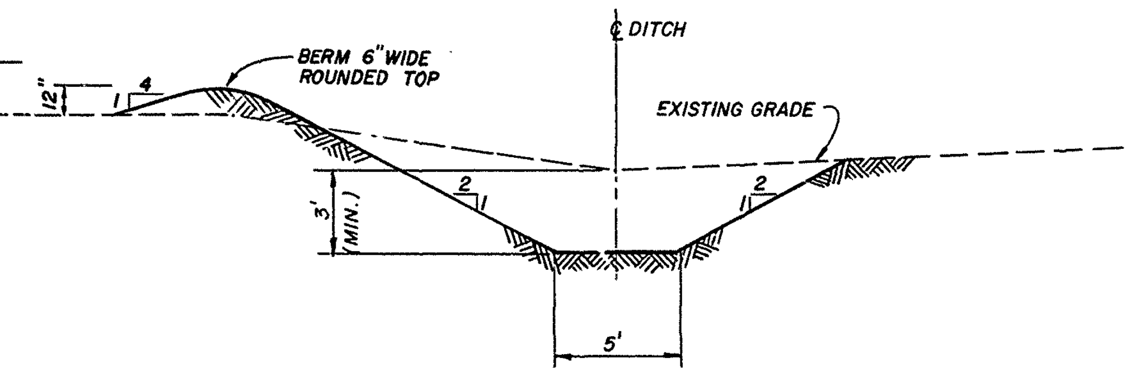
SECTION 3-F

SCALE: 1"=20'
(SEE DWG. 12-3000K)



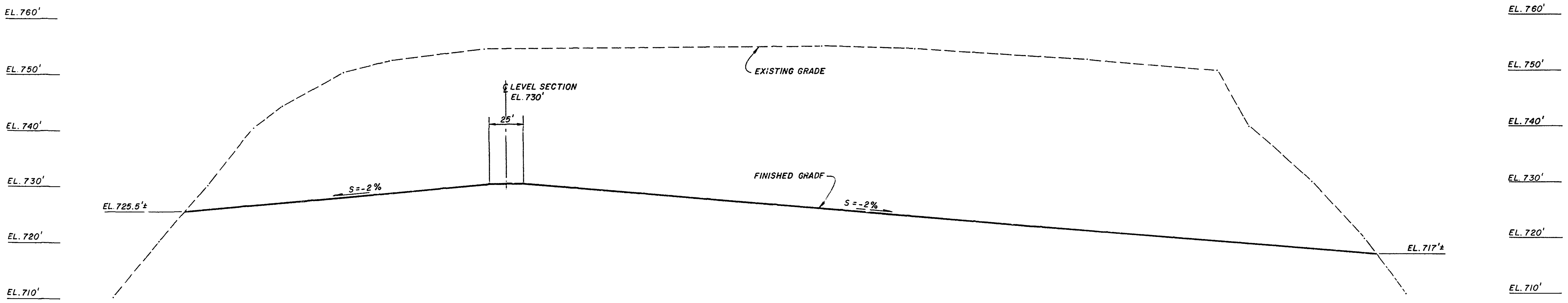
SECTION 6-H

SCALE: 1"=1'-0"
(TYPICAL FOR SIDE & TOE DITCHES)
(SEE DWG. 12-3000K)



SECTION 6-L

SCALE: 1"=5'
(TYPICAL FOR ABUTMENT DITCHES
SEE DWG. 12-3000H)



PROFILE 8-E

℄ EMERGENCY SPILLWAY

SCALES: 1"=40' HOR.
1"=10' VERT.
(SEE DWG. 12-3000Q)

REFERENCE DRAWINGS

12-3000K - LAYOUT & GRADING PLAN.
12-3000Q - EMERGENCY SPILLWAY DAM LAYOUT & GRADING PLAN.
12-3000H - EXISTING CORE TRENCH & NEW EXTENSION PLAN.
12-3000L - SECTIONS & DETAILS SHEET 1

DATE	NO.	DESCRIPTION	APPD.
4/10/89	2	REVISED SECTIONS ~ "AS-BUILT" STATUS. KCM SRM	WM
4/12/88	1	MODIFIED THE INTERNAL CONFIGURATION AS PER RDR-GA-00-3386 M.W. KCM	
5/4/87	0	RELEASED FOR CONSTRUCTION	

REVISIONS

OHIO POWER COMPANY

GAVIN PLANT

CHESHIRE OHIO

STINGY RUN FLY ASH DAM
STAGE II RAISING
SECTIONS & DETAILS
SHEET 2

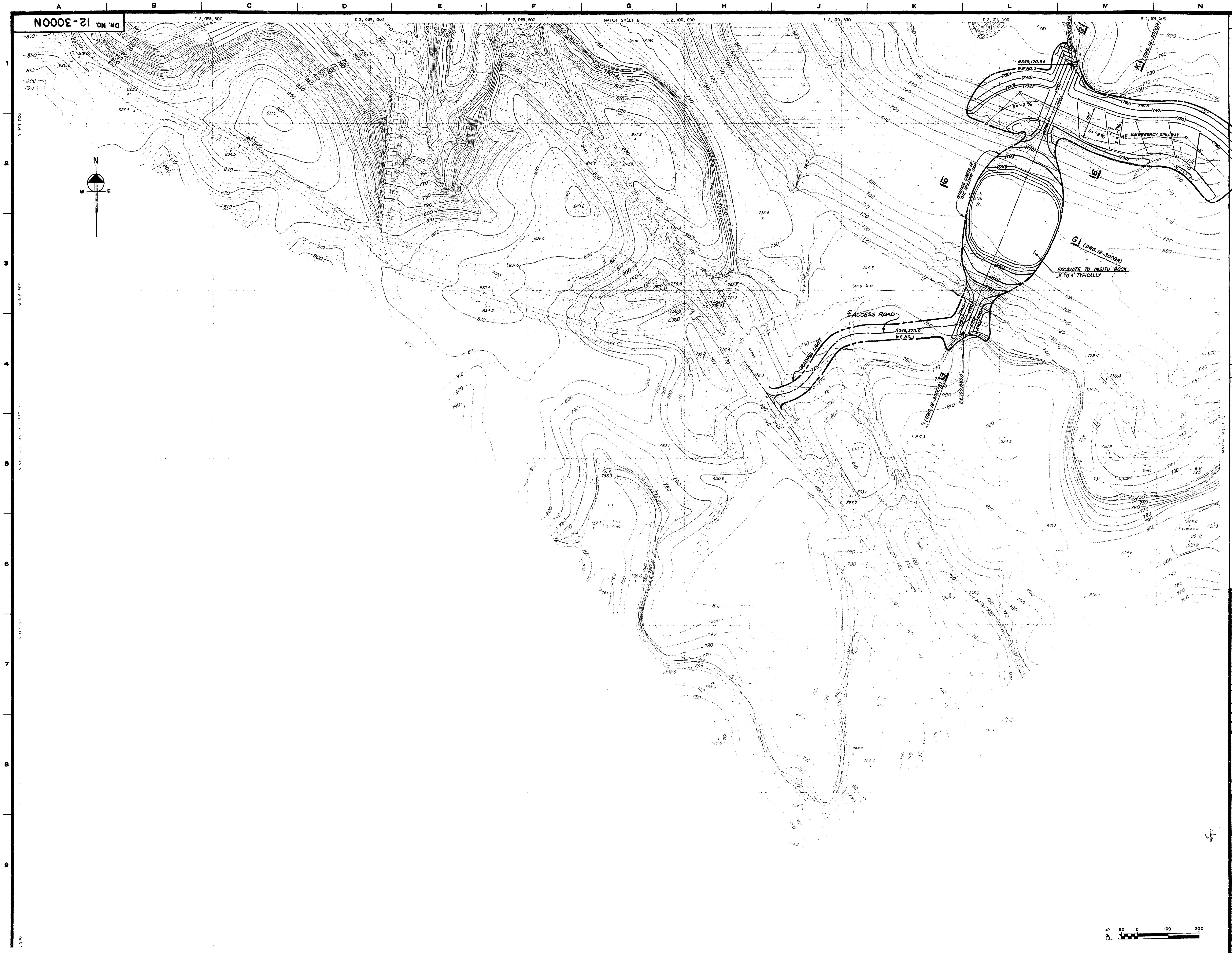
DR. NO. 12-3000M - 2

ARCH.	-	ELEC	-	MECH.	-	STR.	95

SCALE: AS NOTED
DR. R.M./R.D.
CH. R.D.
SQ. LDR RNS
DATE: 12-1-86

APPROVED BY
ENGINEERING DIV B.H. Bennett PKA
DESIGN DIV

DR. NO. 12-3000N



GENERAL NOTES

1. ALL AREAS WITHIN THE GRADING LIMITS OF THE DAM RAISING AND BORROW AREAS ARE TO BE CLEARED AND GRUBBED AND STRIPPED.

REFERENCE DRAWINGS

12-3000Q - EMERGENCY SPILLWAY DAM LAYOUT & GRADING PLAN.

12-3000R - EMERGENCY SPILLWAY DAM PROFILE & SECTIONS.

DATE	No.	DESCRIPTION	APPD.
4/10/89	1	REVISED PLAN — "AS-BUILT" STATUS.	
5/4/87	0	RELEASED FOR CONSTRUCTION	

REVISIONS

OHIO POWER COMPANY

GAVIN PLANT

CHESHIRE OHIO

STINGY RUN FLY ASH DAM
STAGE II RAISING
EMERGENCY SPILLWAY DAM
EXCAVATION PLAN

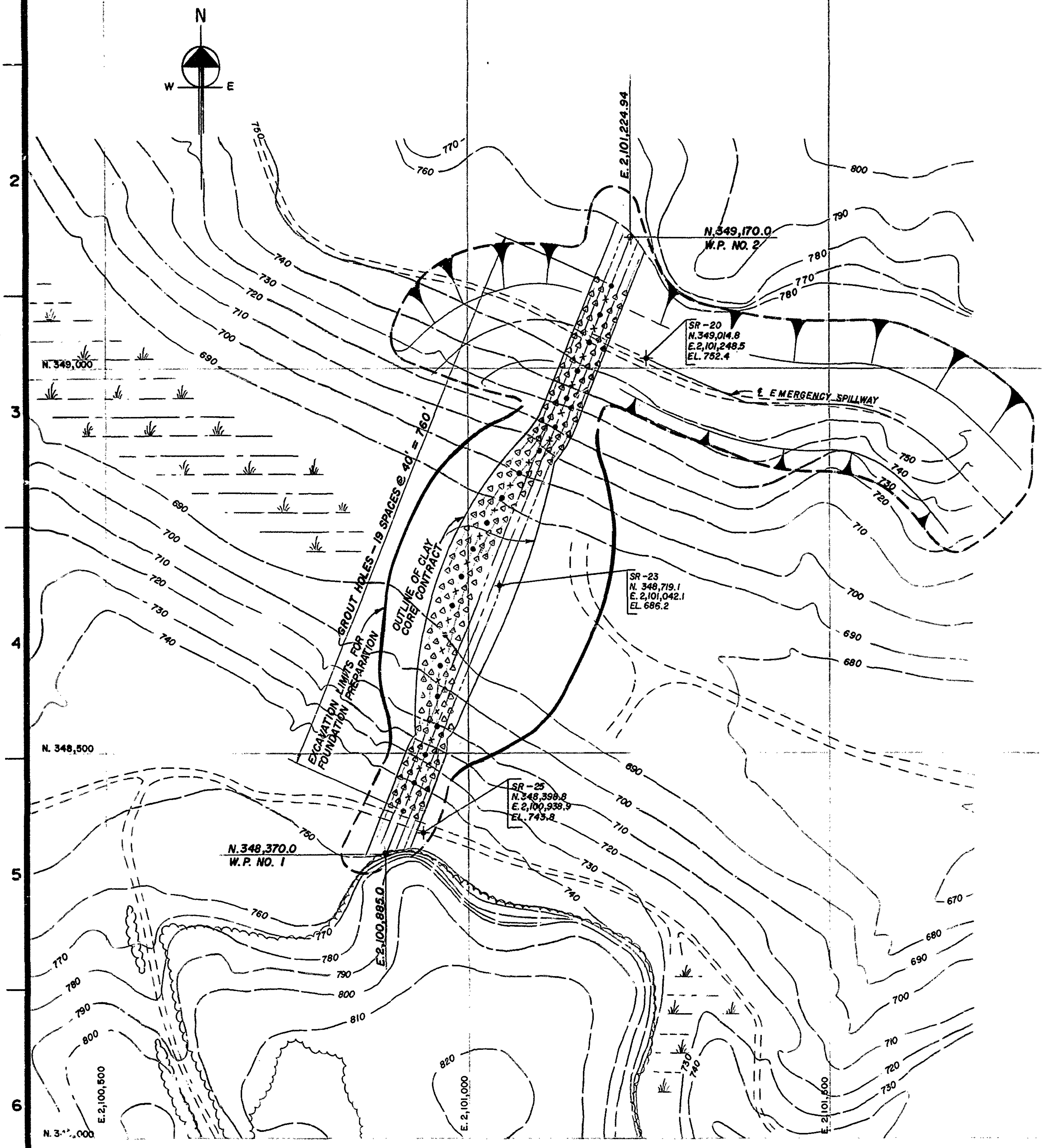
DR. NO. 12-3000N-1

ARCH.	—	ELEC.	—	MECH.	—	STR.	95

SCALE: 1" = 100'
DR. R.G. DURO
CH. KCM
SQ. LDR. BV5
DATE: 2-1-86

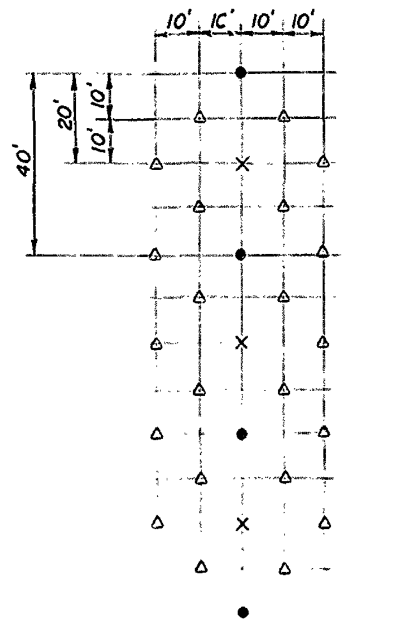
APPROVED BY
ENGINEERING DIV. B.H. Bennett
DESIGN DIV.

DWG. NO. 12-3000P



GROUTING PLAN

SCALE: 1" = 100'



● PRIMARY GROUT HOLES
× SECONDARY GROUT HOLES
△ GROUT CAP HOLES

GROUT PATTERN

"AS-BUILTS" NOTES:

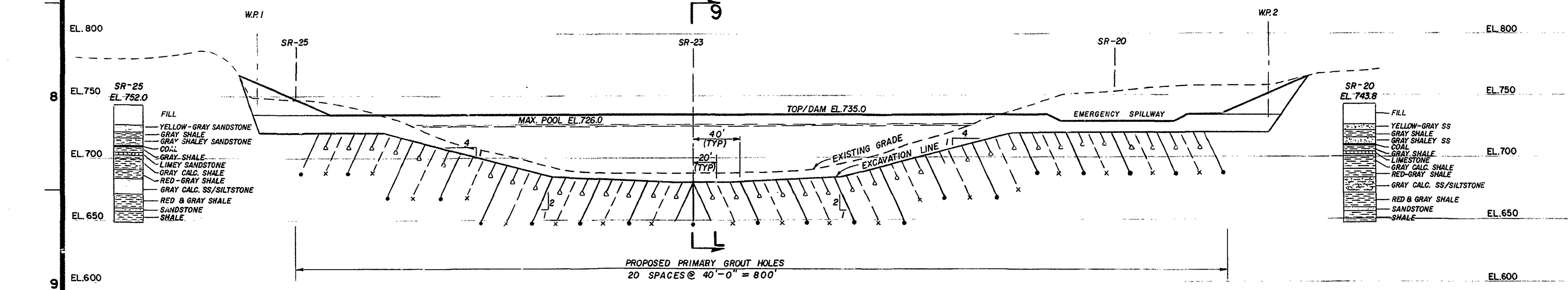
1) GROUTING HOLE ALIGNMENTS AS PER DRAWING.

2) GROUTING SUMMARY REPORT FOUND IN:

MONTHLY CONSTRUCTION REPORT TO O.D.N.R.,
NO. 4 DATED 9/10/87

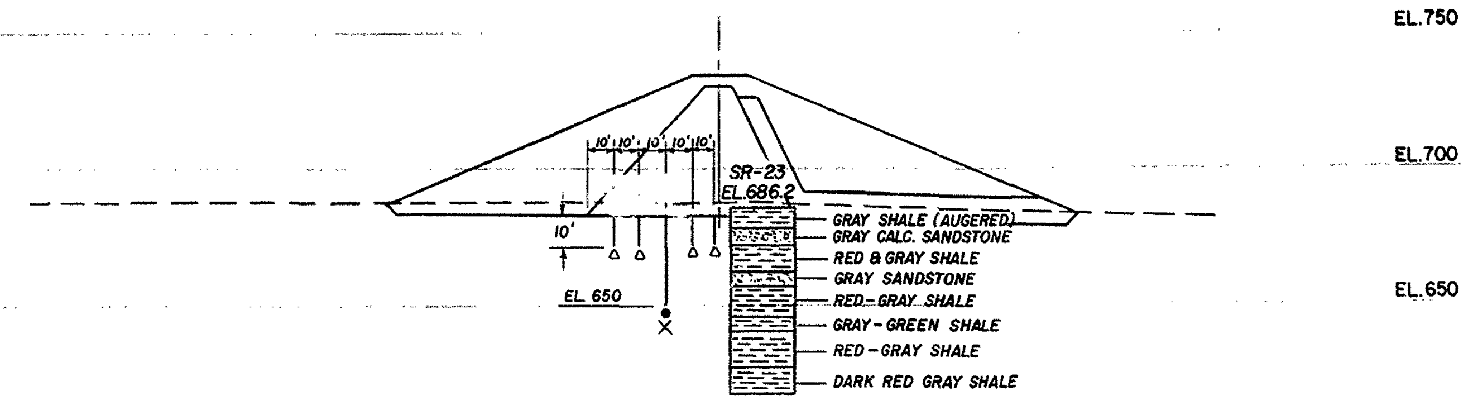
3) GROUTING RAW DATA FOUND IN:

BI-WEEKLY CONSTRUCTION REPORTS TO O.D.N.R.,
NO. 5 DATED 7/21/87
NO. 6 DATED 8/10/87
NO. 7 DATED 8/24/87



℄ PROFILE OF CORE TRENCH

SCALE: 1" = 50'



SECTION 9-L

SCALE: 1" = 50'

GENERAL NOTES

REFERENCE DRAWINGS

.2-3000Q - EMERGENCY SPILLWAY DAM LAYOUT & GRADING PLAN.
12-3000N - EMERGENCY SPILLWAY DAM EXCAVATION PLAN

DATE	NO.	DESCRIPTION	APPD.
4/10/89	1	AS PER RDR NO. 56-043, REV. 1: REVISED PROFILE & ADDED NOTES, "AS-BUILT" STATUS. KCM	
5/4/87	0	RELEASED FOR CONSTRUCTION	

REVISIONS

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OHIO POWER COMPANY

GAVIN PLANT

CHESHIRE OHIO

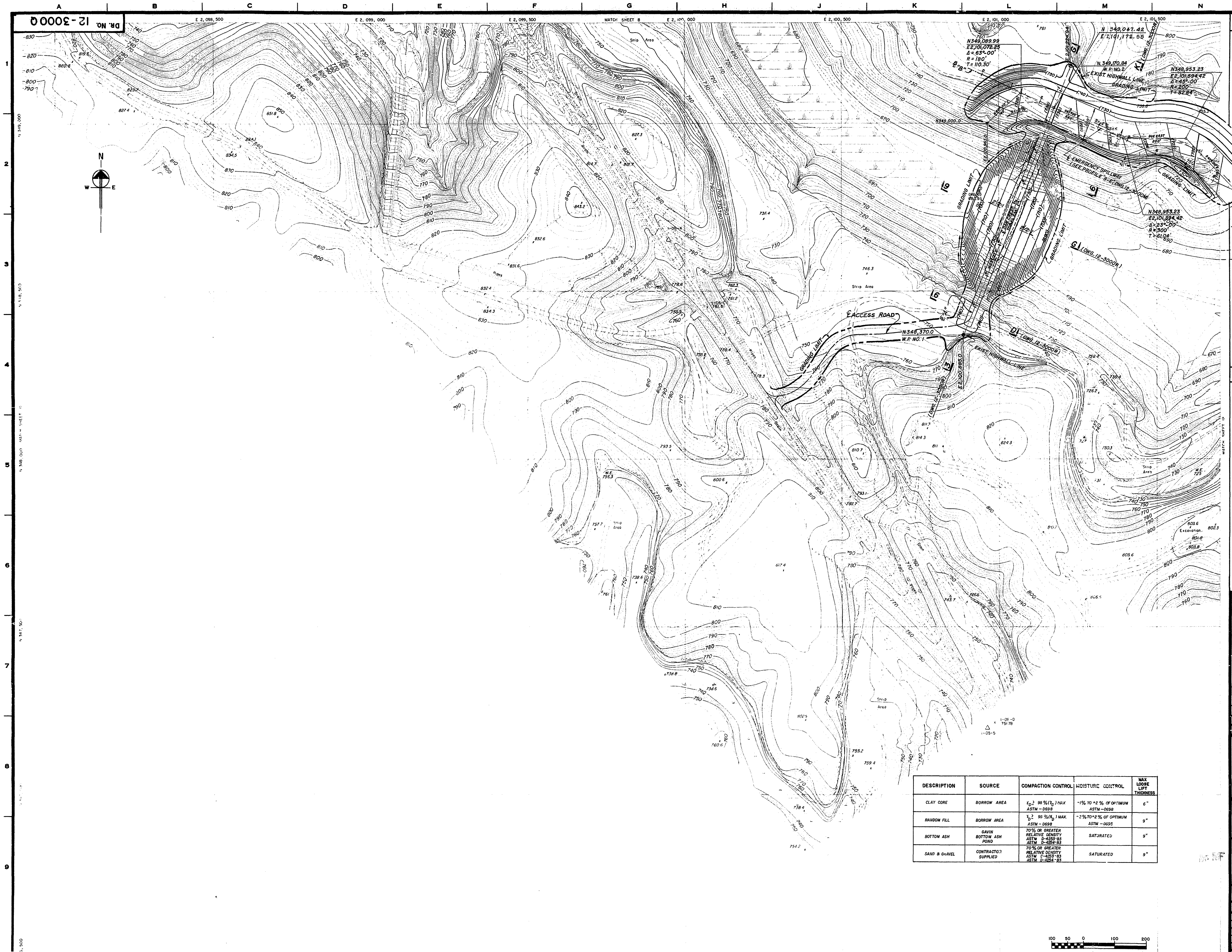
STINGY RUN FLY ASH DAM
STAGE II RAISING
GROUTING PLAN & PROFILE
SPILLWAY DAM

DWG. NO. 12-3000P-1

ARCH.	-	ELEC.	-	MECH.	-	STR.	95

SCALE: AS NOTED
DR: R.M.H.
CH: KCM
SQ. LDR: DNS
DATE: 12-1-86
APPROVED BY
ENGINEERING DIVISION
B.H. Bennett
DESIGN DIVISION

DR. NO. 12-3000Q



GENERAL NOTES

REFERENCE DRAWINGS

12-3000R - EMERGENCY SPILLWAY DAM PROFILE & SECTIONS.

12-3000N - EMERGENCY SPILLWAY DAM EXCAVATION PLAN.

DESCRIPTION	SOURCE	COMPACTION CONTROL	MOISTURE CONTROL	MAX LOOSE LIFT THICKNESS
CLAY CORE	BORROW AREA	$\gamma_D \geq$ 98 % (γ_D) MAX ASTM - D698	-1% TO +2 % OF OPTIMUM ASTM - D698	6"
RANDOM FILL	BORROW AREA	$\gamma_D \geq$ 95 % (γ_D) MAX. ASTM - D698	-2% TO +2 % OF OPTIMUM ASTM - D698	9"
BOTTOM ASH	GAVIN BOTTOM ASH POND	70% OR GREATER RELATIVE DENSITY ASTM D-4253-83 ASTM D-4254-83	SATURATED	9"
SAND & GRAVEL	CONTRACTOR SUPPLIED	70% OR GREATER RELATIVE DENSITY ASTM D-4253-83 ASTM D-4254-83	SATURATED	9"

DATE	No.	DESCRIPTION	APPD.
4/10/89	1	REVISED PLAN ~ "AS-BUILT" STATUS. KCM SRM	
5/4/87	0	RELEASED FOR CONSTRUCTION	

REVISIONS

OHIO POWER COMPANY

GAVIN PLANT

CHESHIRE OHIO

STINGY RUN FLY ASH DAM
STAGE II RAISING
EMERGENCY SPILLWAY DAM
LAYOUT & GRADING PLAN

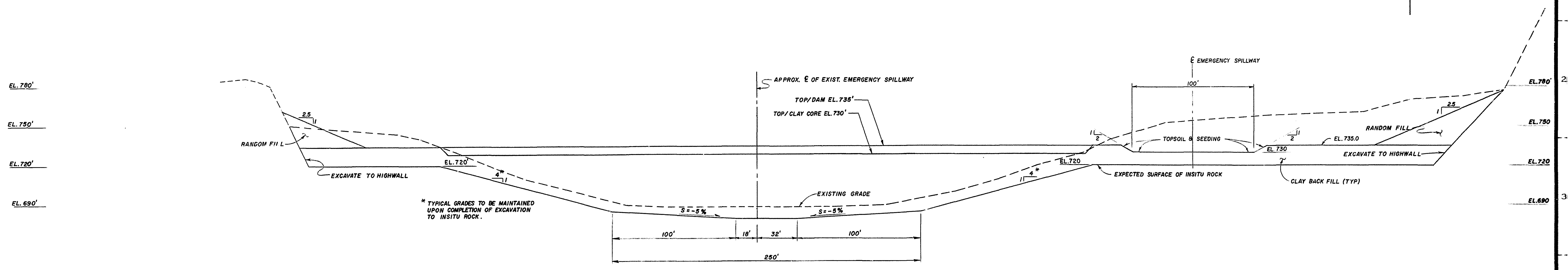
DR. NO. 12-3000Q-1

ARCH.	-	ELEC.	-	MECH.	-	STR.	95

SCALE: 1" = 100'
DR. R.G. DURO
CH. KCM
SQ. LDR. RNS
DATE 12-1-86
APPROVED BY
ENGINEERING DIV. B.H. Bennett
DESIGN DIV.

DR. No. 12-3000R

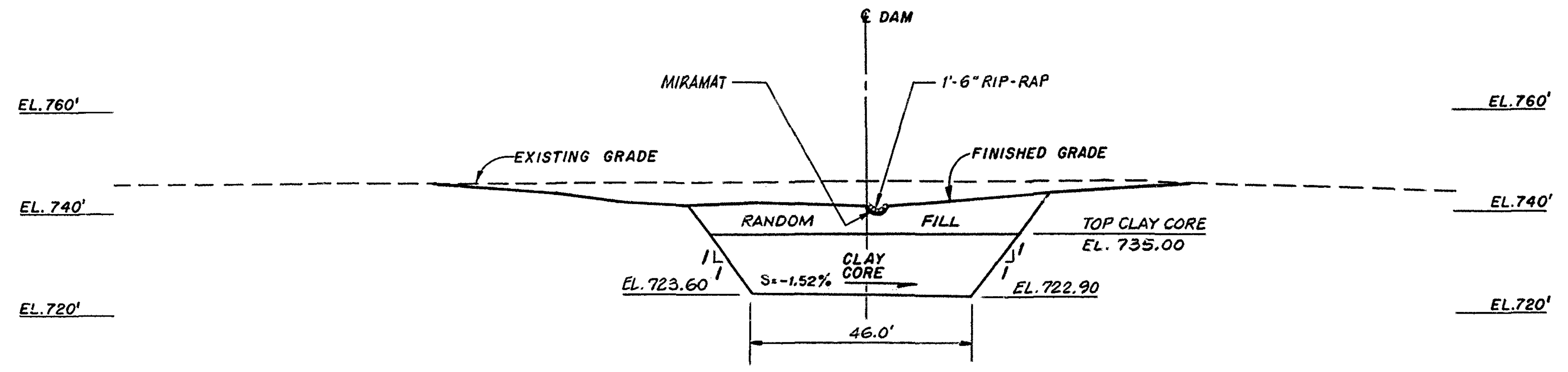
GENERAL NOTES



PROFILE 3-G

SCALE: 1" = 30'

(FOR LOCATION, SEE DWG. 12-3000Q)

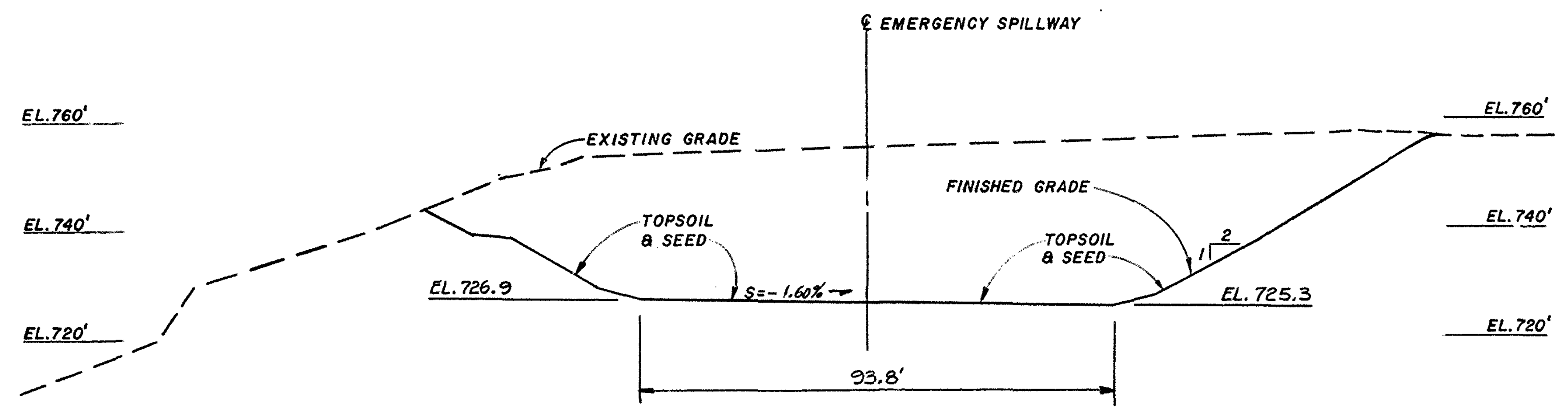


(℄ "A" STA. 6+50.00)

SECTION 6-D

SCALE: 1" = 20'

(FOR LOCATION, SEE DWG. 12-3000Q)

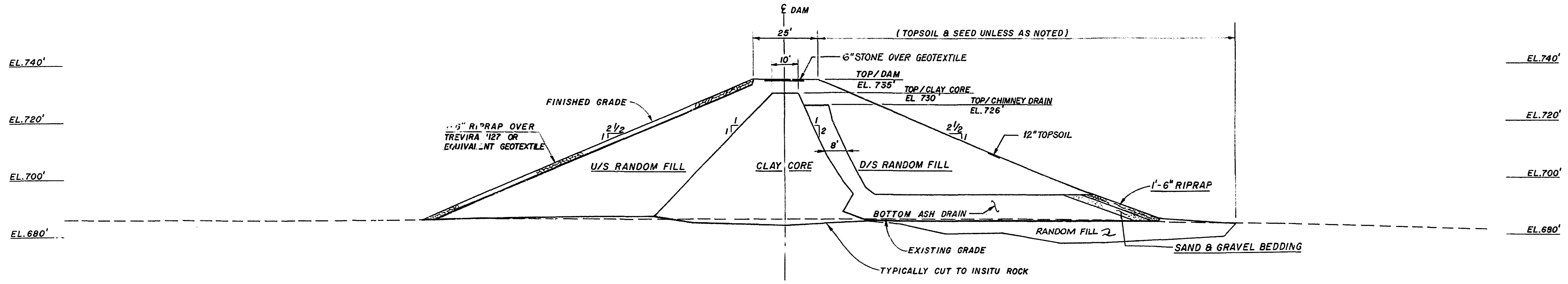


(℄ "B" STA. 4+50.00)

SECTION 6-K

SCALE: 1" = 20'

(FOR LOCATION, SEE DWG. 12-3000Q)



(℄ "A" STA. 3+00.00)

SECTION 9-G

SCALE: 1" = 20'

(FOR LOCATION, SEE DWG. 12-3000Q)

REFERENCE DRAWINGS

12-3000Q - EMERGENCY SPILLWAY DAM LAYOUT & GRADING.

12-3000N - EMERGENCY SPILLWAY DAM EXCAVATION PLAN.

DATE	NO.	DESCRIPTION	APPD.
4/10/89	1	REVISED SECTIONS — "AS-BUILT" STATUS. SRM	
5/4/87	0	RELEASED FOR CONSTRUCTION	

REVISIONS

OHIO POWER COMPANY

GAVIN PLANT

CHESHIRE OHIO

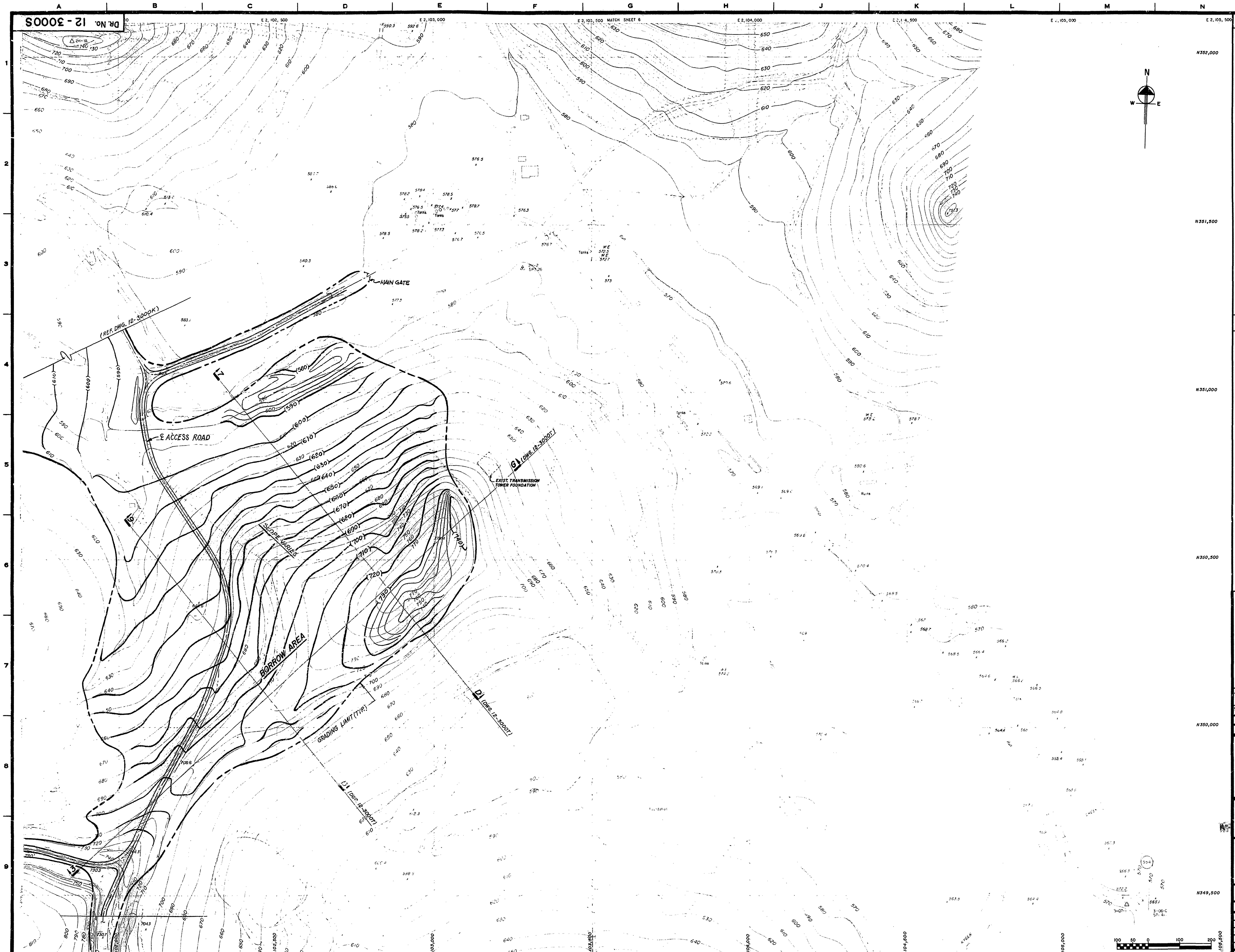
STINGY RUN FLY ASH DAM
STAGE II RAISING
EMERGENCY SPILLWAY
DAM
PROFILE & SECTIONS

DR. NO. 12-3000R-1

ARCH.	—	ELEC	—	MECH.	—	STR.	95

SCALE: 1" = 20'
DR. R.M.
CH. R G. DURO
SQ. LDR AHS
DATE: 12-1-86

APPROVED BY
ENGINEERING DIV B.H. Bennett
DESIGN DIV



GENERAL NOTES

REFERENCE DRAWINGS

12 -30C0T - STINGY RUN FLY ASH DAM STAGE II RAISING - BORRCW SITE SECTS.

DATE	No.	DESCRIPTION	APPD.
4/10/89	1	REVISED PLAN — "AS-BUILT" STATUS. ECM SRM	KBM
5/4/87	0	RELEASED FOR CONSTRUCTION	

REVISIONS

OHIO POWER COMPANY

GAVIN PLANT

CHESHIRE OHIO

STINGY RUN FLY ASH DAM
STAGE II RAISING
BORROW SITE GRADING PLAN

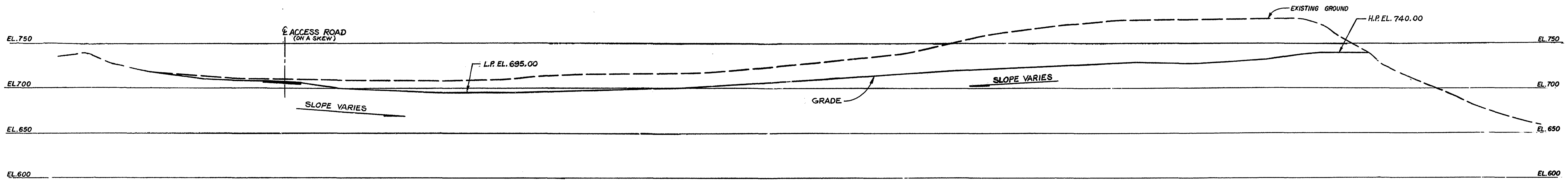
DR. No. 12 - 3000S - 1

ARCH.	—	ELEC.	—	MECH.	—	STR.	95

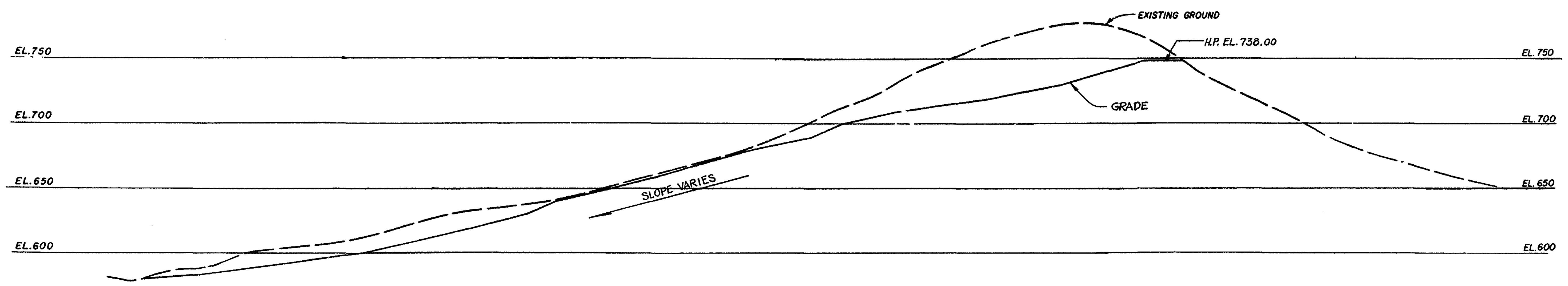
SCALE: 1"=100'
DR. M.HERSHEY
CH. RNS
SQ. LDR. RNS
DATE: 12-1-86

APPROVED BY
ENGINEERING DIV. B.H. Bennett
DESIGN DIV.

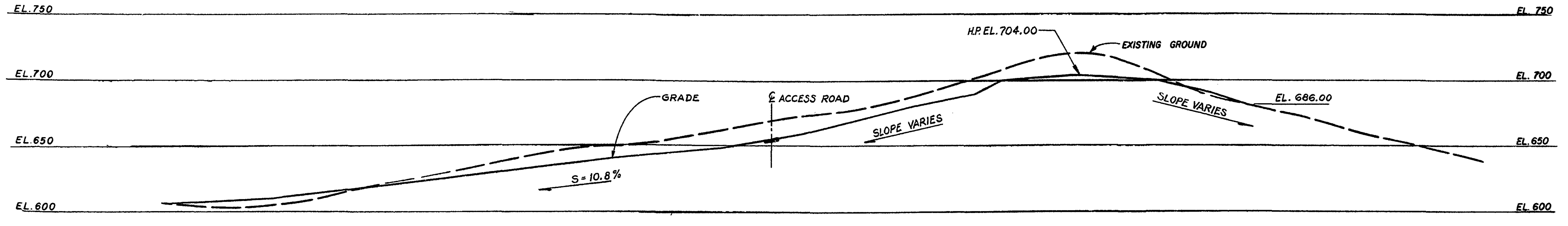
DR. No. 12-3000T



SECTION 3-G
SCALE : 1" = 50'
(SEE DWG. 12-3000S)



SECTION 7-D
SCALE : 1" = 50'
(SEE DWG. 12-3000S)



SECTION 9-D
SCALE : 1" = 50'
(SEE DWG. 12-3000S)

GENERAL NOTES

REFERENCE DRAWINGS

DATE	NO.	DESCRIPTION	APPD.
4/10/89	1	REVISED SECTIONS — "AS-BUILT" STATUS. KCM BRM	
6/4/87	0	RELEASED FOR CONSTRUCTION	

REVISIONS

OHIO POWER COMPANY
GAVIN PLANT
CHESHIRE OHIO

STINGY RUN FLY ASH DAM
STAGE II RAISING
BORROW SITE SECTIONS

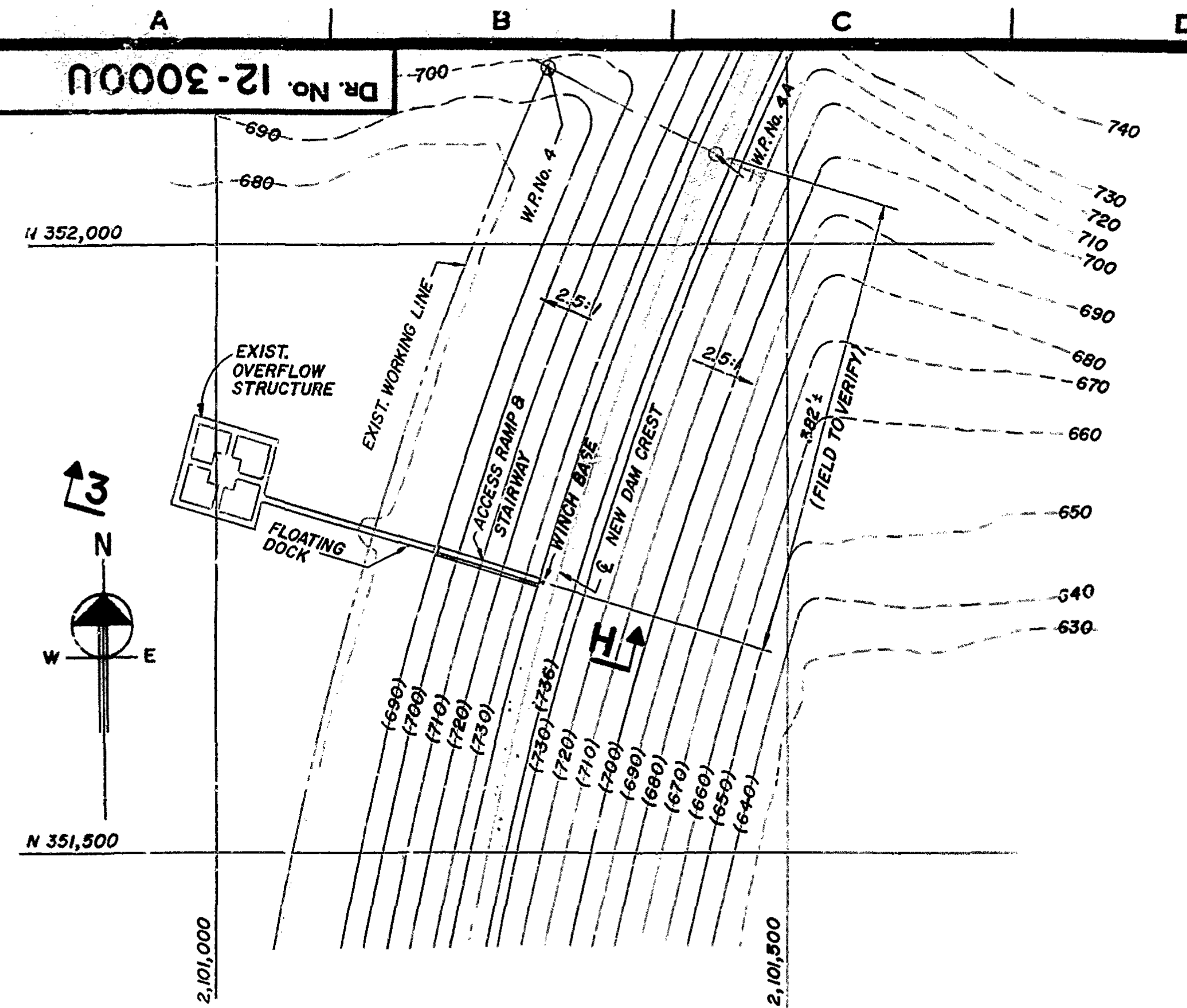
DR. NO. 12 - 3000T-1

ARCH.	-	ELEC	-	MECH.	-	STR.	GS

SCALE: 1" = 50'
DR. M. HERSHEY
CH. RNS
SQ. LDR RNS
DATE: 12-1-86

APPROVED BY
ENGINEERING DIV
R. H. Bennett
DESIGN DIV

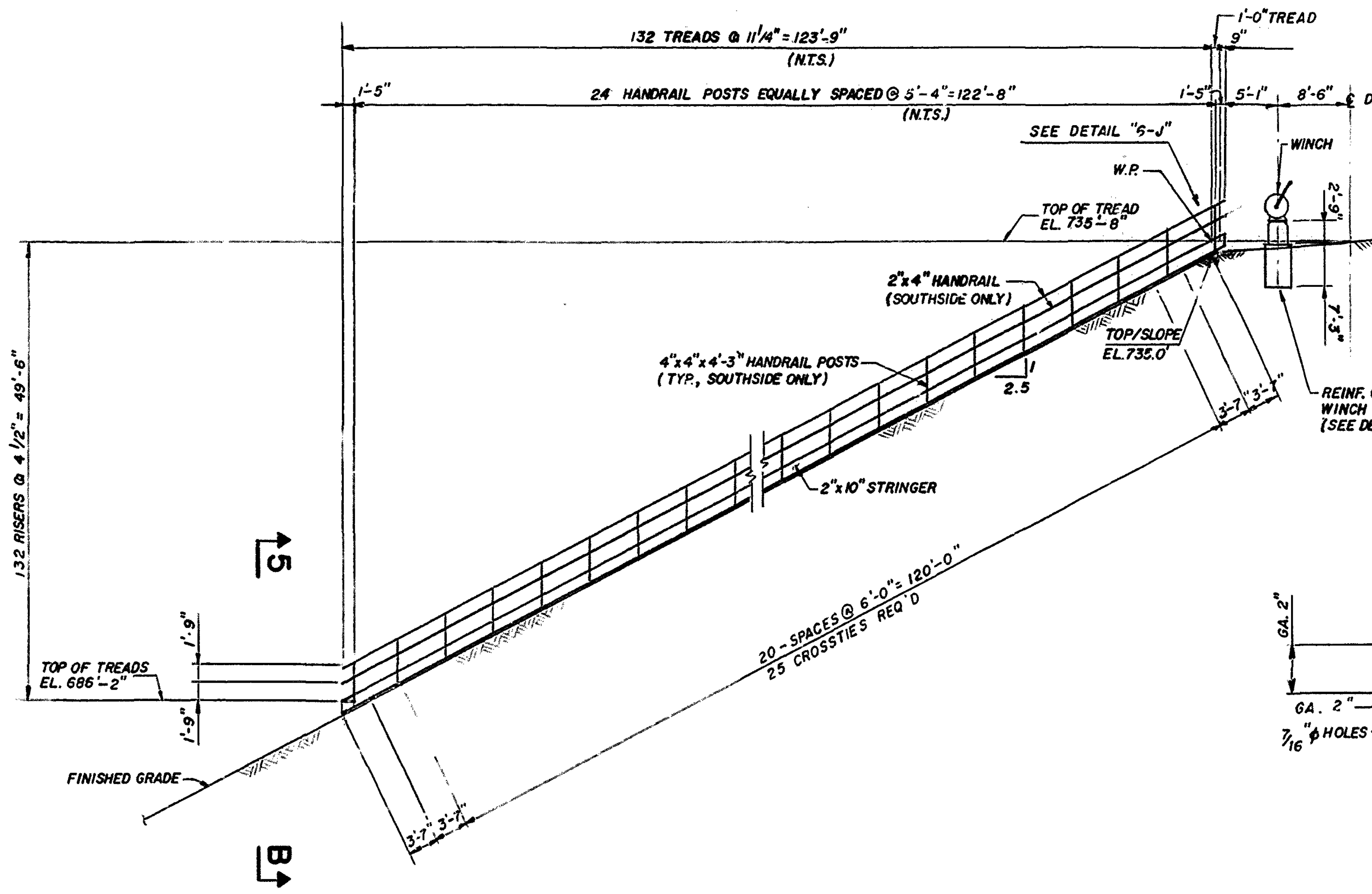
DR. No. 12-3000U



LOCATION PLAN

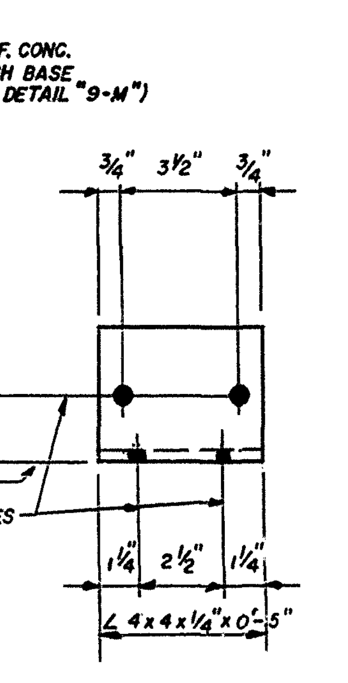
SCALE 1" = 100'-0"

(REFER TO DWG. NO. 12-3000K)



SECTION 3-H

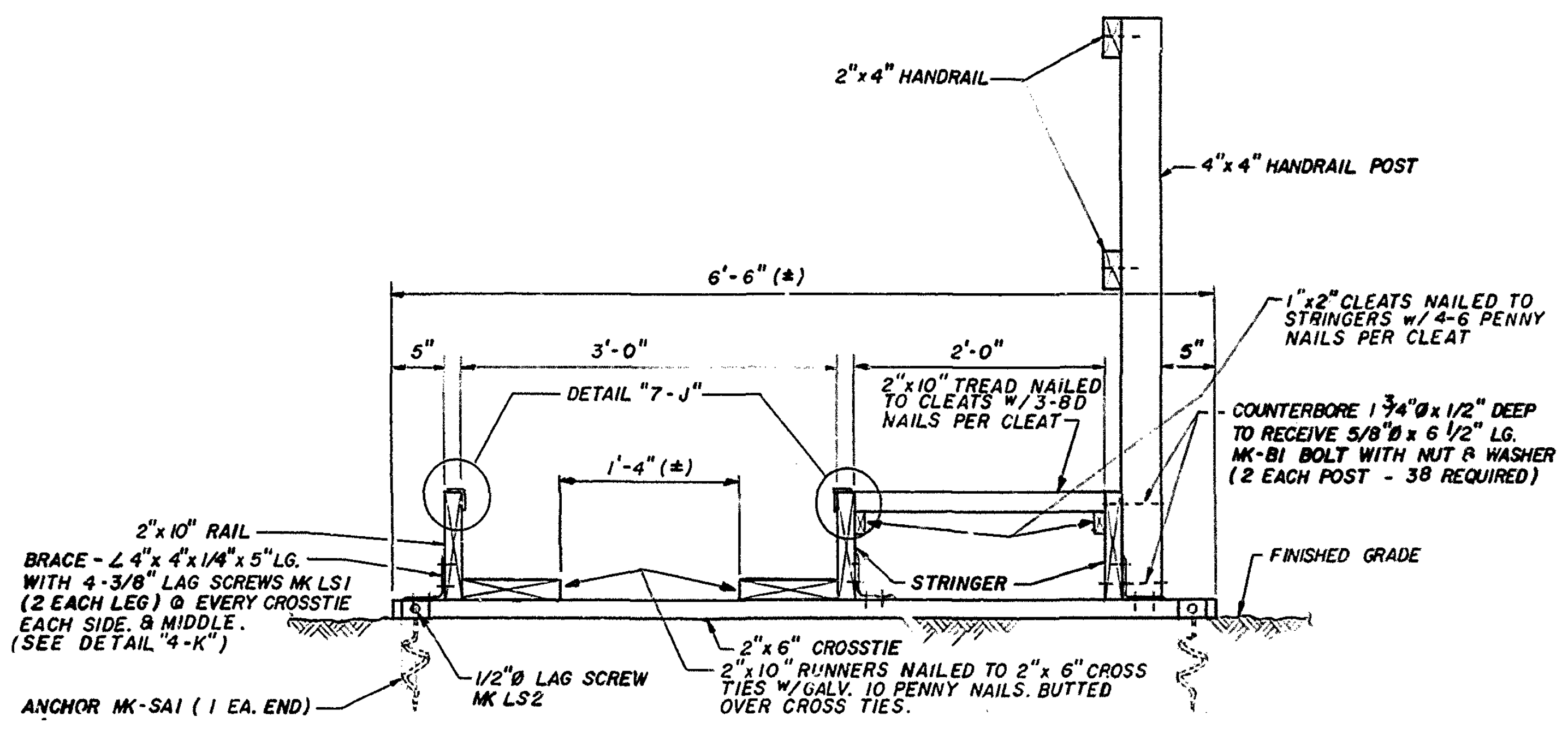
SCALE 3/32" = 1'-0"



DETAIL 4-K

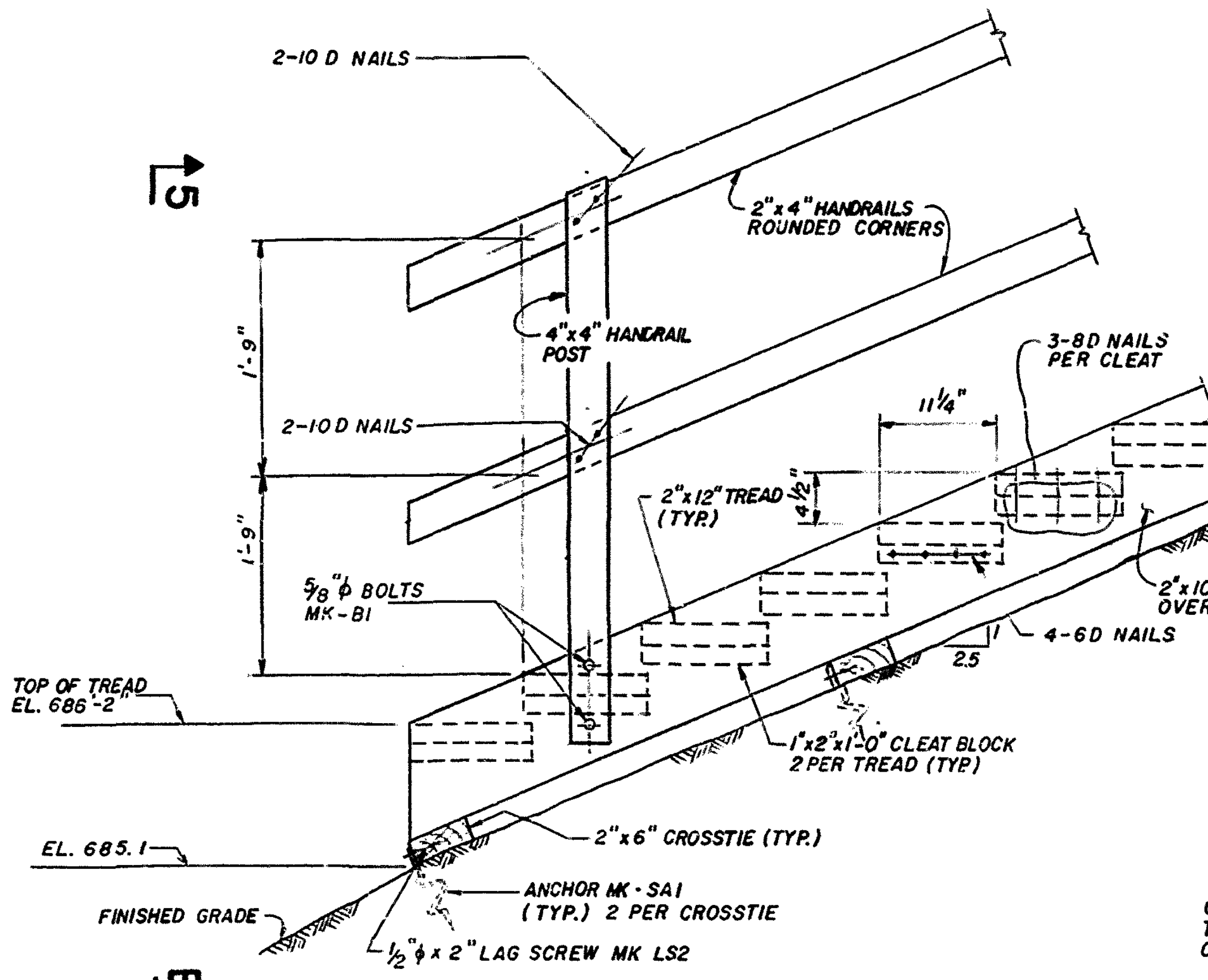
STRINGER BRACE DETAIL

SCALE 3"=1'-0"



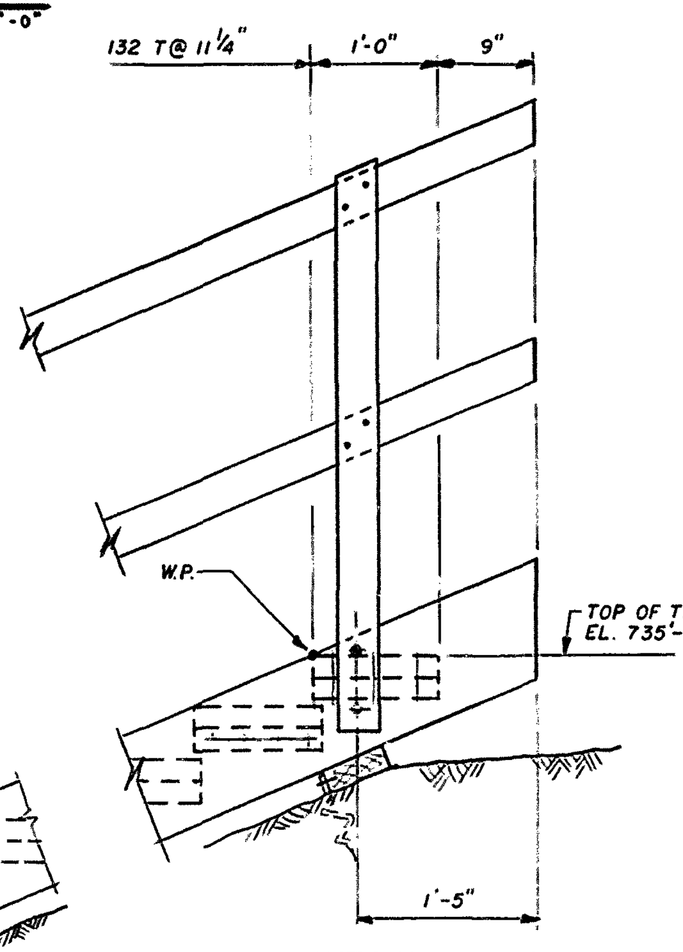
SECTION 5-B

SCALE 1" = 1'-0"



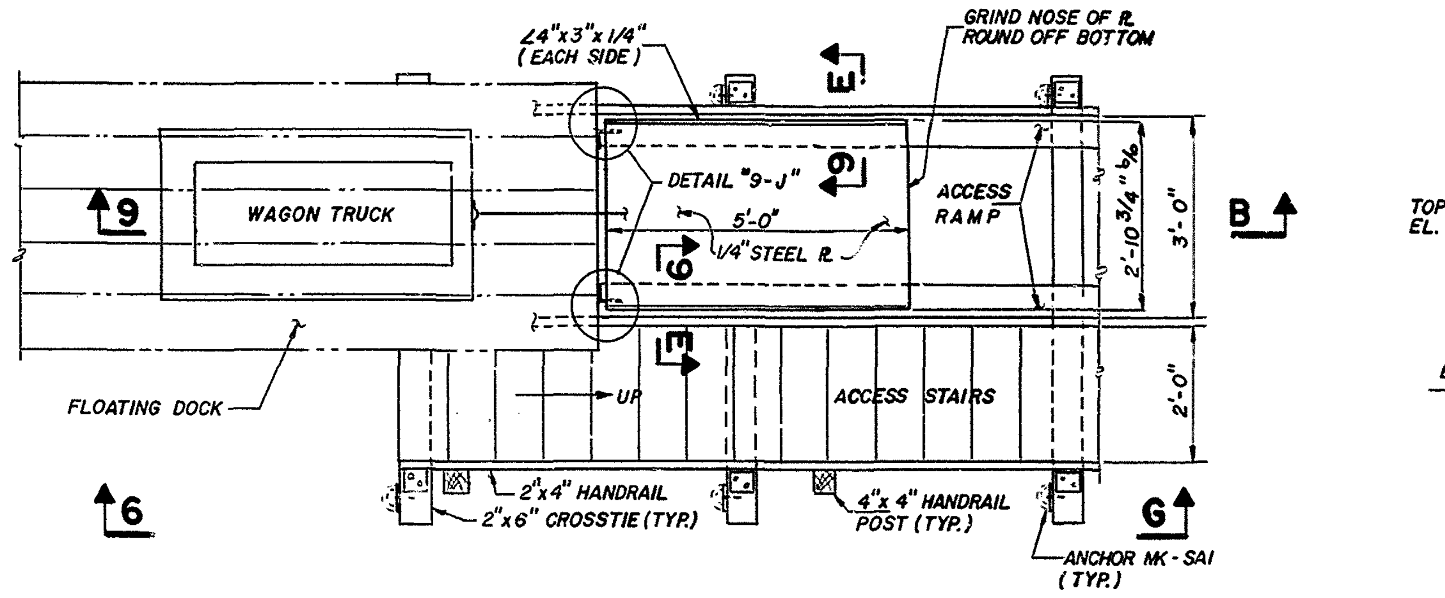
ELEVATION 6-G

SCALE 1" = 1'-0"



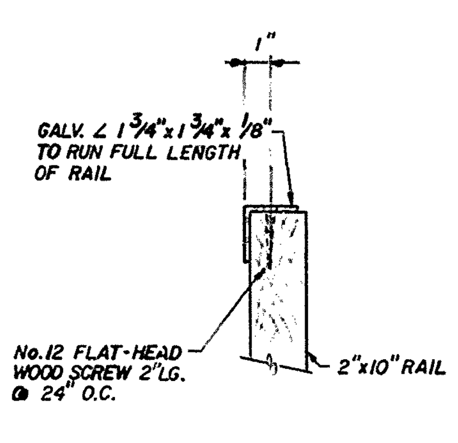
DETAIL 6-J

SCALE 3"=1'-0"



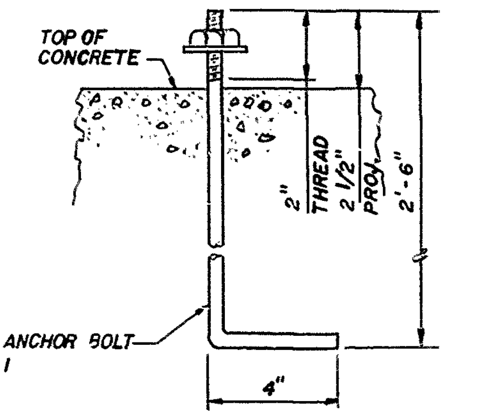
PLAN AT FOOT OF ACCESS RAMP & STAIRWAY

SCALE 1/2" = 1'-0"



DETAIL 7-J

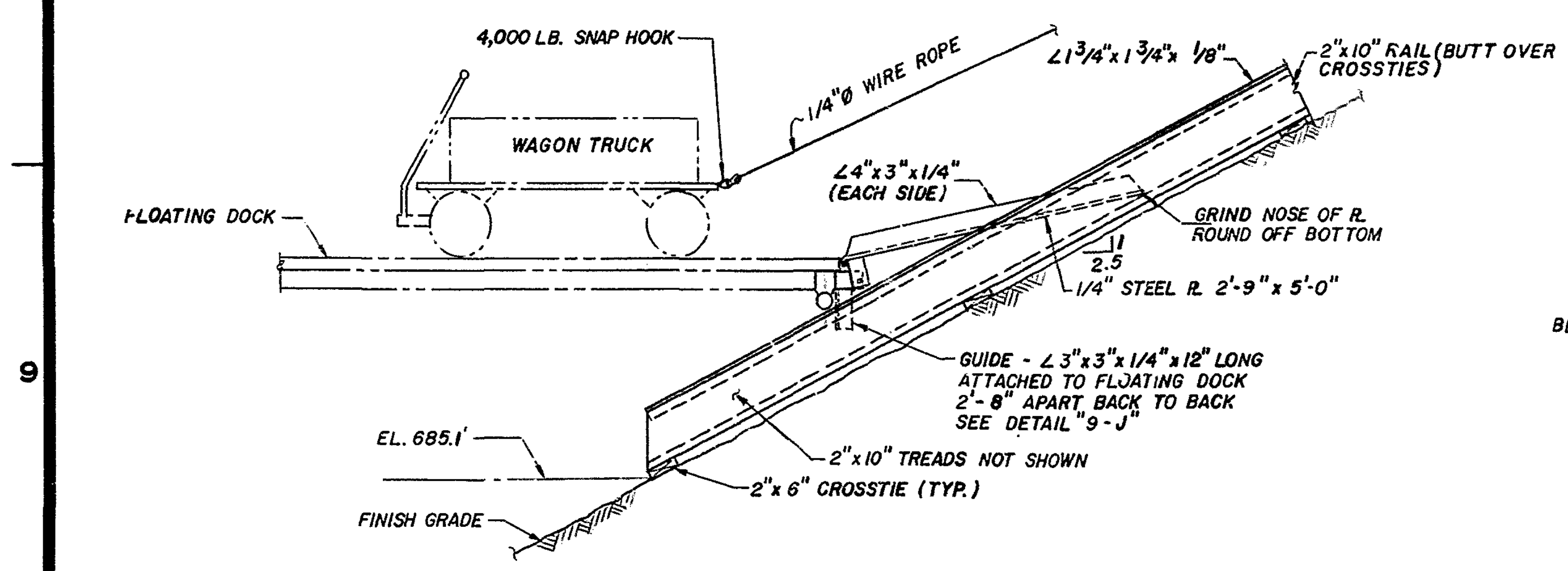
SCALE 3" = 1'-0"



DETAIL 7-K

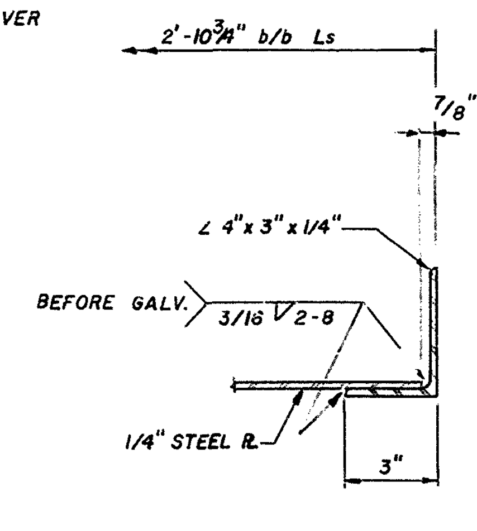
ANCHOR BOLT DETAIL

SCALE 3" = 1'-0" 4 REQUIRED



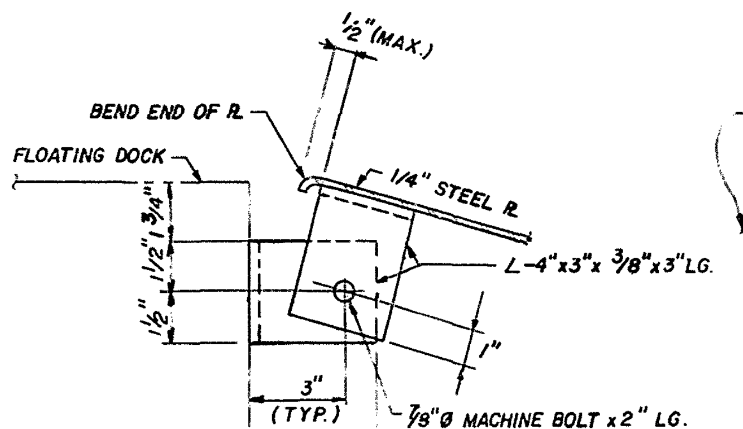
SECTION 9-B

SCALE 1/2" = 1'-0"



SECTION 9-E

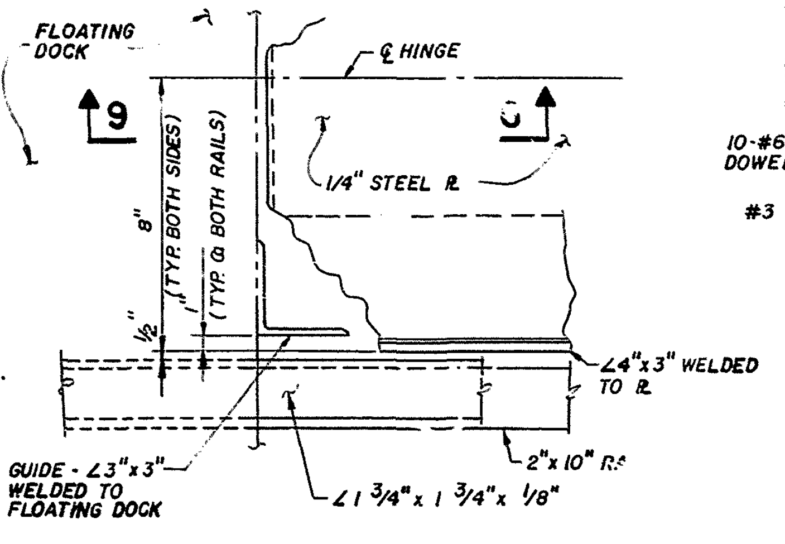
SCALE 3" = 1'-0"



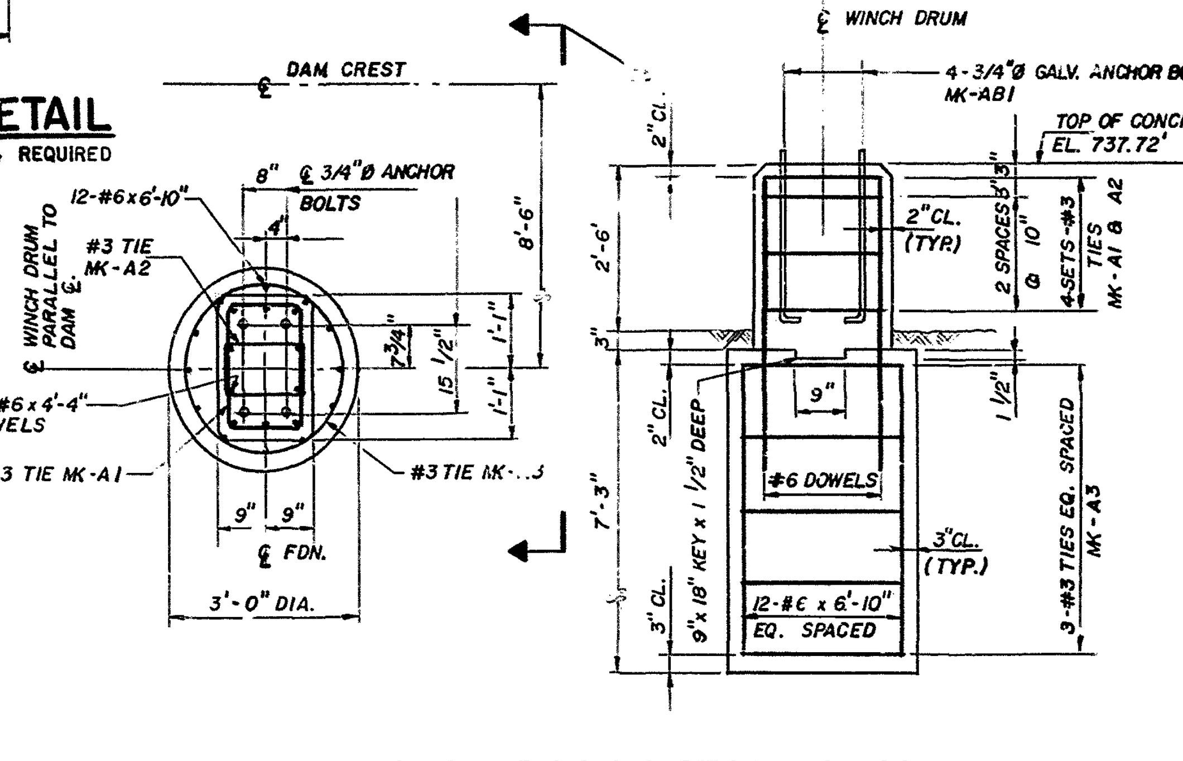
SECTION 9-G

SCALE 3" = 1'-0"

HINGED DETAIL 2 REQUIRED



DETAIL 9-J



WINCH BASE DETAIL 9-M

SCALE 1/2" = 1'-0"

NAIL SCHEDULE

QTY.	SIZE	WT.
330	10 D	5 LB.
798	8 D	8 LB.
2128	6 D	12 LB.

TIMBER SCHEDULE

QTY.	SIZE	LENGTH	TOTAL LENGTH	BD. FT.	REMARKS
2	2"x10"	135'-6"	271'-0"	451.67	STRINGERS
1	2"x10"	135'-6"	135'-6"	225.83	RAILS
133	2"x12"	2'-0"	266'-0"	532.00	TREADS (CUT TO LENGTH)
2	2"x10"	135'-0"	270'-0"	450.00	RUNNERS
25	2"x 6"	6'-8"	166'-8"	166.67	CROSSTIES
24	4"x 4"	4'-3"	102'-0"	136.00	HANDRAIL POSTS
2	2"x 4"	134'-4"	268'-8"	179.11	HANDRAILS
266	1"x 2"	1'-0"	266'-0"	44.33	CLEATS (CUT TO LENGTH)

TOTAL BOARD FEET = 2185.61

BY:

BOLT SCHEDULE

MARK	QTY.	SIZE	LENGTH	TYPE	REMARKS
B-1	50	5/8"Ø	0'-6 1/2"	SQUARE HEAD SQUARE NUT	2 WASHERS 3" THREAD
B-2	2	7/8"Ø	0'-2"	SQUARE HEAD SQUARE NUT	1 LOCKWASHER 1 1/4" THREAD
AB-1	4	3/4"Ø	2'-10"	ANCHOR BOLT WITH HEX NUT & WASHER	SEE DETAIL (7-K)
SA-1	50	1/2"Ø	3'-6"	SCREW-IN ANCHOR w/ 2"x1/4" SQ. WASHERS	
LS-1	315	3/8"Ø	0'-1 1/2"	LAG SCREW	
LS-2	53	1/2"Ø	0'-2"	LAG SCREW	
	145	12	0'-2"	FLAT HEAD WOOD SCREWS	

BY:

STEEL SCHEDULE : A-36 & GALVANIZED

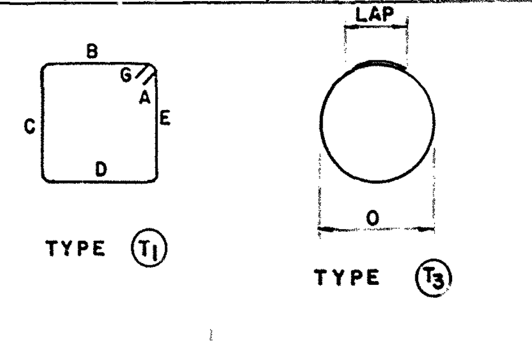
QTY.	SIZE	DESCRIPTION	
2	4"x 3"x 1/4"x 5'-0"	STEEL ANGLES	60 LB.
2	3"x 3"x 1/4"x 1'-0"	STEEL ANGLES	10 LB.
4	4"x 3"x 3/8"x 0'-3"	STEEL ANGLES WITH 15/16"Ø HOLE @ LONGER LEG.	9 LB.
75	4"x 4"x 1/4"x 0'-5"	STEEL ANGLES WITH 2 - 7/16"Ø HOLES IN EACH LEG.	213 LB.
	1 3/4"x 1 3/4"x 1/8"x 268'-0"	STEEL ANGLES CUT TO LENGTH @ RAILS.	400 LB.
1	2'-9"x 5'-0"x 1/4"	STEEL PLATE.	145 LB.

TOTAL 837 LB.

REINF. STEEL SCHEDULE : A-615 (S-1), GRADE 60

MARK	No.	SIZE	LENGTH	TYPE	A	B	C	D	E	G	O
A-1	4	#3	6'-8"	T1	0'-4"	1'-10"	1'-2"	1'-10"	1'-2"	0'-4"	
A-2	4	#3	4'-6"	T1	0'-4"	0'-9"	1'-2"	0'-9"	1'-2"	0'-4"	
A-3	8	#3	9'-0"	T3	L	A P	=	1'-	0"		2'-6"
	10	#6	6'-2"	STR.							
	12	#6	6'-10"	STR.							

TOTAL WEIGHT = 260 LBS.



GENERAL NOTES

1. ALL CONCRETE MATERIALS AND WORKMANSHIP SHALL CONFORM TO A.E.P.S.C. CORP. SPECIFICATIONS # 1000A
2. DIMENSIONS GIVEN FOR REINFORCING STEEL ARE TO CENTER LINE OF BARS.
3. 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.
4. EXPANSION JOINTS MUST BE LOCATED AS SHOWN.
5. ALL EXPOSED EDGES SHALL HAVE A ONE INCH BEVEL.
6. FLOOR FINISH SHALL CONFORM TO THE A.E.P.'s CORP'S SPEC. AND TYPE OF SAME WILL BE DECIDED UPON BY CONCRETE LABORATORY.
7. ALL EXPOSED VERTICAL EXTERIOR CONCRETE SURFACES TO HAVE RUBBED FINISH.
8. ALL REINFORCING BARS SHALL CONFORM TO A-615, (S-1), GRADE 60, DEFORMED TYPE.
9. CONCRETE : f'c = 3,500 p.s.i. @ 28 DAYS.
10. ALL TIMBER SHALL BE DOUGLAS FIR OR SOUTHERN YELLOW PINE, SELECT STRUCTURAL SIZES & TO BE IN ACCORDANCE WITH AMERICAN LUMBER STANDARDS.
11. EXCEPT FOR HANDRAILS, ALL TIMBER SHALL BE CREOSOTED & SHALL CONFORM TO SPEC. A.W.P.A., C-2 LATEST EDITION. HANDRAILS SHALL BE PRESSURE-TREATED LUMBER.
12. ALL STEEL ANGLES, PLATE, BOLTS, WASHERS, NUTS, SCREW-IN ANCHORS, LAG SCREWS, WOOD SCREWS AND NAILS SHALL BE GALVANIZED. ALL EXPOSED AREAS TO RECEIVE ONE FIELD COAT OF BITUMASTIC PAINT.
13. ALL HOLES FOR MACHINE BOLTS TO BE 1/16" OVER BOLT DIAMETER.

MATERIALS

CONCRETE : - 2.23 CU. YD.

WINCH : "THERN" - MODEL No. 492 OR EQUAL - 1 REQ'D.

WAGON TRUCK : "JAKES" 5149 SERIES MODEL No. 5149-P12-2 OR EQUAL - 1 REQ'D.

1/4"Ø WIRE ROPE : - 150.0 LIN. FT.

MISC. NAILS & WOOD SCREWS : (GALVANIZED) 25 LB.

ALL MATERIALS BY CONTRACTOR.

REFERENCE DRAWINGS

12-3000K - LAYOUT & GRADING PLAN.

REVISIONS

DATE	NO.	DESCRIPTION	APPD.
4/10/89	1	AS PER RDR No. 56-043, BELL: "AS-BUILT" STATUS.	
9/18/87	0	RELEASED FOR CONSTRUCTION	

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OHIO POWER COMPANY

GAVIN PLANT

CHESHIRE OHIO

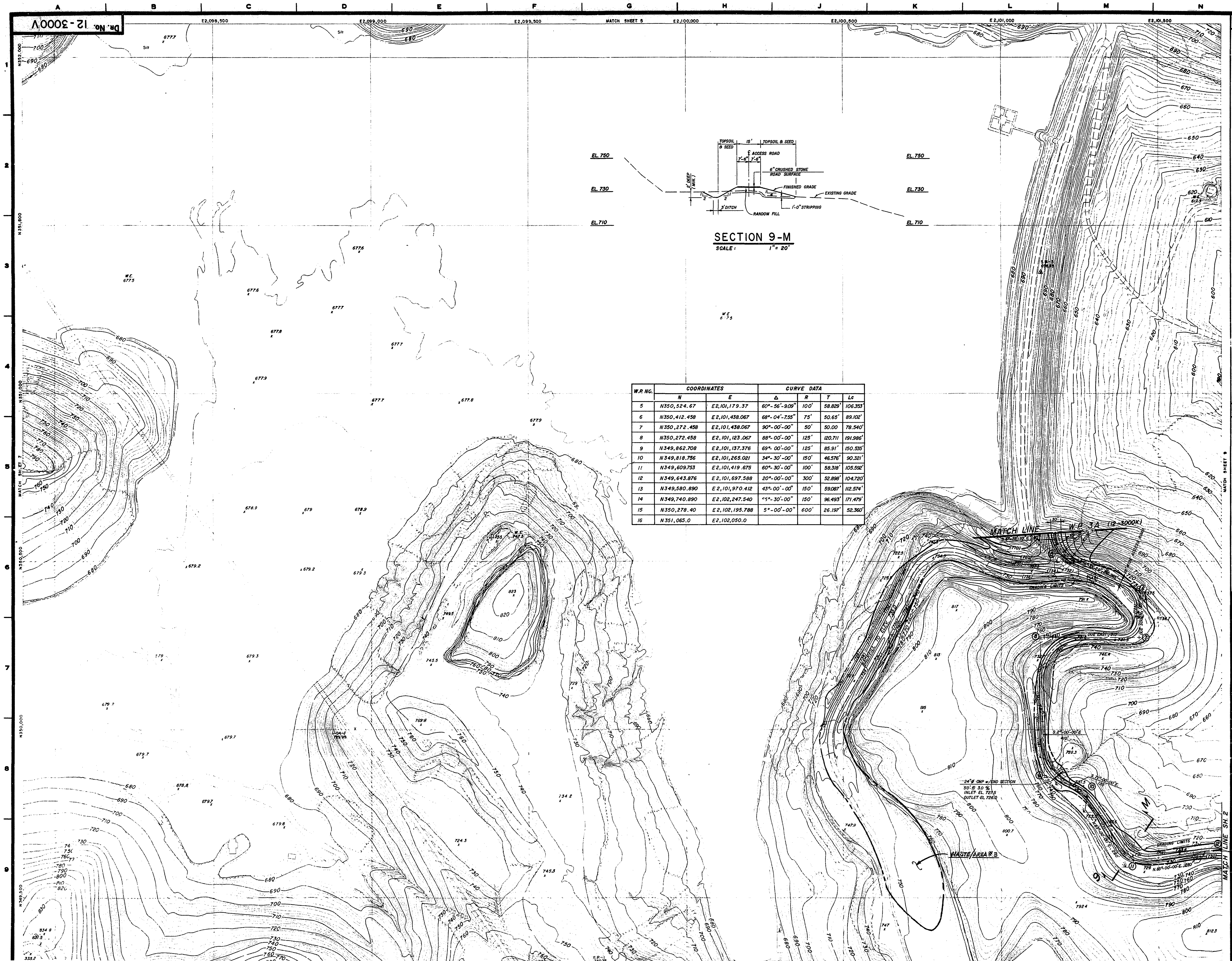
STINGY RUN FLY ASH DAM
STAGE II RAISING
ACCESS STAIRWAY TO SERVICE
SPILLWAY STRUCTURE
PLAN, SECTIONS & DETAILS

DR. NO. 12-3000U-1

ARCH.	—	ELEC	—	MECH.	—	STR.	95

SCALE: AS SHOWN
DR. A.C.Y.
CH. R.D./S.T.
SQ. LDR G.S.
DATE: 12-1-86

APPROVED BY
ENGINEERING DIV
B.H. Barrett
DESIGN DIV



W.P. NO.	COORDINATES		CURVE DATA			
	N	E	Δ	R	T	Lc
5	N350,524.67	E2,101,179.37	60°-56'-9.09"	100'	58.829'	106.353'
6	N350,412.458	E2,101,438.067	68°-04'-7.55"	75'	50.65'	89.102'
7	N350,272.458	E2,101,438.067	90°-00'-00"	50'	50.00	78.540'
8	N350,272.458	E2,101,123.067	88°-00'-00"	125'	120.711	191.986'
9	N349,862.708	E2,101,137.376	69°-00'-00"	125'	85.91'	150.535'
10	N349,818.756	E2,101,265.021	34°-30'-00"	150'	46.576'	90.321'
11	N349,609.753	E2,101,419.675	60°-30'-00"	100'	58.318'	105.592'
12	N349,643.876	E2,101,697.588	20°-00'-00"	300'	52.898'	104.720'
13	N349,580.890	E2,101,970.412	43°-00'-00"	150'	59.087'	112.574'
14	N349,740.890	E2,102,247.540	65°-30'-00"	150'	96.493'	171.479'
15	N350,278.40	E2,102,195.788	5°-00'-00"	600'	26.197'	52.360'
16	N351,065.0	E2,102,050.0				

GENERAL NOTES

REFERENCE DRAWINGS

12-3000V - LAYOUT & GRADING PLAN
12-3000W - ACCESS ROAD SHEET 2

DATE	No.	DESCRIPTION	APPO.
4/10/89	1	AS PER RDR NO. 56-043, REV. 1: REVISED PLAN — "AS-BUILT" STATUS. KCM	
5/4/87	0	RELEASED FOR CONSTRUCTION	

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OHIO POWER COMPANY
GAVIN PLANT
CHESHIRE OHIO

STINGY RUN FLY ASH DAM
STAGE II RAISING
ACCESS ROAD
SHEET 1

DR. No. 12-3000V-1

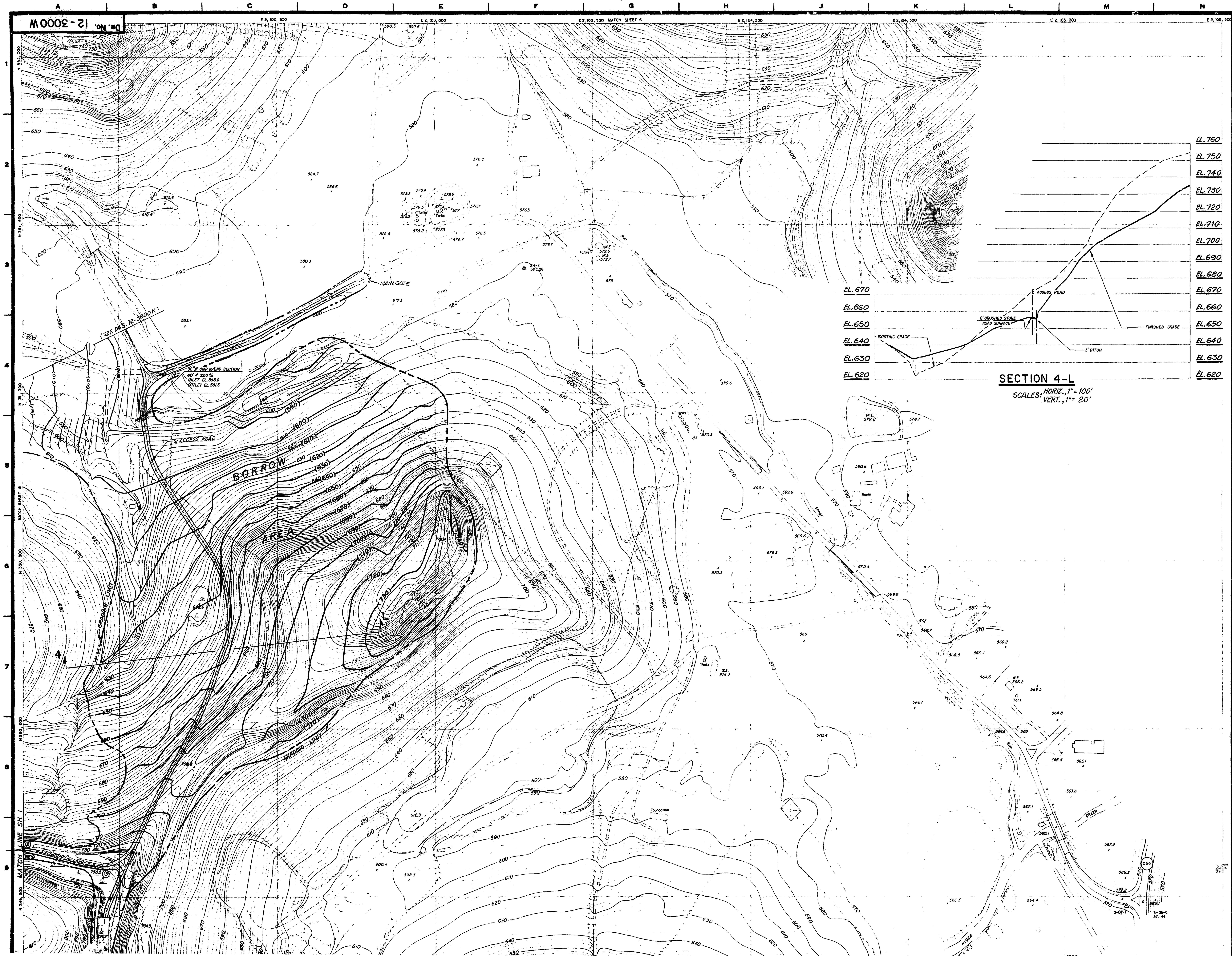
ARCH. – ELEC. – MECH. – STR. GS

SCALE: 1" = 100'
DR. M. HERSHEY
CH. KCM
SQ. LDR. RNS
DATE: 12-1-86

APPROVED BY
ENGINEERING DIV.
B.H. Barrett
DESIGN DIV.
VA Lyne

AMERICAN ELECTRIC POWER SERVICE CORP.

DR. No. 12-3000V



SECTION 4-L

SCALES: HORIZ., 1" = 100'
VERT., 1" = 20'

GENERAL NOTES

REFERENCE DRAWINGS

12-3000K - LAYOUT & GRADING PLAN
12-3000S - BORROW SITE GRADING PLAN
12-3000V - ACCESS ROAD SHEET 1

DATE	No.	DESCRIPTION	APPD.
4/10/89	1	REVISED PLAN & SECTION - "AS-BUILT" STATUS. KCM SRM	
5/4/87	0	RELEASED FOR CONSTRUCTION	

REVISIONS

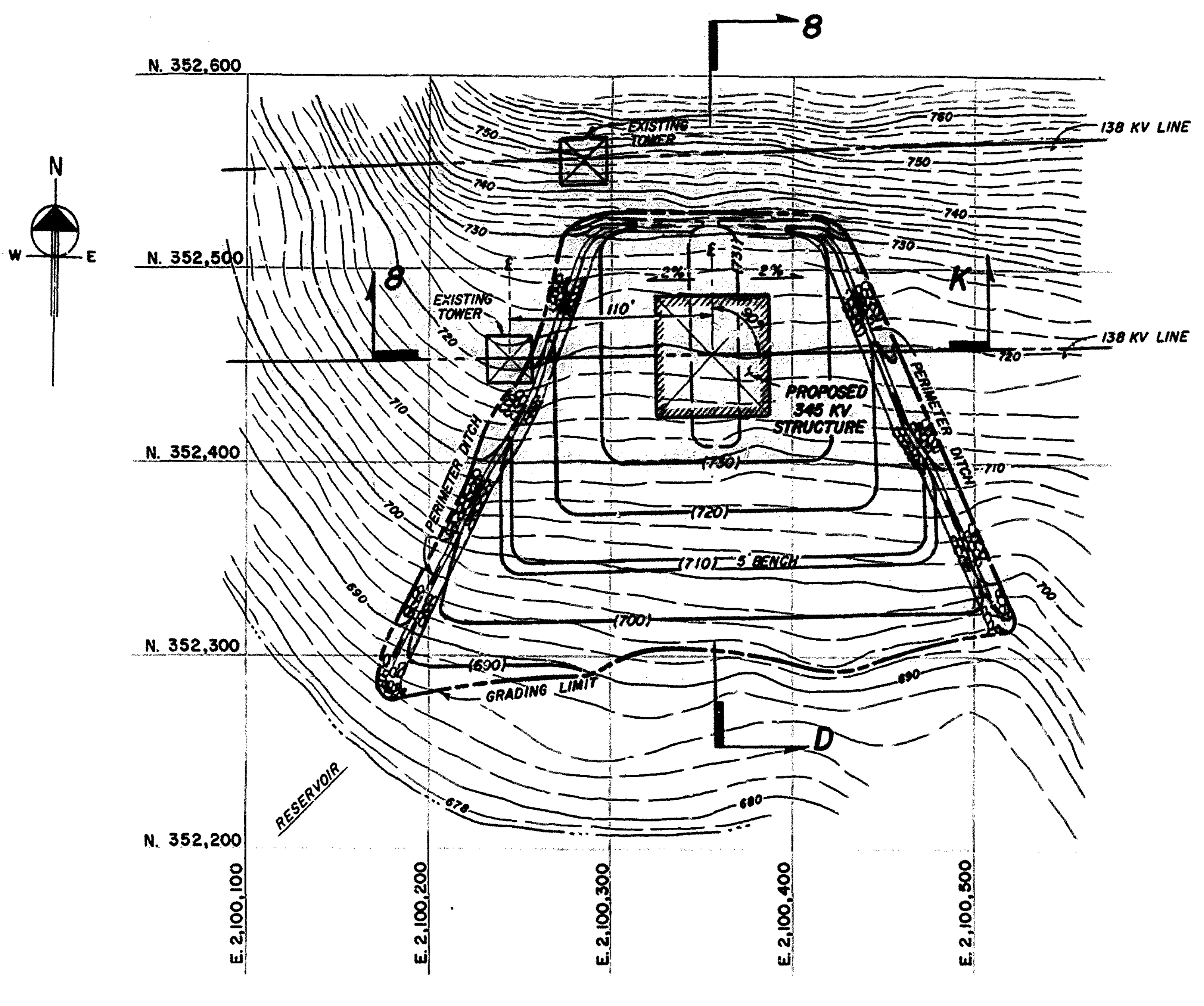
OHIO POWER COMPANY
GAVIN PLANT
CHESHIRE OHIO

STINGY RUN FLY ASH DAM
STAGE II RAISING
ACCESS ROAD
SHEET 2

DR. No. 12 - 3000W - 1

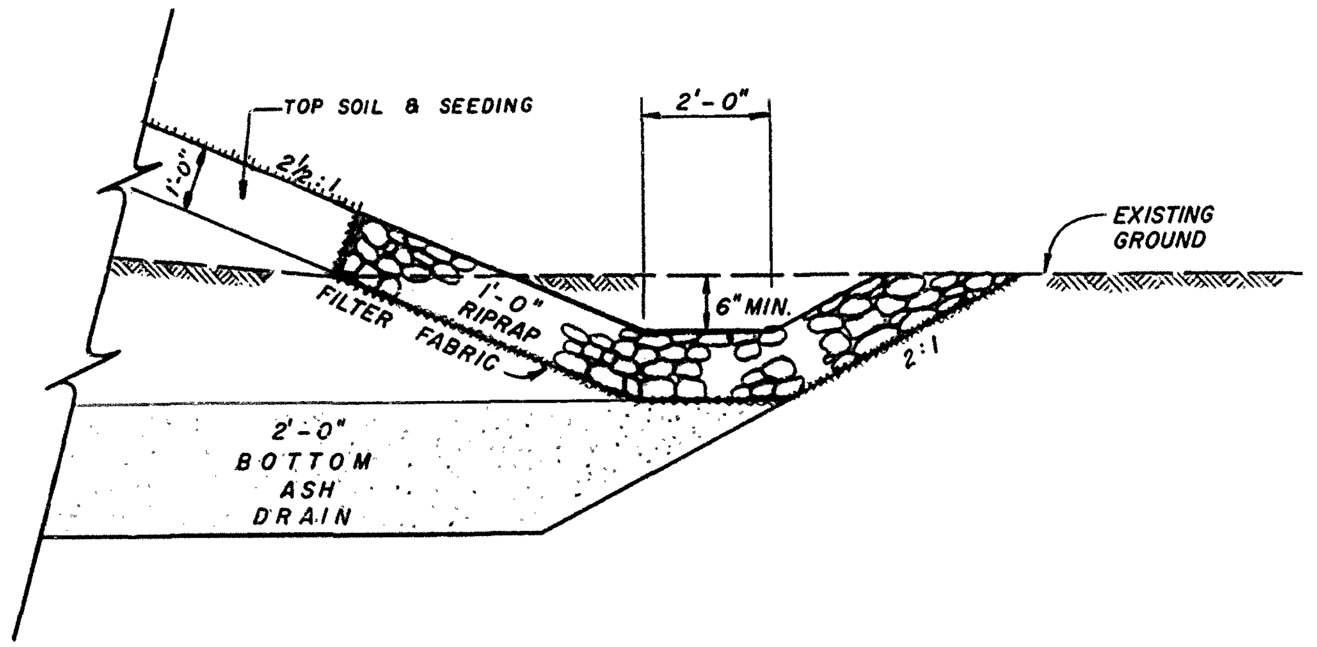
ARCH. - ELEC. - MECH. - STR. GS
SCALE: 1" = 100'
DR. M. HERSHEY
CH. KCM
SQ. LDR. RNS
DATE: 12-1-86
APPROVED BY
ENGINEERING DIV. B.H. Barnett
DESIGN DIV.

DWG. NO. 12 - 3000X



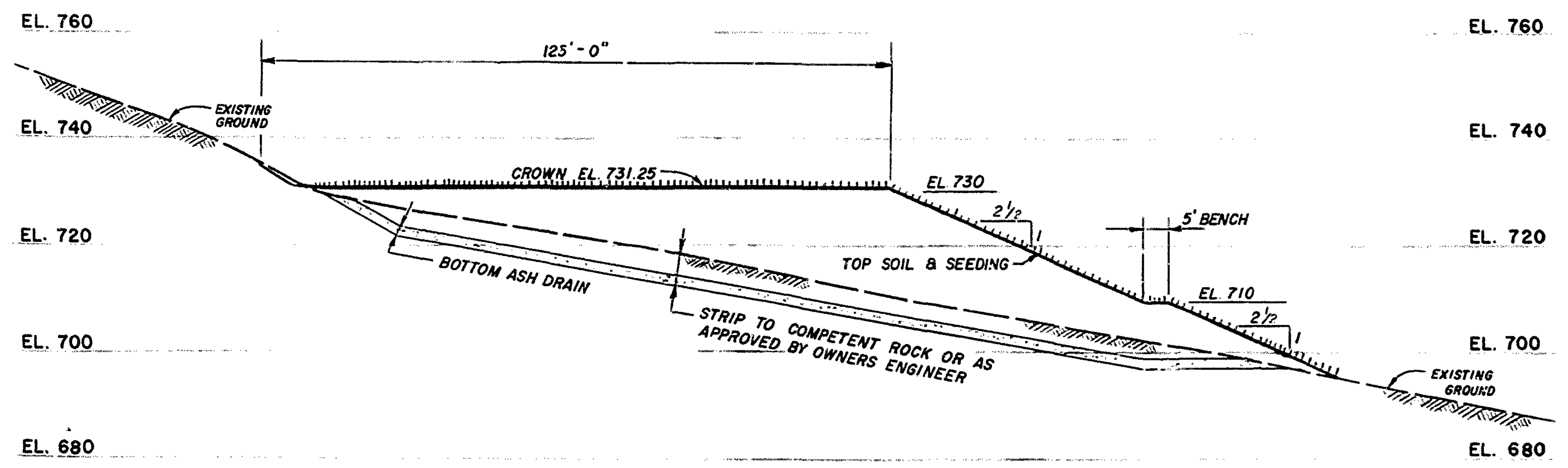
LOCATION PLAN

SCALE: 1" = 50'



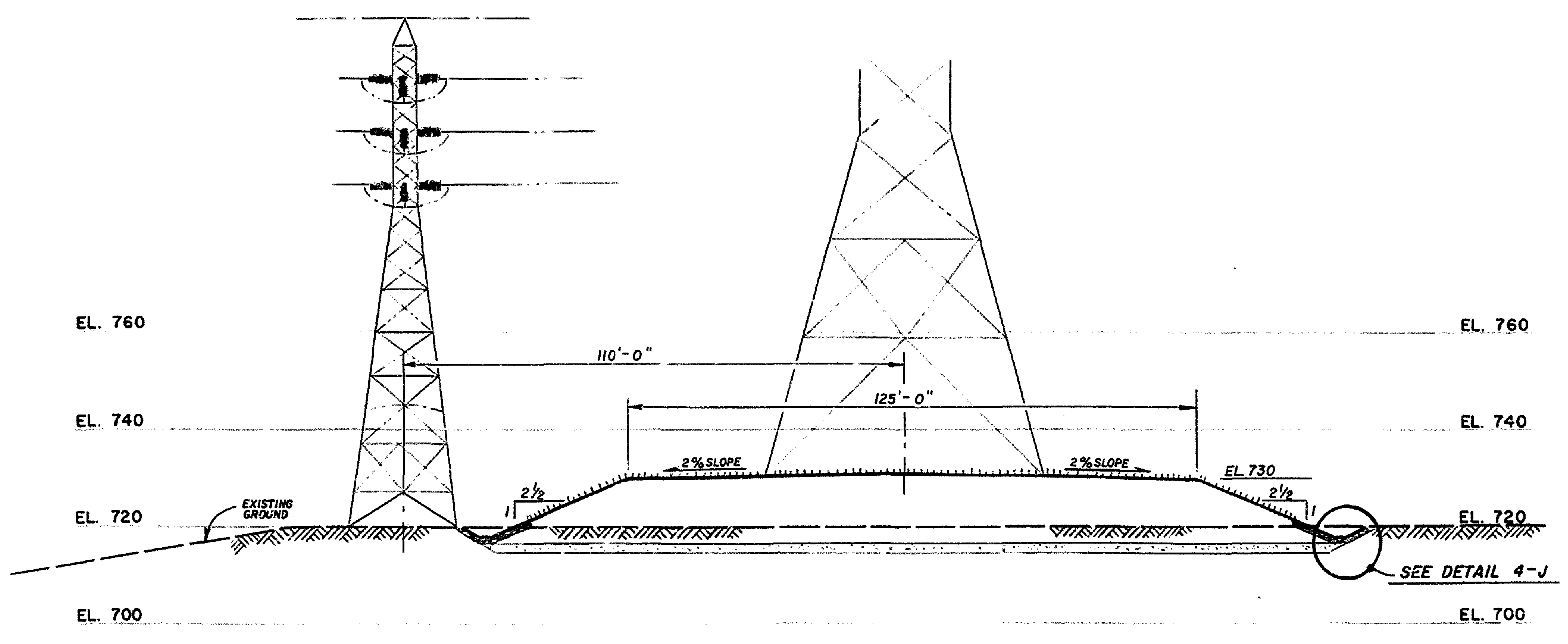
DETAIL 4 - J

SCALE: 1" = 2'



SECTION 8 - D

SCALE: 1" = 20'



SECTION 8 - K

SCALE: 1" = 20'

GENERAL NOTES

1. DO NOT SCALE THIS DRAWING.
2. TOPO ENLARGEMENT DERIVED FROM DWG. 12-3000K.
3. EMBANKMENT SHALL BE WELL COMPACTED AND BE IN ACCORDANCE WITH A.E.P. CIVIL ENGINEERING SPECS.

EARTHWORKS

EXCAVATION :

BOTT. ASH DRAIN AND STRIPPING	9800	CU. YD.
FILL : BOTT. ASH	3920	CU. YD.
EMBANKMENT	21,700	CU. YD.
RIPRAP	250	CU. YD.
SURFACE AREA :	52,925	SQ. FT.
	1.22	ACRES

MATERIALS

FILTER FABRIC :	7,400	SQ. FT.
SEEDING AND MULCHING :	46,255	SQ. FT.

REFERENCE DRAWINGS

12-3000A - GENERAL ARRANGEMENT
12-3000K - LAYOUT & GRADING PLAN

DATE	NO.	DESCRIPTION	APPD.
4/10/89	1	AS PER RDR NO. 56-043, REV. 1: DRAWING REVIEWED ~ "AS-BUILT" STATUS. KCM	Wm
6/1/88	0	RELEASED FOR CONSTRUCTION	

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OHIO POWER COMPANY
GAVIN PLANT
CHESHIRE OHIO

STINGY RUN FLY ASH DAM
STAGE II RAISING

SPORN-SOUTH POINT
TOWER FILL

DWG. NO. 12 - 3000X - 1

ARCH.	-	ELEC.	-	MECH.	-	STR.	95

SCALE: SHOWN
DR: M. HURSEY
CH: KCM
SQ. LDR: LMS
DATE: 5/19/88

APPROVED BY
ENGINEERING DIVISION: BH Bennett PHA
DESIGN DIVISION: VA Lepore

[illegible]

FINAL REPORT

GAVIN FLY ASH DAM

AND

FLY ASH LINE SUPPORT SYSTEM

VOLUME I

[illegible]

FEBRUARY 1975

OHIO ELECTRIC COMPANY

GAVIN POWER PROJECT

GAVIN FLY ASH DAM
AND
FLY ASH LINE SUPPORT SYSTEM
FINAL REPORT

VOLUME I

Prepared by

HARZA ENGINEERING COMPANY

January 1975

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Chapter V - EXHIBITS

(NOTE: EXHIBITS 'E-K' ARE NOT INCLUDED)

REPORT

Chapter I

INTRODUCTION

Scope

Harza Engineering Company's responsibility for the Gavin Power Project and related works included the design of the civil features for two overland coal conveyors and the civil, mechanical and electrical features for a water supply dam, pumping plant and a five-mile pipeline, for a fly ash retention dam and related facilities. The Gavin Power Project is a development of Ohio Electric Company and Southern Ohio Coal Company, which are subsidiaries of American Electric Power Company.

Harza certified to the Division of Water on November 5, 1974 that the dam was built according to plans and specifications, and other than for slow growth of some of the vegetal cover and some resultant gullying, the dam was approved by the State. This, therefore, is Harza's Final Report for fly ash retention dam, the fly ash support systems, and those parts of the fly ash slurry pipes from the generating plant to the reservoir with which Harza was involved.

The dam is named the Gavin Fly Ash Dam and is the property of the Ohio Electric Company. The purpose of the dam is to retain fly ash produced by the burning of pulverized coal at the Gavin Power Plant. The retention dam originally was named "Stingy Run Fly Ash Dam" and some documents were issued bearing that name, but the name was later changed while design was underway.

Previously, Harza supplied reports covering the design of the Gavin Fly Ash Dam in its present form, as follows:

July 24, 1972	-	Coal Handling and Fly Ash Auxiliaries (Summary of New York meeting)
October 10, 1972	-	Relationship of Reservoir Lands to AEP Property
November 10, 1972	-	Cost Estimate

November 29, 1972 - Detailed Cost Estimate (presented at a meeting in New York)
January 1973 - Preliminary Report
February 1973 - Preliminary Design Report
April 1973 - Final Design Report
November 1973 - Supplement to Final Design Report
December 1974 - Operation and Maintenance Manual

This Final Report supplements the above reports, which provide reference material relative to detailed design. This report provides a record of (1) the design and construction of the Gavin Fly Ash Dam, (2) the design of the Fly Ash Line Support System, and (3) the design of the fly ash slurry lines. No previous report has been written by Harza on the design of the Fly Ash Line Support System or the fly ash slurry lines. There were additional reports that were superseded by later decisions by the Owner as described below.

Brief History of Project

The following are significant activities and dates in the design and construction of the Gavin Fly Ash Dam, portions of the Fly Ash Support System and slurry lines with which Harza was involved:

Harza proposal to AEP for design services and resident engineer services during construction	May 19,1972
Ohio Electric Contract 3040 let to Harza for design services of Fly Ash Dam only	May 26, 1972
Harza report submitted to the Owner defining most economical location of Fly Ash Dam	July 19, 1972
Harza requested to design fly ash support systems	July 19, 1972
Owner selection of fly ash damsite	August 3, 1972
Owner revised selection of damsite because of property acquisition problems	September 20, 1972
Contract Documents submitted to Owner for Kyger Creek Ash Line Support System	December 18, 1972

Preliminary Report for Fly Ash Dam submitted by Owner to Ohio Division of Water	February 9, 1973
Amendment #1 to Contract 3040 for Harza to design crossing of Kyger Creek for fly ash slurry pipelines	February 16, 1973
Harza first revised proposal for Resident Engineer services during construction of dam	February 23, 1973
Construction drawings and specifications of dam ready for issuance of requests to bid	March 16, 1973
Requests issued for bidding	March 30, 1973
Final Report prepared for submittal to Ohio Department of Natural Resources, Division of Water	April 3, 1973
Application for Construction Permit with Final Report submitted by Owner to Ohio Division of Water	April 12, 1973
Construction bids received	April 24, 1973
Contract between Harza and General Analytics, Inc. (GAI) for soil testing services during construction	April 27, 1973
Harza requested to design fly ash slurry lines from Gavin Plant to their discharge point into the fly ash retention reservoir	April 30, 1973
Contract between Harza and S. D. Pomeroy and Associates (Pomeroy) for surveying services	May 2, 1973
Construction contract for dam let to Paul C. Coffey Construction Company, Raceland, Kentucky	May 11, 1973
Contract between Harza and Union Boiler Company (Union Boiler) for inspectors	May 25, 1973
Construction permit for dam received from Ohio Division of Water	June 5, 1973
Harza second revised proposal for Resident Engineer services	June 5, 1973

Construction of dam begun	June 18, 1973
Core trench excavation completed	July 12, 1973
Amendment #2 to Contract 3040 for Harza to design and prepare drawings for the fly ash slurry lines from Gavin Plant into the fly ash retention reservoir	August 20, 1973
Harza Resident Engineer's office established at Cheshire, Ohio	September 10, 1973
Foundation grouting completed	October 11, 1973
Supplement to Final Design Report for raising of Fly Ash Dam submitted to Owner	November 16, 1973
Supplement to Final Design Report submitted by Owner to Ohio Division of Water	November 19, 1973
Fly ash dam embankment completed	November 21, 1973
Approval for revised dam given by Ohio Division of Water	December 19, 1973
Ohio Electric Contract 3294 dated as of June 5, 1973 issued to Harza for Resident Engineer services incorporating Harza proposal of June 5, 1973	December 30, 1973
Principal Spillway completed	January 30, 1974
Harza requested to write an "Operation and Maintenance Manual" for the fly ash retention dam and outlet works	March 22, 1974
Coffey's construction plant moved out	April 10, 1974
Riprap placement completed	May 10, 1974
Seeding and mulching completed	May 17, 1974

Engineering Contracts

A summary of the engineering contracts involved in the above history is listed below.

Contract	Between	Date
#3040 - for Engineering Design of the Gavin Fly Ash Dam at Stingy Run	Ohio Electric Co. and Harza	May 26, 1972
Amendment #1	Ohio Electric Co. and Harza	February 16, 1973
Amendment #2	Ohio Electric Co. and Harza	August 20, 1973
#3294 - for Field Engineering During Construction	Ohio Electric Co. and Harza	June 5, 1973
Subcontract #1	Harza and GAI	April 27, 1973
Subcontract #2	Harza and Pomeroy	May 2, 1973
Subcontract #3	Harza and Union Boiler	May 25, 1973

The drilling and boring, seismic survey, and soil testing for the design portion of the fly ash dam were performed at Harza direction under direct contracts between the Owner and the supplier. They are, respectively:

American Drilling and Boring Co., Inc., East Providence, R.I. Weston Geophysical Engineers, Inc., Weston, Mass. General Analytics, Inc., Monroeville, Pa.

Brief Description of Project

The fly ash is transported from the Gavin Power Plant in the form of a fly ash-water slurry through two steel pipelines which run along side of the Gavin Overland Coal Conveyor. Two additional lines have been provided as backup. These lines discharge into the fly ash reservoir as shown on Exhibit A, Feature Location Map. The Gavin Fly Ash Dam, which creates the reservoir, comprises three primary structures. They are: (1) the Dam, (2) the Principal Spillway, and (3) the Emergency Spillway.

The Dam is approximately 104 feet high (settled elevation) and is roughly 1580 feet long at its crest. The Dam is essentially a homogeneous earthfill dam with a total volume of about 1,800,000 c.y. Provisions have been made in the design for future raising by an additional 39 feet (settled) in two stages.

The Principal Spillway is used for discharging from the reservoir all water except for the routed discharge from storm inflows that exceed the runoff from a storm with a return period of 100 years. It is a tower-type decanting structure with a drop-inlet leading to a discharge pipe. The pipe passes under the left abutment of the dam and is supported by a concrete cradle. The pipe is designed to flow partially full for the runoff from the 100-year storm.

The Emergency Spillway is located on the right bank about 1800 feet from the right abutment of the dam in a rock cut. It is of the broad-crested weir type and is used only to discharge outflow in excess of that produced by a storm with a return period of 100 years. The Emergency Spillway is designed to pass safely, the outflow from the Probable Maximum Flood, routed through the reservoir. Riprap is provided to resist erosion.

The Fly Ash Support System described in this report consists of all the necessary earthwork (except that portion which is described in Harza's Report of February 1974 for the Gavin Overland Conveyor) and bridges to support the four 20 inch diameter steel pipelines which will carry the fly ash in the form of a fly ash-water slurry from the Gavin Plant to the Gavin Fly Ash Reservoir. The fly ash lines follow their own right-of-way from the boundary of the Gavin Power Plant to the Transfer Station (G5) of the Gavin Overland Conveyor west of Gravel Hill (see Exhibit A, Feature Location Map). In this portion of their route, the ash lines cross the following major obstructions:

(1) Above one high pressure gasline owned by Ohio Fuel Gas Company;

(2) Above one relatively major stream, Kyger Creek, via Kyger Creek Bridge; and

(3) Under Gravel Hill Road via multiplate arch-type culvert. At the Transfer Station, they join the right-of-way of the Gavin Overland Conveyor and follow it until their discharge points at the fly ash reservoir. Along this portion of their route, the ash lines pass over Turkey Run Road and, in planned future installation, across Stingy Run via the existing Stingy Run Bridge, which is designed to support four pipes. As shown on Exhibit A, the ash lines leave the Conveyorway to two different points to discharge into the reservoir.

Chapter II

DESIGN

Fly Ash Dam Initial Design

In April, 1973, Harza submitted a design report for the Gavin Fly Ash Dam. Additional revised reports were issued in October and November 1973, as is explained in the following sections. Details of the initial design are described in that report, but for convenience, pertinent information is presented here.

The dam was designed for a final settled crest elevation of 679 feet, for a maximum estimated height of 90 feet from the stripped ground surface. The maximum normal water surface elevation was 670, the estimated life of the reservoir was 7.9 years, and the estimated unit cost of storage was $0.28/c.y.

The Dam was essentially an earth fill structure with 3h:1v slopes. Stabilizing berms (8h:1v) were provided on both the upstream and downstream sides. These berms both had a settled top elevation of 640. The crest was 30 feet wide. A core trench extended the full length of the dam into rock. 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 riprap on the exposed 3h:1v upstream slope and seeding on the entire downstream slope.

The Principal Spillway is located at the left abutment under the dam, and contained three major components: an intake structure, a conduit, and an impact basin. The intake was a tower-type decanting structure with stoplogs. The initial decanting elevation is 645. Two concentric floating skimmers are provided to retain particulates from entering the intake. The conduit is a 48-inch diameter prestressed concrete cylindrical pipe with rubber and steel joints. Antiseepage collars are provided at intervals along the pipe. The conduit is supported on a reinforced concrete cradle. A USBR impact-type stilling basin was provided for energy dissipation.

The Emergency Spillway, located on the right bank 1800 feet from the right abutment of the dam, is of the broad-crested weir type. The inlet slope is 1% and the outlet slope 2%. The width of the spillway was a constant 100 feet throughout its length. A 30-foot long crest was at a constant elevation of 671, one foot higher than the maximum normal water surface of 670. The side slopes were 3h:1v in overburden and 2h:1v in rock with a 15-foot berm between these two slopes. Eighteen inches of riprap was provided at the crest and downstream of the crest to resist erosion. Seeding was provided on all the remaining excavated areas.

For additional information on the initial design and design considerations of the Fly Ash Dam, refer to the Final Design Report dated April, 1973.

Fly Ash Dam Design Revision #1

The low bid for construction of the dam was considerably lower than the funds allocated for construction based on the engineer's estimate. Consequently, it was decided to use the remaining funds for raising the elevation of the crest of the dam.

The Dam was redesigned permitting storage of ash to elevation 680, or 10 feet higher than the original design. The dam was designed for a final settled crest elevation of 688, and a total estimated height of 99 feet from the stripped ground surface. The estimated life of the reservoir was increased to 11 years, and the estimated unit cost of ash storage was decreased to $0.23/c.y.

The basic geometry of the Dam remained the same: 3h:1v slopes with 8h:1v stabilizing berms on both the upstream and downstream sides. However, a 30-foot wide berm at a settled elevation of 670 had to be added to the upstream slope of the dam to maintain stability.

The overflow crest elevation of the Emergency Spillway was subsequently raised 10 feet to elevation 681. This reduced the required excavation to less than the amount needed for the fill in the dam. The result was additional borrow excavation within the reservoir, and hence increased storage volume. All other features of the Emergency Spillway remained the same.

Ash Pipeline Costs: 70,000 l.f. x $40/l.f. = $2,800,000 – + Drainage – Road, etc. at Least $3, Million

6.5 Mil X 20% X 13 yrs / 12 Mil Ton Ash = $1.40

No design changes had to be made to the Principal Spillway since it is designed and constructed initially to accommodate the maximum anticipated normal water surface elevation of 725.

A design report entitled "Supplement to Final Design Report" was prepared by Harza in October, 1973 to reflect the revised design. However, this report was superseded due to additional design changes requested by AEP. The October report therefore was never issued. For additional information on design revision #1, refer to Volume II of this Report, Appendix A, where the October report is presented in its entirety.

Fly Ash Dam Final Design

During excavation of the Emergency Spillway, a larger quantity of competent rock than had been anticipated from results of exploration was uncovered. At the request of the Owner, excavation in the Emergency Spillway was halted and Harza again modified the design of the Dam and Spillway so that the invert of the emergency spillway was on top of the competent rock. This modification resulted in substantial savings. It then became unnecessary to place riprap along the invert of the Emergency Spillway since it is on top of sound rock. Time and costs relating to spillway excavation also were saved.

In November, 1973, a second design report, also entitled "Supplement to Final Design Report", was issued by Harza to reflect the final design. Details of the final design are in that report. Herein, only pertinent information is presented.

The Dam is designed for a final settled crest elevation of 692 feet, for a total estimated height of 104 feet from the stripped ground surface. The maximum normal water surface elevation is 685.5, the fly ash storage is 12,320,000 cu. yd., the estimated life of the reservoir is 13.6 years, and the estimated unit cost of storage is $0.19/c.y.

Because of the increased height of the dam, stability problems were encountered and some significant modifications to the original cross-section were necessary. The Dam remained essentially an earth fill

13 yr Life
10% Int
5% [illegible]
Annual Carrying Ch.

Not Correct

$2.7 –
Eng. design @ 19% .52
3.22
Int @ .38
3.60

$3.5 Mil X 20% X 13 yrs / 12 Mil = 75¢/c.y.

structure, with upstream side slopes of 3h:1v and with an 8h:1v stabilizing berm to a settled top elevation of 640. However, a 30-foot wide berm on the upstream slope of the dam at a settled elevation of 670 was added. The downstream side was steepened to 2.5h:1v slopes with an 8h:1v stabilizing berm to a settled top elevation of 650. In addition, a 15-foot deep by 3-foot wide trench backfilled with sand was added on the downstream side of the dam at the intersection of the 2.5:1 and 8:1 slopes to increase stability during the steady state seepage condition.

The Emergency Spillway also was changed significantly. The minimum width of the spillway at the control section is 220 feet. The crest elevation varies locally between 685 and 687.5 (this is due to excavation prior to the revised design). The flow control section is 200 feet wide instead of the originally planned 36 feet. The inlet and outlet slopes are unchanged - 1% and 2%, respectively. The side slopes are now all 3h:1v and the 15 foot berm that was intended to go between the 3:1 and 2:1 slopes in the original design has been eliminated. Since a great portion of the invert of the spillway now rests on competent sandstone, no riprap is provided except on the side slopes. The operating head above the crest is reduced, and velocities are reduced correspondingly.

As was previously noted in design revision #1, no design changes had to be made to the Principal Spillway since it was originally designed to be operable up to a maximum normal water surface elevation of 725.

Fly Ash Line Support System

For the purposes of this report, the portion of the fly ash support system from the Gavin Plant coal storage area to Gravel Hill Road and the portion along the Gavin Overland Conveyor from Gravel Hill Road to the fly ash reservoir will be considered separately.

Gavin Plant to Gravel Hill Road

The initial segment of the ash line support system extends from the Gavin Plant coal storage area to Gravel Hill Road. It consists of two earth embankments and a bridge. Prior to finalizing this design, Harza was requested to study four alternative schemes which involved replacing part or all of the earth embankments and bridge with individual structural bents to support the ash lines. However, cost considerations indicated that the initial design of support by earth embankments and bridge was the least expensive.

The embankments consist of random fill material with a 4h:1v slope on the upstream side of Kyger Creek and a 3h:1v slope on the downstream side. The maximum height of fill is approximately 25 feet above the stripped ground surface. A 25-foot crest has been provided to accommodate a 15-foot right-of-way for the fly ash pipes and a 10-foot wide access road. The total quantity of fill is about 80,000 cubic yards. Riprap is provided at the toe of the fills in the vicinity of Kyger Creek to resist erosion at times of flooding.

The bridge which supports the fly ash pipes across Kyger Creek is called the Kyger Creek Bridge. It is a through truss steel bridge consisting of three 111'-2" spans supported at each end by reinforced concrete piers. The piers are supported by Monotube friction piles. The overall length of the bridge is 341.5 feet; the bottom chord elevation is 575.0 for a total clear height above Kyger Creek of about 25 feet. The bottom chord elevation was chosen with considerations given to the overall profile of the support system and to providing an adequate waterway opening for runoff from a storm with a 100-year return period. The 100 year storm will produce a maximum water surface elevation of 570.0±. Therefore, there will be 5.0± of clearance from the bottom chord of the Kyger Creek Bridge during this storm. The bridge was designed to carry the load from four 20-inch slurry lines and one 16-inch waterline plus the usual dead load, snow loads and wind loads. The waterline, however, was not installed.

Supervision of the construction of this segment of the ash line support system was done by others. Therefore, details of the construction and construction drawings are not presented in this Report.

Gravel Hill Road to Fly Ash Reservoir

The fly ash pipes cross underneath Gravel Hill Road via a multiplate arch-type culvert which was designed by others. At the exit of the culvert, the ash pipes enter transfer station G5 of the Gavin Overland

Conveyor. From there, they run along the conveyorway on a 10-foot wide berm until reaching the fly ash reservoir.

Two conveyor structures are crossed by the ash lines between transfer station G5 and the fly ash reservoir, Turkey Run Bridge and Stingy Run Bridge. Both bridges were designed for the slurry lines as well as the conveyor. Details of these bridges can be found in the "Final Report - Gavin Overland Conveyor and Meigs Intermine Conveyor", February 1974. The ash lines cross both bridges along the top truss.

Two discharge points are provided at the reservoir (see Exhibit A, Feature Location Map). The first discharge point is for three of the four ash pipes which diverge at conveyor station 453+00 and carry the slurry to a discharge point approximately 400 feet from the conveyor centerline to the south end of the fly ash reservoir. The fourth ash line crosses Stingy Run Bridge and discharges at the northwest end of the reservoir near conveyor station 390+50±.

Harza also designed and supervised the construction of all the necessary earthwork required by the ash line support system along the Conveyor between Transfer Station G5 and the fly ash reservoir as part of a separate contract (No. 3012 - Field engineering services for construction of Gavin Overland Conveyor). Details of this work can be found in the "Final Report Gavin Overland Conveyor and Meigs Intermine Conveyor," February 1974.

The rollers and ties which support the ash pipes across the bridges and along the conveyorway and the steel pipelines all were designed by others, and therefore details concerning them are not presented herein.

Chapter III

CONSTRUCTION

General

The Gavin Fly Ash Retention Dam is located on the Stingy Run Creek approximately 3,000 feet east of the Stingy Run Bridge on the Gavin Overland Conveyor. The Dam site is accessible by road from Ohio State Route 554 along the Stingy Run Road.

The Contractor, Paul Coffey Construction Company, started the construction operations on June 18, 1973. Excavation in the core trench was essentially completed by July 12, 1973 enabling the Contractor to proceed with drilling and grouting of the rock foundation in the core trench. During the period of curtain grouting, the fill operations essentially were limited to the material available from required emergency spillway excavation. Grouting operations on the north abutment were completed on September 17, 1973 and on the south abutment by October 11, 1973. Full scale fill construction operations commenced thereafter with over 90% of the fill material coming from borrow areas. The dam embankment was completed on November 21, 1973. The Contractor had sufficient equipment and manpower to maintain a work schedule of two 9 hour shifts during the period of major earthwork construction.

Concrete work was initiated on September 8, 1973 and was completed on January 30, 1974. The concrete for the top 3 lifts on the Principal Spillway and the top lift on the impact basin was placed during winter months. The Contractor avoided placing concrete during freezing weather, thus minimizing the need for protection during and after concrete placement.

Placing of bedding and riprap on the upstream face of the dam was initiated on November 30, 1973 and was essentially completed on February 15, 1973. Riprap in the Emergency Spillway and outlet channel was placed in April, 1974.

The supply of skilled and unskilled labor was adequate and no downtime was experienced due to strikes.

Sequence of Construction

The sequence of construction was essentially the same as for any earthfill dam of similar magnitude and design. The following were the major activities in the construction of the Project:

1) Clearing and grubbing.
2) Care of water.
3) Excavation of Core Trench.
4) Treatment of the surface of the bottom of the core trench and grouting the rock under the core bed.
5) Excavation in the Emergency Spillway.
6) Construction of Principal Spillway.
7) Embankment construction.
8) Placing of riprap and road surfacing.
9) Slope dressing top soil and seeding.
10) Clean up.

Construction sequence generally followed that listed above although activities 2, 4, 5, occurred concurrently.

Diversion And Care of Water

The Contractor excavated the core trench for the full length of the dam, except near the Stingy Run stream bed, by utilizing CAT 637 scraper units supported by D8 Dozers. During this portion of his operation, the stream flowed in its natural channel. The material from dam excavation was used for constructing the upstream berm on both sides of the stream. A sump was constructed in the bottom of the core trench to collect rain water during foundation treatment operations. Excavation of the sump was executed with a Bucyrus Erie 30 B Dragline. Water from the sump was pumped out by maintaining in operation 3 inch and 10 inch water pumps. To minimize siltation resulting from construction activities, a sediment pond was constructed and maintained at the end of the outlet channel.

When the upstream berm on both sides of the stream bed was complete, a cofferdam was constructed across the stream by utilizing two scrapers and one dozer. Water was ponded by this structure until the portion of the upstream berm in the stream bed was raised to the level of the adjacent berms. The berm fill material came from Emergency Spillway and core trench excavation. Thereafter, the upstream berm acted as a cofferdam impounding the water. To prevent overtopping of the cofferdam until the core trench was backfilled and the dam embankment was raised to elevation 635, the Contractor positioned one 10 inch pump on the upstream berm. Inflow into the reservoir remained below elevation 620 during the construction of the dam and as such neither the emergency pump nor the temporary outlet in the Principal Spillway was needed for diversion purposes.

Drilling and Grouting

Construction of the grout curtain below the core trench was done by two subcontractors:

B. H. Mott & Sons Drilling Co. Huntington, West Virginia	North Abutment
Pennsylvania Drilling Co. Pittsburgh, Pennsylvania	South Abutment

The drilling operations on the north abutment were initiated on July 12, 1973 and grouting was completed on September 17, 1973. Sixteen AX holes with a total length of 952 feet and seventeen NX holes with a total length of 1,247 feet were drilled and grouted.

The drilling operations on the south abutment were initiated on July 31, 1973 and grouting was completed on October 11, 1973. Fifty AX holes with a total length of 3,526 feet were drilled and grouted.

All holes were drilled at an angle of 30° from the vertical with the exception of 1-60° hole and 2-vertical holes in the south abutment and 2-60° holes and 1 vertical hole in the north abutment.

Essentially the drilling subcontractors used Tricone drills where core recovery was not required; for other holes requiring core recovery, diamond carbon bits were used. The average rate of production was 47 feet of drilled hole per day on the north abutment and 68 feet of drilled hole per day on the south abutment.

The following tabulation shows the manhours used for drilling, coring, pressure testing, washing and grouting.

Location	Total Length of Holes	Manhours	Manhours/ Ft. of Hole
North Abutment	2,199 ft	2,340	1.06
South Abutment	3,526 ft	2,912	0.83

Drawings 670C181A and 670C181B show grout takes and pressure test results for the north and south abutments.

Given below are the major items of plant and equipment used in the drilling and grouting work.

1) 2 Air Tracs
2) 2 Sled Rigs
3) 4 600 CFM Compressors
4) 2 Slush Pumps
5) 2 Grouting Plants
6) 2 Water Pumps
7) 2 Water Tanks
8) 2 John Deere 450 Dozers

Excavation and Embankment Construction

Earthwork operations were initiated on June 18, 1973; the majority of this work was completed on November 21, 1973 except for Emergency Spillway excavation which was completed a month later on December 21, 1973.

During the peak production months, the Contractor worked two nine hour shifts. Each shift had two to four spreads of earthmoving equipment. Portable light plants were used during night shifts. All but one earthmoving spread was composed of CAT 631 and CAT 637 scrapers of 20 cubic yards struck capacity, supplemented with dozers and graders. Excavation of the sandstone in the Emergency Spillway required drilling and blasting. Material was loaded on CAT 983 front-end loaders and hauled by two CAT 769 dump trucks.

One self-propelled CAT 825 Sheepsfoot roller and one Hyster 450 Compactor roller were used for compaction of the embankment fills. Graders worked the fills and haul roads as required to provide a good trafficable surface for the hauling equipment. One 8,000 gallon water truck equipped with a 6 inch pump was used to bring the moisture content to the required limits and was also used for dust control.

Listed below are the quantities excavated and their utilization.

Source	Total Excavation	Utilization Fill	Utilization Wasted
Emergency Spillway Excavation	834,000 C.Y.	326,000 C.Y.	508,000 C.Y.
Principal Spillway Excavation	61,000 C.Y.	30,000 C.Y.	31,000 C.Y.
Dam			
Excavation	162,000 C.Y.	150,000 C.Y.	12,000 C.Y.
Stripping	62,000 C.Y.	20,000 C.Y.	42,000 C.Y.
Borrow Excavated (Placed as Fill)	1,150,000 C.Y.	1,150,000 C.Y.	
Totals	2,269,000 C.Y.	1,676,000 C.Y.	593,000 C.Y.

Principal Spillway

The Principal Spillway has a 100-foot high intake tower located over the upstream end of a 48-inch diameter prestressed concrete conduit which discharges into an impact basin approximately 750 feet downstream of the intake. The intake tower is equipped with floating platforms and skimmers.

Construction of the Principal Spillway was initiated on August 8, 1973 with the excavation of the trench for the prestressed conduit. The conduit has ten anti-seep collars and is supported by a continuous concrete cradle. The foundation for the concrete cradle was generally a clay shale. The cradle was placed under the conduit, which was temporarily supported on concrete blocks. The Contractor placed about 80 linear feet of cradle per day. The cradle and the anti-seep collars were placed in two weeks, from September 21 through October 1, 1973.

The concrete work for the impact basin was started on October 18, 1973 and was completed on November 20, 1973. Concrete was placed in two lifts.

The construction of the tower was initiated on September 18, 1973 and was completed on January 30, 1974 by placing concrete in 7 lifts above the foundation slab.

The following manhours were used in constructing the Principal Spillway concrete work:

Location	Concrete C.Y.	Manhours	Manhours/C.Y. of Concrete
Riser	380.2	15,800	41.6
Cradle and Anti-seep Collars	305.5	2,394	7.8
Impact Basin	43	2,600	60.5

Riprap and Bedding

The placement of riprap and bedding on the upstream face of the dam was initiated on November 30, 1973 and was completed on February 15, 1974.

The riprap material was hauled by trucks from Standard Slag Company Plant at Logan, Ohio, 43 miles from the dam site. Each day's supply was placed on the upstream face of the dam by utilizing one D5 CAT-dozer, one 977 CAT front-end loader and one G12 CAT-Grader. The maximum rate of placing riprap and bedding was 1,000 C.Y. per 8 hours day, with an average placement rate of 560 C.Y. per 8 hours day.

Riprap and bedding for the Outlet Channel and riprap for the Emergency Spillway was hauled and stockpiled during February, 1967 and was placed in April, 1974.

A total of 16,173 cubic yards of riprap and 10,118 cubic yards of bedding was placed.

Production Rates

The following production rates are derived from daily inspector's reports, weekly reports and the Contractor's records and reflect an average production rate over the length of time which was chosen to be most representative of production activity on that item.

Excavation	28,000 C.Y./day
Embankment	18,000 C.Y./day
Drilling and Grouting	57 feet/day
Riprap and Bedding	560 C.Y./day

Contractor's Equipment

The equipment used by the Contractor is listed below:

No.	Equipment Type
1	CAT D5 Dozer
2	Caterpillar Model D8 Dozers
6	Caterpillar Model D9 Dozers
7	Caterpillar Model 631 Scrapers
4	Caterpiller Model 637 Scrapers
1	Caterpillar Model G16 Grader
1	Caterpillar Model 825 Sheepsfoot Roller
1	Hyster Model 450 Compactor Roller
1	Link-belt LS318 Crane
6	Light Plants
1	8,000 Gals. Water Wagon
1	Caterpillar Model G12 Grader
1	JD 450 Backhole
2	Caterpillar Model 769 End Dump Trucks
1	CAT 977 Front-end Loader
1	Caterpillar Model 983 Front-end Loader
1	600 CFM Joy Compressor
1	30B Bucyrus Erie Crane with Dragline
3	Pick-ups
1	Mechanics Truck
1	3 Inch Pump
1	10" Yeager Pump

Contractor's Organization

Paul Coffey Construction Company maintained a field office at the Project site. One general superintendent managed the job with the aid of several support supervisory personnel.

Listed below are the Contractor's supervisory personnel and support staff:

1 General Superintendent
1 Project Engineer
1 Earthwork Superintendent
1 Concrete Superintendent
4 Foremen
1 Office Staff

The following table shows the number of operators, craftsmen and laborers employed by the Contractor at the end of each month.

Month	Operators	Craftsmen	Laborers	Total
June, 1973	7	2	3	12
July	23	7	2	32
August	32	15	15	62
September	40	14	8	62
October	44	15	18	77
November	43	26	12	88
December	11	17	9	37
January, 1974	5	15	4	24
February	3	7	4	14

Construction Inspection

The Resident Engineer and his supporting staff as described below were provided by Harza Associates of Ohio. The resident Engineer directed all inspection activities except placement of concrete (which inspection was performed by American Electric Power Civil Engineering Laboratory) coordinated planning with the Contractor, and checked quantities involved in the Contractor's invoices.

Construction inspectors were provided through a sub-contract between Harza and Union Boiler Company, and were supervised by Harza Associates of Ohio. The source of manpower, which was provided by local communities, proved adequate and men with previous construction experience were available as required.

The size and disposition of the inspection force was dependent upon the number of separate operations in progress. An inspector was always placed on each operation involving the following: embankment construction, concrete work, placing of riprap and bedding. Operations such as drilling and grouting work were covered with the inspection force consistent with the work involved. The usual distribution of the inspection force was as follows:

Earth Work	2
Drilling and Grouting	2
Concrete Work	1

General Analytics, Inc., of Pittsburgh, Pennsylvania, under sub-contract with Harza Associates of Ohio, provided soils testing services and equipment for the Project. Their operations were housed in a mobile trailer equipped with testing facilities and located at the Project site. All compaction tests were taken with a Nuclear Surface Moisture Density Gauge. During the months of large earthwork production, the number of Soil Technicians totalled 3 and were supervised by one Soils Engineer. Soils Compaction Tests were used for control of dam embankment, corrugated metal culvert placement and backfill around structures.

General Analytics submitted a weekly summary of tests completed, listing explicitly locations, elevations, type of material and results.

Compaction tests were taken at intervals which allowed no more than one to three feet of material placement between tests. Generally, material types and compaction procedures which may have been of doubtful quality were cause for greater frequency of tests; in more easily compacted materials, experience was sufficient to allow a lesser frequency of tests. Such procedures led to a rate of 30 tests per day during peak production or 600 cubic yards of embankment per test recorded.

Engineer's Organization

Harza's Resident Engineer and his field organization provided services on several other construction contracts simultaneously with the Gavin Fly Ash Dam work. These contracts were:

1. Construction of the water supply dam and pumping station at Mine No. 1.
2. Construction of the water line between Mines 1 and 2.
3. Gavin Overland Conveyor.
4. Meigs Intermine Conveyor.

The organization for the work is listed below:

1	Resident Engineer
2	Deputy Resident Engineers
3	Field Engineers
1	Office Engineer
2	Office Staff

Additionally, Harza's Chicago Office provided personnel for consultation, training, and general supervision as required during drilling and grouting operation in the core trench.

Construction Records

Daily reports of construction activities and quantities were prepared by the Field Inspectors. Weekly reports were prepared by the field office staff and submitted to administrative and supervisory personnel in AEP, N.Y., Harza, Chicago, and others concerned with the work. Internal reports were compiled in the field office during the construction period on various items of work, testing and inspection of materials and workmanship, weather and other items concerning the work. Periodic photographs of the construction work were taken by a professional photographer. Upon completion of construction, the records were turned over to Ohio Electric Company. Construction drawings were revised as required and upon completion of the work to show construction changes.

CHAPTER IV
COSTS

Itemized quantities and construction costs for the Gavin Fly-Ash Dam are tabulated on the following pages. Costs are not available for the Fly-Ash Line Support System since supervision of construction was done by others.

GAVIN FLY ASH RETENTION DAM

FINAL QUANTITIES AND COSTS

Item No.	Description	Quantity	Unit	Unit Price (Dollars)	Amount (Dollars)
1	Diversion and Care of Water	All	Lump Sum		15,000.00
2	Clearing and Grubbing	55	Acre	400.00	22,000.00
3	Transportation of Core Boxes	All	Lump Sum		
4	Stripping	61,801	C.Y.	0.60	37,080.60
5	Principal Spillway Excavation	61,061	C.Y.	1.50	91,591.50
6	Dam Excavation	152,554	C.Y.	1.70	259,341.80
7	Emergency Spillway Excavation	834,000	C.Y.	0.68	567,120.00
8	Borrow Excavation	1,258,266	C.Y.	0.58	729,794.28
9	Dental Excavation	10	C.Y.	20.00	200.00
9A	Resin Coating	1,012	S.Y.	1.35	1,366.20
10	Impervious Fill	665,014	C.Y.	0.10	66,501.40
10A	Upstream Berm Fill	172,943	C.Y.	0.40	69,177.20
11 12	Rolled Shale and Random Fill	847,458	C.Y.	0.10	84,745.80
13	Bedding	10,413	C.Y.	10.00	104,130.00
14	Rip-Rap - Type I	2,737	C.Y.	10.50	28,738.50
15	Rip-Rap - Type II	16,602	C.Y.	11.00	182,622.00
16	Rip-Rap - Type II, Hand Placed	364	C.Y.	25.00	9,100.00
17	Road Surfacing	1,555.6	C.Y.	12.50	19,445.00
18	Top Soil	4,056	C.Y.	5.00	20,280.00
19	Additional Rolling For Compaction		1000 S.Y.	12.00	
20	Commercial Fertilizer	26	Ton	140.00	3,640.00

GAVIN FLY ASH RETENTION DAM

FINAL QUANTITIES AND COSTS

Item No.	Description	Quantity	Unit	Unit Price (Dollars)	Amount (Dollars)
21	Agriculture Liming	111	Ton	45.00	4,995.00
22	Seeding and Mulching	318,557	S.Y.	0.12	38,226.84
23	Surface Settlement Monument	3	No.	200.00	600.00
24	Settlement Gage	3	No.	1,500.00	4,500.00
25	PVC Pipe - Perforated	372	L.Ft.	3.50	1,302.00
26	PVC Pipe - Unperforated	258.5	L.Ft.	2.25	581.63
27	Setting Up Cover Grout, Check Exploratory and Observation Well Holes	117.0	No.	30.00	3,510.00
28	Drilling AX Holes	5,724.6	L.Ft.	5.00	28,623.00
29	Drilling NX Holes	249.5	L.Ft.	8.95	2,233.03
30	Drilling 4" Overburden Holes	256.5	L.Ft.	8.80	2,257.20
31	Core Recovery "AX" Holes	3,798.8	L.Ft.	3.00	11,396.40
32	Core Recovery "NX" Holes	249.5	L.Ft.	6.00	1,497.00
33	Drilling Grout From Holes	83.0	L.Ft.	5.00	415.00
34	Connections to Drilled Grout Holes	65.0	No.	15.00	975.00
35	Processing, Mixing, and Injecting Pressure Grout	2,408	No. of Sacks	3.30	7,946.40
36	Cement in Grout	2,466	No. of Sacks	3.00	7,398.00
37	Sand in Grout		C.Y.	20.00	
38	Fly Ash in Grout	30,150	Lbs.	0.20	6,030.00
39	Bentonite in Grout		Lbs.	0.20	

GAVIN FLY ASH RETENTION DAM

FINAL QUANTITIES AND COSTS

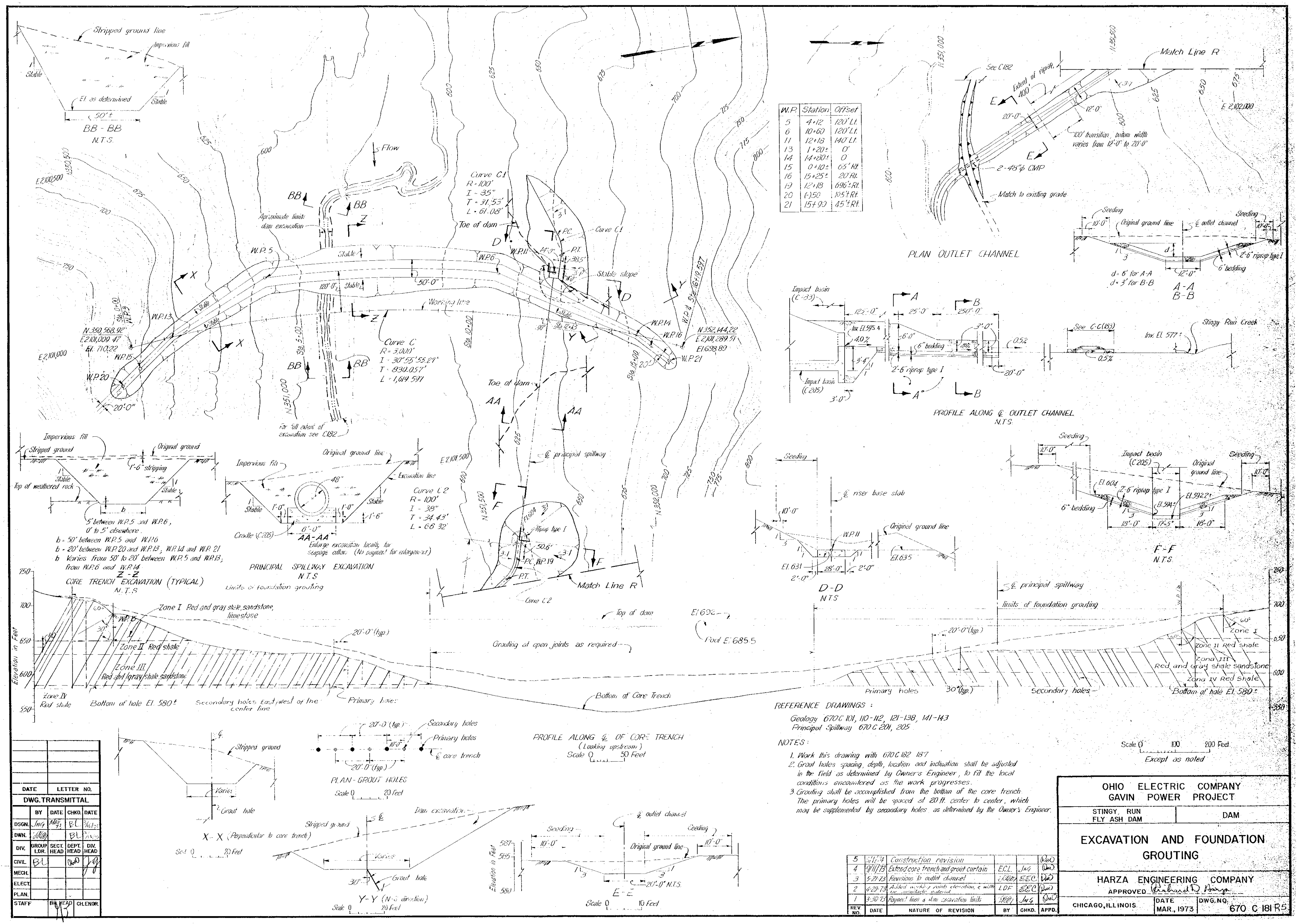
Item No.	Description	Quantity	Unit	Unit Price (Dollars)	Amount (Dollars)
40	Pressure Tests	245	No.	40.00	9,800.00
41	Corrugated Metal Pipe 48-in. Dia.	120	L.Ft.	50.00	6,000.00
42	Pipe Bedding	70	C.Y.	12.00	840.00
43	Concrete, 2500 Psi	346.8	C.Y.	60.00	20,808.00
44	Concrete, 4000 Psi	601.2	C.Y.	65.00	39,078.00
45	Concrete, 6000 Psi	7.0	C.Y.	150.00	1,050.00
46	Grade 60, Steel Reinforcement	116,214	Lb.	0.22	25,567.08
47	Straight Forms	14,466.2	S.Ft.	4.00	57,864.80
48	Curved Forms	1,443.1	S.Ft.	7.50	10,823.25
49	Prestressed Pressure Pipe Conduit - 48 in.	753	L.Ft.	100.00	75,300.00
50	Preformed Expansion Joint Filler	1,758.4	Lbs.	1.20	2,110.08
51	Miscellaneous Metal Work	7,133	Lbs.	1.00	7,133.00
52	Floating Platforms and Skimmer		Lump Sum		
Total					$2,690,734.99

EXHIBITS

DRAWINGS SHOWING CONSTRUCTION CHANGES

GAVIN FLYASH DAM DRAWINGS

Drawing No. 670C181R5	Excavation and Foundation Grouting
Drawing No. 670C181A	Drilling and Grouting North Abutment
Drawing No. 670C181B	Drilling and Grouting South Abutment
Drawing No. 670C182R7	Plan of Dam
Drawing No. 670C183R7	Sections and Details of Dam
Drawing No. 670C191R4	Emergency Spillway - Plan
Drawing No. 670C192R4	Emergency Spillway - Sections
Drawing No. 670C201R3	Principal Spillway - Plan and Sections
Drawing No. 670C205R3	Principal Spillway - Conduit and Impact Basin
Drawing No. 670C206AR1	Principal Spillway - Floating Skimmer



W.P.	Station	Offset
5	4+12	120' Lt.
6	10+60	120' Lt.
11	12+18	140' Lt.
13	1+20±	0'
14	14+80±	0
15	0+10±	65' Rt.
16	15+25±	20' Rt.
19	12+18	696'± Rt.
20	(-)50	105'± Rt.
21	15+90	45'± Rt.

REFERENCE DRAWINGS :

Geology 670 C 101, 110-112, 121-138, 141-143
Principal Spillway 670 C 201, 205

NOTES :

1. Work this drawing with 670 C 182 187
2. Grout holes spacing, depth, location and inclination shall be adjusted in the field as determined by Owner's Engineer, to fit the local conditions encountered as the work progresses.
3. Grouting shall be accomplished from the bottom of the core trench. The primary holes will be spaced at 20 ft. center to center, which may be supplemented by secondary holes as determined by the Owner's Engineer.

REV. NO.	DATE	NATURE OF REVISION	BY	CHKD.	APPD.
5	5/1/74	Construction revision			
4	9/11/73	Extend core trench and grout curtain	E.C.L.	JNG	
3	5-21-73	Revisions to outlet channel		EEC	
2	4-20-73	Added working points elevation, & width for unsuitable material	LDF	EEC	
1	3-30-73	Payment lines & dam excavation limits		JNG	

OHIO ELECTRIC COMPANY
GAVIN POWER PROJECT

STINGY RUN FLY ASH DAM | DAM

EXCAVATION AND FOUNDATION GROUTING

HARZA ENGINEERING COMPANY
APPROVED Richard D. Harza
CHICAGO, ILLINOIS
DATE MAR., 1973
DWG. NO. 670 C 181 R5

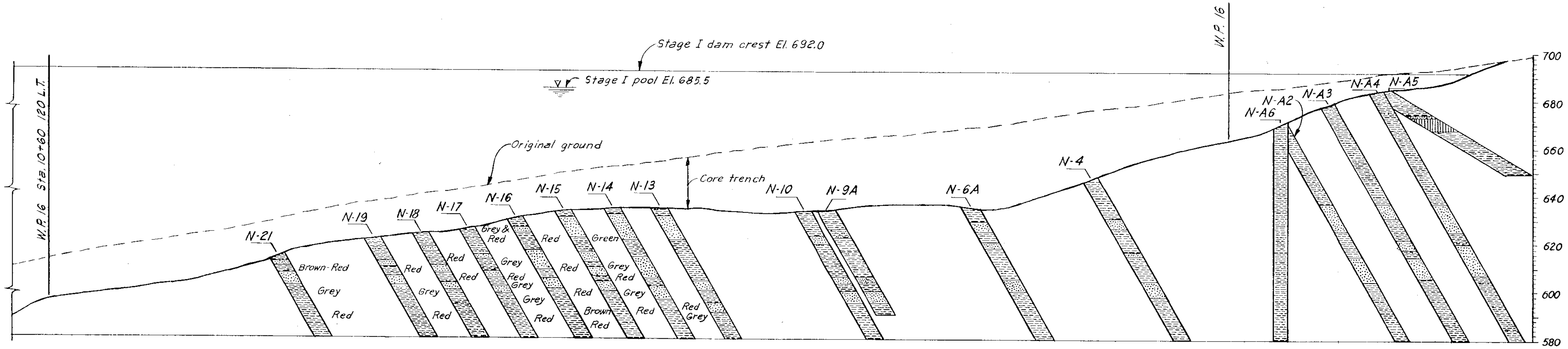
DATE	LETTER NO.

DWG. TRANSMITTAL

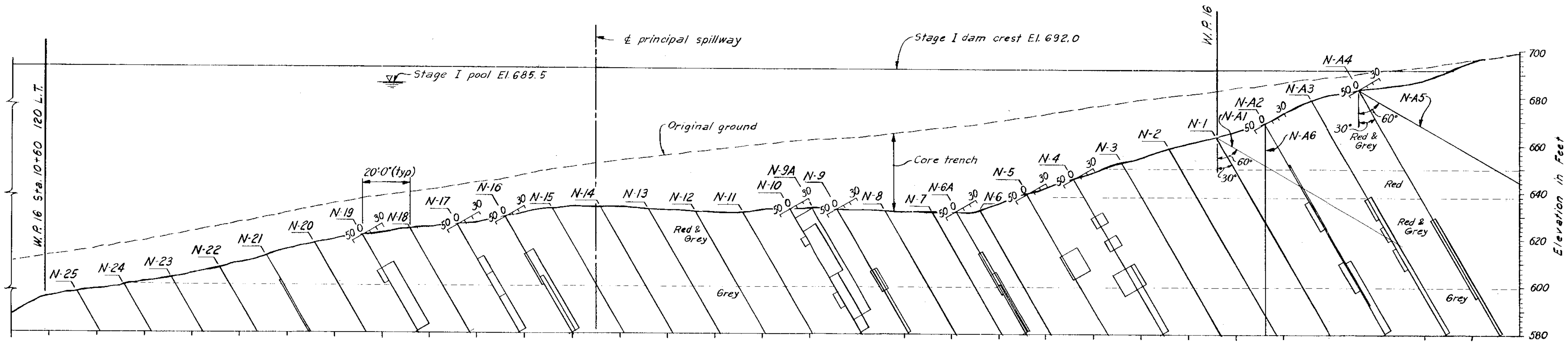
	BY	DATE	CHKD.	DATE
DSGN.	JNG	MAY 73	BL	3/16/73
DWN.			BL	3/ /73

DIV.	GROUP LDR.	SECT. HEAD	DEPT. HEAD	DIV. HEAD
CIVIL	BL			
MECH.				
ELECT.				
PLAN.				

STAFF | BR. HEAD | CH. ENGR.



PROFILE ALONG CENTER LINE
CORE TRENCH



PROFILE ALONG CENTER LINE
CORE TRENCH
(From W.P.6 Towards North)

LEGEND:
— — — Original ground
- - - - - Rock contacts
——— Core bed
W.P. Working point
▨ Shale
▨ Sandstone
▥ Clay
▨ Sandy shale
50 0 30 Grout take, cubic yards
Water pressure test results, lugeons

Scale 0 20 40 Feet
Except as noted

NOTES:
1. Cement and Fly Ash in grout are shown in cubic yards. Beside drill holes, water takes are shown in Lugeons (LU) for intervals tested.
2. Main rock types are labelled. Minor interbeds are omitted.
3. Strata labelled shale includes Red, Green, Brown Grey and Blue shale.
4. Holes N-1, N-A1, N-2, N-3, N-5 through N-9, N-11 N-12, N-20 and N-22 through N-25 were not cored.

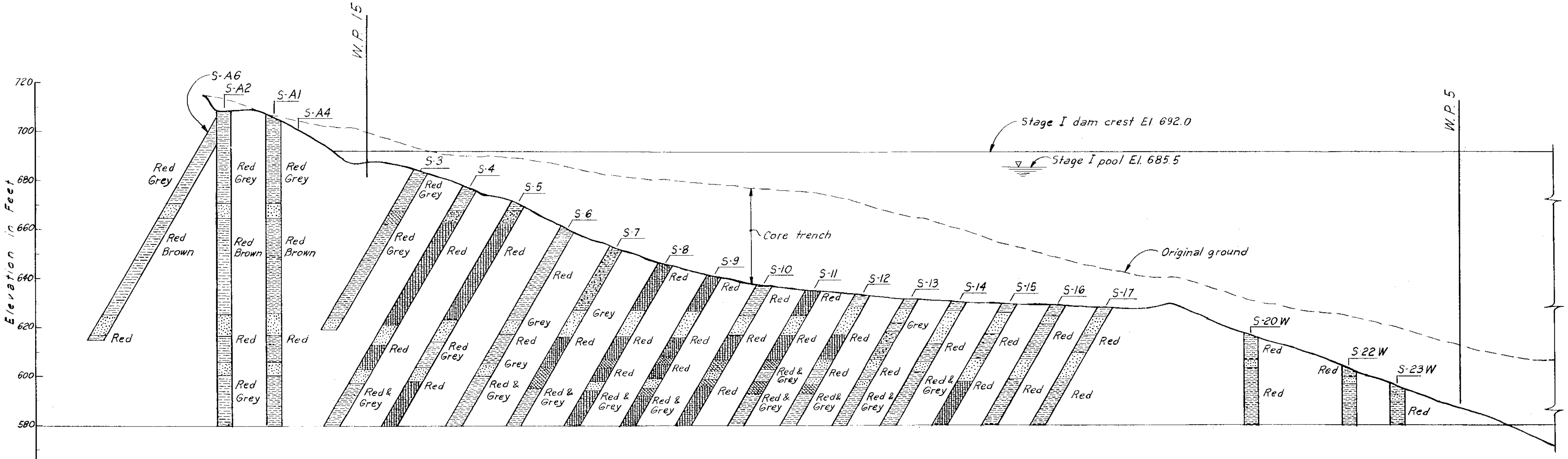
DATE		LETTER NO.		
DWG. TRANSMITTAL				
	BY	DATE	CHKD	DATE
DSGN.				
DWN.				
DIV.	GROUP LDR.	DEPT. HEAD	DIV. HEAD	BR. HEAD
CIVIL				
MECH.				
ELECT				
PLAN.				
STAFF			CH.ENGR.	

REV. NO.	DATE	NATURE OF REVISION	BY	CHKD.	APPD.

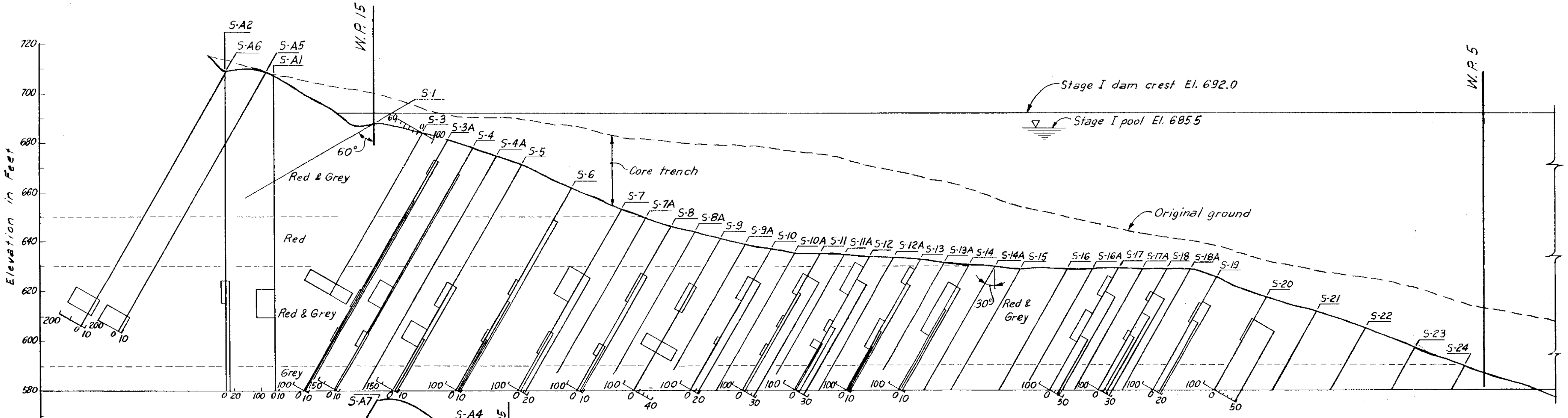
OHIO ELECTRIC COMPANY
GAVIN POWER PROJECT
GAVIN FLY ASH DAM
DAM

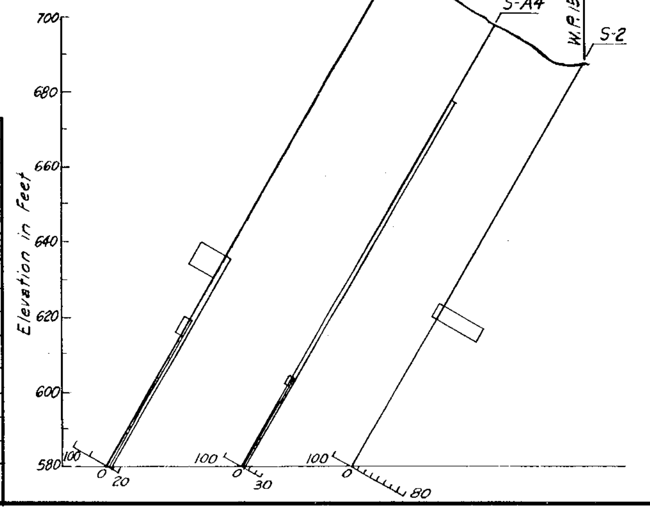
DRILLING AND GROUTING
NORTH ABUTMENT

HARZA ENGINEERING COMPANY
APPROVED
CHICAGO, ILLINOIS
DATE JAN. 1975
DWG. NO. 670C181A



PROFILE ALONG CENTER LINE
CORE TRENCH





PROFILE ALONG CENTER LINE
CORE TRENCH
(From W.P. 5 Towards South)
Looking up-stream

LEGEND:

— — —	Original ground
- - - -	Rock contacts
———	Core bed
W.P.	Working point
▤	Shale
▨	Sandy shale
▩	Clay shale
▧	Silt shale
░	Sandstone
▨	Siltstone
100	Grout take, cubic yards
50	Water pressure test results, lugeons

NOTES:
1. Cement and Fly Ash in grout is shown in cubic yards. Water takes are shown in Lugeons (LU) for intervals tested.
2. Major rock types are labelled and minor interbeds are omitted.
3. Holes S-2, S-A4 and S-A7 are separated to show the water takes and solids grouted.
4. Holes S-A4, S-A5, S-A7, S-2, S-3A, S-4A, S-7A through S-14A, S-16A through S-18A and S-18 through S-24 were not cored.

Scale 0 20 40 Feet
Except as noted

OHIO ELECTRIC COMPANY
GAVIN POWER PROJECT
GAVIN FLY ASH DAM
DAM

DRILLING AND GROUTING
SOUTH ABUTMENT

HARZA ENGINEERING COMPANY
APPROVED________________
CHICAGO, ILLINOIS
DATE JAN. 1975
DWG NO 670C 181B

REV. NO.	DATE	NATURE OF REVISION	BY	CHKD.	APPD.

DATE	LETTER NO.			
DWG. TRANSMITTAL				
	BY	DATE	CHKD.	DATE
DSGN.				
DWN.				
DIV	GROUP LDR.	DEPT. HEAD	DIV. HEAD	BR. HEAD
CIVIL				
MECH.				
ELECT.				
PLAN.				
STAFF			CH ENGR	

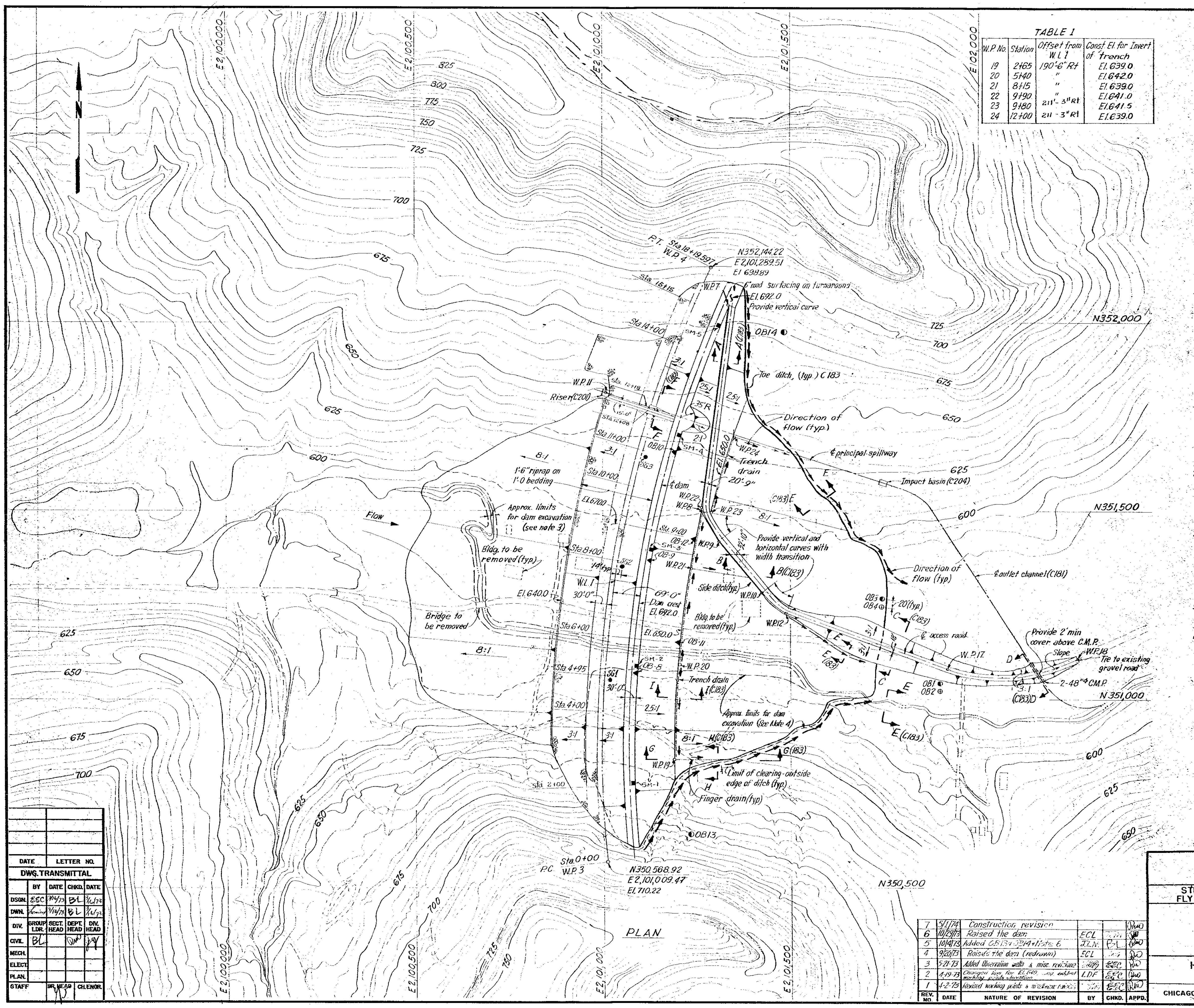


TABLE 1

W.P. No.	Station	Offset from W.L.1	Const. El. for Invert of trench
19	2+65	190'-6" Rt	El. 639.0
20	5+40	"	El. 642.0
21	8+15	"	El. 639.0
22	9+90	"	El. 641.0
23	9+80	211'-3" Rt	El. 641.5
24	12+00	211-3" Rt	El. 639.0

CURVE DATA OF W.L. 1

R=3,000.0'
I=30°55'55.27"
T=830.057'
L_{arc}=1,619.597'
D=1°54'35.49"

W.P. No.	Sta.	Offset from W.L.1	Construction Elevation
7	15+45	84'-0" RT	691.50
8	9+80	189'-0" RT	654.26
9	9+09	253'-0" RT	646.20
10	7+78	390'-0" RT	627.67
11	12+18	140.0' LT	For horizontal control only
12	7+27	462.0 RT	617.27
17	6+08	909.0' RT	587.0
18	7+18	1252± RT	582±

LEGEND

■ SM1 Surface settlement monument
● SG1 Settlement gage
◐ OB1 Observation well, Type I
⊕ OB2 Observation well, Type II
▲ OB8 - OB12 Observation well, Type II (see Note 6)

SURVEY CONTROL

Working points 3 and 4 were established by the Ohio Power Company. These points shall be used to establish construction survey by the Contractor.

REFERENCE DRAWINGS

Geology — 670 C101, C110 thru C112
C121 thru C138
C141 thru C143
Soil tests — 670C151 thru C153, C156
C161 thru C164, C167
Outlet works 670C191 thru C193
C201 thru C206

NOTES

1. Work this drawing with 670C181, C183 thru C187.
2. Topography on this drawing shall not be used for payment purposes. See note 3 on C184
3. Approximately 10' unsuitable material, mainly silt, along the stream bed is expected to be removed. Exact limits and depths may vary and will be determined by the Owner's Engineer.
4. Approximately 2' to 16' strip mine disposal material is distributed in this area. The upper 1'-6" shall be stripped and is to be defined as Stripping. The strip mine disposal material below 1'-6" is expected to remain, subject to approval by the Owner's Engineer. If removal is required, payment will be made as Dam Excavation.
5. All elevations shown are settled elevations except as noted. All slopes shown are settled slopes except as noted.
6. All observation wells will be installed by contractor.
7. All ponded water within strip mine areas within the reservoir to be drained.
8. All stationing along the dam refers to W.L. 1
9. All P.V.C. pipe shall be Schedule 40 and conform to ASTM D1785

Scale 0 100 200 Feet

REV. NO.	DATE	NATURE OF REVISION	BY	CHKD.	APPD.
7	5/1/74	Construction revision			
6	10/29/73	Raised the dam	ECL		
5	10/4/73	Added OB13 + OB14 + Note 6	JLN	BL	
4	9/20/73	Raised the dam (redrawn)	ECL		
3	5-21-73	Added Observation wells & misc. revisions		EEC	
2	4-19-73	Changed line for El. 640 and added working points elevation	LDF	EEC	
1	4-2-73	Revised working points & misc. revisions		EEC	

OHIO ELECTRIC COMPANY
GAVIN POWER PROJECT

STINGY RUN FLY ASH DAM | DAM

PLAN

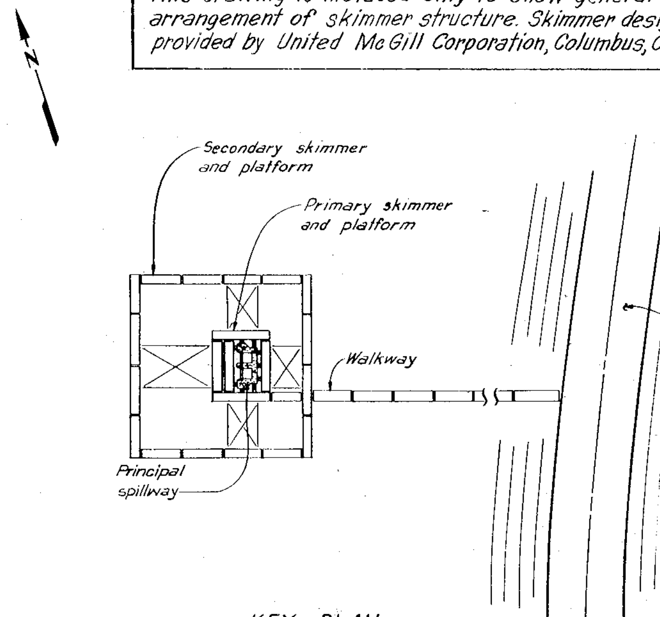
HARZA ENGINEERING COMPANY
APPROVED Richard D. Harza
CHICAGO, ILLINOIS
DATE MAR., 1973
DWG. NO. 670-C182 R7

DATE	LETTER NO.

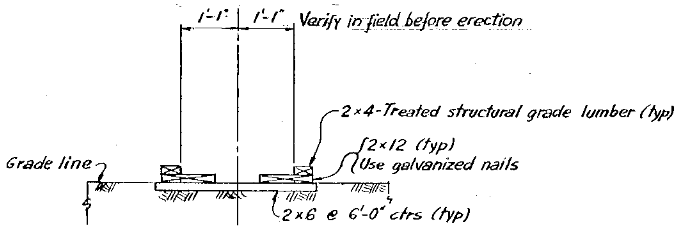
DWG. TRANSMITTAL

	BY	DATE	CHKD.	DATE
DSGN.	EEC	3/16/73	BL	3/16/73
DWN.		3/16/73	BL	3/16/73
DIV.	GROUP LDR.	SECT. HEAD	DEPT. HEAD	DIV. HEAD
CIVIL	BL			
MECH.				
ELECT.				
PLAN.				
STAFF	BR. HEAD	CH. ENGR.		

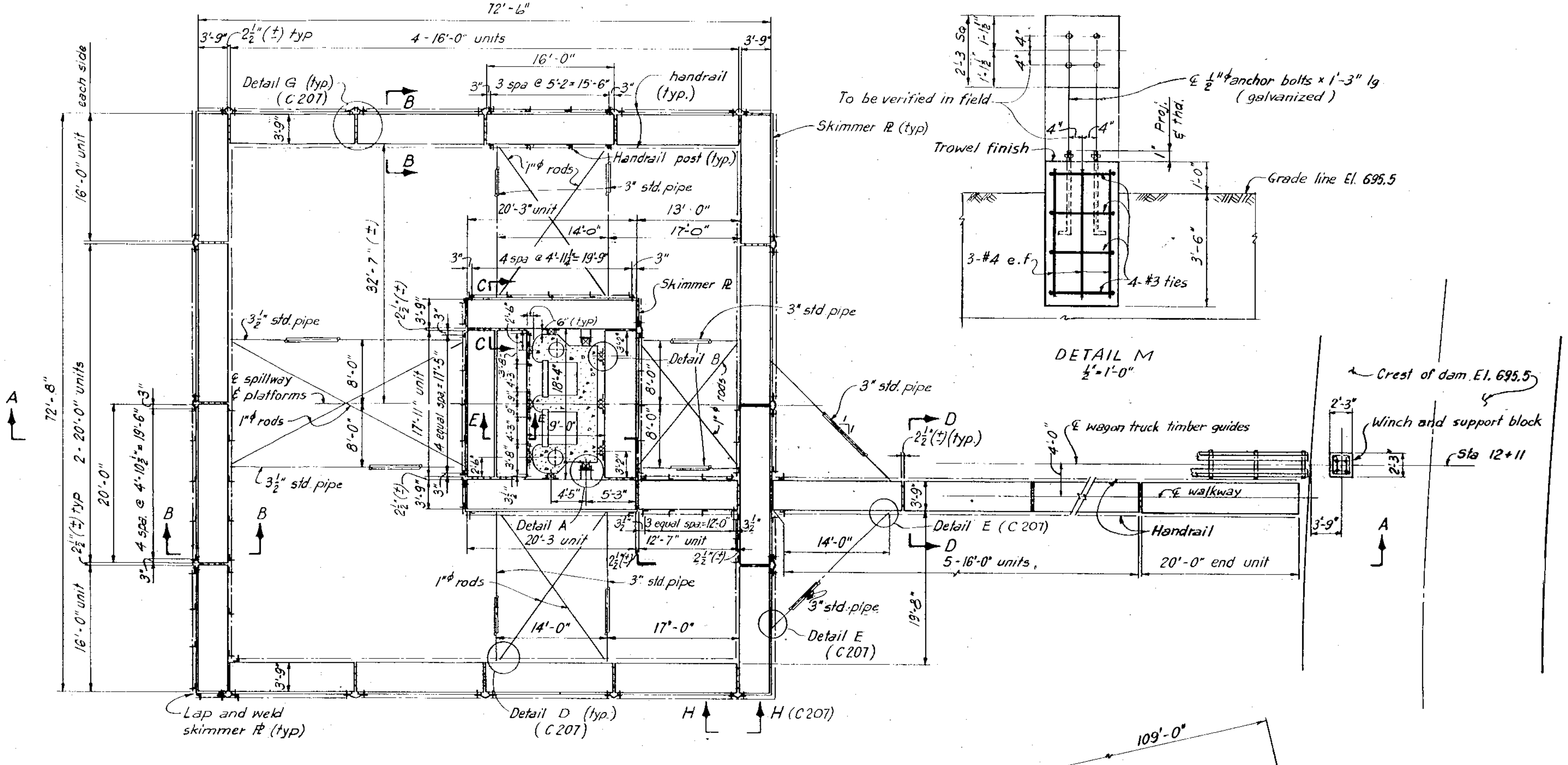
This drawing is included only to show general arrangement of skimmer structure. Skimmer designed and provided by United McGill Corporation, Columbus, Ohio.



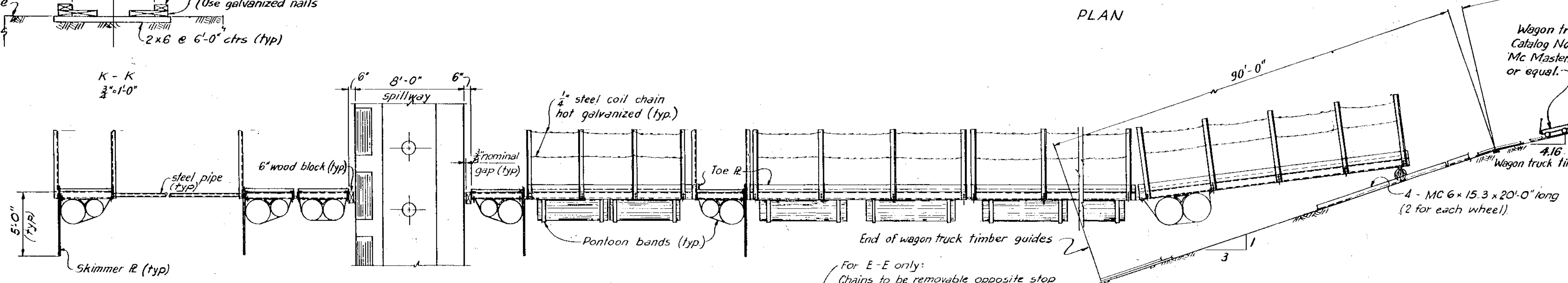
KEY PLAN
Scale 0 30 Feet
1" = 30'

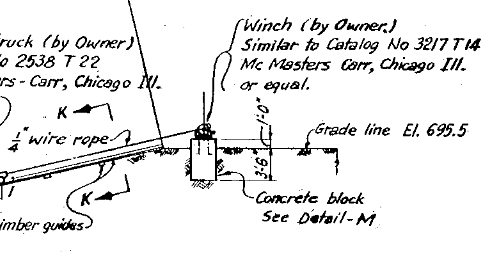


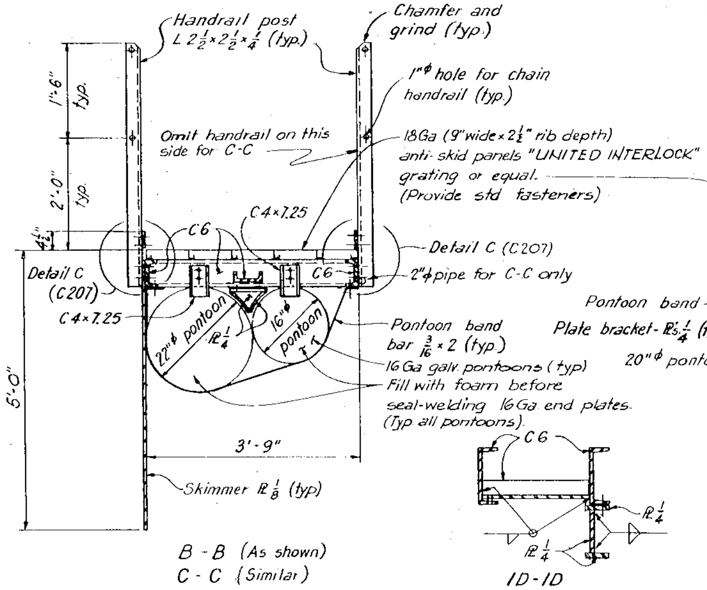
K - K
3/4" = 1'-0"



PLAN

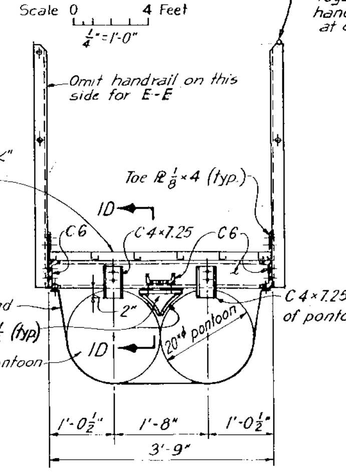






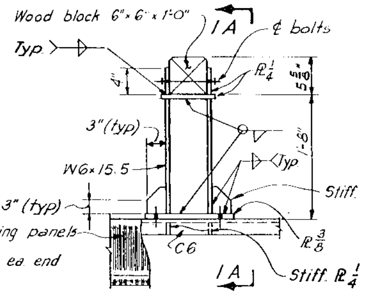
B - B (As shown)
C - C (Similar)
Scale 0 1 Foot
3/4" = 1'-0"

1D - 1D
Scale 0 1 Foot
1 1/2" = 1'-0"

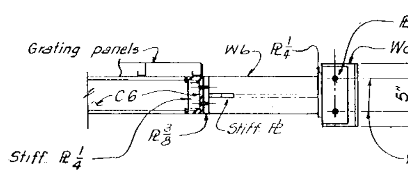


A - A
Scale 0 4 Feet
1/4" = 1'-0"

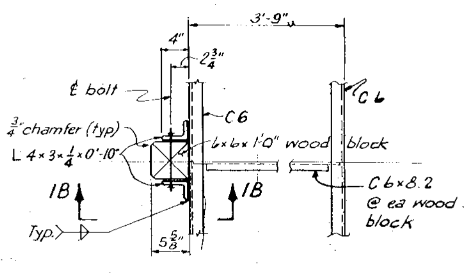
D - D (As shown)
E - E (As shown & noted)
3/4" = 1'-0"



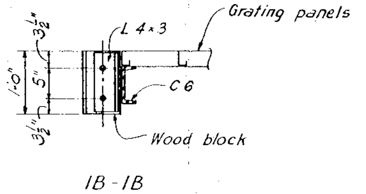
DETAIL A
(2 Req'd)
1" = 1'-0"



1A - 1A
1" = 1'-0"



DETAIL B
(8 - Req'd)
Scale 0 1 Foot
1" = 1'-0"



1B - 1B
1" = 1'-0"

REFERENCE DRAWINGS:

Work this dwg. with 670 C 207 and Principal Spillway, Plan and Sections 670 C 201

NOTES:

1. Bolts 3/4"ϕ, open holes 13/16"ϕ unless noted.
2. All material shall be galvanized structural steel ASTM - A36 except as noted.
3. All welds shall be continuous 3/16" fillet welds unless noted
4. **Wood blocks to be dense, select structural grade**

The Floating Platform and Skimmer will be furnished by Owner. For installation requirements see specifications HECO No. 670 C-3
This drawing supersedes dwg. C 206

5. Wagon truck guides to be structural grade lumber, pressure creosoted with minimum retention of 10lbs/cu ft.

RICHARD D. HARZA
E-27062

Scale 0 8 16 Feet
1/8" = 1'-0"
Except as noted

REV. NO.	DATE	NATURE OF REVISION	BY	CHKD.	APPD.
R1	Jan. 1974	Added winch, support, wagon truck, and guides	B.J.	M.Ba.	RPW

OHIO ELECTRIC COMPANY
GAVIN POWER PROJECT

STINGY RUN FLY ASH DAM | OUTLET WORKS

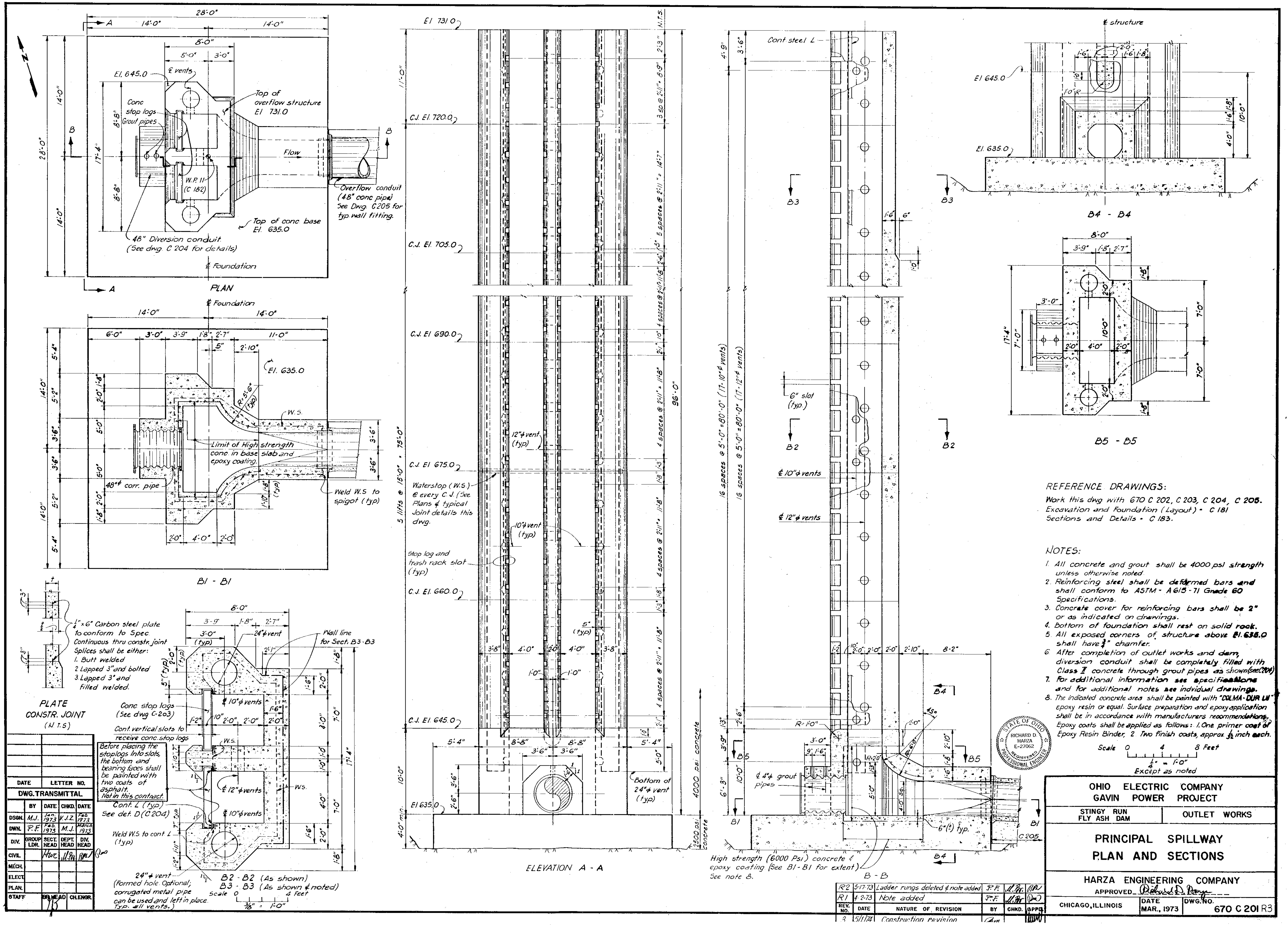
PRINCIPAL SPILLWAY
FLOATING PLATFORMS AND SKIMMER
SHEET 1

HARZA ENGINEERING COMPANY
APPROVED Richard D. Harza
CHICAGO, ILLINOIS
DATE AUG. 1973
DWG. NO. 670 C206A R1

DATE	LETTER NO.

DWG. TRANSMITTAL

	BY	DATE	CHKD.	DATE
DSGN.	M.J.	March 1973	L.P.	March 1973
DWN.	M.D.	March 1973	M.J.	March 1973
DIV.	GROUP LDR.	SECT. HEAD	DEPT. HEAD	DIV. HEAD
CIVIL			M.Ba.	RPW
MECH.				
ELECT.				
PLAN.				
STAFF		BR. HEAD	CH. ENGR.	
		WOG		



REFERENCE DRAWINGS:

Work this dwg with 670 C 202, C 203, C 204, C 205.
Excavation and Foundation (Layout) - C 181
Sections and Details - C 183.

NOTES:

1. All concrete and grout shall be 4000 psi strength unless otherwise noted.
2. Reinforcing steel shall be deformed bars and shall conform to ASTM - A615 - 71 Grade 60 Specifications.
3. Concrete cover for reinforcing bars shall be 2" or as indicated on drawings.
4. Bottom of foundation shall rest on solid rock.
5. All exposed corners of structure above El. 635.0 shall have ¾" chamfer.
6. After completion of outlet works and dam, diversion conduit shall be completely filled with Class II concrete through grout pipes as shown (see C204)
7. For additional information see specifications and for additional notes see indvidual drawings.
8. The indicated concrete area shall be painted with "COLMA-DUR LV" epoxy resin or equal. Surface preparation and epoxy application shall be in accordance with manufacturers recommendations. Epoxy coats shall be applied as follows: 1. One primer coat of Epoxy Resin Binder, 2. Two finish coats, approx 1/32 inch each.

Scale 0 4 8 Feet
¼" = 1'-0"
Except as noted

STATE OF OHIO
RICHARD D. HARZA
E-27062
REGISTERED PROFESSIONAL ENGINEER

REV. NO.	DATE	NATURE OF REVISION	BY	CHKD.	APPD.
3	5/1/74	Construction revision	Am		
R2	5-17-73	Ladder rungs deleted & note added	P.F.	M.Br.	RDH
R1	4-2-73	Note added	P.F.	M.Br.	DnO

DATE	LETTER NO.

DWG. TRANSMITTAL

	BY	DATE	CHKD.	DATE
DSGN.	M.J.	Jan. 1973	V.J.Z.	Feb 1973
DWN.	P.F.	Feb. 1973	M.J.	March 1973
DIV.	GROUP LDR.	SECT. HEAD	DEPT. HEAD	DIV. HEAD
CIVIL		Hwz	M.Br.	RDH
MECH.				
ELECT.				
PLAN.				

STAFF	BR. HEAD	CH. ENGR.
	WB	

OHIO ELECTRIC COMPANY
GAVIN POWER PROJECT

STINGY RUN
FLY ASH DAM

OUTLET WORKS

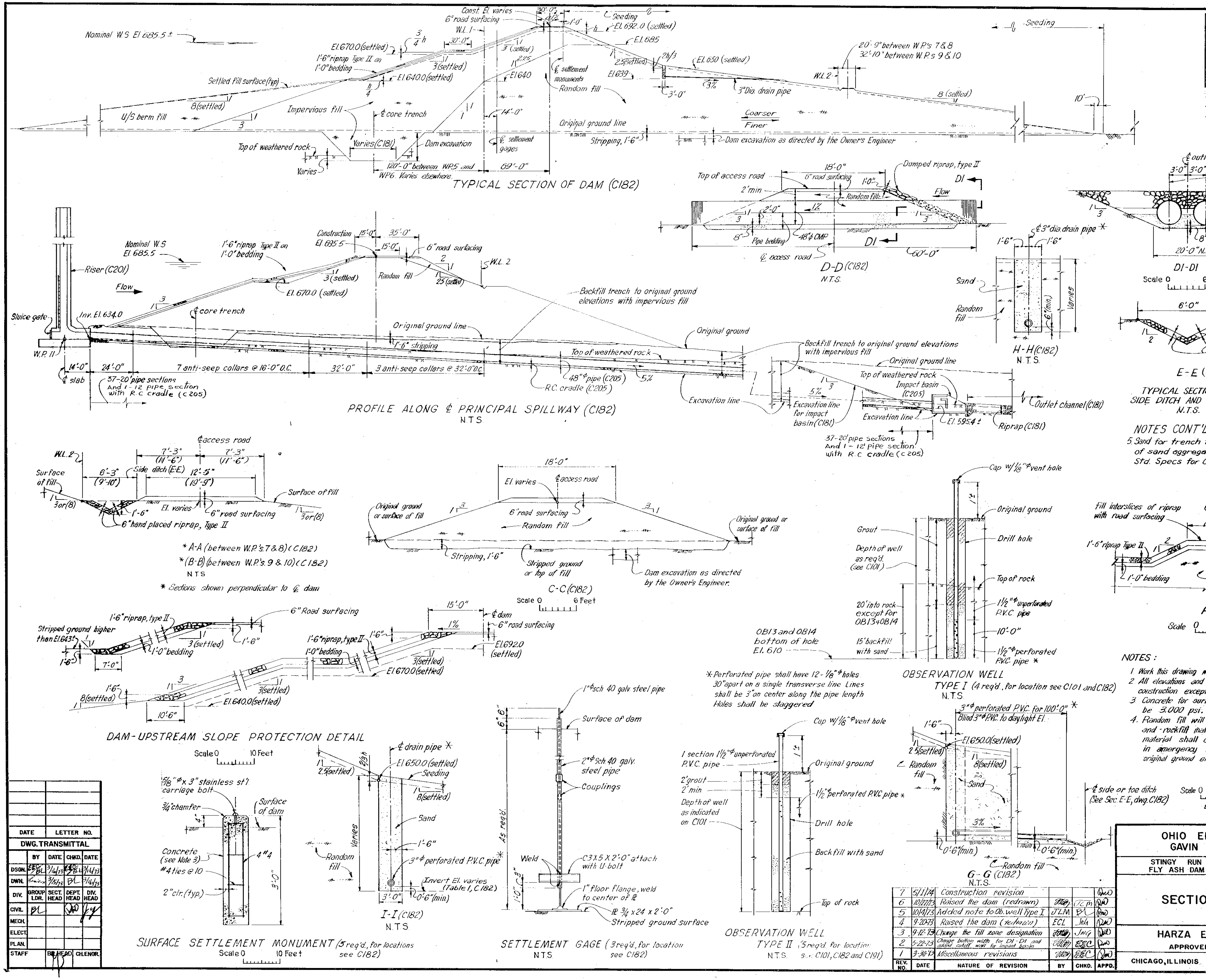
PRINCIPAL SPILLWAY
PLAN AND SECTIONS

HARZA ENGINEERING COMPANY
APPROVED Richard D. Harza

CHICAGO, ILLINOIS
DATE MAR., 1973
DWG. NO. 670 C 201 R3

CONSTRUCTION ELEVATIONS			
STATION	OFFSET	CONSTR'N.EL.	h
0+30±	69'RT	692.0	0
4+85	69'RT	699.0	7'
9+29	69'RT	699.0	7'
15+45	69'RT	692.0	0
16+05	69'RT	692.0	0

Note: h varies linearly between Sta 0+30± to Sta 4+85 & Sta 9+29 to Sta.15+45



NOTES CONT'D.:

5. Sand for trench and finger drains shall consist of sand aggregate as specifed in ASTM C-33, Std. Specs for Concrete Aggregate.

NOTES:

1. Work this drawing with 670 C181, C182, C184-C187
2. All elevations and slopes shown shall be used for construction except those indicated as settled.
3. Concrete for surface settlement monuments shall be 3.000 psi.
4. Random fill will consist of soil, weathered rock and rockfill materials, separately or mixed. This material shall come from required excavation in emergency spillway, principal spillway above original ground elevation 600 and borrow areas.

Scale 0 40 80 Feet
Except as noted

RICHARD D. HARZA E-2[illegible]

DATE	LETTER NO.			
DWG. TRANSMITTAL				
	BY	DATE	CHKD.	DATE
DSGN.	EEC/BL	3/16/73	EEC/BL	3/16/73
DWN.	Lemieux	3/15/73	BL	3/16/73
DIV.	GROUP LDR.	SECT. HEAD	DEPT. HEAD	DIV. HEAD
CIVIL	BL		RwD	Jy
MECH.				
ELECT.				
PLAN.				
STAFF	BR. HEAD	CH. ENGR.		

REV. NO.	DATE	NATURE OF REVISION	BY	CHKD.	APPD.
7	5/1/74	Construction revision			RwD
6	10/27/73	Raised the dam (redrawn)	[illegible]	JLM	RwD
5	10/4/73	Added note to Ob. well Type I	JLM	BL	RwD
4	9-20-73	Raised the dam (redrawn)	ECL	JnG	RwD
3	9-12-73	Change the fill zone designation	[illegible]	JNG	RwD
2	5-22-73	Change bottom width for DI-DI and added cutoff wall for impact basin	[illegible]	EEC	RwD
1	3-30-73	Miscellaneous revisions	[illegible]	EEC	RwD

OHIO ELECTRIC COMPANY
GAVIN POWER PROJECT

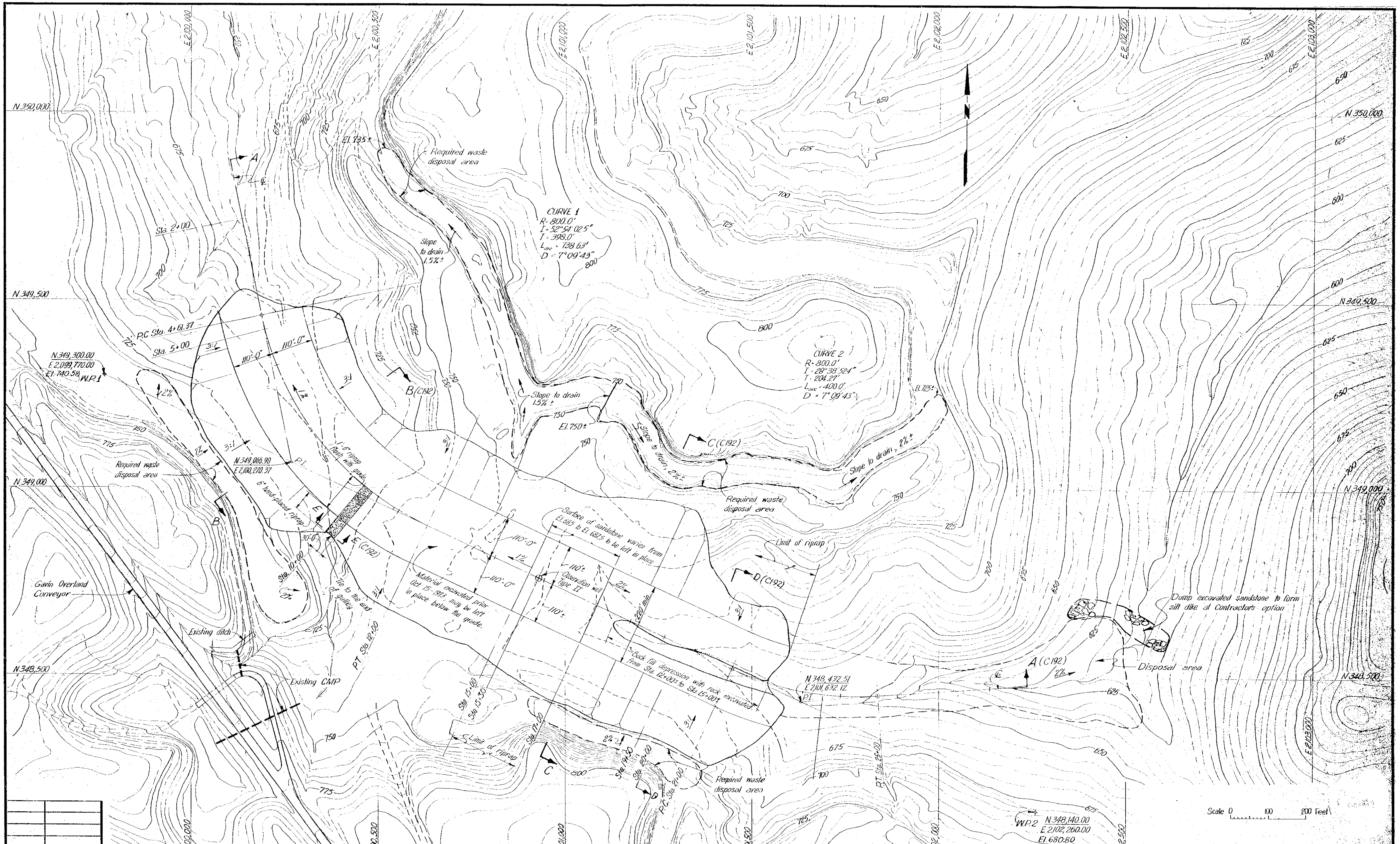
STINGY RUN
FLY ASH DAM

DAM

SECTIONS AND DETAILS

HARZA ENGINEERING COMPANY
APPROVED Richard D. Harza

CHICAGO, ILLINOIS
DATE MAR., 1973
DWG. NO 670-C183 R7



REFERENCE DRAWINGS:

Geology 670 C 101, 110-112
670 C 121-138, 141-143
Soil Tests 670 C 151-153, 156
670 C 161-164, 167
Dam 670 C 181-187

NOTES:

1. Work this drawing with 670 C 192 and 193
2. The topography on this sheet is not to be used for payment purposes. See Note 4 on 670 C 193
3. Disposal area at downstream limit of emergency spillway to be used only after required disposal areas are filled.

SURVEY CONTROL

Working points 1 and 2 were established by the Ohio Power Company. These points shall be used to establish construction survey by the Contractor.

DATE	LETTER NO.
DWG. TRANSMITTAL	

	BY	DATE	CHKD.	DATE
DSGN.	EEC	MARCH 73	BL	3/14/73
DWN.		MARCH 73	EEC	MARCH 73
DIV.	GROUP LDR.	SECT. HEAD	DEPT. HEAD	DIV. HEAD
CIVIL	BL			
MECH.				
ELECT.				
PLAN.				
STAFF		BR. HEAD	CH. ENGR.	

REV. NO.	DATE	NATURE OF REVISION	BY	CHKD.	APPD.
4	5/1/74	Construction revision			
3	10/29/73	Raised the invert, widened the spillway and deleted riprap on the bottom		BL	
2	9/20	Raised the invert elevation			
1	4-18-73	Added working points elevation	LDF	EEC	

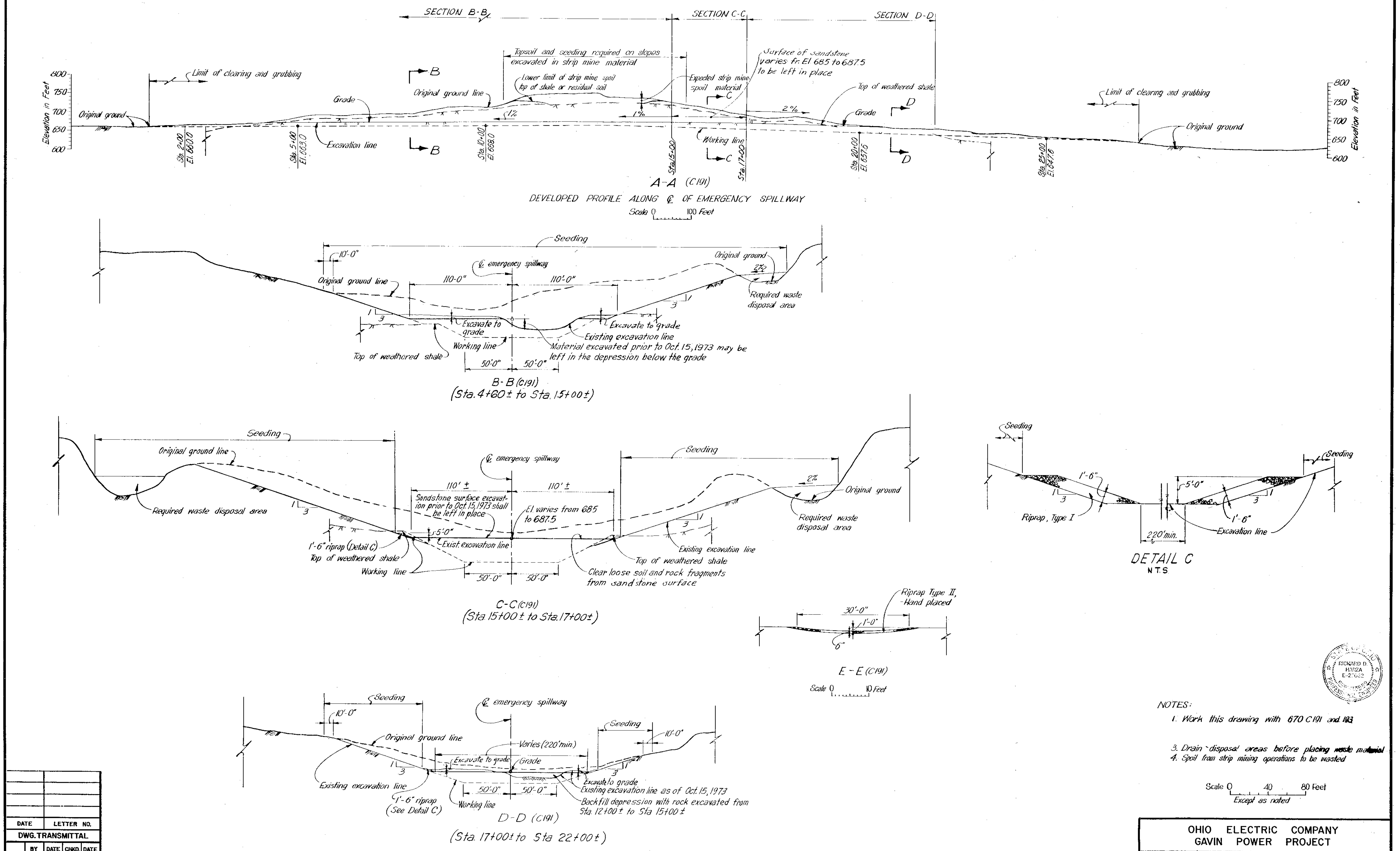
OHIO ELECTRIC COMPANY
GAVIN POWER PROJECT

STINGY RUN
FLY ASH DAM

OUTLET WORKS

EMERGENCY SPILLWAY
PLAN

HARZA ENGINEERING COMPANY
APPROVED Richard D. Harza
CHICAGO, ILLINOIS
DATE MAR., 1973
DWG. NO. 670 C 191 R4



A-A (C191)

DEVELOPED PROFILE ALONG ℄ OF EMERGENCY SPILLWAY

Scale 0 100 Feet

B-B (C191)
(Sta. 4+60 ± to Sta. 15+00 ±)

C-C (C191)
(Sta 15+00 ± to Sta. 17+00 ±)

D-D (C191)
(Sta. 17+00 ± to Sta 22+00 ±)

DETAIL C
N.T.S.

E-E (C191)
Scale 0 10 Feet

RICHARD D. HARZA
E-27062

NOTES:

1. Work this drawing with 670 C191 and 193

3. Drain disposal areas before placing waste material
4. Spoil from strip mining operations to be wasted

Scale 0 40 80 Feet
Except as noted

DATE	LETTER NO.

DWG. TRANSMITTAL

	BY	DATE	CHKD.	DATE
DSGN.	EEC	MARCH 73	BL	3/14/73
DWN.		MARCH 73	EEC	MARCH 73
DIV.	GROUP LDR.	SECT. HEAD	DEPT. HEAD	DIV. HEAD
CIVIL	BL			
MECH.				
ELECT.				
PLAN.				

STAFF	BR. HEAD	CH. ENGR.

REV. NO.	DATE	NATURE OF REVISION	BY	CHKD.	APPD.
4	5/1/74	Construction revision			
3	10/29/73	Raised invert, widened channel, deleted riprap at the bottom	ECL	BL	
2	10/1/73	Miscellaneous		JWG	
1	9/11/73	Raised the invert by 10 feet, change excavation and riprap detail		BL	

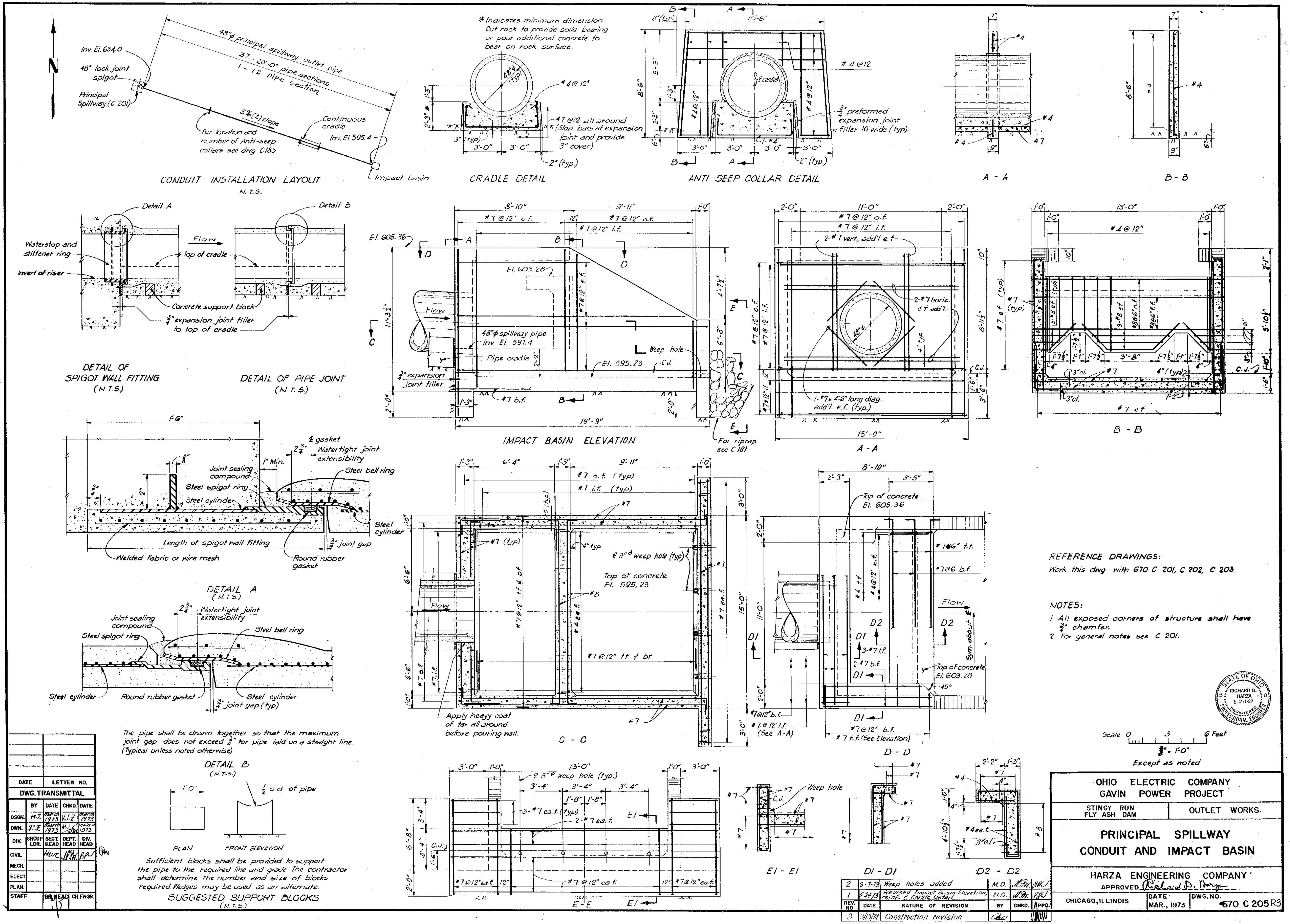
OHIO ELECTRIC COMPANY
GAVIN POWER PROJECT

STINGY RUN
FLY ASH DAM

OUTLET WORKS

EMERGENCY SPILLWAY
SECTIONS

HARZA ENGINEERING COMPANY
APPROVED Richard D. Harza
CHICAGO, ILLINOIS
DATE MAR., 1973
DWG. NO. 670 C 192 R4



The pipe shall be drawn together so that the maximum joint gap does not exceed ¾" for pipe laid on a straight line. (Typical unless noted otherwise)

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

REFERENCE DRAWINGS:
Work this dwg with 670 C 201, C 202, C 203.

NOTES:
1. All exposed corners of structure shall have ¾" chamfer.
2. For general notes see C 201.

RICHARD D. HARZA
E-27062

Scale 0 3 6 Feet
⅜" = 1'-0"
Except as noted

REV. NO.	DATE	NATURE OF REVISION	BY	CHKD.	APPD.
2	6-7-73	Weep holes added	M.D.	M.Pr.	RDH
1	5-24-73	Revised Impact Basin Elevation, reinf. & Cradle Detail	M.D.	M.Pr.	RDH
3	3/13/74	Construction revision			

OHIO ELECTRIC COMPANY
GAVIN POWER PROJECT

STINGY RUN FLY ASH DAM
OUTLET WORKS.

PRINCIPAL SPILLWAY
CONDUIT AND IMPACT BASIN

HARZA ENGINEERING COMPANY
APPROVED Richard D. Harza
CHICAGO, ILLINOIS
DATE MAR., 1973
DWG. NO. 670 C 205 R3

DATE	LETTER NO.
DWG. TRANSMITTAL	

	BY	DATE	CHKD.	DATE
DSGN.	M.S.	March 1973	V.J.Z.	March 1973
DWN.	P.F.	March 1973	M.J.	MARCH 1973
DIV.	GROUP LDR.	SECT. HEAD	DEPT. HEAD	DIV. HEAD
CIVIL		Hwz	M.Pr.	RDH
MECH.				
ELECT.				
PLAN.				
STAFF		BR. HEAD	CH. ENGR.	

$3 Million

Eng. O.H. & Field Supv. – 25% $3.75

[illegible] 10% .37

$4.12

Say $4 Mill.

FINAL REPORT

GAVIN FLY ASH DAM
AND
FLY ASH LINE SUPPORT SYSTEM

VOLUME II

1975

GAVIN FLY ASH DAM APPENDICES

APPENDIX A

DESIGN REVISION NO. 1 REPORT

OHIO ELECTRIC COMPANY
GAVIN POWER PROJECT
GAVIN (STINGY RUN)
FLY ASH DAM
Supplement
To
FINAL DESIGN REPORT

PREPARED BY
HARZA ENGINEERING COMPANY
OCTOBER 1973

INDEX

Introduction

This report is a supplement to the final design report submitted April, 1973, with the application for a construction permit in accordance with the requirements of Rules NRD-5-03 through NRD-5-07 of the Administrative Rules for Issuing Construction Permits for Dams, Dikes and Levees. The purpose of this Report is to describe and analyze a revised first stage dam permitting storage of water to an elevation 10 feet higher than that described in the Report of April, 1973.

A preliminary report in conjunction with the April application was submitted to the Ohio Department of Natural Resources, Division of Water, by Ohio Electric Company in February, 1973. Approval of the preliminary report was given in a letter from Mr. Roy Winkle, Chief, Division of Water on March 12, 1973. A construction permit No. 73-68 was issued by the Division on June 5, 1973, for stage 1, which permitted storage to elevation 670.

Covered in this report are the following major items:

1) Description of the revisions in the first stage dam.
2) Stability studies for the revised dam.
3) Cost estimate of the revised project.

The retention dam is designed to be capable of being built in three stages over a period of several years, with the highest stage providing liquid storage to Elevation 725. This application for a revised construction permit applies only to the revised first stage dam with a maximum nominal pool elevation of 680. In references to the dam in this report, only the revised first stage dam is being considered, unless otherwise stated.

Revised Dam

The revised dam will be built to a final settled elevation of 688 feet for a total estimated height of 99 feet from the stripped ground surface. The maximum nominal water surface elevation is 680. Based upon Section NRD-13-01 of the Ohio Division of Water's "Administrative Rules," the dam will fall into Class I.

The dam will be essentially an earth fill structure and will have 3h:1v slopes with 8h:1v stabilizing berms on both the upstream and downstream sides. These berms will reach a settled elevation of 640±.

A 30 ft. wide berm with a settled elevation of 670± has been added to the upstream slope of the dam.

Drawings 670C 182R5 and 183R5, enclosed, show the revised dam plan, sections and details. (Appendix D)

Soil Strength Parameters

Strength parameters are unchanged from those used in the April, 1973 Report except that the required excavation has provided insufficient shale for construction of the rolled shale zone in the dam, making it necessary to change the random fill and downstream berm to impervious material, and requiring the use of corresponding strengths. A summary of the effective shear strengths that were used for the revised stability analyses is given in Table 1.

Table 1

Material	Unconsolidated Strengths		Consolidated Strengths	
	Ø*	c'**	Ø'*	c'**
Foundation	12	300	23	0
Impervious Fill	15	400	22	0
Core Trench	10	500	20	300
Upstream Berm	1	0	1	0
Sandstone	32	0	32	0

* Effective angle of internal friction in degrees.

** Effective cohesion in pounds per square foot.

Stability

Because of the change in materials for the dam and the increase in the stage 1 reservoir elevation to 680, the stability study of the April 1973 Report is revised.

In the revision, the effective shear strength parameters stated in Table 1 were used. Table 2 summarizes the minimum safety factors computed for four different loading conditions of the revised stage 1 and of each of the two possible future stages.

The Morgenstern-Price stability program was used to determine the minimum factor of safety for each case studied. The stability of all three stages was investigated. The individual studies are presented in Appendix A, Exhibits 1 - 9.

Table 2 shows that the lowest factor of safety exists during first stage construction except for earthquake with full reservoir. The construction condition usually is more critical for a dam built on a soft foundation than any other steady state condition.

Table 2

Study	Construction Stage	Case	Minimum Factor of Safety
1	1	Construction case upstream (Reservoir empty)	1.18
2	1	Construction case downstream (Reservoir empty)	1.16
3	1	Steady seepage downstream (Reservoir El. 680)	1.57
4	1	Steady seepage downstream (with 0.05g earthquake) (Reservoir El. 680)	1.21
5	2	Construction case upstream (Reservoir El. 680)	1.40
6	2	Construction case downstream (Reservoir El. 680)	1.27
7	2	Steady seepage downstream (Reservoir El. 705)	1.43
8	2	Steady seepage downstream (with 0.05g earthquake) (Reservoir El. 705)	1.14
9	3	Construction case upstream (Reservoir El. 705)	1.54
10	3	Construction case downstream (Reservoir El. 705)	1.44
11	3	Steady seepage downstream (Reservoir El. 725)	1.35
12	3	Steady seepage downstream (with 0.05g earthquake) (Reservoir El. 725)	1.12

Freeboard

For the nominal water surface elevation of 670, flood routing studies (see preliminary report) have shown that the maximum water surface due to the probable maximum flood is Elevation 677.6. No flood routing studies were done for the revised dam height. However, by changing the water surface elevation to 680 the probable maximum flood would crest at an elevation lower than elevation 687.6 because of the increased reservoir area. The reservoir area increases from 164 acres at El. 670 to 206 acres at El. 680. A settled dam crest of elevation 688 is, therefore, adequate. Computations for the determination of settlement of the dam crest are shown in Appendix C.

Emergency Spillway

The emergency spillway for the initial stage of development is designed to pass safely the maximum probable flood, which produces a maximum reservoir inflow computed as 9,200 cfs. Flood routing studies were presented in the preliminary report, and showed maximum spillway outflow of 4,428 cfs. Because of the larger reservoir area the maximum outflow will be reduced, although the spillway crest length has not been shortened. The emergency spillway, located on the right bank 1,800 feet from the right abutment of the dam, will be of the broad-crested weir type. The inlet slope will be 1% and the outlet slope 2%. The width of the spillway will remain a constant 100 feet throughout its length. A 30-foot long crest will be at a constant elevation of 681, one foot higher than the maximum normal water surface of 680. The side slopes will be 2h:1v in rock and 3h:1v in overburden. Eighteen inches of riprap will be provided at the crest and downstream of the crest to prevent erosion. Seeding will be provided on all the remaining excavated areas. The revised spillway is shown on drawings 670C 191R2 and 192R2, included with this report. (Appendix D)

Cost Estimate

The final detailed cost estimate of the dam and all appurtenances thereto is estimated to be $2,463,964, and is presented in Appendix B.

Summary

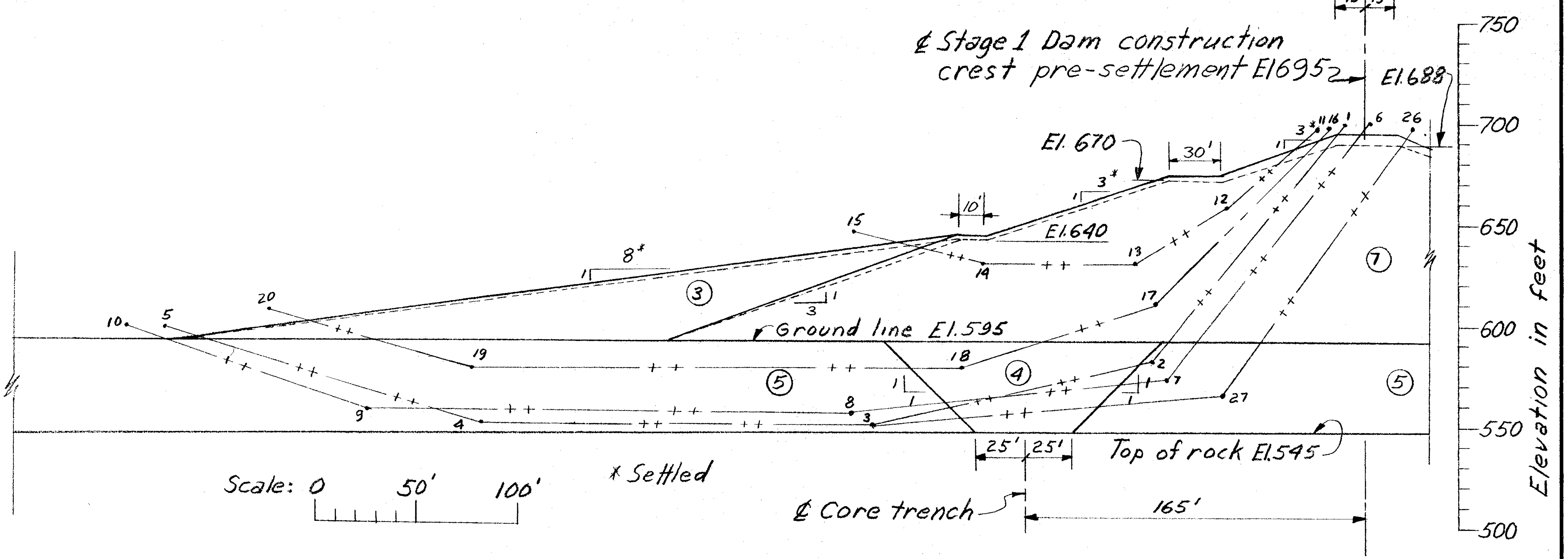
This supplement to the final report of April, 1973 is to be used in conjunction with the final report of April, 1973, the preliminary report, the technical specifications dated April, 1973 (Volume I of the Contract Documents) and the contract drawings dated April, 1973 (Volume II of the Contract Documents). All drawing that needed revision due to the changes as mentioned herein have been revised and are included with this report.

In this supplement to the final report, the subjects in the final report were not restated here unless appropriate or unless revisions were made in which case this report will govern.

APPENDIX A

Stability Analysis

APPENDIX A, EXHIBIT 1



Potential Failure Surface	Points Defining Failure Surface	Factor of Safety
1	1-2-3-4-5	1.25
2	6-7-8-9-10	1.22
3	11-12-13-14-15	1.61
4	16-17-18-19-20	1.36
5	26-27-3-4-5	1.18 ←

Soil Parameters

Material Type	γ_{wet} #/ft³	γ_{sat} #/ft³	C' #/ft²	Ø'	Pore Pressure
⑦ Impervious fill*	130	131	400	15°	20%
⑤ Foundation *	130	131	300	12°	40%
③ Upstream Berm*	130	131	0	1°	0
④ Core Trench *	130	131	500	10°	40%

* Unconsolidated Strength

Notes:
1. Method of analysis: Morgenstern-Price Computer Program
2. Reservoir Empty
3. Type of analysis for soil parameters: Effective Stress.
4. Post settlement dam crest elevation 688.

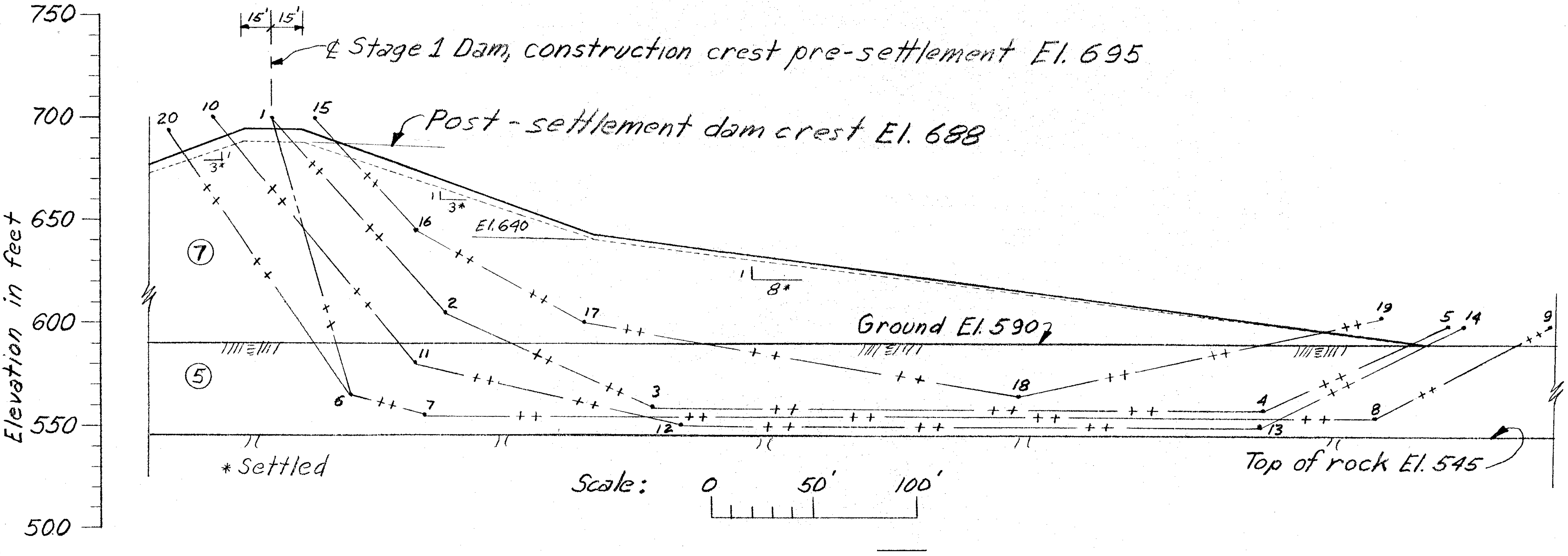
GAVIN POWER PROJECT
STINGY RUN FLY ASH DAM

CONSTRUCTION CASE
UPSTREAM - STAGE 1

HARZA ENGINEERING CO., CHICAGO
APPROVED........................
DATE OCT. 1973 DWG.NO. 6705KC301

R1

APPENDIX A, EXHIBIT 2



Potential Failure Surface	Points Defining Failure Surface	Factor of Safety
1	1-2-3-4-5	1.27
2	1-6-7-8-9	1.33
3	10-11-12-13-14	1.16
4	15-16-17-18-19	1.48
5	20-6-7-8-9	1.17

Soil Parameters

Material Type	γwet #/ft³	γsat #/ft³	C' #/ft²	Ø'	Pore Pressure
⑦ Impervious fill*	130	131	400	15°	20%
⑤ Foundation*	130	131	300	12°	40%

*Unconsolidated Strength

Notes:
1) Method of analysis: Morgenstern-Price Computer Program
2) Reservoir Empty
3) Type of analysis for soil parameters: Effective Stress.

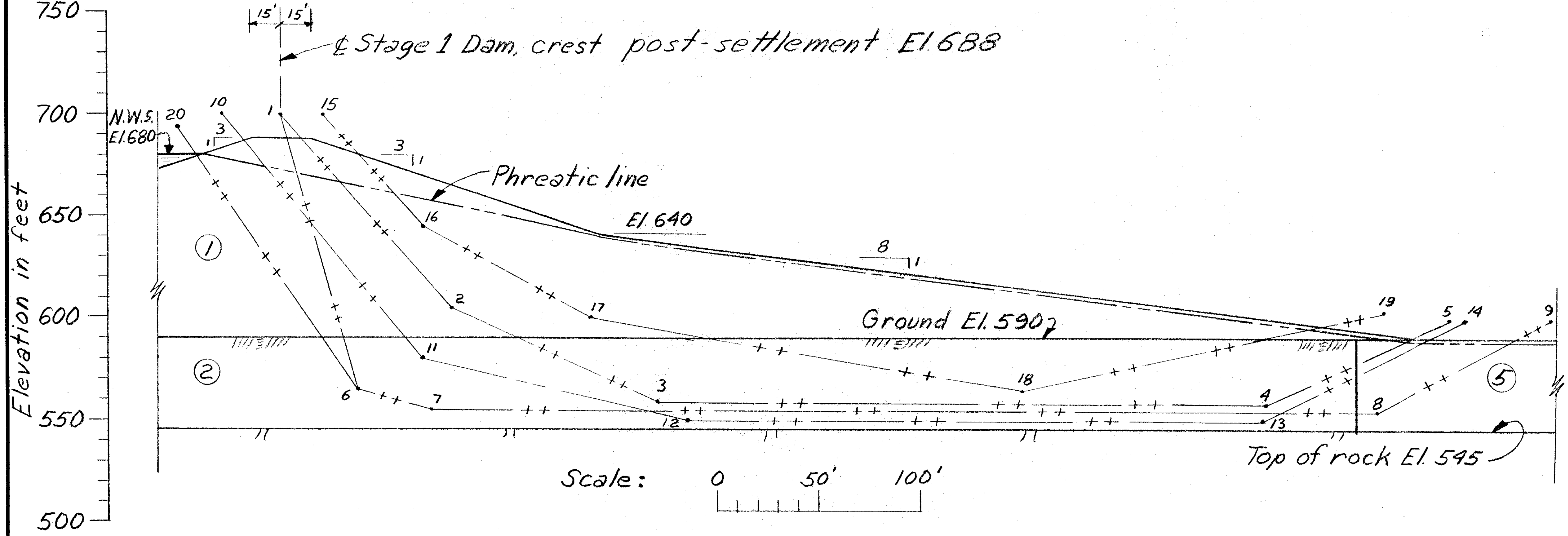
GAVIN POWER PROJECT
STINGY RUN FLY ASH DAM

CONSTRUCTION CASE
DOWNSTREAM - STAGE 1

HARZA ENGINEERING CO., CHICAGO
APPROVED........................
DATE OCT. 1973 DWG.NO. 6705KC302

R1

2



Potential Failure Surface	Points Defining Failure Surface	Factor of Safety Without Earthquake	Factor of Safety With Earthquake
1	1-2-3-4-5	1.62	1.23
2	1-6-7-8-9	1.86	1.41
3	10-11-12-13-14	1.57	1.21
4	15-16-17-18-19	1.61	1.22
5	20-6-7-8-9	1.65	1.27

Soil Parameters					
Material Type	γ_{wet} #/ft³	γ_{sat} #/ft³	C' #/ft²	Ø'	Pore Pressure
① Impervious fill ‡	130	131	0	22°	from Phreatic line
② Foundation ‡	130	131	0	23°	"
⑤ Foundation *	130	131	300	12°	"

‡ Consolidated Strength
* Unconsolidated Strength

Notes:
1) Method of analysis: Morgenstern - Price Computer Program.
2) Reservoir El. 680
3) Earthquake factor equal to 0.05 g.
4) Type of Analysis: for soil parameters: Effective Stress

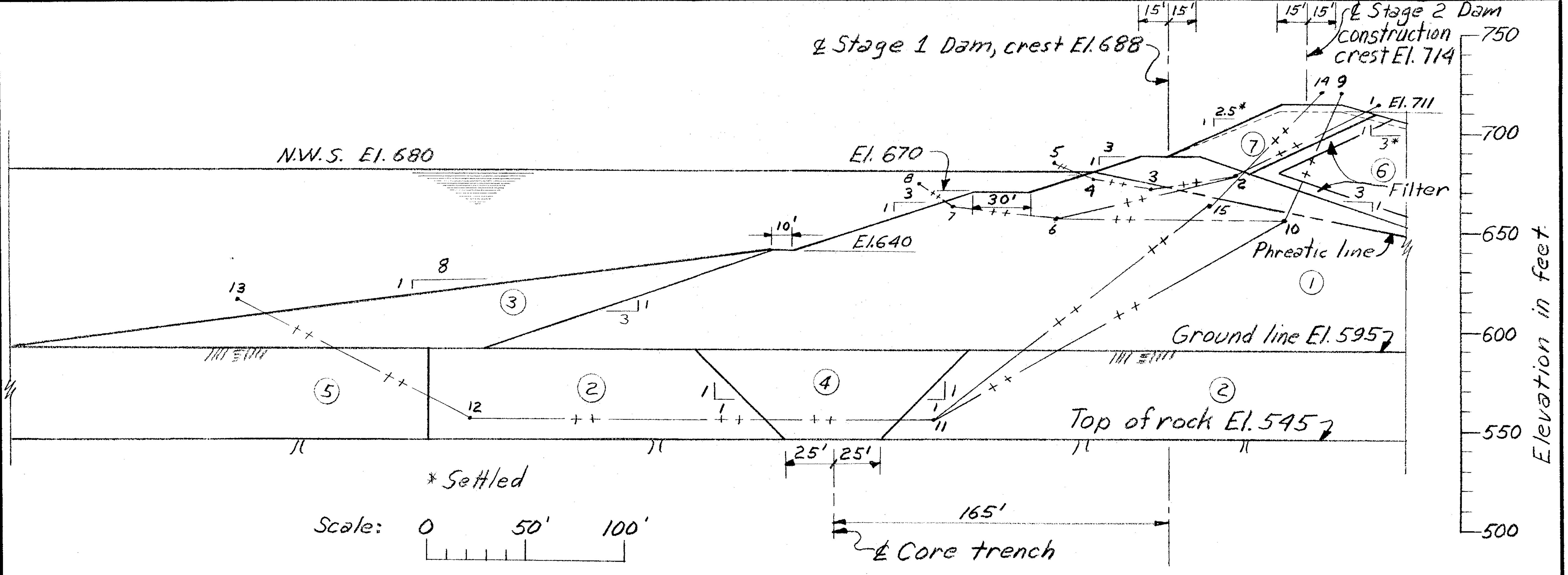
GAVIN POWER PROJECT
STINGY RUN FLY ASH DAM

STEADY SEEPAGE CASE
DOWNSTREM - STAGE 1

HARZA ENGINEERING CO., CHICAGO
APPROVED........................
DATE OCT. 1973 DWG.NO. 670SKC303

R-1

APPENDIX A, EXHIBIT 4



Potential Failure Surface	Points Defining Failure Surface	Factor of Safety
1	1-2-3-4-5	1.64
2	1-2-6-7-8	1.40 ←
3	9-10-6-7-8	2.30
4	9-10-11-12-13	1.40
5	14-15-11-12-13	1.40

Soil Parameters					
Material Type	γ_{wet} #/ft³	γ_{sat} #/ft³	C' #/ft²	Ø'	Pore Pressure
① Impervious fill ‡	130	131	0	22°	from Phreatic line
② Foundation ‡	130	131	0	23°	"
③ Upstream Berm ‡	130	131	0	1°	"
④ Core trench ‡	130	131	300	20°	"
⑤ Foundation *	130	131	300	12°	"
⑥ Sandstone *	130	131	0	32°	0
⑦ Impervious fill *	130	131	400	15°	20%

‡ Consolidated Strength
* Unconsolidated Strength

Notes:
1) Method of analysis: Morgenstern-Price Computer Program
2) Reservoir El. 680
3) Type of Analysis for soil parameters: Effective Stress.
4) Post-settlement dam crest El. 711

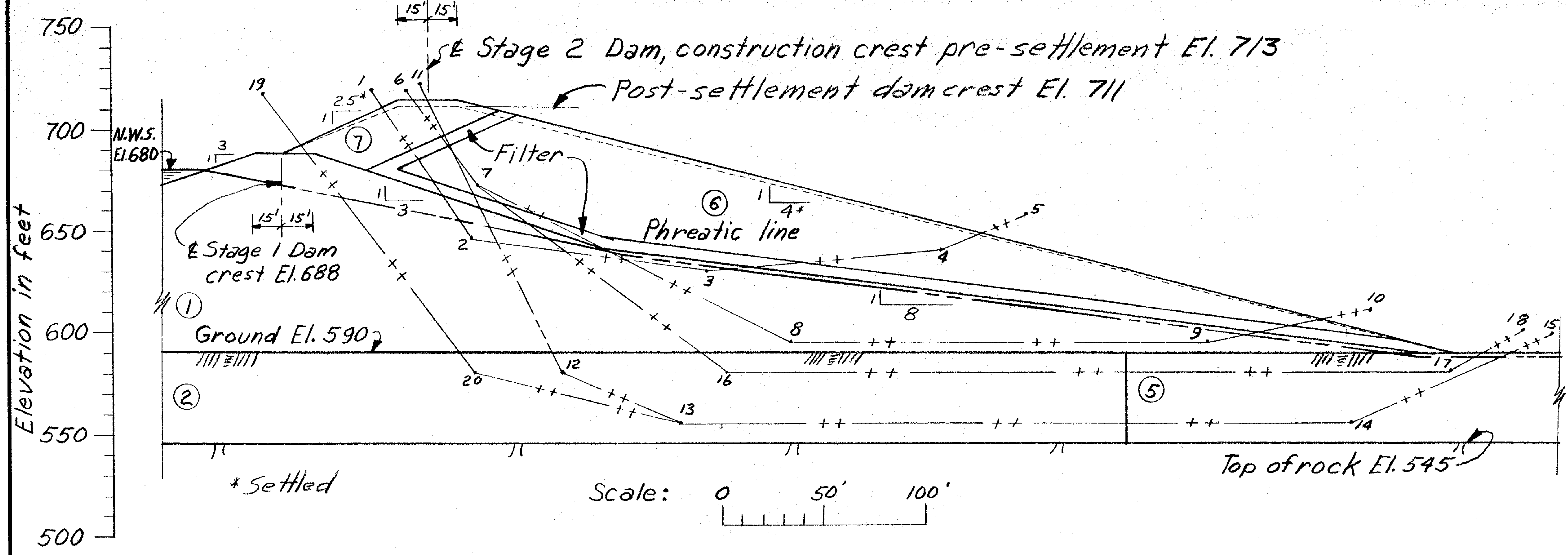
GAVIN POWER PROJECT
STINGY RUN FLY ASH DAM

CONSTRUCTION CASE
UPSTREAM - STAGE 2

HARZA ENGINEERING CO., CHICAGO
APPROVED........................
DATE OCT. 1973 DWG.NO. 670 SKC 304

R1

APPENDIX A, EXHIBIT 5



Potential Failure Surface	Points Defining Failure Surface	Factor of Safety
1	1-2-3-4-5	1.91
2	6-7-8-9-10	1.34
3	6-7-16-17-18	1.27
4	11-12-13-14-15	1.32
5	19-20-13-14-15	1.36

Soil Parameters

Material type	γwet #/ft³	γsat #/ft³	C' #/ft²	Ø'	Pore Pressure
① Impervious fill ‡	130	131	0	22°	from Phreatic line
② Foundation ‡	130	131	0	23°	〃
⑤ Foundation *	130	131	300	12°	〃
⑥ Sandstone *	130	131	0	32°	0
⑦ Impervious fill *	130	131	400	15°	20%

‡ Consolidated strength
* Unconsolidated strength

Notes:
1) Method of analysis: Morgenstern Price Computer Program
2) Reservoir El. 680
3) Type of Analysis for soil parameters: Effective Stress.

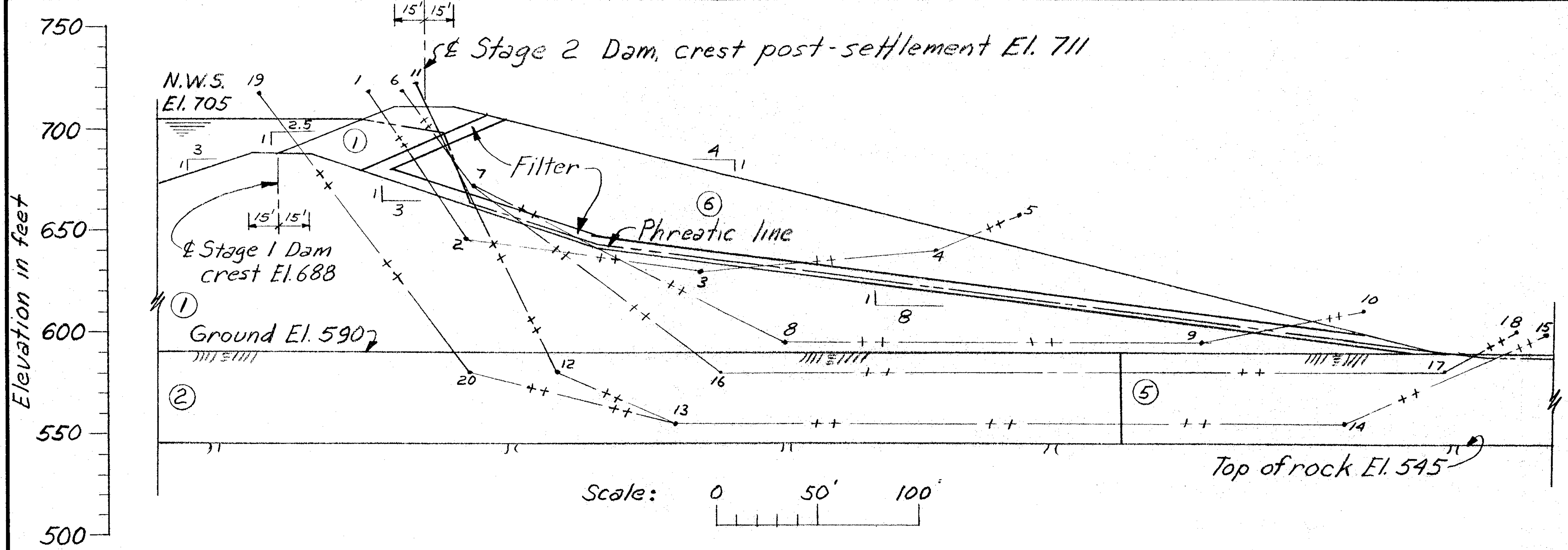
GAVIN POWER PROJECT
STINGY RUN FLY ASH DAM

CONSTRUCTION CASE
DOWNSTREAM - STAGE 2

HARZA ENGINEERING CO., CHICAGO
APPROVED........................
DATE OCT. 1973 DWG.NO. 670SKC305

R1

APPENDIX A, EXHIBIT 6



Potential Failure Surface	Points Defining Failure Surface	Factor of Safety Without Earthquake	Factor of Safety With Earthquake
1	1-2-3-4-5	2.45	1.99
2	6-7-8-9-10	1.78	1.45
3	6-7-16-17-18	1.48	1.21
4	11-12-13-14-15	1.52	1.23
5	19-20-13-14-15	1.43	1.14 ←

Soil Parameters					
Material Type	γ_{wet} #/ft³	γ_{sat} #/ft³	C′ #/ft²	ϕ'	Pore Pressure
① Impervious fill ǂ	130	131	0	22°	from Phreatic line
② Foundation ǂ	130	131	0	23°	"
⑤ Foundation *	130	131	300	12°	"
⑥ Sandstone ǂ	130	131	0	32°	0

ǂ Consolidated Strength
* Unconsolidated Strength

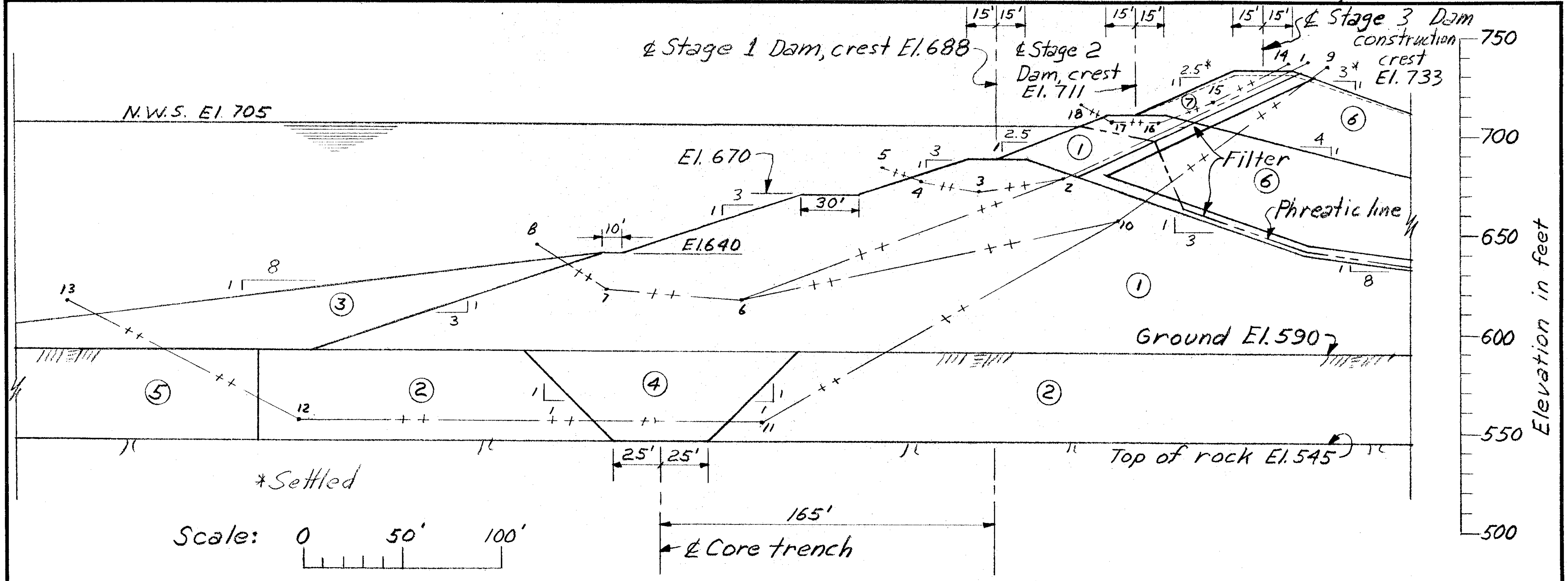
Notes:
1) Method of analysis: Morgenstern Price Computer Program.
2) Reservoir El. 705
3) Earthquake factor equal to 0.05g.
4) Type of Analysis for soil parameters Effective stress

GAVIN POWER PROJECT
STINGY RUN FLY ASH DAM

STEADY SEEPAGE CASE
DOWNSTREAM-STAGE 2

HARZA ENGINEERING CO., CHICAGO
APPROVED........................
DATE OCT. 1973 DWG.NO. 6705KC306
R1

APPENDIX A, EXHIBIT 7



Potential Failure Surface	Points Defining Failure Surface	Factor of Safety
1	1-2-3-4-5	1.54 ←
2	1-2-6-7-8	1.72
3	9-10-6-7-8	1.79
4	9-10-11-12-13	2.34
5	14-15-16-17-18	2.13

Soil Parameters

Material type	γ_{wet} #/ft³	γ_{sat} #/ft³	C' #/ft²	Ø'	Pore Pressure
① Impervious fill ‡	130	131	0	22°	from Phreatic line
② Foundation ‡	130	131	0	23°	"
③ Upstream Berm ‡	130	131	0	1°	"
④ Core Trench ‡	130	131	300	20°	"
⑤ Foundation *	130	131	300	12°	"
⑥ Sandstone *	130	131	0	32°	0
⑦ Impervious fill *	130	131	400	15°	20%

‡ Consolidated Strength
* Unconsolidated Strength

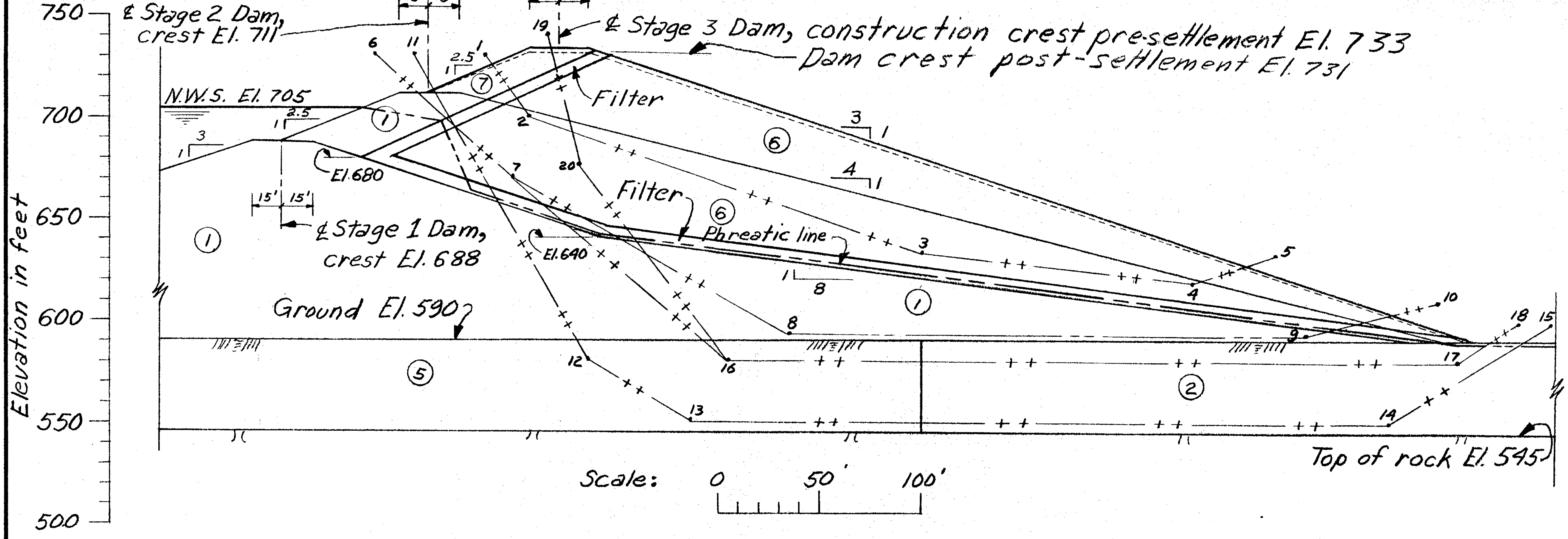
Notes:
1) Method of analysis: Morgenstern-Price Computer Program
2) Reservoir El. 705
3) Type of Analysis for soil parameters: Effective Stress.
4) Post-settlement dam crest El. 731

GAVIN POWER PROJECT
STINGY RUN FLY ASH DAM

CONSTRUCTION CASE
UPSTREAM - STAGE 3

HARZA ENGINEERING CO., CHICAGO
APPROVED..........................
DATE OCT. 1973
DWG.NO. 670 SKC 307
R1

APPENDIX A, EXHIBIT 8



Potential Failure Surface	Points Defining Failure Surface	Factor of Safety
1	1-2-3-4-5	2.36
2	6-7-8-9-10	1.87
3	6-7-16-17-18	1.46
4	11-12-13-14-15	1.48
5	19-20-16-17-18	1.44 ←

Soil Parameters					
Material Type	γwet #/ft³	γsat #/ft³	C' #/ft²	Ø'	Pore Pressure
① Impervious fill ‡	130	131	0	22°	from Pheatic line
② Foundation ‡	130	131	0	23°	"
⑤ Foundation *	130	131	300	12°	"
⑥ Sandstone *	130	131	0	32°	0
⑦ Impervious fill *	130	131	400	15°	20%

‡ Consolidated Strength

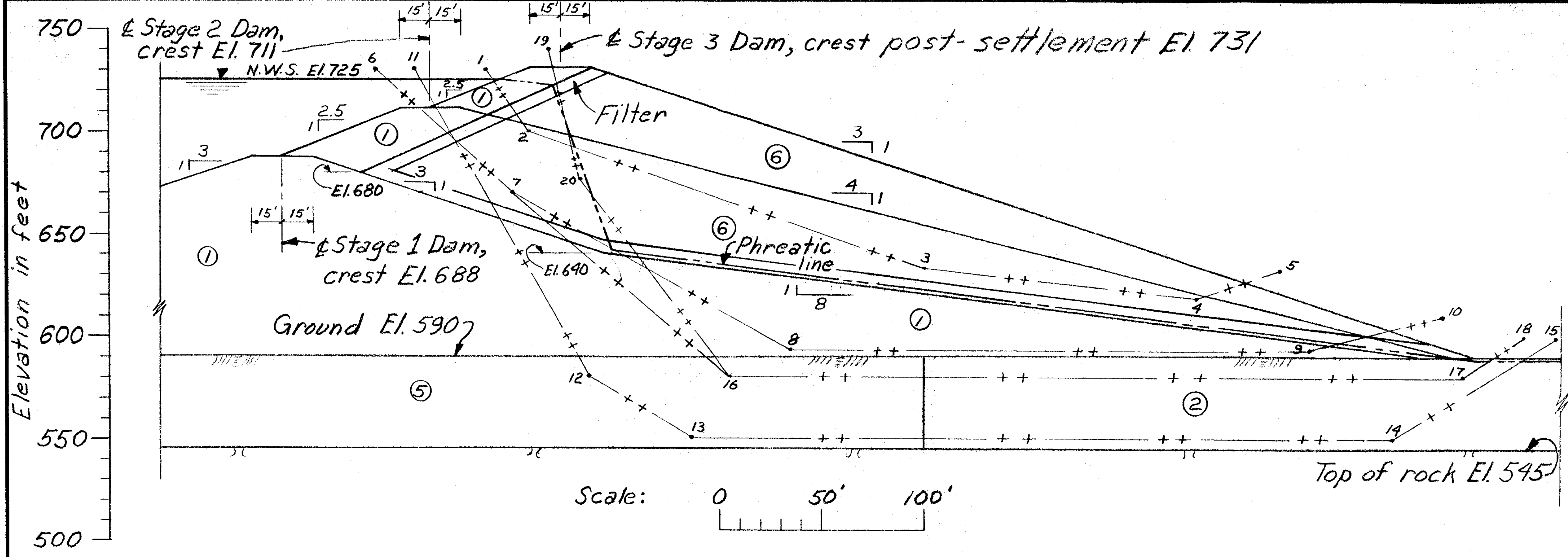
* Unconsolidated Strength

Notes:
1) Method of analysis: Morgenstern-Price Computer Program
2) Reservoir El. 705
3) Type of Analysis for soil parameters: Effective Stress

GAVIN POWER PROJECT
STINGY RUN FLY ASH DAM

CONSTRUCTION CASE
DOWNSTREAM-STAGE 3

HARZA ENGINEERING CO., CHICAGO
APPROVED..........................
DATE OCT. 1973 DWG.NO. 670SKC308
R1



Potential Failure Surface	Points Defining Failure Surface	Factor of Safety Without Earthquake	Factor of Safety With Earthquake
1	1-2-3-4-5	2.37	2.00
2	6-7-8-9-10	1.82	1.51
3	6-7-16-17-18	1.39	1.15
4	11-12-13-14-15	1.35	1.12 ←
5	19-20-16-17-18	1.45	1.23

Soil Parameters					
Material Type	γ_{wet} #/ft³	γ_{sat} #/ft³	C' #/ft²	Ø'	Pore Pressure
① Impervious fill ‡	130	131	0	22°	from Phreatic line
② Foundation ‡	130	131	0	23°	"
⑥ Sandstone ‡	130	131	0	32°	0
⑤ Foundation *	130	131	300	12°	from Phreatic line

‡ Consolidated Strength
* Unconsolidated Strength

Notes:
1) Method of analysis: Morgenstern-Price Computer Program
2) Reservoir El. 711
3) Earthquake factor equal to 0.05g.
4) Type of Analysis for soil parameters Effective stress.

GAVIN POWER PROJECT
STINGY RUN FLY ASH DAM

STEADY SEEPAGE CASE
DOWNSTREAM-STAGE 3

HARZA ENGINEERING CO., CHICAGO
APPROVED........................
DATE OCT. 1973 DWG.NO. 670 SKC 309
R1

APPENDIX B

Schedule of Bid Items

Appendix B

REVISED SCHEDULE OF BID ITEMS

Item Number	Reference Section	Description	Quantity	Unit	Unit Price	Amount
1	1	Diversion and Care of Water	all	lump sum		15,000
2	2	Clearing and Grubbing	55'	acre	400.00	22,000
2a		Reservoir Clearing	162	acre	300.00	48,600
3	2	Transportation of Core Boxes		lump sum		2,200
4	3	Stripping	76,000	cu yd	0.60	45,600
5	3	Principal Spillway Excavation	50,000	cu yd	1.50	75,000
6	3	Dam Excavation	115,000	cu yd	1.70	195,500
7	3	Emergency Spillway Excavation	730,000	cu yd	0.68	496,400
8	3	Borrow Excavation	905,000	cu yd	0.58	524,900
9	3	Dental Excavation	100	cu yd	20.00	2,000
9a	3	Resin Coating	700	sq yd	1.35	945
10	4	Impervious Fill	575,000	cu yd	0.10	57,500
10a	4	Upstream Berm Fill	150,000	cu yd	0.40	60,000
11	4	Rolled Shale		cu yd	0.10	0
12	4	Random Fill	900,000	cu yd	0.10	90,000
13	4	Bedding	8,800	cu yd	10.00	88,000
14	4	Riprap - Type I	8,000	cu yd	10.50	84,000
15	4	Riprap - Type II	14,000	cu yd	11.00	154,000
16	4	Riprap, Type II, Hand Placed	400	cu yd	25.00	10,000
17	4	Road Surfacing	1,400	cu yd	12.50	17,500
18	4	Topsoil	550	cu yd	5.00	2,750
19	4	Additional Rolling for Compaction	300	1000 sq yd	12.00	3,600
20	4	Commercial Fertilizer	19	ton	140.00	2,660

Item Number	Reference Section	Description	Quantity	Unit	Unit Price	Amount
21	4	Agriculture Liming	95	ton	45.00	4,275
22	4	Seeding and Mulching	190,000	sq yd	0.12	22,800
23	4	Surface Settlement Monument	3	No.	200.00	600
24	4	Settlement Gage	3	No.	1500.00	4,500
25	4	PVC Pipe - Perforated	360	lin. ft.	3.50	1,260
26	4	PVC Pipe - Unperforated	180	lin. ft.	2.25	405
27	5	Setting Up Over Grout, Check, Exploratory and Observation Well Holes to be Drilled	170	No.	30.00	5,100
28	5	Drilling AX Holes	6,500	lin. ft.	5.00	32,500
29	5	Drilling NX Holes	280	lin. ft.	8.95	2,506
30	5	Drilling 4" Overburden Holes	260	lin. ft.	8.80	2,288
31	5	Core Recovery, "AX" Holes	6,500	lin. ft.	3.00	19,500
32	5	Core Recovery, "NX" Holes	280	lin. ft.	6.00	1,680
33	5	Drilling Grout from Holes	3,300	lin. ft.	5.00	16,500
34	5	Connections to Drilled Grout Holes	170	No.	15.00	2,550
35	5	Processing, Mixing, and Injecting Pressure Grout	10,000	No. of 94# Sack	3.30	33,000
36	5	Cement in Grout	10,000	No. of 94# Sack	3.00	30,000
37	5	Sand in Grout	1,000	cu yd	20.00	20,000
38	5	Fly Ash in Grout	80,000	lbs	0.20	16,000
39	5	Bentonite in Grout	8,000	lbs	0.20	1,600
40	5	Pressure Tests	325	lumber	40.00	13,000

Item Number	Reference Section	Description	Quantity	Unit	Unit Price	Amount
41	6	Corrugated Metal Pipe 48" Dia.	120	in ft	50.00	6,000
42	6	Pipe Bedding	70	cu yd	12.00	840
43	7	Concrete, 2500 psi	130	cu yd	60.00	7,800
44	7	Concrete, 4000 psi	650	cu yd	65.00	42,250
45	7	Concrete, 6000 psi	10	cu yd	150.00	1,500
46	7	Grade 60, Steel Reinforcement	100,000	lbs	0.22	22,000
47	7	Straight Forms	14,500	sq ft	4.00	58,000
48	7	Curved Forms	1,650	sq ft	7.50	12,375
49	8	Prestressed Pressure Pipe Conduit - 48 inch	760	lin ft	100.00	76,000
50	8	Preformed Expansion Joint Filler	1,650	lbs	1.20	1,980
51	9	Miscellaneous Metalwork	5,000	lbs	1.00	5,000
52	9	Floating Platforms and Skimmer		lump sum		2,000
		TOTAL				$2,463,964

APPENDIX C

Settlement Computations

HARZA ENGINEERING COMPANY CHICAGO	SUBJECT Crest and Constr. El.	PROJECT FLY ASH DAM
		FILE NO. 6706
	COMPUTED JNG CHECKED JLM	DATE Oct 9, 73 PAGE 1 OF 2 PAGES

PURPOSE :-

Compute the crest and construction elevation at NWS. El. 680

CREST ELEVATION -

Use 688

See the attached APPX 1

CONSTRUCTION ELEVATION

REFERENCE FINAL DESIGN REPORT APPX. C.

Ground El.	590±
Crest El.	679
Const El.	685

Constr. surcharge as in the above report = 6 ft

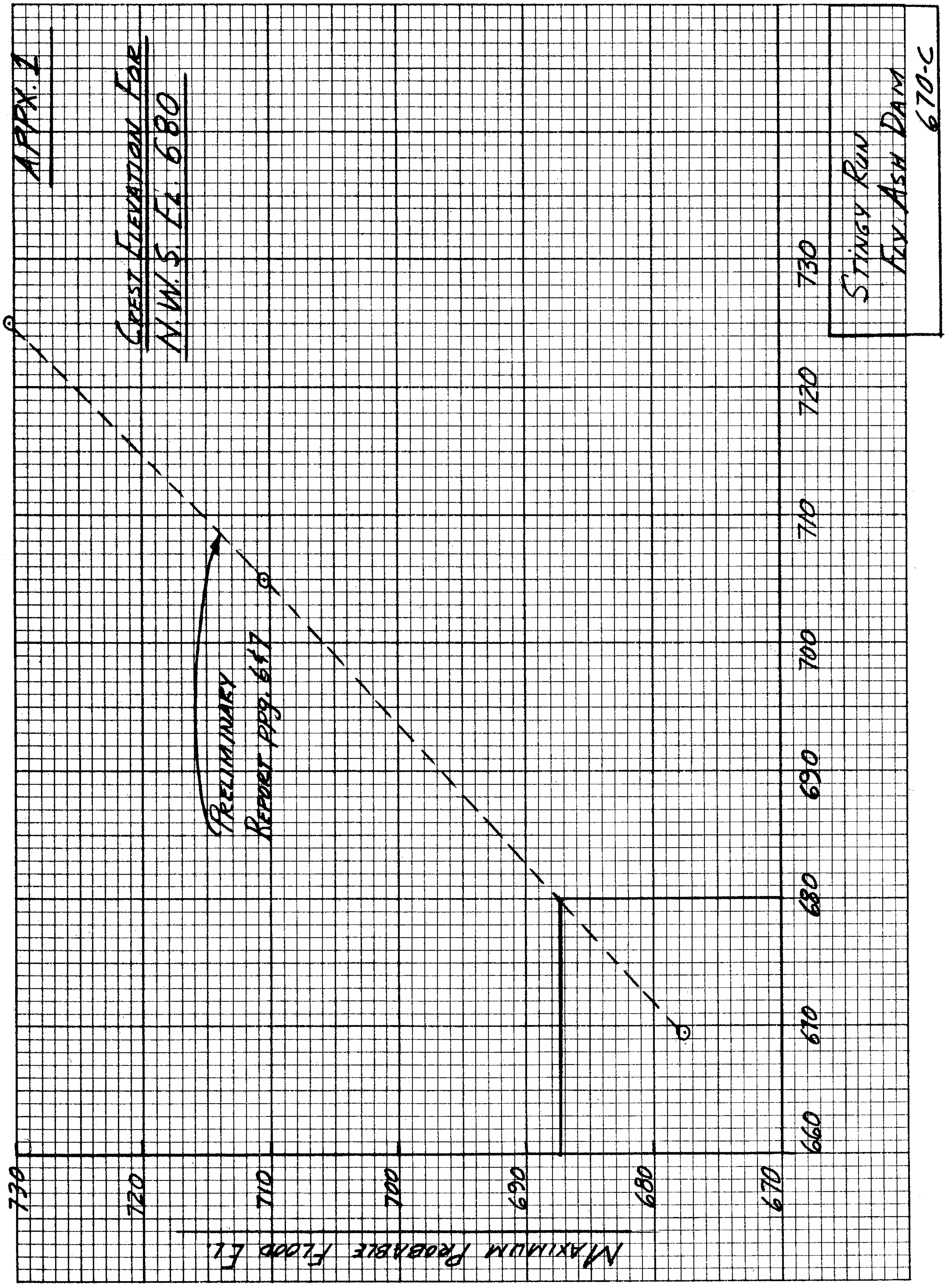
Present Crest El. = 688

Constr. Surcharge $\frac{6 \times 98}{89} = 6.6$ ft

Construction El. = 688 + 6.6 ft

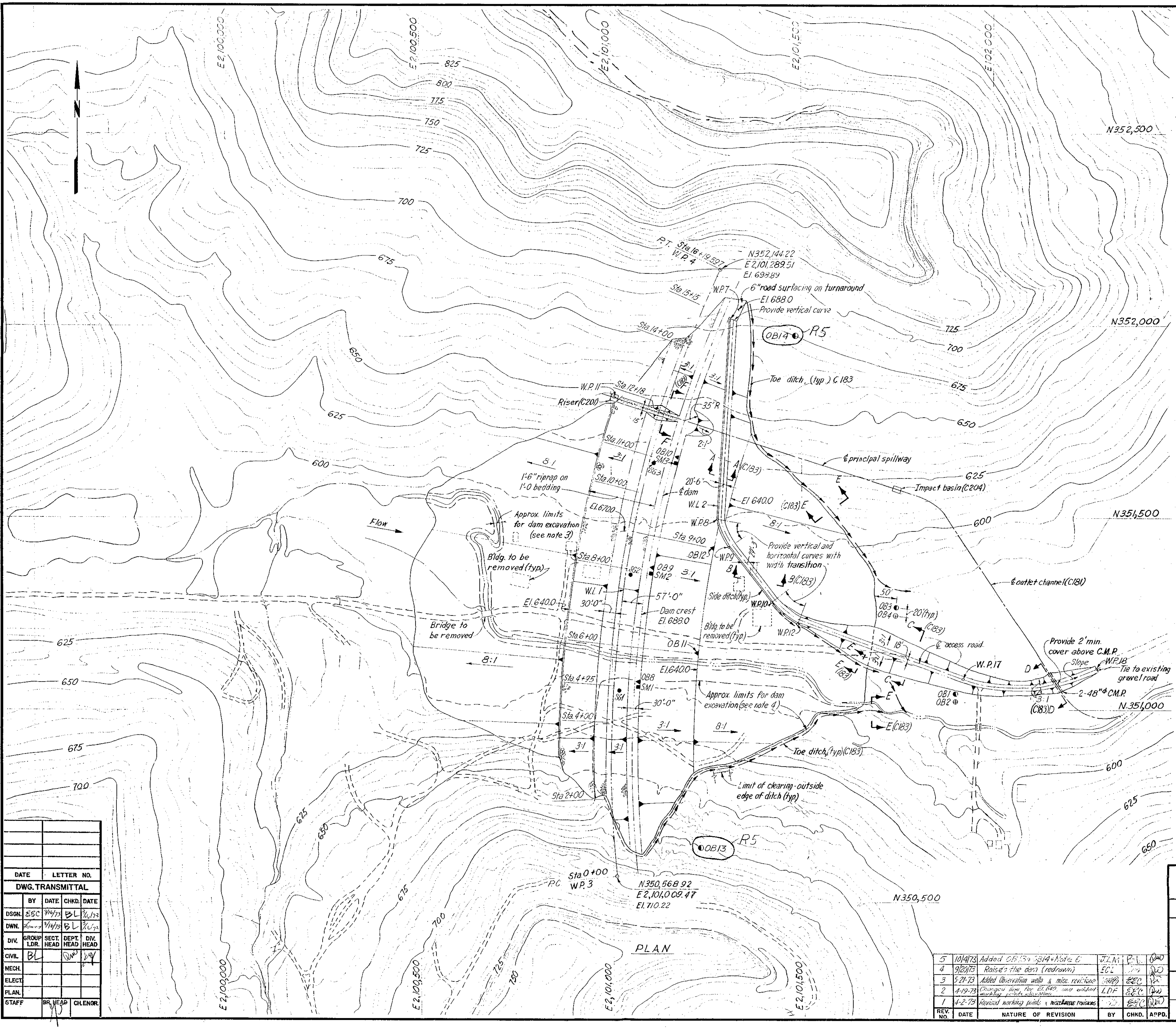
= 694.6 ft

Use 695 ft.


APPX. I
CREST ELEVATION FOR
N.W.S. EL. 680
STINGY RUN
FLY ASH DAM
670-C
PRELIMINARY
REPORT PPG. 6&7
MAXIMUM PROBABLE FLOOD EL.
660
670
680
690
700
710
720
730
670
680
690
700
710
720
730

APPENDIX D

Drawings



CURVE DATA OF W.L. 1

R=3,000.0'
I=30°55'55.27"
T=830.057'
L arc = 1,619.597'
D=1°54'35.49"

W.P. No.	Sta.	Offset from W.L. 1	Construction Elevation
7	15+15	73.5' RT	687.50
8	9+80	215.55' RT	643.20
9	9+09	253.0' RT	638.55
10	7+78	390.0' RT	620.42
11	12+18	140.0' LT	For horizontal control only
12	7+27	462.0 RT	611.80
17	6+08	909.0' RT	587.0
18	7+18	1252'± RT	582±

LEGEND

■	SMJ	Surface settlement monument
●	SG1	Settlement gage
◐	OB1	Observation well, Type I
⊕	OB2	Observation well, Type II
▲	OB8 - OB12	Observation well, Type II (see Note 6)

SURVEY CONTROL

Working points 3 and 4 were established by the Ohio Power Company. These points shall be used to establish construction survey by the Contractor

REFERENCE DRAWINGS

Geology — 670 C101, C110 thru C112
C121 thru C138
C141 thru C143
Soil tests — 670C151 thru C153, C156
C161 thru C164, C167
Outlet works 670C191 thru C193
C201 thru C206

NOTES

1. Work this drawing with 670C181, C183 thru C187
2. Topography on this drawing shall not be used for payment purposes. See note 3 on C184
3. Approximately 10' unsuitable material, mainly silt, along the stream bed is expected to be removed. Exact limits and depths may vary and will be determined by the Owner's Engineer
4. Approximately 2' to 16' strip mine disposal material is distributed in this area. The upper 1'-6" shall be stripped and is to be defined as Stripping. The strip mine disposal material below 1'-6" is expected to remain, subject to approval by the Owner's Engineer. If removal is required, payment will be made as Dam Excavation.
5. All elevations shown are settled elevations except as noted. All slopes shown are settled slopes except as noted.
6. All observation wells will be installed by contractor. R5
7. All ponded water within strip mine areas within the reservoir to be drained.
8. All stationing along the dam refers to W.L 1

Scale 0 100 200 Feet

DATE	LETTER NO.

DWG. TRANSMITTAL				
	BY	DATE	CHKD.	DATE
DSGN.	EEC	3/16/73	BL	3/16/73
DWN.	[illegible]	3/16/73	BL	3/16/73
DIV.	GROUP LDR.	SECT. HEAD	DEPT. HEAD	DIV. HEAD
CIVIL	BL		RW	[illegible]
MECH.				
ELECT.				
PLAN.				
STAFF		BR. HEAD		CH. ENGR.

REV. NO.	DATE	NATURE OF REVISION	BY	CHKD.	APPD.
5	10/4/73	Added OB13 + OB14 + Note 6	JLM	BL	RW
4	9/20/73	Raised the dam (redrawn)	ECL	[illegible]	RW
3	5-21-73	Added Observation wells & misc. revisions	[illegible]	EEC	RW
2	4-19-73	Changed line for El. 649, and added working points elevation	LDF	EEC	RW
1	4-2-73	Revised working points & miscellaneous revisions	[illegible]	EEC	RW

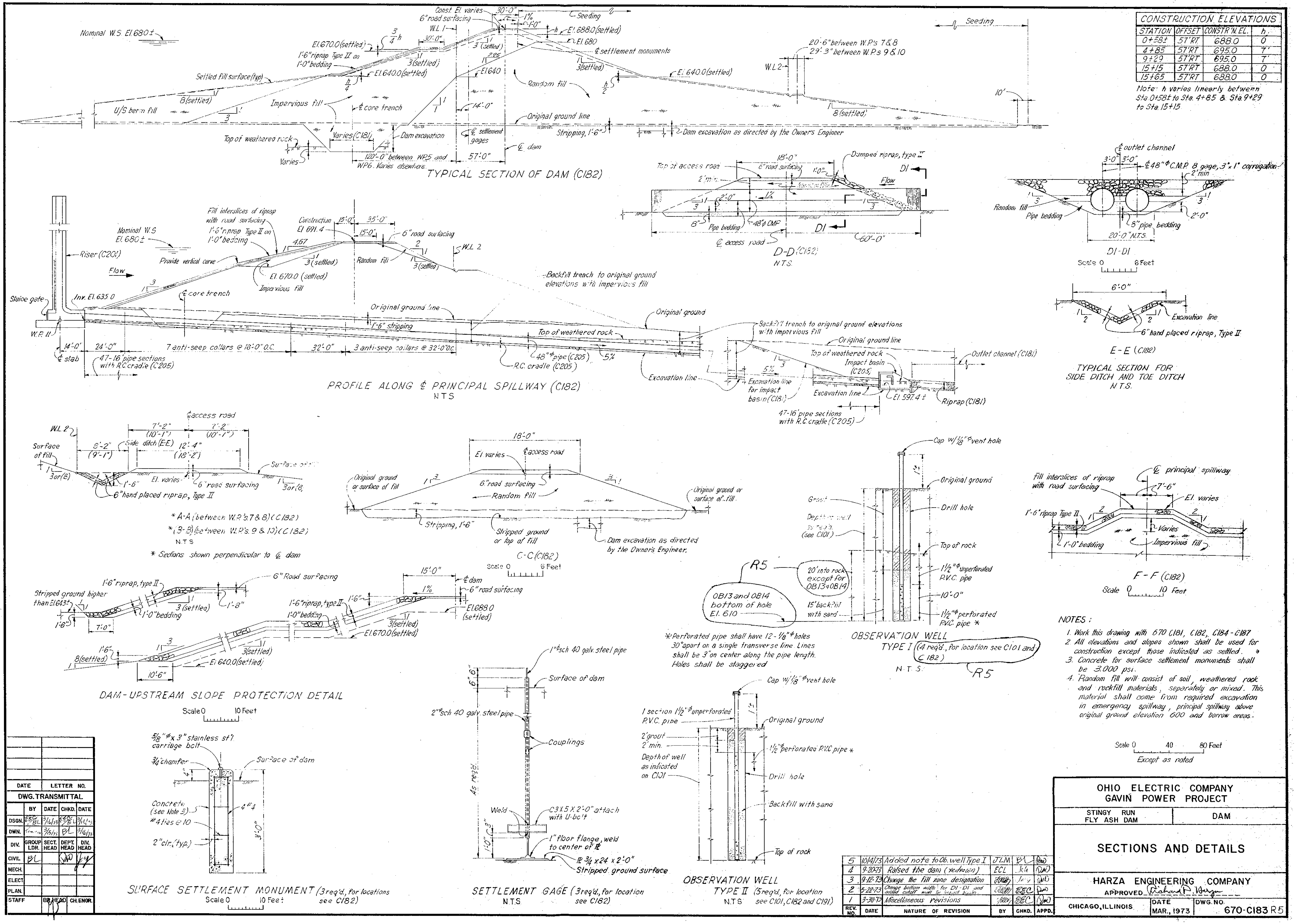
OHIO ELECTRIC COMPANY
GAVIN POWER PROJECT

STINGY RUN FLY ASH DAM | DAM

PLAN

HARZA ENGINEERING COMPANY
APPROVED Richard D. Harza

CHICAGO, ILLINOIS | DATE MAR., 1973 | DWG. NO. 670-C182 R5



CONSTRUCTION ELEVATIONS

STATION	OFFSET	CONSTR'N. EL.	h
0+58±	57'RT.	688.0	0
4+85	57'RT	695.0	7'
9+29	57'RT	695.0	7'
15+15	57'RT	688.0	0
15+65	57'RT	688.0	0

Note: h varies linearly between Sta. 0+58± to Sta. 4+85 & Sta. 9+29 to Sta. 15+15

NOTES:

1. Work this drawing with 670 C181, C182, C184-C187
2. All elevations and slopes shown shall be used for construction except those indicated as settled.
3. Concrete for surface settlement monuments shall be 3,000 psi.
4. Random fill will consist of soil, weathered rock and rockfill materials, separately or mixed. This material shall come from required excavation in emergency spillway, principal spillway above original ground elevation 600 and borrow areas.

Scale 0 40 80 Feet
Except as noted

DATE	LETTER NO.
DWG. TRANSMITTAL	

	BY	DATE	CHKD.	DATE
DSGN.	EEC/BL	3/16/73	EEC/BL	3/16/73
DWN.		3/15/73	BL	3/16/73
DIV.	GROUP LDR.	SECT. HEAD	DEPT. HEAD	DIV. HEAD
CIVIL	BL			
MECH.				
ELECT.				
PLAN.				
STAFF		BR. HEAD	CH. ENGR.	

REV. NO.	DATE	NATURE OF REVISION	BY	CHKD.	APPD.
5	10/4/73	Added note to Ob. well Type I	JLM	BL	
4	9-20-73	Raised the dam (redrawn)	ECL		
3	9-12-73	Change the fill zone designation			
2	5-22-73	Change bottom width for DI-DI and added cutoff wall for impact basin		EEC	
1	3-30-73	Miscellaneous revisions		EEC	

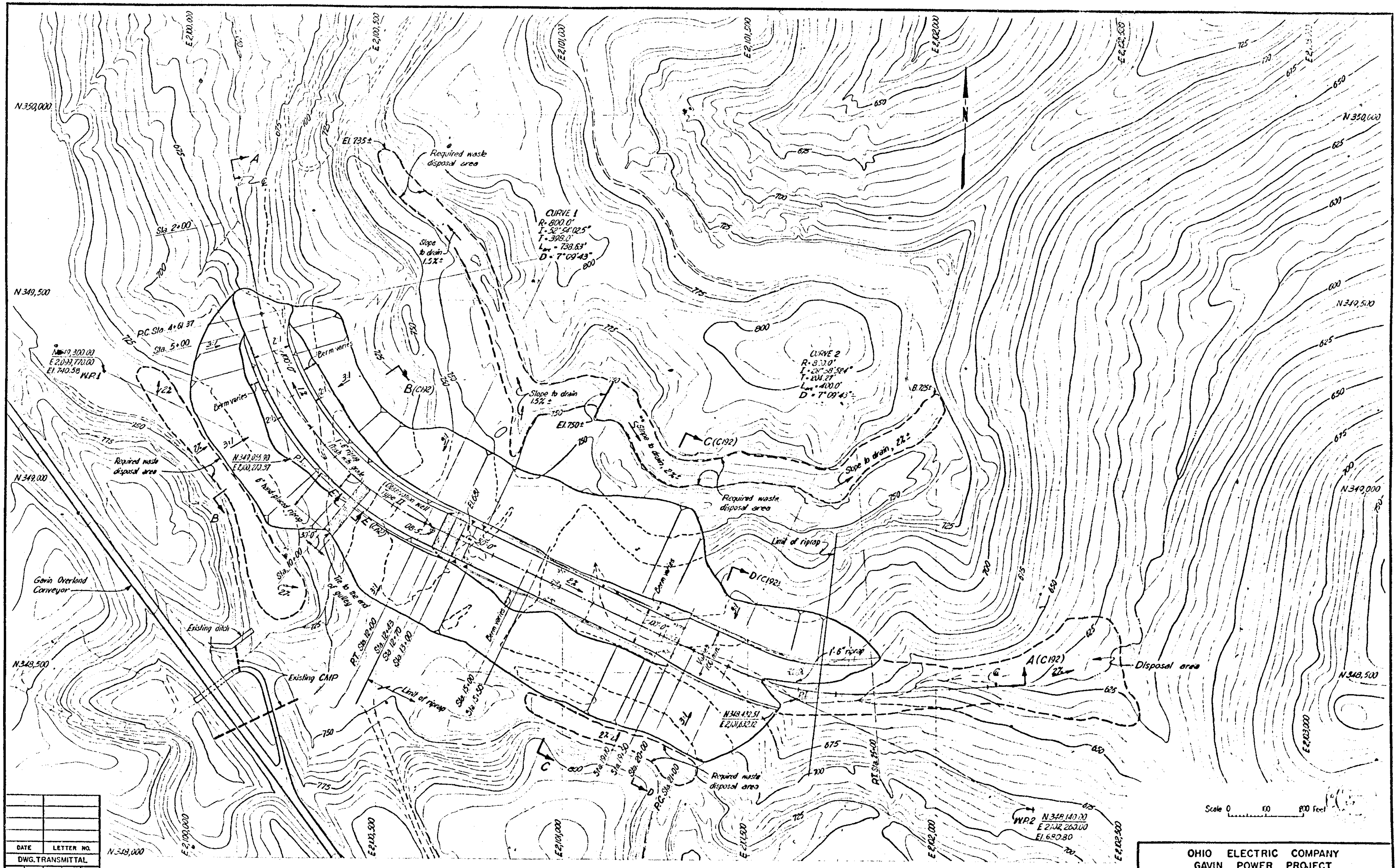
OHIO ELECTRIC COMPANY
GAVIN POWER PROJECT

STINGY RUN FLY ASH DAM | DAM

SECTIONS AND DETAILS

HARZA ENGINEERING COMPANY
APPROVED Richard P. Harza

CHICAGO, ILLINOIS | DATE MAR., 1973 | DWG. NO. 670-C183 R5



DATE	LETTER NO.

DWG. TRANSMITTAL

	BY	DATE	CHKD.	DATE
DSGN.	EEC	[illegible]	BL	[illegible]
DWN.	[illegible]	[illegible]	EEC	[illegible]
DIV.	GROUP LDR.	SECT. HEAD	DEPT. HEAD	DIV. HEAD
CIVIL	BL		[illegible]	[illegible]
MECH.				
ELECT.				
PLAN.				
STAFF		BR. HEAD [illegible]	CH. ENGR.	

REFERENCE DRAWINGS:

Geology 670 C 101, 110-112
670 C 121-138, 141-143
Soil Tests 670 C 151-153, 156
670 C 161-164, 167
Dam 670 C 181-187

NOTES:

1. Work this drawing with 670 C 192 and 193
2. The topography on this sheet is not to be used for payment purposes. See Note 4 on 670 C 193
3. Disposal area at downstream limit of emergency spillway to be used only after required disposal areas are filled.

SURVEY CONTROL

Working points 1 and 2 were established by the Ohio Power Company. These points shall be used to establish construction survey by the Contractor.

REV. NO.	DATE	NATURE OF REVISION	BY	CHKD.	APPD.
2	9/20	Raised the weir elevation	[illegible]	[illegible]	DO
1	4-18-73	Used working points elevat[illegible]	[illegible]	[illegible]	[illegible]

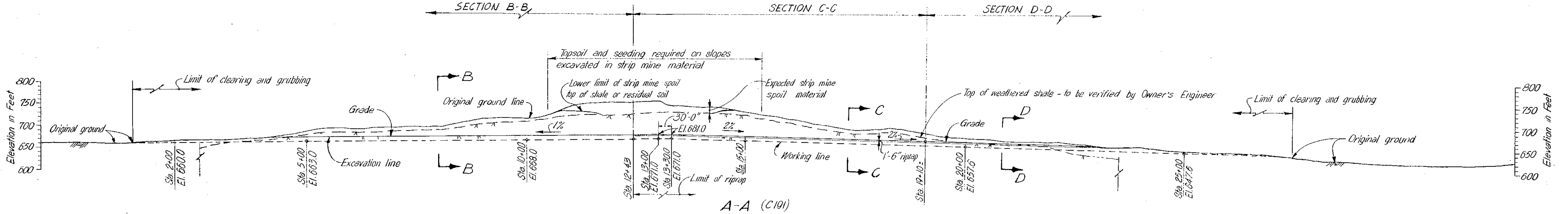
OHIO ELECTRIC COMPANY
GAVIN POWER PROJECT

STINGY RUN FLY ASH DAM — OUTLET WORKS

EMERGENCY SPILLWAY
PLAN

HARZA ENGINEERING COMPANY
APPROVED [illegible]

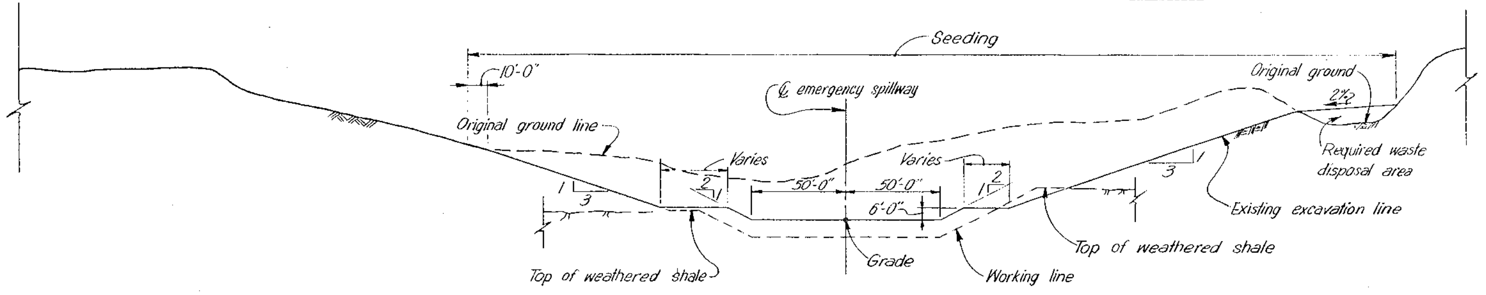
CHICAGO, ILLINOIS — DATE MAR, 1973 — DWG. NO. 670 C 191 R2



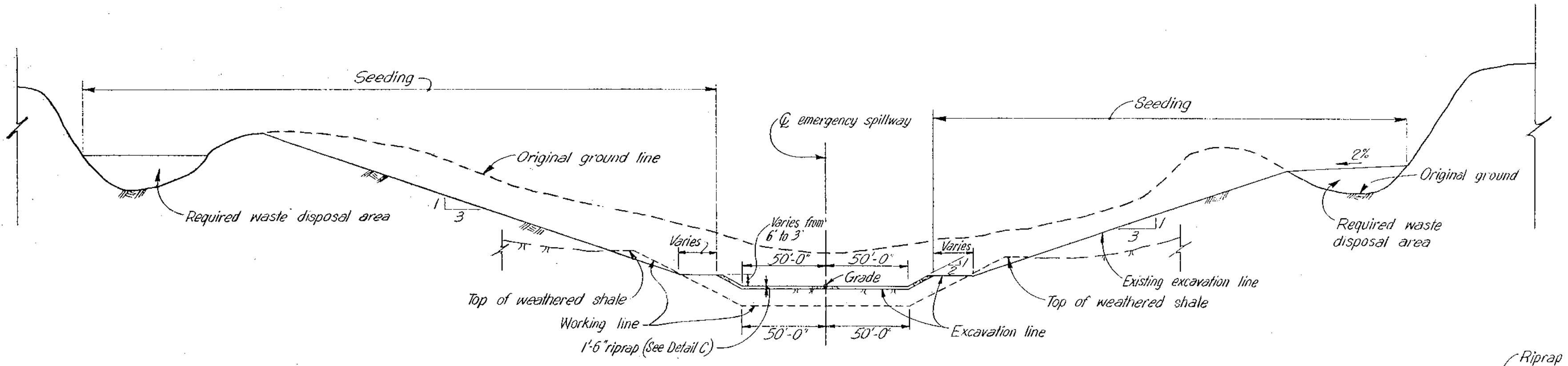
A-A (C191)

DEVELOPED PROFILE ALONG ℄ OF EMERGENCY SPILLWAY

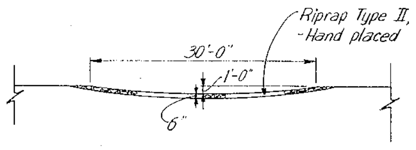
Scale 0 100 Feet



B-B (C191)

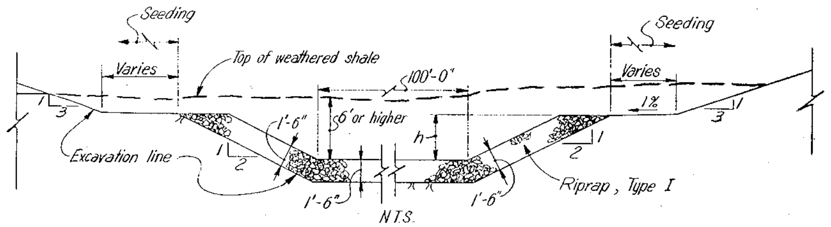


C-C (C191)



E-E (C191)

Scale 0 10 Feet



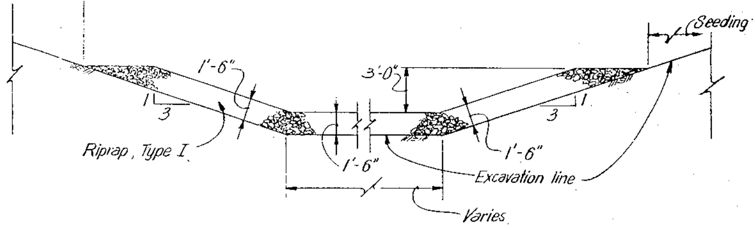
N.T.S.

DETAIL C

h = 6'-0" from Sta. 12+43 to Sta. 15+00

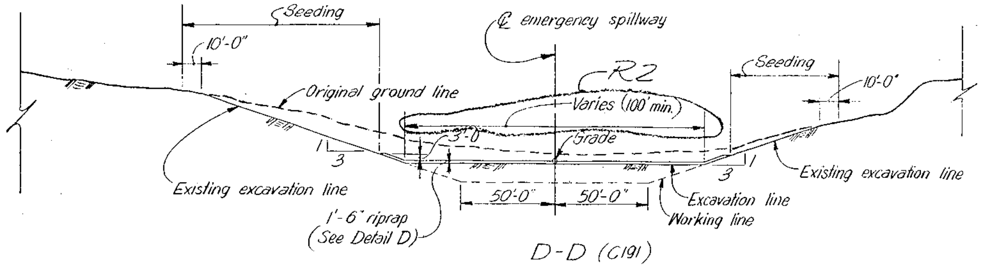
h = varies from 6'-0" to 3'-0" from Sta. 15+00 to Sta. 15+30

h = 3'-0" from Sta. 15+30 and beyond



DETAIL D (For Sta. 19+10± and beyond)
N.T.S.

R2



D-D (C191)

NOTES:

1. Work this drawing with 670 C191 and 193
2. Top of weathered shale shall be verified by Owner's Engineer
3. Drain disposal areas before placing waste material
4. Spoil from strip mining operations to be wasted

Scale 0 40 80 Feet
Except as noted

DATE		LETTER NO.		
DWG. TRANSMITTAL				
	BY	DATE	CHKD.	DATE
DSGN.	EEC	MARCH 73	BL	3/14/73
DWN.		MARCH 73	EEC	MARCH 73
DIV.	GROUP LDR.	SECT. HEAD	DEPT. HEAD	DIV. HEAD
CIVIL	BL			
MECH.				
ELECT.				
PLAN.				
STAFF		BR. HEAD		CH. ENGR.

REV. NO.	DATE	NATURE OF REVISION	BY	CHKD.	APPD.
2	10/1/73	Miscellaneous		JLG	
1	9/11/73	Raised the invert by 10 feet, change excavation and riprap detail		BL	

OHIO ELECTRIC COMPANY
GAVIN POWER PROJECT

STINGY RUN
FLY ASH DAM

OUTLET WORKS

EMERGENCY SPILLWAY
SECTIONS

HARZA ENGINEERING COMPANY
APPROVED Richard D. Harza

CHICAGO, ILLINOIS

DATE MAR., 1973

DWG. NO. 670 C 192 R2

APPENDIX B

SOIL TESTING BY GENERAL ANALYTICS, INC.

Ohio Power Company

Gavin Plant

Cheshire, Ohio

Proposed Dam Raising For Phase II Stingy Run Fly Ash Retention Pond

1501:21-5-04 Final Design Report

Appendices D-I

APPROVED
PERMIT NO. 87-159
FEB 19 1987
Robert L. Goettemoeller

December, 1986

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

APPENDICES D-I

STATE OF OHIO
DEPARTMENT OF NATURAL RESOURCES
DIVISION OF WATER
DAM PERMITS
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DIVISION OF WATER
DAM SAFETY

Ohio Power Company

Gavin Plant

Cheshire, Ohio

Proposed Dam Raising
For
Phase II Stingy Run Fly Ash
Retention Pond

1501:21-5-04 Final Design Report

Appendix - D

Piezometer Falling Head Tests

December, 1986

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

APPENDICES D-I

PIEZOMETER INSTALLATION REPORT

Project Gavin Plant - Stingy Run Fly Ash Dam P-1-85

Piez Type 5' PVC Well Screen	Depth. 63.5	Riser Desc. 1" PVC
Mat'l @ Tip	Sample #	Boring Dia. 4"

Method of Installation 4 " Casing Advancer

Type of Grnd Protection 4" Steel Pipe set in concrete

Grnd Elev 636.80	Riser Elev. 638.40	Piez Tip Elev. 573.3

Filter Material Sand from Elev. 579.1 to Elev 572.3

Seal Material Bentonite from Elev. 581.3 to Elev 579.1

Installed By Roush - Lambert

Date Installed 3-1-85	Date Tested 4-18-86

Method of Testing Piez

Time	Elapsed Time	Depth to Water	Time	Elapsed Time	Depth to Water	Time	Elapsed Time	Depth to Water
10:05	0	0	11:35	90	1.0			
	.5	.05	12:05	120	1.25			
10:12	7	.15	12:35	150	1.50			
10:21	16	.25	1:05	180	1.70			
10:50	45	.6	2:05	240	2.0			
11:05	60	.75						

REMARKS

Cement Bentonite Grout from 636.8 to 581.3

Water Depth at Start 22.2 Ft. Below Top of Standpipe

$$K = \frac{d^2 \ln (ML/D)}{8L(T_2-T_1)} \ln(H1/H2) \div 60 * 30.48 \rightarrow \text{cm/sec}$$

m= kv/kn

k_{240} $6.808 * 10^{-8}$ cm/sec

Time	K (C/S)
.5	$7.805 * 10^{-7}$
7	$1.204 * 10^{-7}$
16	$8.741 * 10^{-8}$
45	$9.593 * 10^{-9}$
60	$8.040 * 10^{-8}$
90	$8.763 * 10^{-8}$
120	$6.843 * 10^{-8}$
240	$6.808 * 10^{-8}$

LAG TIME ≈ 48 hrs

$$T = \frac{A}{FK} \qquad A = \frac{\pi d^2}{4}$$

$$F = \frac{2 \pi L}{\ln(L/D)}$$

PIEZOMETER INSTALLATION REPORT

Project Gavin Plant - Stingy Run Fly Ash Dam P-2-85

Piez Type 5' PVC Well Screen | Depth. | Riser Desc. 1" PVC

Mat'l @ Tip | Sample # | Boring Dia. 4"

Method of Installation 4" Casing Advancer

Type of Grnd Protection 4" Steel Casing set in concrete

Grnd Elev 617.43 | Riser Elev. | Piez Tip Elev. 564.43

Filter Material Sand from Elev. 570.73 to Elev 563.43

Seal Material Bentonite from Elev. 573.23 to Elev 570.73

Installed By Roush - Lambert

Date Installed 3-12-85 | Date Tested 4-18-86

Method of Testing Piez.

Time	Elapsed Time	Depth to Water	Time	Elapsed Time	Depth to Water	Time	Elapsed Time	Depth to Water
9:40	0	0	11:10	90	24.8			
	.5	.35'	11:40	120	28.2			
9:47	7	3.35'	12:10	150	30.8			
9:56	16	7.2'	12:40	180	32.4			
10:25	45	16.4'	1:10	210	33.9			
10:40	60	19.65'	2:40	300	36.0			

REMARKS

Cement Bentonite Grout from 617.43 to 573.23

Water Depth at Start 3817 Ft. Below Top of Standpipe

$$K = \frac{d^2 \ln (ML/D)}{8L (T_2 - T_1)} \ln (H_1/H_2) \div 60 * 30.48 \rightarrow \text{cm/sec}$$

$$m = kv/kh$$

Time	K(C/s)
.5	$3.007*10^{-6}$
7	$2.074*10^{-6}$
16	$2.120*10^{-6}$
45	$1.971*10^{-6}$
60	$1.738*10^{-6}$
90	$1.738*10^{-6}$
120	$1.547*10^{-6}$
180	$1.409*10^{-6}$
300	$1.168*10^{-6}$

K ave $1.468* 10^{-6}$ cm/sec
$2.892* 10^{-6}$ ft/min

LAG TIME ≈ 2. hours

$T = \frac{A}{Fk}$ $A = \frac{\pi d^2}{4}$

$F = \frac{2 \pi L}{\ln (L/D)}$

PIEZOMETER INSTALLATION REPORT

Project Gavin Plant - Stingy Run Fly Ash Dam P-3-85

Piez Type 5" PVC Well Screen | Depth. | Riser Desc. 1" PVC

Mat'l @ Tip | Sample # | Boring Dia. 4"

Method of Installation 4" Casing Advances

Type of Grnd Protection 4" Steel Casing set in concrete

Grnd Elev 588.38 | Riser Elev. | Piez Tip Elev. 555.38

Filter Material Sand from Elev. 561.28 to Elev 554.38

Seal Material Bentonite from Elev. 563.08 to Elev 561.28

Installed By Roush - Lambert

Date Installed 3-4-85 | Date Tested 4-18-86

Method of Testing Piez

Time	Elapsed Time	Depth to Water	Time	Elapsed Time	Depth to Water	Time	Elapsed Time	Depth to Water
9:13	0	0						
	.5	9.4						
9:20	7	10.25						
9:29	16	11.0						
2:15		11.0						

REMARKS

Cement Bentonite Grout from 588.38 to 563.08

Water Depth At Start 11.0 Ft. Below Top of Standpipe

$$K = \frac{d^2 \ln(ML/D)}{8L(T_2-T_1)} \ln(H_1/H_2) \div 60 * 30.48 \rightarrow \text{cm/sec}$$

$$M = KV/KH$$

Time	K (cm/sec)
.5	$6.612*10^{-4}$
7	$1.999*10^{-5}$
16	$8.227*10^{-5}$*

K ave → $6.579*10^{-5}$ cm/sec
→ $1.295*10^{-4}$ ft/min

T = 3 min

$$T = \frac{A}{Fk} \qquad A = \frac{\pi d^2}{4}$$

$$F = \frac{2\pi L}{\ln(L/D)}$$

*May have reached 11' prior to 16 min. reading.

Ohio Power Company

Gavin Plant

Cheshire, Ohio

Proposed Dam Raising For Phase II Stingy Run Fly Ash Retention Pond

1501:21-5-04 Final Design Report

December, 1986

APPROVED
PERMIT NO. 87-159
FEB 19 1987
Robert L. Goettemoeller

Prepared by

Geotechnical Engineering Section
Civil Engineering Division
American Electric Power Service Corporation

STATE OF OHIO
DEPARTMENT OF NATURAL RESOURCES
DIVISION OF WATER
DAM PERMITS
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DAM SAFETY

Ohio Power Company

Gavin Plant

Cheshire, Ohio

Proposed Dam Raising For Phase II Stingy Run Fly Ash Retention Pond

1501:21-5-04 Final Design Report

December, 1986

Prepared by

Geotechnical Engineering Section
Civil Engineering Division
American Electric Power Service Corporation

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A - Boring Logs & Pressure Tests

B - Laboratory Testing & Stability Analyses

C - Hydrology

Final Design Report:

D - Piezometer Falling Head Tests

E - Seepage Analysis Calculations

- E.1 Dam Raising
- E.2 Spillway Dam

F - Stability Analysis - Supplimental

- F.1 Dam Raising EOC
- F.2 Dam Raising SSS
- F.3 Spillway Dam EOC
- F.4 Spillway Dam SSS

G - Cost Estimate

H - Estimated Construction Schedule

I - Instructions to Bidders (Partial)

J - Civil Engineering Division Technical Specifications for Material and Construction

1.0 INTRODUCTION

The Ohio Power Company is proposing that the Stingy Run Fly Ash Dam be raised from a average crest elevation of 696 feet to an elevation of 735 feet. In conjunction with the raising of the existing dam, a new dam will be constructed in the existing emergency spillway channel and a new secondary spillway constructed. On March 27, 1986, the Preliminary Design Report was submitted to the ODNR in compliance with Ohio Law 15:21-5-02. The Preliminary Design Report included most of the requirements for the Final Design Report. Therefore, this document will report only the supplemental information requested in the ODNR letter of August 4, 1986, the additional information stated in Section 11.0 of the Preliminary Design Report, and any other pertinent information. The supplimental information is grouped into the following 5 areas:

1. Design Changes from the Preliminary Design Report

2. Seepage Analysis

3. Grouting Program

4. Stability Analyses

5. Instrumentation

6. Additional Hydrologic Investigations

2.0 DESIGN CHANGES

The following changes have been made in the dam raising project since submittal of the Preliminary Design Report:

1. Grout holes have been shortened and a grout cap has been added.

2. The downstream random fill zone configuration has been changed to simplify construction.

3. Rip Rap bedding material has been replaced with a geotextile to reduce cost.

4. The limits of the bottom ash drain have been better defined as requested in the August 4, 1986, ODNR letter.

5. The contact between the upstream clay core and the bottom ash drain has been fixed at a constant slope to make the core thickness uniform.

6. The location of the access road has been relocated to reduce costs and improve road stability.

7. The borrow area has been modified to show a reduced scope of required borrow material.

8. The highwall fill has been added upstream of the south abutment.

The following changes have been made to the spillway dam since submittal of the Preliminary Design Report:

9. The Option B grass lined spillway has been selected.

10. The cutoff trench has been eliminated in favor of wider clay core contact.

11. The rip-rap bedding material has been replaced with a geotextile to reduce cost.

2.1 Grout Curtain:

The purpose of a grout curtain is to increase the seepage flow path length and therefore, minimize the piping potential. The final length of the grout holes was selected to ensure a gradient no larger than 1 and to restrict flow through a sandstone seam with a base elevation at 665 on the south abutment and elevation 670 on the north abutment. This criteria was met with shorter grout holes than shown on the preliminary design report drawings. The orientation of the grout holes was shifted to a North-South direction and angled into the slopes at 40°.

The pressure tests and field examinations indicate that permeability of near surface rock increases with elevation. The increase in permeability is probably due to stress relief and weathering along the ridge tops. Therefore, a 10 foot deep grout cap will be placed along the contact area with the clay core. This will minimize the piping potential along this interface. Actual grouting techniques will be discussed in Section 4.0 of this report. The grouting changes are shown on drawings numbered 12-3000I, Grouting Plan & Profile - North Abutment, and 12-3000J, Grouting Plan & Profile - South Abutment.

2.2 Random Fill Configuration

The downstream random fill from the crest to the bench has been altered from a variable to a constant thickness. This was done to simplify construction. The proposed thickness has an 8 foot horizontal width to facilitate placement with standard compaction equipment. The 8 foot horizontal width will provide an erosional protection blanket of 3 foot thickness parallel to the slope. This change is reflected on profiles shown on drawing numbers 12-3000F, 12-3000L, and 12-3000M.

2.3 Rip Rap Bedding Material:

The rip rap bedding material has been replaced with a geotextile. The geotextile will be an 8 oz. syd. non-woven fabric which will furnish adequate filter protection and puncture resistance to the rip rap.

2.4 Bottom Ash Drain Boundaries:

The ODNR letter of August 4, 1986, requested that the limits of the bottom ash be defined on the drawings Drawing Number 12-3000-H, has been modified to show the extent of the bottom ash material.

2.5 Clay/Bottom Ash Interface Slope:

Drawings 12-3000L and 12-3000M were changed to have a consistent clay core thickness along the entire crest of the dam. The change was accomplished by fixing the slope of the interface between the clay core and bottom ash drain at a 2h:1v slope.

2.6 Access Road Location:

The proposed access road shown on drawing numbers 12-3000B and 12-3000K, Layout and Grading Plan, has been relocated. The relocation eliminates the large cut and fill scheme shown on the preliminary drawings. In addition, a large portion of the road was located on the east side of the south abutment. This side of the abutment is on the down dip bedding planes and is regionally susceptible to sliding. Therefore, the road was relocated on the west side of the adjoining ridge within the proposed borrow area. This will ensure a competent base and reduce the area of disturbance.

2.7 Borrow Area Grading:

The borrow area drawings 12-3000S and 12-3000T have been modified to show the reduced scope of work in the slope of the interface between the clay core and bottom ash drain at a 1h:1v slope.

2.6 Access Road Location:

The proposed access road shown on drawing numbers 12-3000B and 12-3000K, Layout and Grading Plan, has been relocated. The relocation eliminates the large cut and fill scheme shown on the preliminary drawings. In addition, a large portion of the road was located on the east side of the south abutment. This side of the abutment is on the down dip bedding planes and is regionally susceptible to sliding. Therefore, the road was relocated on the west side of the adjoining ridge within the proposed borrow area. This will ensure a competent base and reduce the area of disturbance.

2.7 Borrow Area Grading:

The borrow area drawings 12-3000S and 12-3000T have been modified to show the reduced scope of work in the borrow area. This reduction results from the greater use of bottom ash in the dam raising.

2.8 Highwall Fill:

The highwall between the proposed dam raising and the proposed spillway dam has areas of exposed highwall below the elevation of 735 feet. These areas will be backfilled with random fill. The random fill will be at least 30 feet wide and will support the access road from the existing dam to the new spillway and spillway dam. The maximum operating pool of the reservoir will not reach this random fill, but major storms occurring at the end of the reservoir's service life may encroach on the bottom 1-2 feet of this fill. The highwall fill is shown on drawing 12-3000K.

2.9 Spillway Option:

The option B grass lined spillway was selected as the secondary spillway for this project. The option B spillway will provide secondary spillway capacity at the end of the reservoir life. This option provides the potential use of the excavated material as random fill in the spillway dam. The spillway is shown on drawings numbered 12-3000N and 12-3000Q.

2.10 Cut-Off Trench

The cut off trench has been eliminated in the spillway dam. The trench is replaced by a wider clay core at the base and a wide grout cap along the upstream edge of the rock/clay core contact. With this scheme the maximum exit gradients along the rock/clay core interface will be less than 0.5. This plan includes the excavation to insitu rock along the entire contact zone with the clay core. The maximum elevation of the clay core was reduced to elevation 730. This change is shown on drawing 12-3000R.

2.11 Spillway Dam-Rip Rap Bedding Material

As in the dam raising the upstream rip rap bedding material has been replaced by a non-woven geotextile.

2.12 General Slope Reductions

All 2h:1v non-impounding slopes have been re-sloped to 2.5h:1v for maintenance reasons. The effective angle of the regional soils is in the mid to upper twenties degrees. Therefore, the factor of safety for saturated surficial sloughs approaches 1.0 for 2h:1v slopes.

3.0 SEEPAGE ANALYSIS

The seepage analysis is reported for the dam raising and the spillway dam. In either case the seepage effecting the embankment is generated by reservoir water seeping through the core of the dam and water seeping from the abutments. In the case of the spillway dam, foundation seepage may also contribute. In both embankments the bottom ash drains extend up the adjoining abutments to intercept the abutment seepage.

3.1 Dam Raising Seepage Analysis

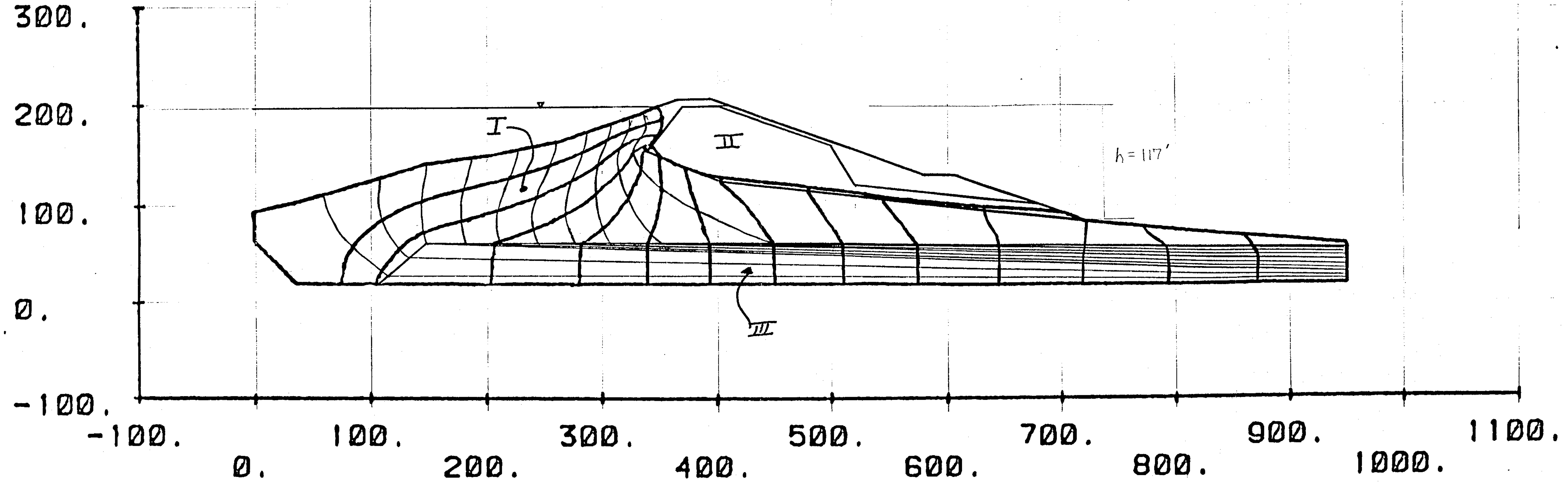
The upstream clay core is inclined downstream over the highly permeable bottom ash. The existing foundation materials are more permeable than the existing embankment materials. Both of these characteristics control the seepage flow through the dam.

The upstream clay core sloping over the bottom ash will produce a zone of percolation or partially saturated flow in the bottom ash. The permeability for the clay is $1.0*10^{-8}$ cm/sec and for the bottom ash $1.0*10^{-2}$ cm/sec. Therefore, the bottom ash is roughly a million times more permeable than the clay core. As a result of this ratio of permeabilities and the extent of the drain/clay core contact, a very low flow depth in the bottom ash is anticipated.

The foundation materials appear to function as a drain for the embankment. At least 2 predominantly sand layers have been detected in the foundation materials by the site investigation boring program. Falling head and drawdown tests have been performed on piezometers P-1, P-2, and P-3, which were installed in the underlying sands. Piezometers P-1 has behaved erratically and will be replaced with a pneumatic piezometer as discussed in section 6 of this report. Piezometers P-2 and P-3 were located in predominately sand layers and have yielded permeabilities of $1.5*10^{-6}$ and $6.5*10^{-5}$ cm/sec respectively. Therefore, the predominately sand layers represent a continuous layer of soil material that is at least 1 order of magnitude more permeable than the overlying clay core or random fill. Appendix D contains the completed Piezometer Installation Reports with the falling head permeability tests.

A finite element program was used to model the embankment. The "Plane and Axisymmetric Finite Element Program for Steady-State Seepage Problems" by Fred T. Tracy was acquired form the U.S. Army Engineer Waterways Experiment Station (WES) in Vicksburg, Miss. This program is very similar to the program, FEDAR, developed at the University of California at Berkeley by Taylor and Brown. The program handles anisotropic permeabilities, layers with different permeabilities, free surface flow problems, and pressure-dependent permeabilities.

Figure 1 shows the flow net from this analysis. The overall model was simplified into 3 material types. The clay portion of the dam was modeled with an isotropic permeability of $1*10^{-8}$ cm/sec to bound the maximum expected flow of seepage. The bottom ash portion of the dam was modeled with an isotropic permeability of $1*10^{-6}$ cm/sec. This is 10,000 times less permeable than what the actual permeability is, but is 100 times more permeable than the adjacent clay. Using a permeability no more than 2 orders of magnitude larger the adjacent material was required to make the model work, but does not adversely effect the results of the model. The foundation portion of the dam was modeled with an anisotropic permeability with a vertical permeability of $1*10^{-7}$ cm/sec and a horizontal



Equipotential Lines by COE Finite Element Program

$q = \frac{N_f}{N_e} kh = \frac{9}{12} * 2.33*10^{-9} ft/sec * 117 ft$

$= 2.0446*10^{-7} ft^2/sec/ft$

* Ave of 1200' width

= .88 cfhr

= 6.61 gph

≅ 160 gpd

Finite Element Analysis → $3.148*10^{-7} ft^2/sec/ft$ q

≅ 240 gpd

Zone	Description	Permeability	
I	CLAY CORE	$1*10^{-8}$ cm/sec	
II	BOTTOM ASH	$1*10^{-6}$ cm/sec	
III	FOUNDATION	$1*10^{-7}$ cm/sec	VERT.
		$2*10^{-7}$ cm/sec	HORIZ.

Figure 1

GAVIN PLANT
STINGY RUN FLY ASH DAM
DAM RAISING - SEEPAGE ANALYSIS
SCALE: HORIZ 1" = 100'
VERT 1" = 116.7'
12-5-86
KCM

permeability of $2*10^{-7}$ cm/sec. The foundation portion of the analysis was set up to model a material with a permeability 1 order of magnitude less than the overlying embankment. The anisotropic permeability reflects the expected behavior of the stratified sands and clays which make up the foundation material.

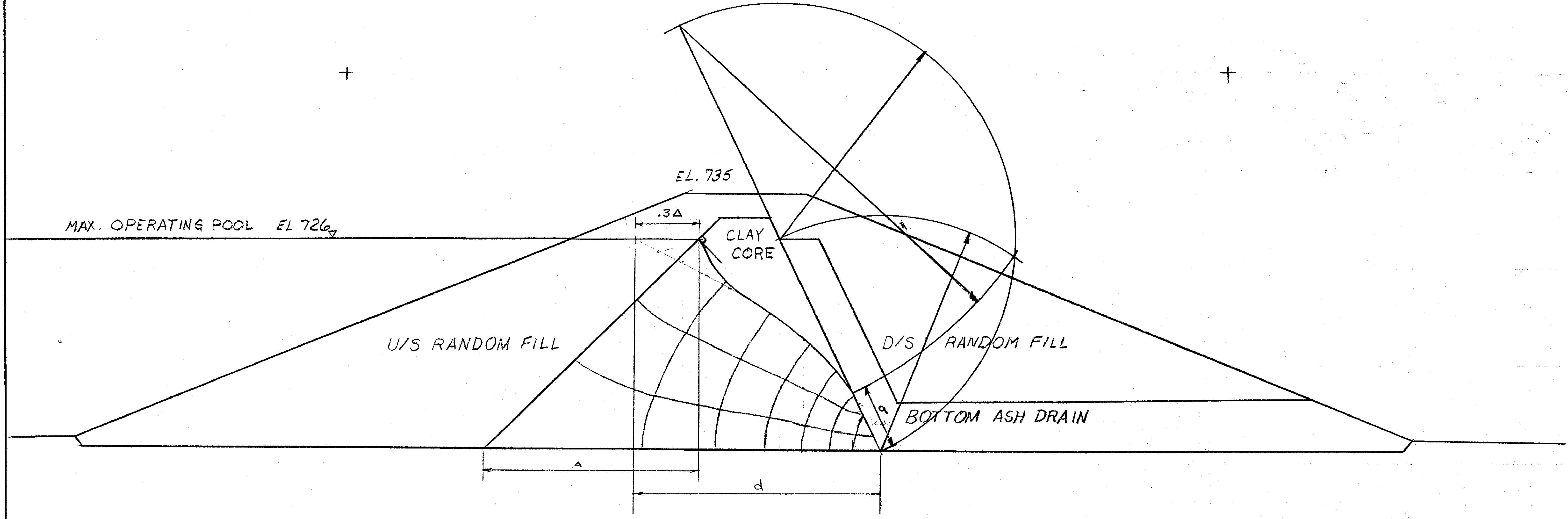
Based upon the flow net and finite element analysis approximately 160 to 240 gpd of steady state seepage is expected through the embankment portion of the dam at the maximum operating pool level.

Abutment flows can be determined utilizing Darcy's Law, using a maximum gradient of .7, and permeabilities from the pressure tests reported in the preliminary design report. The gradient estimates were based upon several combinations of maximum head potentials and minimum flow lengths in the adjoining ridges. A gradient of 0.7 represents an upper bound of these combinations. The areas are determined by a horizontal projection of the bottom ash blanket on to the adjoining rock abutments. From these analyses a flow of 36,000 gpd is expected from the south abutment and 29,400 gpd from the north abutment. Therefore, at the maximum operating pool with the adjacent hillsides completely saturated, a total estimated flow of 65,640 gpd (45.58 gpm) could occur. This flow would require approximately 4 feet of flow depth in the 10 foot deep drainage blanket. The backup calculations to these analyses may be found in Appendix E.

3.2 Spillway Dam Seepage Analysis:

The spillway dam represents a more conventional geometry. It was assumed for the clay core analysis that the embankment is founded on an impermeable material. The L. Casagrande's Solution (Harr, 1962) includes a graphical procedure for establishing the unconfined phreatic surface. This procedure was used to create the flownet found on Figure 2. The expected flow through the clay core into the chimney drain is approximately 25 gpd at the maximum operating pool.

The expected seepage flow from the abutments was estimated in the same manner as for the dam raising analysis. The pressure tests and the projection of the bottom ash blanket on the adjoining abutments were used to estimate the quantity of seepage flow. An overall gradient of 0.7 was used for the spillway dam. The north abutment will yield an estimated seepage flow of 6055 gpd and the south abutment 2300 gpd at the maximum operating pool. It is estimated that the foundation sandstones would yield up to 37,320 gpd of seepage into the blanket drain at maximum operating pool without a grout curtain. The grout curtain extends completely through the sandstone into the underlying shales. Estimating that the curtain will provide a minimum of an 80% reduction in flow through the sandstone the foundation would contribute approximately 7500 gpd into the drainage blanket. Therefore, a total flow of 15,880 gpd (11.0 gpm) will be required to be passed by the bottom ash drain at the maximum operating pool level. Using Darcy's Law, a flow depth of approximately 3 feet would be required to handle this flow in the drain. Backup calculations for this analyses are found in Appendix E.



$q = \frac{N_f}{N_e} k h$

$\left(\frac{2.50}{5.75}\right) 2.33 * 10^{-9}$ ft/sec $* 40'$

$8.47 * 10^{-8}$ cfs /ft of dam

ave 425' of Dam

$Q = 3.6 * 10^{-5}$ cfs

or

$\approx$ 23 GPD

L Casagrande's Solution

$q = k \cdot a \cdot \sin^2 \alpha$

$= 2.42 * 10^{-8}$ cfs /ft of Dam

ave of 425' of Dam

$Q = 1.0285 * 10^{-5}$ cfs

or

$\approx$ 7 GPD

$\alpha = 63°26'6''$ (63.435°)
$a = 13'$
$d = 51'$
$\Delta = 44'$
$k = 7.09 * 10^{-8}$ cm/sec
or
$2.33 * 10^{-9}$ ft/sec

FIGURE 2

GAVIN PLANT
STINGY RUN FLY ASH DAM
SPILLWAY DAM - SEEPAGE ANALYSIS
SCALE 1" = 20'   11-6-86
KCM

4.0 GROUTING PROGRAM

The overall grouting plan for the dam raising and the spillway dam is the same. This is because both structures are in the same rock units and have roughly the same geometric orientation. The grouting plan calls for a roughly 40 foot wide grout cap formed by 10 foot deep, 10 foot by 10 foot spaced grout holes. The purpose of the grout cap will be to reduce the permeability of near surface rock which has opened up due to stress release and weathering. This will mitigate the potential for piping along the clay/rock contact. The grout cap shall be in place prior to placing the grout curtain. The grout cap will be split with a single row of primary and secondary grout holes to establish the grout curtain at depth. The grout curtain will be down staged initially. Tertiary holes shall be added to the grout curtain as necessary.

The actual grouting will be preceded by a 15 min. water pressure test to wet the groutable zones. The pressure test will be immediately followed with grout mix of 1 part water to 1 part portland cement by volume.
If grout hole pressure cannot be generated, the mix will be

changed to 1 part water to 1 part sand to 1 part cement by volume. The maximum pressure to be applied will be 1 psi per foot of vertical depth.

The grouting program for the dam raising is shown on drawings numbered 12-3000I and 12-3000J. The grouting program for the spillway dam is shown on drawing 12-3000P.

5.0 STABILITY ANALYSES

The internal cross sections of the dam raising and spillway dams were changed from the configurations analyzed for the preliminary design report. New stability analyses have been performed to reflect these changes. In addition, all long term conditions have been analyzed for seismic stability using a psuedo-static method. This method applied a 0.05, force both horizontally outward and vertically upward from the slope. The Stingy Run Fly Ash Dam is in the Zone 1 seismic assessment area of Ohio. The SCS publication T.R. No. 60 recommends a minimum seismic coefficient of 0.05 for the horizontal force in a quasistatic analysis in Zone 1 and a minimum factor of safety of 1.1.

The following stability analyses will be addressed:

1. Dam Raising - End of Construction Stability
2. Dam Raising - Long Term Stability
3. Spillway Dam - End of Construction Stability
4. Spillway Dam - Long Term Stability

A summary is provided at the end of this section.

5.1 Dam Raising - End of Construction Stability:

The ODNR letter of August 3, 1986, pointed out that the factor of safety reported in the preliminary design report for the downstream, end of construction case was lower than that recommended by the Corps of Engineers in EM-1110-2-1902, "Engineering and Design Stability of Earth and Rock-fill Dams". The Corps of Engineers recommends a factor of safety of 1.4 for end of construction conditions in embankments over 50 feet high on relatively weak foundations.

Furthermore, there was some discussion with the consultant, John Lowe III, about the proper undrained strength to use in the foundation materials. These discussions were mentioned in the July 15, 1986 meeting with ODNR personnel. Therefore, the undrained shear strength was calculated by using the effective friction envelopes developed by the drained direct shear tests and the undrained shear strength calculated using the existing effective overburden pressure as the total confining pressure (see Appendix F). This is the same as assuming a Skempton A parameter of 1.0. Using this procedure an end of construction factor of safety of 1.27 was calculated for the downstream slope. However, this approach does not take into account the dissipation of excess pore pressures during the construction. The dissipation of excess pore pressures is the result of consolidation which yield higher shear strengths. Therefore, the factor of safety at the end of construction is expected to be higher than 1.27. Two approaches were analyzed in order to evaluate the dissipation of excess pore pressures on the end of construction stability of the dam raising.

The embankment was evaluated to see at what height of the new construction the factor of safety fell below 1.4. Assuming no dissipation of pore pressures, and therefore no increase in strength due to consolidation, the dam raising could be built to elevation 720 before a factor of safety below 1.4 was calculated (see figure 3).

It was recognized that this dam raising will take place over a period of 2 years. This will allow time for the foundation clays to consolidate and therefore gain shear strength. The consolidation tests in the foundation clays were examined to estimate the time required to dissipate excess pore pressure due to surcharge loading. The consolidation coefficients ranged from .002 to .008 cm^2/sec. Next the borings were examined to determine the maximum thickness of clay between sand layers. Two borings of continuous Shelby tube samples were used to evaluate this. A maximum of 15 feet of the weak gray clay was determined. Using these values, the time estimated for 100% consolidation using 1-D theory ranged from 75 to

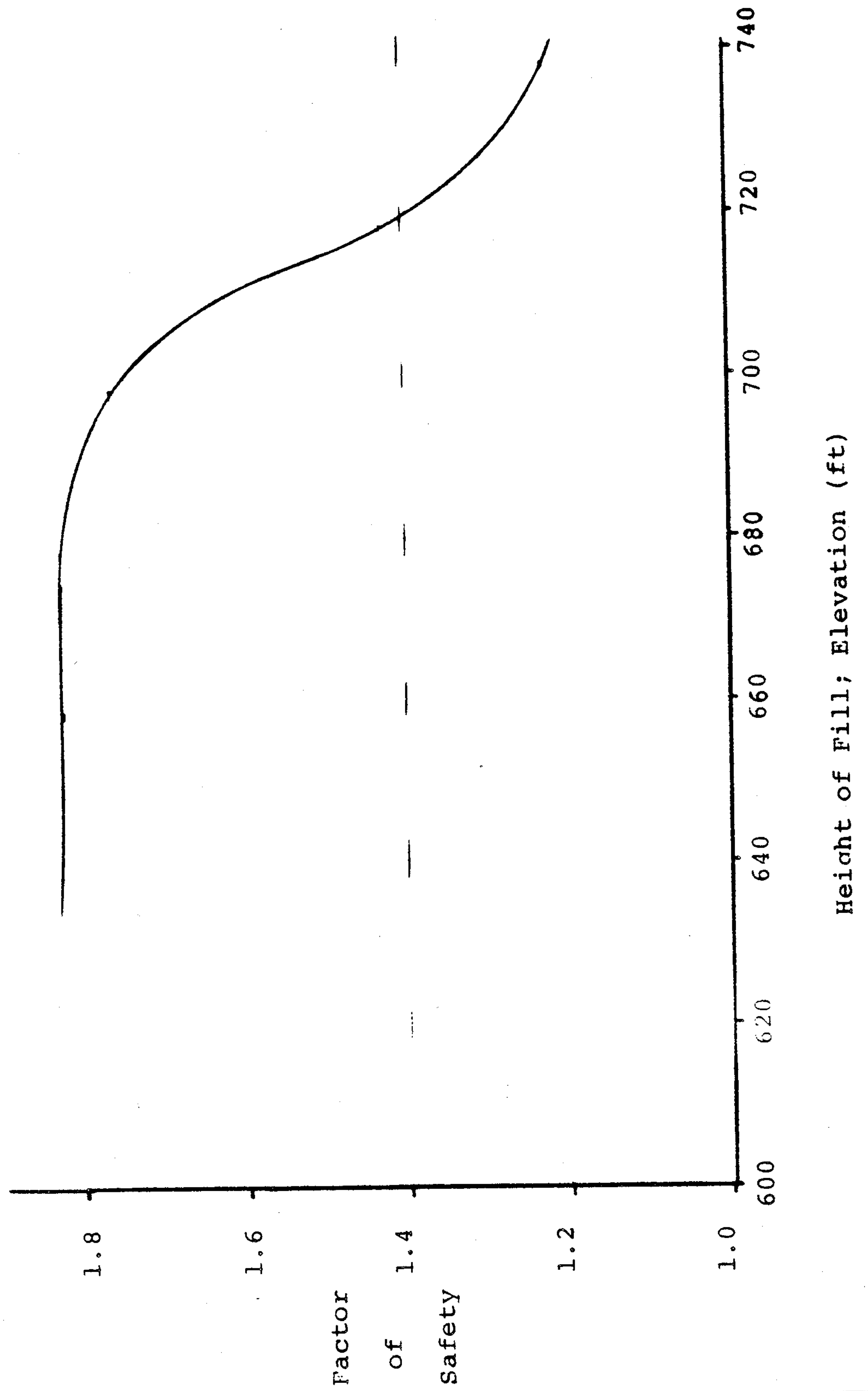


Figure 3
Height of Fill vs. Factor of Safety

Gavin - Stingy Run
Fly Ash Dam Raising
Final Design Report

300 days. Therefore, it is realistic to expect sufficient dissipation of pore pressures during the 2 year construction period to allow complete construction.

To monitor the pore pressures during construction, piezometers will be located in the foundation layers. Section 6.0 will discuss the location of these piezometers. To determine the criteria to monitor the piezometers, the final embankment height was evaluated with effective parameters and a series of Ru values. Figure 4 shows that as long as Ru values over .328 are not generated the factor of safety will be maintained above 1.4.

Therefore, the dam raising can be constructed to elevation 720 and maintain a factor of safety of at least 1.4. The dam raising may continue to the final elevation as long as the measured Ru values do not exceed 0.328 and maintain a factor of safety of at least 1.4. The upstream analysis remains unchanged from the preliminary design report.

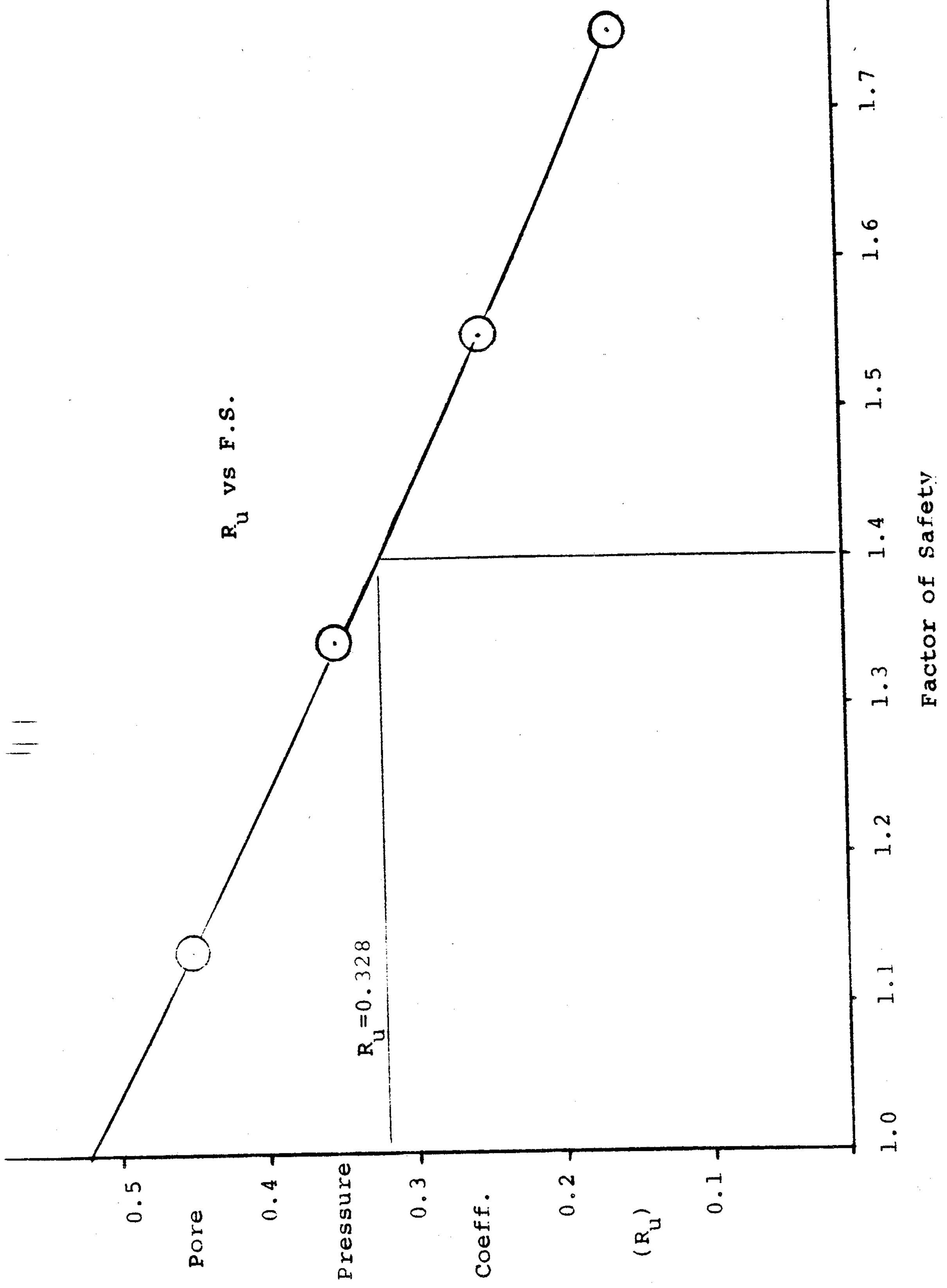


Figure 4
Pore Pressure Coeff. vs. Factor of Safety
Final Configuration

Gavin - Stingy Run
Fly Ash Dam Raising
Final Design Report

5.2 Dam Raising - Long Term Stability

The Preliminary Design Report lists two cases in which the factor of safety drops below the Corps of Engineers' recommended criteria; (1) the upstream partial pool slope, and (2) the downstream, full pool deep sliding slope. In both cases the Corps of Engineers recommend a factor of safety of 1.5. The minimum partial pool slope was reported to be 1.42 which occurred at an elevation of 685 feet. The downstream deep sliding surface was reported to be 1.46.

The partial pool analysis for the Preliminary Design Report assumed a phreatic surface which varied linearly from pool level to the toe of the existing 2.5h:1v downstream slope. In reality the piezometers show an existing phreatic surface that drops to a point 60 feet below this point. This occurs when the pool is at at elevation of 680 feet. Paralleling the existing surface until it intercepts the new drain provides a more realistic solution (see Fig 5). Using this the

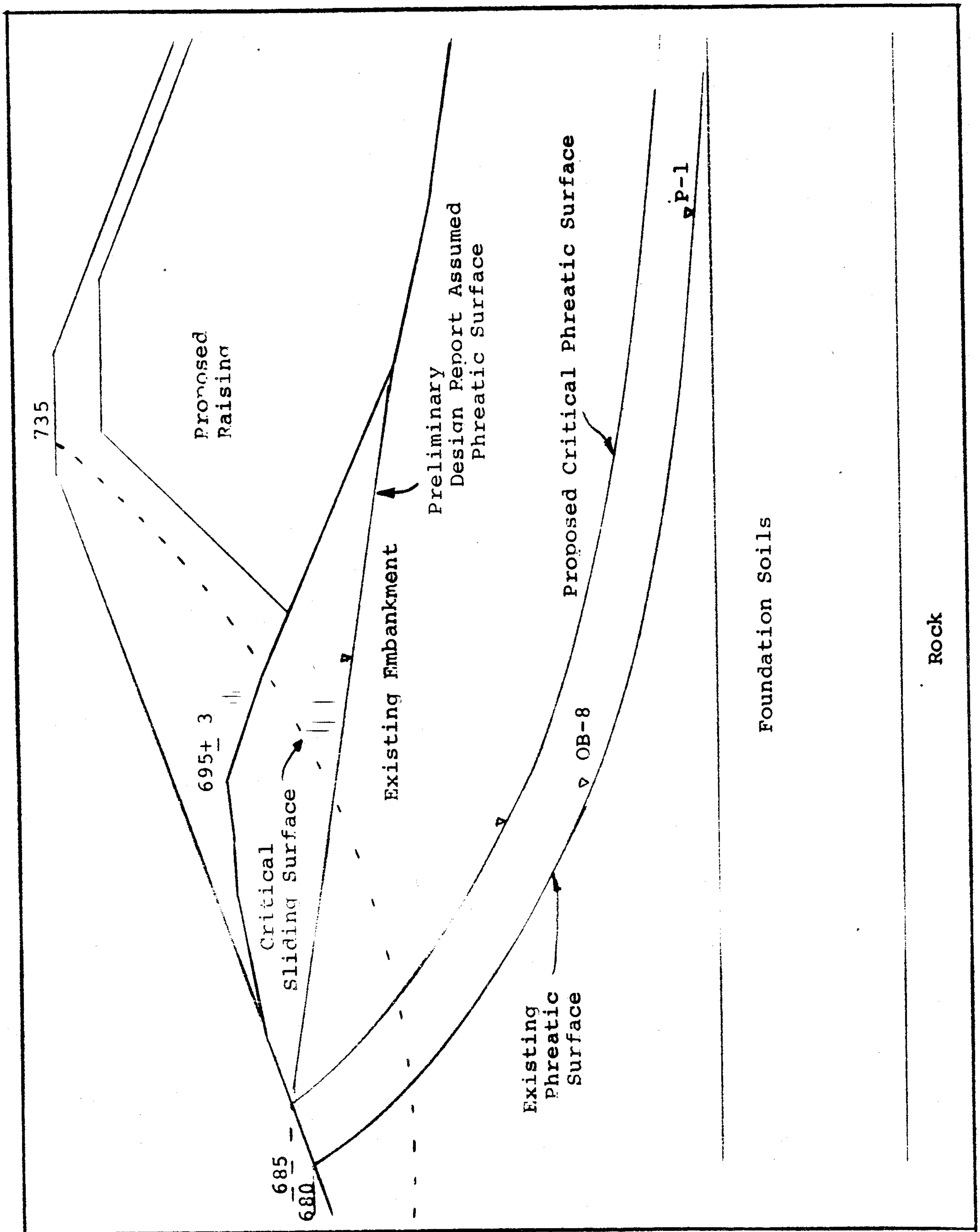


Figure 5
Existing Phreatic Surface &
Phreatic Surface for Critical
u/s Partial Pool Stability Analysis

Gavin - Stingy Run
Fly Ash Dam Raising
Final Design Report

minimum factor of safety for the upstream partial pool is 1.53 at elevation 685 feet. The pool will reach this elevation within 2 years after the completion of the dam raising. From this point on the upstream slope will become increasingly more stable with the weight of the water and downstream seepage gradients.

The upstream seismic analysis was performed on the full pool configuration. The analysis yielded a factor of safety which decreased to 1.37. This is well above the Corps of Engineers' recommended criteria of 1.1.

The downstream deep sliding surface was previously reported to have a factor of safety of 1.46. The minor configuration changes did not significantly alter this factor of safety. However, both the unit weight and shear strength parameters were re-evaluated.

The laboratory shear strength test for the random fill material indicated an effective friction angle of 27 degrees. A preliminary effective friction angle of 25 degrees was used in the initial analyses. The density of the random fill used in the analysis was 130 pcf, moist, and 133 pcf saturated. The laboratory tests in the random fill had an averaged insitu unit weight of 136 pcf. Using 136 pcf as the saturated unit weight of the random fill combined with an effective friction angle of 27 degrees yields a factor of safety of 1.5 .

When the seismic load was applied to the downstream, deep seated failure the minimum factor of safety was 1.22. Shallower failure surfaces yielded a minimum seismic factor of safety of 1.27. These are above the Corps of Engineers criteria of 1.1.

5.3 Spillway Dam - End of Construction Stability

The spillway dam internal configuration was changed to incorporate a wider clay core. Therefore, this design is based upon placement of all core material at a moisture content up to 3% greater than optimum. This design will allow some tolerance of wet conditions. The allowable factor of safety by Corps of Engineers criteria is 1.3. The minimum upstream end of construction factor of safety is 1.33 and the downstream is 1.62. Both factors of safety are within acceptable limits.

5.4 Spillway Dam - Long Term Stability

The long term analysis for the spillway dam includes static upstream and downstream analysis for full pool, upstream analysis for partial pool, and seismic analysis for upstream and downstream, full pool conditions. All factors of safety exceed the Corps of Engineers criteria of 1.5.

5.5 Stability Summary

The following summarizes the revised stability analyses:

Dam Raising

Downstream Wedge Analysis

min. F.S. = 2.35 EOC (Shallow Slide)*

min. F.S. = 1.40 EOC (Deep Slide)**

min. F.S. = 1.52 SSS (Shallow Slide)

min. F.S. = 1.50 SSS (Deep Slide)

min. F.S. = 1.27 SSSQ (Shallow Slide) Seismic

min. F.S. = 1.22 SSSQ (Deep Slide) Seismic

<u>Upstream - Modified Bishop Analysis</u>

min. F.S. = 2.59 EOC *

min. F.S. = 1.84 SSS (Full Pool)*

min. F.S. = 1.53 SSS (Partial Pool @ 685 Ft.)

min. F.S. = 1.37 SSSQ Seismic

* Denotes no change from Preliminary Design Report

** Denotes F.S. dependent upon the measured pore pressures during construction having an Ru value 0.328.

<u>Spillway Dam</u>

<u>Downstream - Modified Bishop Analysis</u>

min. F.S. = 1.62 EOC

min. F.S. = 2.05 SSS

min. F.S. = 1.78 SSSQ Seismic

Upstream - Modified Bishop Analysis

min. F.S. = 1.33	EOC
min. F.S. = 2.08	SSS (Full Pool)
min. F.S. = 1.58	SSS (Partial Pool at Elev. 706 ft.)
min. F.S. = 1.59	SSSQ Seismic

EOC = End of Construction Parameters

SSS = Steady State Seepage Parameters

6.0 **INSTRUMENTATION:**

The instrumentation program consists of 51 piezometers and 18 surface monuments to monitor the conditions of both dams during both the construction period and service life of the embankment structures. The piezometers will be either pneumatic operated devices or open tube standpipes. The surface monuments will be pre-cast or cast-in-place concrete topped with a permanently identifiable object. Reading schedules for all instruments will be dependent upon life of the structure and pool elevation, and will be specifically addressed in the operations manual.

The existing instrumentation will be completely replaced with the exception of instruments located outside of the construction area and observation wells OW-18 and OW-19. The existing monitoring devices were established to monitor a homogeneous earthfill without any distinctive zonal characteristics. The new piezometer instrumentation plan will attempt to distinguish the seepage flow characteristics of the core materials, abutment, foundation, and drains. To make these distinctions piezometers, and not observation wells, are required. All instrumentation locations are shown on drawings numbered 12-3000C, 12-3000D, and 12-3000E.

6.1 Piezometer Types:

There will be three basic types of piezometers used in this project; (1) a standard pneumatic piezometer, (2) a push point pneumatic piezometer, and (3) an open tube standpipe type piezometer.

The standard pneumatic piezometer will be placed in a borehole with the tip developed in a 1 to 2 foot column of sand. These will be installed only in saturated materials. The devices will be 1 inch diameter, 3 tube, low diaphragm displacement piezometer equivalent to a SINCO 514178 or Petur P-100.

The push point piezometer will be the same basic devices as described above, but will be installed in well point tips. These tips will be pressed into the base of a borehole. This will minimize the volume of filter space to be re-watered when installed in non or partially saturated materials. These piezometers will have high air entry pressure filters such as a Coors Alumina filter.

The open tube piezometers will be 1 inch I.D. PVC pipe with 2 to 5 foot lengths of porous High Density Polyethylene tubing. A sand column will be used to develop the zone desired with the pourous H.D.P.E. tubing at the base of the sand column. Where open tube piezometers are developed in bottom ash zones the columns will be developed with bottom ash.

6.2 Dam Raising - Foundation Piezometers:

A total of 12 pneumatic piezometers will be installed in the foundation soils beneath the existing structures. These devices will be numbered NP-1 thru NP-12. These piezometers will be installed prior to construction in order to monitor the excess pore water pressure changes during construction. Devices NP-1 to NP-4 will be placed in the predominately sand layer above the clay layer. Devices NP-5 to NP-8 will be placed within the clay layer itself. Devices NP-9 to NP-12 will be placed in the sandy layer below the clay layer. Because valley alluvial deposits vary to a large extent, it is not expected that the diamond shaped plan view pattern will be able to place all of the piezometers in exactly the same material types. However, the layering of sands and silts should generate an areal response to the construction loads. Shelby tube samples will be obtained in the development zone of each device so that the material of implantation can be accurately defined.

To serve as secondary reading devices during construction and operational life, observation wells OW-18 and OW-19, and piezometers P-2 and P-3 will be left in place. All four of these instruments were installed in 1985. These instruments extend into the foundation sands.

6.3 Dam Raising - Existing Embankment Piezometers:

Pneumatic piezometers NP-13 thru NP-24 will be installed in the existing embankment immediately prior to construction. Many of these piezometers will be installed in partially saturated earth materials. Therefore, accurate pore pressure readings will not be obtained until either the phreatic surface rises or the material compresses to the point of saturation. For this reason, these piezometers will be the point driven style of piezometer to minimize the volume space around the instrument that will be required to re-saturate prior to giving accurate readings.

Instruments NP-13 thru NP-18 will monitor the pore pressures in the existing embankment along potential downstream shallow sliding planes.

Instruments NP-19 thru NP-24 will monitor the pore pressures along potential upstream sliding surfaces. The deeper instruments will monitor pore pressures along potential end of construction failure planes and the higher instruments will measure pore pressures along potential long term failure planes. These instruments will supply actual seepage pressure information throughout the life of the structure that would be otherwise unavailable due to their upstream location.

The downstream pneumatic devices will be backed up with open tube piezometers OB-20 thru OB-22 and OB-28 thru OB-30. These piezometers will be placed after construction in the same zones as the pneumatics. The open tube piezometer will have 20 ft. sections developed in the zone of interest.

6.4 Dam Raising - Drainage Blanket Piezometers:

The bottom ash drainage blanket will be monitored with open tube piezometers OB-23 thru OB-27. The porous nature of the bottom ash does not require pneumatic devices. These instruments will have 10 foot sections developed and will be placed at the bottom ash/embankment interface to measure the flow depth of the drainage blanket. These piezometers will be installed upon completion of the dam.

6.5 Dam Raising - Clay Core Piezometers:

Pneumatic piezometers NP-25 thru NP-27 will be installed in the new clay core material during construction. These instruments will be located to measure pore pressure along potential upstream failure planes. These devices will also have to be the point driven style piezometer because this will be a partially saturated zone for many years. However, because it is an upstream face clay core open tube piezometer would not remain accessible due to the rising pool.

6.6 Dam Raising - Abutment Piezometers:

The upper sandstone zones in both abutments will have open tube piezometers. Piezometers OB-32 thru OB-36 will be installed in the abutments upon completion of the dam. The soil portion of these piezometers will be cased to rock to mitigate the potential for shearing at the soil/rock interface. These piezometers will be used to monitor the flow potential from the abutments into the blanket drain of the dam.

6.7 Spillway Dam - Clay Core Piezometers:

The clay core will have two open tube piezometers numbered OB-37 and OB-38. These piezometers will have 25 foot sections developed beginning at the base of the clay core section. The instruments will monitor the rise in the phreatic surface as the operating pool increases.

6.8 Spillway Dam - Drainage Blanket Piezometers:

The bottom ash drain/filter will be monitored at three points with open tube piezometers. Piezometers OB-39 and OB-41 will be located near the abutments. Piezometer OB-40 will be located along the center line of the dam. The elevational difference in readings between the abutment and centerline instruments should provide some indication of the inflow from the abutments. The developed sections will be based at the rock/bottom ash contact and extend through the bottom ash.

6.9 Spillway Dam - Foundation and Abutment Piezometers:

The predominate sandstone seams in the abutments and foundations will be monitored with piezometers OB-42 thru OB-46. These piezometers will be open tube piezometers 10 to 25 feet of section developed in the sandstone seams. These piezometers will be used to monitor seepage from the abutment and foundation areas of the dam.

6.10 Dam Raising - Settlement Monuments:

A total of 15 permanent settlement monuments will be placed during the dam raising construction. These monuments will be placed as the dam progresses in elevation. The settlement monuments are expected to provide a more global representation of the performance of the dam. The monuments will provide supplemental information to evaluate the pore pressure dissipation in the foundation materials.

6.11 Spillway Dam - Settlement Monuments:

The spillway dam will have three permanent settlement monuments along the crest of the dam. The limited number is due to the rock foundation of this embankment. The only settlements expected are due to the elastic compression and consolidation resulting from self weight of the embankment.

7.0 ADDITIONAL HYDROLOGIC INVESTIGATION

Additional hydrologic studies were undertaken at the request of our independent consultant reviewing the project and as to serve as additional technical suport to the two spillway options addressed in the report. The hydrologic studies addressed the following conditions.

1. Since the capacity of the existing principal spillway is such that it takes greater than 7 days to drain the flood waters, what are the consequences if the 6-hour PMF occurs back to back, assuming Option A? If a 24-hour storm is more hydrologically critical, will the dam be overtopped?

2. If Option B is selected, should the crest of the emergency spillway be placed lower than elevation 730 feet? Back to back 6-hour PMF's were modeled to address this concern.

Item 1. The spillway design Option A utilizes the principal spillway and reservoir storage to handle the design flood. No provisions are made for an emergency outlet.

Flood routings were conducted assumintg two 6-hour PMF's occurred within a 24 hour period. The initial water surface was set at 726.0, maximum normal pool. The second storm was assumed to begin when the pool elvel receded to elevation 730 from its maximum level of 731.1 feet during passage of the first storm. This modeling resulted in a maximum pool level of 734.7 feet.

The PMP for a 24 hour duration is 35 inches of rainfall. Assuming no discharges through the principal outlet, this flood event can be completely contained within the reservoir. The water surface will rise to elevation 732.7 feet.

In either condition, selection of Option A for the emergency spillway will not result in overtopping of the dam at its proposed final crest of 735 feet.

Item 2. Option B consists of a 100-foot grassed lined emergency spillway at elevation 730.0.

As described under Item 1, two 6-hour PMF's were routed through the reservoir. Discharges through the principal spillway were neglected for this modeling. Results of the flood routing showed the storm scenario would increase the water level to elevation 733.7 feet when the second storm began with the reservoir level at 730 feet.

In conclusion, the reservoir and appurtanences in Option B will provide adequate protection against overtopping, as designed.

(NOTE: APPENDICES NOT INCLUDED HEREIN)

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Ohio Power Company

Gavin Plant

Cheshire, Ohio

Proposed Dam Raising For Phase II Stingy Run Fly Ash Retention Pond

APPENDIX A

A-1 Dam Raising Soil Boring and Rock Core Logs

A-2 Cone Penetration Boring Logs

A-3 Dam Raising Abutment Pressure Tests

A-4 Piezometer Installation Reports

A-5 Spillway Dam Soil Borings and Rock Core Logs

A-6 Spillway Dam Abutment and Foundation Pressure Tests

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

March, 1986

Prepared By
Geotechnical Engineering Section
~~Civil Engineering~~ Division
American Electric Power Service Corporation

PRELIMINARY DESIGN REPORT
APPROVED
Robert L. Gottemoeller
Chief, Division of Water
CLASS I
Date: AUG 4 1986

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Proposed Dam Raising
For
Phase II Stingy Run Fly Ash
Retention Pond

1501:21-5-02 Preliminary Design Report

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Proposed Dam Raising for Phase II Stingy Run Fly Ash Retention Pond

1501:21-5-02. Preliminary Design Report

1501:21-5-02 Preliminary Design Report

1.0 General Project Description

Since 1975 the Ohio Power Company's 2600 megawatt Gavin Power Plant has been serviced by the Stingy Run Fly Ash Dam. Shortly after 1988 the current fly ash facility will reach its maximum storage capacity. It is proposed that the existing dam be raised by approximately 40 feet, a new dam be built in the existing emergency spillway channel, and new emergency flood routing methods be provided. This raising is part of the phased construction which was planned when the dam was originally designed and built in 1973. This work is planned to provide additional fly ash storage capacity into the year 2012. This report presents the preliminary design of these new facilities.

1.1 Existing Conditions:

The existing dam site is located in the Stingy Run watershed. This watershed is in the northwestern portion of Gallia County in section 15 of Township 6N, Range 14W, approximately 2 miles northwest of the Gavin Power Plant. The center of the crest of existing dam has the State coordinates of N 351,360 and E 2,101,120. The center of the existing emergency spillway has the state coordinates of N 348,720 and E 2,101,040. The dam is accessed from Cheshire Township Road 17 approximately 4600 feet west of State Highway 554.

The dam is an earth fill structure with a rip rap covered upstream face and grass covered downstream slopes. The upstream slopes are 3h:1v with an 8h:1v stabilizing berm which begins at elevation 640 feet. A 30 foot wide berm was constructed at elevation 670 feet on the upstream slope. The downstream slopes were constructed at 2.5h:1v with an 8h:1v stabilizing berm beginning at elevation 650 feet.

The maximum height above original ground along the centerline of the dam is 105 feet. The crest of the dam is approximately 1600 feet long. The dam was constructed to arch upstream with a 3000' radius. The crest of the dam was constructed with a high point in the center of the valley to compensate for expected settlements. A total of 8 feet of settlement was expected of which less than 2 feet has occurred since construction was completed 10 years ago. As a consequence, the crest elevation varies from a height of 698.8 feet at the center to a low of 693.6 feet at the north abutment.

The emergency spillway is an open channel earth and rock cut and has a minimum crest elevation of 684.6 feet and a minimum control section width of 220 feet. The inlet and outlet slopes are approximately 1% and 2%, respectively. The side slopes of the channel are 3h:1v. The existing emergency spillway will only be activated by storm inflows resulting from a 50 year storm event or greater.

The principal spillway is presently used to discharge all water from the reservoir. It is a tower-type decanting structure with a concrete drop-inlet leading to a 48 inch RCPP discharge pipe. The pipe passes under the north abutment of the dam. The pipe was installed in an incised trench and is supported by a concrete cradle. The pipe was designed to flow partially full for pool hydraulic heads resulting from routing a 50-year storm or less. These two spillway structures safely pass all storm water runoff resulting from the 1.5 square mile watershed.

According to Chapter 12 - Classification and Design Criteria, 1501:21-13-01. Classification of Dams, Section A(1), of the Ohio Laws and Administrative Rules the existing structure is a Class I dam. Several residential dwellings begin 2200 feet downstream of the centerline of the dam. Therefore, failure of the dam would result in probable loss of human life and serious damage to homes. The existing dam has a storage volume greater than 5000 acre feet and has a height greater than 60 feet. According to Ohio Law and these facts the existing dam is classified as Class I.

1.2 Proposed Dam Raising:

It is proposed that the crest of the existing dam be raised from its current elevation of 696± 3 feet to an elevation of 735 feet. The proposed raising will have 2.5h:1v upstream and downstream slopes, a tapered upstream clay core, a wide bottom ash internal zone, and a tapered downstream compacted shale shell with a 30 foot wide berm. The upstream clay core will taper from a maximum horizontal width of 75 feet at an elevation of 695 feet to 30 feet at an elevation of 726 feet. The proposed raising will align with the existing upstream arch. The bottom ash internal zone will function as a drain, a filter, and structural fill. The bottom ash will abut the clay core material, the downstream abutments and the downstream compacted shale shell. The downstream compacted shale shell will taper from 3 feet to 35 feet thick (perpendicular to the slope) and will function as an erosion protection blanket for the internal bottom ash fill and downstream stability berm.

The upstream slopes will be protected with rip rap. The rip rap will be an extension of that which was placed on the existing dam. The downstream slopes will be topsoiled and seeded.

At the downstream toe of the proposed raising a seepage collection ditch will be constructed. All seepage that passes through the drain will be collected in this ditch and directed to the existing discharge basin downstream of the principal spillway outlet structure on the north abutment of the dam.

The proposed dam raising will utilize the existing principal spillway that was installed to its maximum elevation of 731 feet. However, at the conclusion of this raising, access stairs to the spillway structure will be furnished on the downstream slope.

The proposed dam raising will not change the classification of the dam. The downstream residential areas have not decreased, the dam is being raised higher and the volume storage becomes greater. Therefore, the classification according to Ohio Law remains class I.

1.3 Spillway Dam and New Emergency Spillway

The existing dam is serviced by a 230 foot wide open channel emergency spillway. The control section of this channel has a minimum elevation of 684.6 feet. Therefore, to increase the storage capacity of the reservoir, this channel must be closed off and other means to route excess storm runoff determined. It is proposed that the closing be accomplished by constructing a earth fill embankment in the spillway channel.

The spillway dam embankment would be constructed to an elevation of 735 feet to match the dam raising on the existing embankment. The proposed dam would have 2.5h:1v upstream and downstream slopes. The upstream slopes will be lined with rip rap and the downstream slopes will be vegetated. The proposed dam will have a random fill upstream zone, central clay core zone, downstream random fill, and a bottom ash chimney drain and drainage blanket. The existing spillway is cut through rock. The proposed spillway dam will be founded on this competent rock.

The volume of storage from the base of the proposed spillway dam at elevation 686 feet to the maximum operating pool at elevation 726 feet is 13,400 acre-feet. The large storage volume classifies this structure as a class I dam according to the Ohio Law criteria.

After the existing spillway channel has been closed with the proposed spillway dam other means for safe passage of the design storm shall be provided. There are two options proposed in this report. Option A proposes a variance to store the runoff from a PMP event and empty 80% of the runoff within 10 days. Option B proposes a 100 ft. wide grass lined, trapezoidal channel to store and pass the PMF event. Both of these options are discussed in greater detail in Section 6.1 of this report.

2.0 MAPS

The cover sheet entitled "Design Drawings, Proposed Dam Raising for Phase II Stingy Run Fly Ash Retention Pond" has a composite of U.S.G.S. 7.5 minute quadrangle maps showing the location of the proposed structures. The project site is contained on the Addison, Ohio - W.Va. 7.5' quadrangle. However, a portion of the Cheshire, W.Va.-Ohio 7.5' quadrangle is added to show the full extent of the property lines and reference the disposal site with the plant site.

3.0 Geologic and Physiographic Setting

The geologic information reported herein results from field reconnaissance and the references listed at the end of this section.

3.1 Area Geology

The Stingy Run Fly Ash Dam lies within the Appalachian Plateaus Physiographic Province. The site region is underlain by a thick sequence of Pennsylvanian Age sedimentary rocks overlying Precambrian crystalline rocks. The thickness of sedimentary rocks exceeds 5,000 feet and dips gently to the east - southeast.

3.2 Regional Geology

The region of interest lies on the eastern limit of the Cincinnati Arch. Bedrock consists of shales, sandstones, coals, and limestone of Pennsylvanian age. In general the rocks thicken and dip toward the Appalachian geosyncline to the east. Therefore, the oldest rocks are exposed along the Arch, striking north from Cincinnati, and progressively younger rocks outcrop to the east. These rocks strike north-south to northeast-southwest. The dip is low and averages about 30 feet per mile.

This region is unglaciated but several stages of erosion resulting from outwash during preglacial, glacial, and postglacial times have carved into the bedrock surface. The drainage patterns of today essentially follow the erosion patterns results from the outwash of the later glacial stages. Residual soils remain over bedrock highlands where the outwash did not erode. Preglacial drainage patterns were markedly changed during the Pleistocene by glacial outwash sediments. Valleys formed during preglacial time were filled with unconsolidated deposits. Depths to bedrock, therefore, may vary considerably over short distances from bedrock highlands to buried valleys.

3.3 Site Geology

Bedrock under the existing dam consists of a series of clay shale, siltstone, sandstone and thin limestone beds. These beds are essentially flat lying although there are minor structural and lithologic variations across the existing dam axis.

Clay shales are the predominant rock type in the region. These consist of red or maroon clay shales and clay shales variously described as gray, blue, or green. The shales are interbedded with variagated strata containing thin beds of sandstone, red and gray shale, and some limestone. A generalized stratigraphic section as found in the original project drilling and verified for the proposed dam raising project is as follows:

Unit No.	Rock Description	Average Thickness in feet
1.	Sandstone, brown to gray (caps hilltops)	60
2.	Shale, silty, gray, sandstone partings	15
3.	Coal, Redstone No. 8A (underclay)	4
4.	Shale, sandy, gray on left abutment; sandstone, argillaceous, gray, with thin gray shale on right abutment	12
5.	Shale, red and gray, sandstone interbeds to 10' limestone interbed to 10' at top	60
6.	Shale, red	30
7.	Sandstone, argillaceous, gray, and interbedded shale, red; limestone interbed	35

8.	Shale, red, remolded (at top of rock in valley floor)	50
9.	Shale, gray, local limestone interbed to 8'	20
10.	Shale, red, remolded, local thin lenses of gray sandstone and shale	35
11.	Shale, gray, sandy or silty with local red shale interbeds; locally siltstone, gray	25
12.	Shale, gray; limestone, green; shale, red	10
13.	Sandstone, argillaceous, gray, locally shale, sandy, gray	15 limit of original exploration program

Detailed lithologic descriptions of the aforementioned stratigraphic units are listed below by Unit number.

(Another stratigraphic column of bedrock in the area, taken from a published bulletin, giving formation names and showing deeper stratigraphy, will be found in Table 3.1 at the end of this chapter.)

Unit 1: A massive sandstone unit, (unit 1 listed above in the stratigraphic column) 60 feet thick, caps the hilltops in the area. It is light gray to buff and weathers to a rusty brown. It is very fine-grained to medium-grained, is porous and is slightly to well cemented. It is cross-bedded and thin laminae of black organic material are sometimes found along cross-beds and bedding planes. Bedding thicknesses reach 10 feet. The sandstone appears to have a unconformable contact with the underlying shale. The contact is irregular and in some areas around the reservoir several feet of shale appear to be eroded.

Unit 2: The sandstone is underlain by 12 to 15 feet of sandy or silty gray shale with sandstone partings (2). This bed immediately overlies the Redstone No. 8A coal seam (3). The coal bed, which is about three feet thick, was reported to have been strip mined and augered by closely spaced horizontal holes which extend several hundred feet into the bed.

Unit 3: Underclay was reported under the coal bed. Core loss in the coal sections makes determination of stratigraphic continuity of the underclay unit impossible. A 3 point plot of the coal floor elevation in the reservoir area determined a region strike of directly north with a dip of 1° to the East.

Unit 4 and 5: Underlying the coal and underclay are two variable units (4 and 5) consisting of relatively thin interbedded clay shales and sandstones with a single limestone unit. The upper part of this section consists of sandy gray shale and argillaceous, gray sandstone (4). The thickness is about 12 feet. The unit is mainly shale on the left abutment and sandstone on the right abutment.

Below this sandy zone is a limestone bed, and gray and red shale beds interbedded with sandstone. The correlative limestone unit is found in both dam abutments at about elevation 715 feet. Its thickness varies from 6 feet on the left abutment to 10 feet in the area of the right abutment. The limestone is blue-gray and argillaceous. The shales in this unit are mottled red and gray and are interbedded with thin beds of sandstone.

Where Unit (5), is near the top of rock the sandstone zones within it are associated with water takes in the original pressure tests of nearly 30 lugeons. Where the same unit is considerably below the rock surface, the water takes appear to be negligible. The top of the hill above the left abutment is one such area.

The lower shalier section of this unit gave water takes of less than five lugeons. The weathered and open nature of this sandstone and underlying shale indicates probable high permeability.

Unit 6: A 30-feet thick red clay shale unit (6) is found underlying the interbedded shales and sandstones. It is locally weathered and shows yellowish mottling. The unit is stratigraphically equivalent to the bluish gray clay shale which is slightly sandy.

Unit 7: A third variable unit (7) is found below the red clay shale. This unit is similar to unit 5 and consists of thin sandstone beds interbedded with thin red and gray shale beds. The shale and thin sandstone beds in this unit appear to have been remolded. This will be discussed more fully in the description of unit 8.

Unit 7 contains some zones of high water takes in water pressure tests. These high takes appeared on both abutments where this unit is very close to top of rock. The lower portion of this unit penetrated showed negligible water takes. Water takes in holes near maximum pool elevation, where the rock unit is covered by overlying rock strata show a range of permeabilities. Originally, in this unit the left abutment the water takes were less than 5 lugeons. On the right abutment the water takes ranged from 8 to 17 lugeons for rock sections measuring 56 to 24 feet respectively.

Unit 8: Below the variable unit is a thick red clay shale section (8). This unit is at the top in the valley floor. The unit is 50 feet thick. In several of the original borings the entire thickness of this unit appears to have been remolded. In the original borings in the valley floor remolding appears to be absent or in thin zones within the unit.

The term "remolded" has been used to indicate rock which appears to have been disturbed and which has a brecciated appearance. The red clay shales appear to have been particularly susceptible to this type of disturbance but the same process has obscured natural bedding in thin gray shale and sandstone beds associated with the red shales. The disturbed bedding is frequently associated with weathering and oxidation giving a color mottling to the rock in addition to a textural mottling. The brecciation in the rock is now healed and the rock is firm. The original horizontal bedding planes have been obliterated in this process. The original core borings that were drilled at 30° angle from the vertical, break perpendicularly to the core axis, rather than at the expected bedding angle of 30° to the core axis. Joints in the remolded rock commonly have a slickensided appearance. One four-inch specimen of rock displays four differently oriented planes of apparent remolded slickensiding.

Unit 9 thru Unit 13: Below the red clay shale section are additional beds of gray and red shales. The upper unit is gray to blue silty shale or siltstone (9). This unit is 20 feet thick. Below this a 35 foot thick red clay shale unit (10) is encountered. Underlying the red shale is a 25-foot thick unit of sandy or silty gray shale or a shaley gray siltstone (11), a 10-foot thick variable unit (12) containing red and gray shales and a continuous thin limestone bed, and a 15-foot thick sandstone bed (13). This bed is an argillaceous, gray sandstone and marks the vertical limit of exploration in the floor of the valley.

The original water pressure tests of these predominantly shale units in the floor of the valley show negligible water takes in both vertical and angle holes. The upper few feet of the rock were not pressure tested, however, and may be more permeable due to open joints.

The clay shale sections, both red and gray, are relatively soft, weak rock units. Their bearing capacity, however, was sufficient to support the load imposed by the existing dam and will sufficient to support the load of the raising. These beds are highly susceptible to slaking, as shown by dried samples of clay shales which break down immediately upon immersion in water. An example of this can be seen where weathering of the gray clay shales underlying the sandstone caprock has occurred. Shales beneath the dam must be expected to behave in similar manner. Care must, therefore, be taken immediately upon excavation of the overburden and weathered rock, under the extension of the impervious core section, to protect the newly exposed rock surface from losing its moisture.

Joints could be observed in outcrop only in the massive sandstone unit on the hilltops and in the gray shale immediately underlying the sandstone. The units showed three major sets of joints. One set strikes approximately N 50° W and dips 80° SW. Another set strikes N 70° E with a vertical dip. The third set strikes N 30° - 40° E and dips about 80° - 85° SE. The joints do not extend throughout the entire exposure. They are open and continuous, however, across the sandstone-shale contact. Joints are generally spaced 10 to 20 feet apart but are locally spaced a few feet apart.

Open joints and bedding planes in the subsurface are observed in the cores to the full depth of exploration, but water pressure tests indicate that the shales in the valley floor are relatively impermeable. Staining is found along some joints especially in the sandstone beds. Weathering is common in zones in the clay shales throughout the 150 feet drilled beneath the dam in the valley floor. These weathered zones are particularly prominent in the remolded red clay shales.

No evidence of faulting was found within the reservoir and no faults have been reported in the area.

3.4 <u>Seismic Conditions</u>

Southeastern Ohio is an area which is not subject to frequent seismic disturbances. Earthquakes have been recorded in the area, however, with an intensity of up to VII on the Modified Mercalli Intensity Scale of 1931 (Table 3.2 at the end of this chapter). (Ref. 10).

3.5 Groundwater Conditions

In general, ground-water levels are found to be high in both the valley floor and in the reservoir rim. These levels are generally higher than the proposed maximum operating pool of el. 726 ft. Therefore, seepage outward from the reservoir would be limited to the fly ash dam areas.

Ground-water in the area is of very poor quality and limited in quantity. Changes taking place in ground-water levels are presently being monitored throughout the life of the project.

3.6 Geological Engineering Considerations

Overburden thicknesses will be greatest in the valley floor and will thin going up the reservoir slopes. The highest reservoir elevations will encroach on mine spoil materials on the upper slopes. The thickest strata of alluvial sands occur along the valley bottom.

The overburden consists of alluvial, residual, and spoil materials, whose interrelationships are complicated by local sliding in many areas. The permeability of the slope materials is therefore, likely to change considerably over small distances.

A portion of the overburden from large areas of the reservoir floor and slopes upstream of the existing dam were stripped for the construction of existing impervious core material. Overburden from a ridge downstream of the dam will be used to construct the proposed dam raising and emergency spillway dam. As discussed in the lithologic descriptions, water pressure test data show that the clay shales in the reservoir area are relatively impermeable. Thin beds of sandstone found in two rock units (5 and 7) contain open joints and are permeable, especially when the units are found at the bedrock surface. These may provide a path for potential seepage from the reservoir particularly in the areas of thin divides. Therefore some of the natural overburden blanket was left to protect the stability of these slopes as far as possible.

As the beds are essentially flat-lying and no faulting is indicated, the bedrock stratigraphy will resemble closely that found along the dam axis. Holes drilled around the reservoir rim to determine structure on top of the Redstone No. 8A coal, show that the coal has an south-easterly dip of about 28 feet per mile. At its lowest elevations the coal bed dips below the stage II maximum flood pool elevation during the routing of the flood event's runoff. The coal and its associated underclay are likely to provide potential paths for seepage from the reservoir where the beds are below flood elevation. This occurs in the southern half of the hilltop between the right abutment and the existing emergency spillway. The effects of this permeable zone will be minimized by maintaining a maximum operating pool below the coal seam. The high permeability of the coal horizon is due to the extensive fracturing of the coal from mining its inherent nature and the characteristically open nature of the contact between the coal and the underclay. The permeability of this zone has been shown to be approximately 6 Lugeons as was recorded in previous water pressure tests. In addition, around most of the reservoir coal was mined by large diameter horizontal holes augered several hundred feet into the hilltops. This zone, if below pool elevation, could produce leakage from the reservoir. However, the mining is felt to be so extensive that the maximum operating pool is to be maintained below the minimum coal seam elevation. The pool resulting from flood conditions will be transient and should not significantly increase seepage in the mined areas.

Two thin continuous beds of limestone have been found in the dam foundation. The thickness of each of these beds is less than 10 feet. One of these is found in the valley floor in the area of the dam axis at a depth of 140 feet (within unit 12). The second bed is found high in the abutments near the crest elevation of the stage II dam. The elevations of these beds will rise to the west across the reservoir. The upper limestone bed would therefore be higher, in the western reaches of the reservoir, than maximum pool elevation, and the lower bed would still be protected by overlying shale beds. The limestone in these beds has shown no solution effects and has shown negligible water losses in water pressure tests. The thickness, elevation, and apparent watertight nature of the limestone indicate that they present no potential problem of solution or seepage to the dam foundation or to the reservoir.

TABLE 3.1

Stingy Run

Generalized Section of Bedrock

	Ft.	In.	
Pennsylvanian system			
Monongahela series			
Sandstone, massive	30	1	-caps hilltops
Shale, gray, siliceous	3	5	
Coal and partings, Redstone, Pomeroy, or 8a	2	10	
Clay shale, calcareous, with local deposits of thin limestone, Redstone	3	6	
Shale, with sandstone lenses	21	5	
Coal and parting, Pittsburgh or No. 8	2	4	
Conemaugh series			
Limestone, nodular, or in layers interlain with clay, Pittsburgh	4	0	
Shale, sandy	45	0	Reservoir Section
Sandstone, massive to shaly, Connellsville	30	0	
Coal blossom	--	--	
Limestone, nodular	-	8	
Shale, sandy, with red beds	38	0	Clarksburg
Clay, red, with concretions of hematite	28	0	
Shale, sandy	12	0	
Limestone, impure and sandy in places, Ames	1	6	
Shale, sandy	10	0	
Clay shale, red, with nodular limestone, Round Knob	34	0	KNOWN FOR SLIDES
Coal, thin, Anderson	-	-	
Shale, sandy	33	0	
Limestone, dark gray, dense fossiliferous, local, Cambridge	1	6	
Shale, sandy	20	0	
Limestone and fossiliferous shale. In northern part cherty limestone in two parts separated by 18 feet of fossiliferous shale; in southern part two or more beds of rusty gray limestone interlain with sandy shale, Brush Creek member	30	0	
Shale, sandy	5	0	
Coal, Mason	-	6	
Sandstone, shaly to massive	22	0	
Coal, thin, Mahoning	-	-	
Clay, with ferruginous limestone at top	3	0	
Sandstone, massive, Mahoning	33	0	
Allegheny series			
Coal and black shale, Upper Freeport or No. 7	-	8	
Clay, gray, impure, with occasional nodules of limestone	4	0	
Sandstone, grading laterally to shale	37	6	
Coal, thin to wanting, Lower Freeport or No. 6a	-	-	
Clay, gray, impure, with occasional nodules of limestone	3	0	
Sandstone, grading laterally to shale	50	0	
Coal, persistent, Middle Kittanning or No 6	-	10	

From Lamborn, R. E., 1951, Limestones of Eastern Ohio, Ohio Div. of Geol. Surv., 4th Series, Bull. 49 pp. 204-205.

TABLE 3.2

MODIFIED MERCALLI INTENSITY SCALE OF 1931

(ABRIDGED)

I. Not felt except by a very few under specially favorable circumstances. (I Rossi-Forel scale.)

II. Felt only by a few persons at rest, especially on upper floors of buildings. Delicately suspended objects may swing. (I to II Rossi-Forel scale.)

III. Felt quite noticeably indoors, especially on upper floors of building, but many people do not recognize it as an earthquake. Standing motorcars may rock slightly. Vibration like passing of truck. Duration estimated. (III Rossi-Forel scale.)

IV. During the day felt indoors by many, outdoors by few. At night some awakened. Dishes, windows, doors disturbed; walls make creaking sound. Sensation like heavy truck striking building. Standing motorcars rocked noticeably. (IV to V Rossi-Forel scale.)

V. Felt by nearly everyone, many awakened. Some dishes, windows, etc., broken; a few instances of cracked plaster; unstable objects overturned. Disturbances of trees, poles, and other tall objects sometimes noticed. Pendulum clocks may stop. (V to VI Rossi-Forel scale.)

VI. Felt by all, many frightened and run outdoors. Some heavy furniture moved; a few instances of fallen plaster or damaged chimneys. Damage slight. (VI to VII Rossi-Forel scale.)

VII. Everybody runs outdoors. Damage negligible in buildings of good design and construction; slight to moderate in well-built ordinary structures; considerable in poorly built or badly designed structures; some chimneys broken. Noticed by persons driving motorcars. (VIII Rossi-Forel scale.)

VIII. Damage slight in specially designed structures; considerable in ordinary substantial buildings with partial collapse; great in poorly built structures. Panel walls thrown out of frame structures. Fall of chimneys, factory stacks, columns, monuments, walls. Heavy furniture overturned. Sand and mud ejected in small amounts. Changes in well water. Persons driving motorcars disturbed. (VIII+ to IX− Rossi-Forel scale.)

IX. Damage considerable in specially designed structures; well-designed frame structures thrown out of plumb; great in substantial buildings, with partial collapse. Buildings shifted off foundations. Ground cracked conspicuously. Underground pipes broken. (IX+ Rossi-Forel scale.)

X. Some well-built wooden structures destroyed; most masonry and frame structures destroyed with foundations; ground badly cracked. Rails bent. Landslides considerable from riverbanks and steep slopes. Shifted sand and mud. Water splashed (slopped) over banks. (X Rossi-Forel scale.)

XI. Few, if any, (masonry) structures remain standing. Bridges destroyed. Broad fissures in ground. Underground pipelines completely out of service. Earth slumps and land slips in soft ground. Rails bent greatly.

XII. Damage total. Waves seen on ground surfaces. Lines of sight and level distorted. Objects thrown upward into air.

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7. Lamborn, R. E., 1951 *Limestones of Eastern Ohio*, Ohio Div. Geol. Surv., Fourth Ser., Bull. 49.
8. Patton, F. D., Oct. 23, 1970, Preliminary Slope Stability Study for Conveyor, Wellston Coal Mine, Ohio, Tech. Letter No. 1.
9. Stout, W. and Lamb, G. F., 1939, repr. 1968, Physiographic Features of Southeastern Ohio, Ohio Geol. Surv., Reprint Ser. No. 1.
10. U. S. Dept. of Commerce, 1965, *Earthquake History of the United States, Part 1*.

3.7 Area Topography

The project site lies on the Western edge of the Appalachian Plateaus Physiographic Province in the Unglaciated Allegheny Plateaus Section. The area can be described as a maturely-dissected plateau with a maximum relief of approximately 360'. Remnant ridges of the plateau rise to 850 feet above sea level and are cut by relatively broad, steep-sided valleys down to near the Ohio River base level of 540' above sea level.

3.8 Cultural Features

The fly ash dam was originally constructed in a rural farming area. This proposed raising of the dam does not change or impact any cultural features that are readily apparent.

4.0 CONFIGURATION OF PROPOSED STRUCTURES

4.1 Fly Ash Dam Raising

The plan and sections of the proposed dam raising are shown on the following drawings:

Drawing Number	Title
12-3000	General Arrangement
12-3000B	Geol. Profile along C.L. of Dam
12-3000C	Geol. Profile along Crest of the Dam
12-3000F	Existing Core Trench & New Extension
12-3000G	Grouting Plan & Profile; North Abutment
12-3000H	Grouting Plan & Profile, South Abutment
12-3000J	Layout and Grading Plan
12-3000K	Sections and Details Sheet 1
12-3000L	Sections and Details Sheet 2

4.2 Spillway Dam

The plan and sections of the proposed spillway dam are shown on the following drawings:

Drawing Number	Title
12-3000	General Arrangement
12-3000D	Spillway Dam Layout and Grading Plan
12-3000E	Spillway Dam Profile and Sections
12-3000M	Spillway Dam Excavation Plan - Option A
12-3000MA	Spillway Dam Excavation Plan - Option B

5.0 PROJECT BORING LOGS

5.1 Initial 1972 Borings

The original borings that were performed for the existing dam were submitted to ODNR in February 1973 in the preliminary report prepared by the Harza Engineering Company. Therefore, this report does not contain the original boring logs or pressure tests.

5.2 Dam Raising Boring Program

The following boring programs were performed in 1985 in preparation for the proposed dam raising:

Appendix	Title
A.1	Dam Raising Soil Borings and Rock Core Logs

A.2	Cone Penetration Logs
A.3	Dam Raising Abutment Pressure Tests
A.4	Piezometer Installation Reports

The soil borings contain visual descriptions from split spoon samples and standard penetration test results. Shelby tube samples were obtained for shear strength and consolidation testing. The NQ rock core logs contain visual descriptions, RQD, and % recovery. The cone penetration tests were performed with a Hogentogler Dutch Cone Penetrometer with a 10 sq. cm cone and 150 sq. cm friction sleeve. Pressure tests were performed in accordance with procedures outlined in the U.S. Department of the Interior; Water and Power Resources Service's Ground Water Manual. Multiple pressure tests were performed to characterize the probable conditions of the rock strata tested. Single point, single tube, open system type piezometers were installed in foundation sands below the existing dam and along its profile.

5.3 Spillway Dam Boring Program

The following boring programs were performed in 1985 in preparation for the proposed construction of the spillway dam:

Appendix	Title
A.5	Spillway Dam Soil Borings and Rock Core Logs
A.6	Spillway Dam Abutment and Foundation Pressure Tests

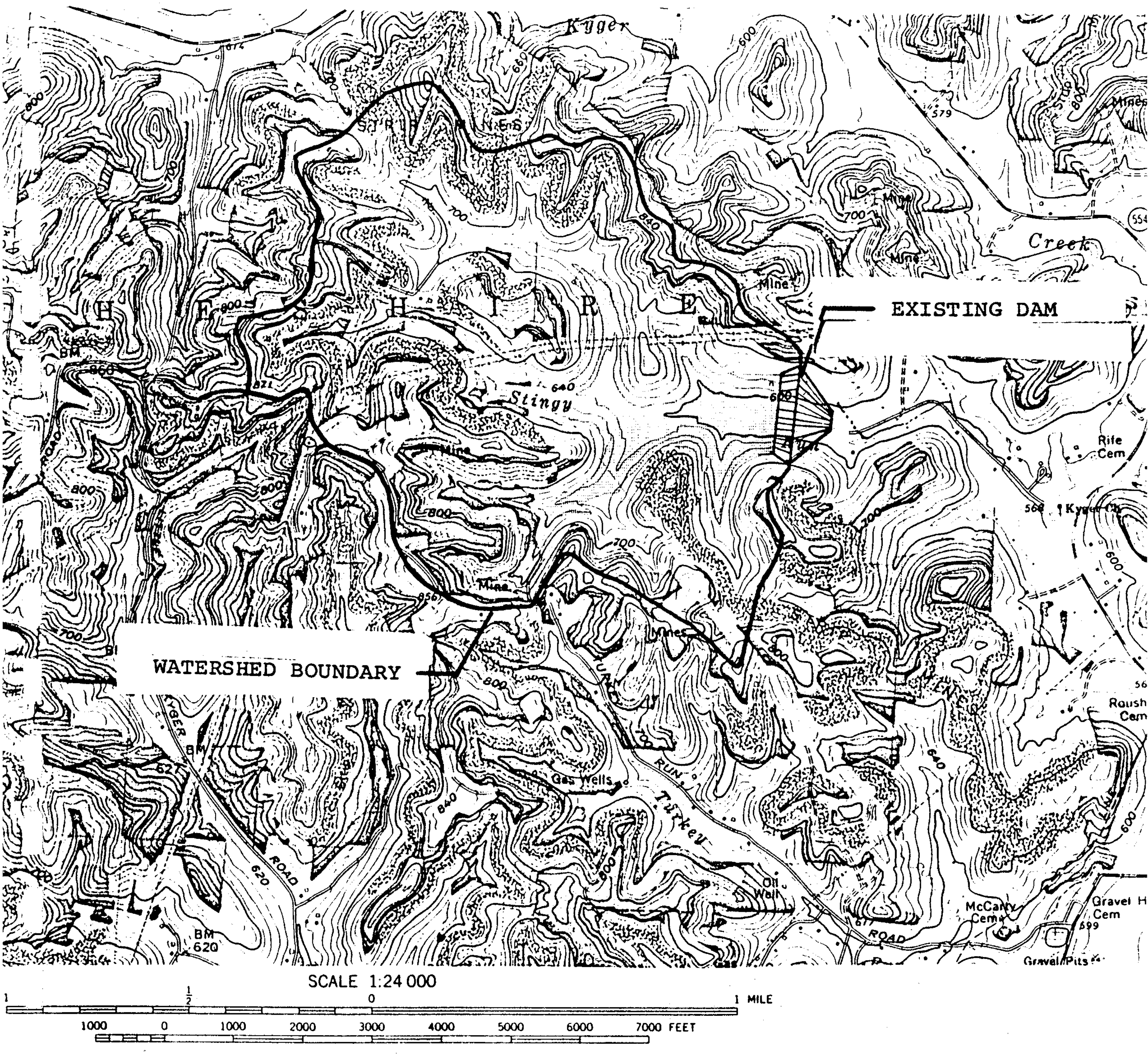
The soil borings contain visual descriptions from split spoon samples and standard penetration test results. Both abutments of the spillway dam were subject to strip mining and reclaim filling. Therefore, continuous standard penetration tests (SPT) were performed in the abutments to differentiate fill from intact materials. NQ rock core was obtained from each abutment and from the emergency spillway floor. The abutments and foundation were pressure tested with the aforementioned methods.

6.0 HYDROLOGY

6.1 GENERAL

The existing hydrologic features of the Stingy Run Fly Ash Dam and watershed are depicted on Figure 6.1. Runoff from the area is conveyed by Stingy Run to Kyger

Creek, approximately 4500 feet downstream of the dam. Kyger Creek eventually empties into the Ohio River.



REFERENCE: USGS 7.5 MINUTE QUADRANGLE
ADDISON, OHIO - W.VA.

FIGURE 6.1 WATERSHED MAP
STINGY RUN FLYASH DAM - PHASE II

Presently, the crest of the dam varies between elevation 693 at the abutments to elevation 698 near the center. Discharge is regulated by the 100-foot high intake tower of the principal spillway, which is connected to a 48-inch diameter reinforced concrete pressure pipe (RCPP). Stop logs are placed in the tower as necessary to maintain settling action of the fly ash and act as an overflow weir to establish operating pool. The emergency spillway is an open channel cut through rock on the right abutment (looking downstream). Flow through this spillway occurs when the reservoir level reaches elevation 686.4 feet. The emergency spillway outlet channel intersects Stingy Run just upstream of the confluence with Kyger Creek.

The proposed raising of the dam will necessitate construction of a dam in the existing emergency spillway channel. Based upon height, storage volume and downstream hazards, the fly ash dam is and the proposed spillway dam will be classified as Class 1 dams.

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

6.2 BASIN CHARACTERISTICS

Figure 6.1 shows the limits of the watershed boundary for the fly ash dam. A review of available topographic maps and aerial photos was made to determine essential basin characteristics. 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.

Present land use within the watershed is limited to abandoned strip mine areas, some woodlands, and the fly ash pond. From previous in-house hydrologic studies, the three land uses occur in equivalent portions of the drainage area. Raising the dam crest will increase the surface area of the pond. The proposed maximum normal operating level of the lake surface is elevation 726 feet. This lake surface represents approximately forty percent of the total catchment.

A detailed soil survey for Gallia County has not been published. From a generalized soil map of Ohio, revised 1962, the area of the watershed is mapped as Muskingum - Upshur soils. These soils fall under the hydrologic soil group C as classified by the Soil Conservation Service (SCS) of the U.S. Department of Agriculture. The table below lists the basin characteristics for the project.

Drainage Area	950 acres
Average Land Slope	12.3%
Hydrologic Soil Group	C
SCS Curve Number	80 (Excluding the Pond)
Time of Concentration	0.20 HR (Pond Length Neglected)

Based on the arrangement shown on Drawing No. 12-3000, the fly ash pond will have the following surface areas and storage capacities.

	ELEVATION (FT. NGVD)	AREA (AC)	STORAGE (AC-FT)
MIN CREST-EXISTING	693.0	253	9,500
PROPOSED MAX OPERATING POOL	726.0	374	19,855
PROPOSED CREST-RAISED	735.0	400	23,365

Figure 6.2 presents the stage-area-volume relationship for the fly ash pond.

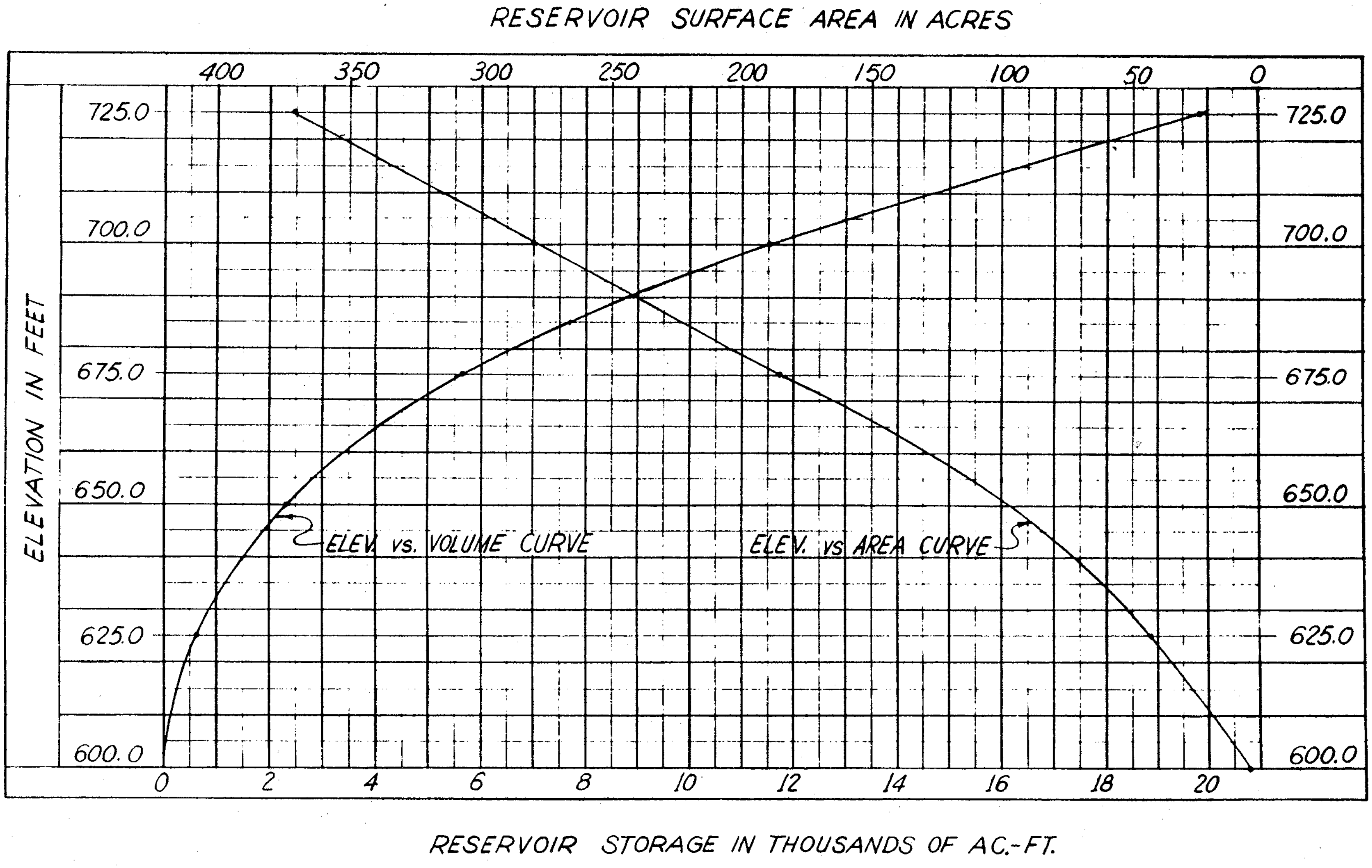


FIGURE 6.2 STAGE-AREA-VOLUME CURVE
STINGY RUN FLYASH DAM - PHASE II

6.3 ASSUMPTIONS AND DESIGN REQUIREMENTS

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

6.3.1 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 of use of the emergency spillway is predicted to be less than once in fifty years. The principal spillway shall also have the capacity to release 80% of the flood waters retained in the reservoir within ten days following the design flood.

It was assumed that a 50-year rainfall event would produce the 50-year flood. The estimated precipitation, 3.75 inches, was obtained from the National Weather Bureau report TP-40. A 6-hour storm duration and average soil moisture conditions were assumed for developing the inflow.

6.3.2 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 overtopping 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, Hydrometeorological Report No. 51. For the study area, a 6-hour PMP of 27.5 inches was used as the design rainfall event. The antecedent moisture conditions of the soil cover were assumed to be average.

6.4 HYDROLOGIC ANALYSIS

All reservoir flood routings were conducted using the U.S. Army Corps of Engineers 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.

6.4.1 PRINCIPAL SPILLWAY

Daily discharges from the reservoir will continue to be regulated by the existing principal spillway structure with no modifications. To comply with Section 1501:21-13-04, Pipe Conduit Spillways, General Requirements, of the regulations, the capability of the reservoir to store the 50-year design flood was examined. This approach was selected over hydrologic routing since the pond area is large and freeboard between the emergency spillway crest and normal pool level is expected to provide storage for floods up to the 50 year event. Therefore, no hydrologic flood routings of the 50-year flood were conducted.

6.4.2 EMERGENCY SPILLWAY

Two options were considered and analyzed during the design of the emergency spillway. Option A considers no emergency spillway - only the storage capacity of the reservoir and discharge from the principal spillway. Option B includes the design of an open channel emergency spillway.

Option A - The probable maximum flood was routed through the reservoir. The initial pond level at the beginning of the storm was set at the maximum operating level. Discharge of the flood waters occurred solely through the principal spillway structure. Option A is being considered because the reservoir will not be at maximum pool until immediately prior to closure of the reservoir.

Option B - 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 reservoir was assumed to be at the maximum operating level at the beginning of the storm. Reservoir flood routings were conducted for two situations - (a) the principal spillway discharging and (b) neglecting those discharges.

Initial routings set the required bottom width of a trapezoidal open channel. An approach channel was designed and a backwater analysis was conducted to determine 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. The stage-discharge curve for the emergency spillway was based on calculations of critical depth at the control section and the results of the backwater curves. Several flood routings were done to determine the crest of the spillway needed to pass the PMF and allow sufficient freeboard for wave height and run-up.

Discharges from the emergency spillway are routed away from the spillway dam through an excavated and seeded earth channel. Design of the channel was based on the expected peak discharge from the reservoir. A minimum depth of channel, equal to the elevation difference between the crests of the spillway and dam was maintained along the channel length. A water surface profile for the outlet channel was developed to examine the depth of flow and average velocity during peak discharge.

6.5 RESULTS

6.5.1 PRINCIPAL SPILLWAY

The existing principal spillway consists of a 100-foot high intake tower connected to a 48-inch diameter reinforced concrete pressure pipe. Discharge is controlled by stop logs placed in the tower and decants down two square vertical shafts. The concrete outlet pipe is buried within the abutments of the earthen dam. An impact-type energy dissipator is provided at the outlet of the principal spillway. See Drawing No. 12-3000J for the existing condition.

The maximum operating level has been established at elevation 726 feet, NGVD. This level has been determined mainly due to the geologic constraints of the abandoned coal mines rather than hydrologic constraints. Assuming 100 per cent runoff during the 50-year flood, the pond level will rise only

0.8 feet above the operating level, neglecting outflow.

6.5.2 EMERGENCY SPILLWAY

The development of the PMF hydrograph indicates a peak inflow of 19104 cfs for the raised dam conditions. The hydrograph is plotted in Figure 6.3 and represents the combined runoff from the pond surface and remaining drainage area (including a base flow of 30 cfs to simulate the ash sluicing water).

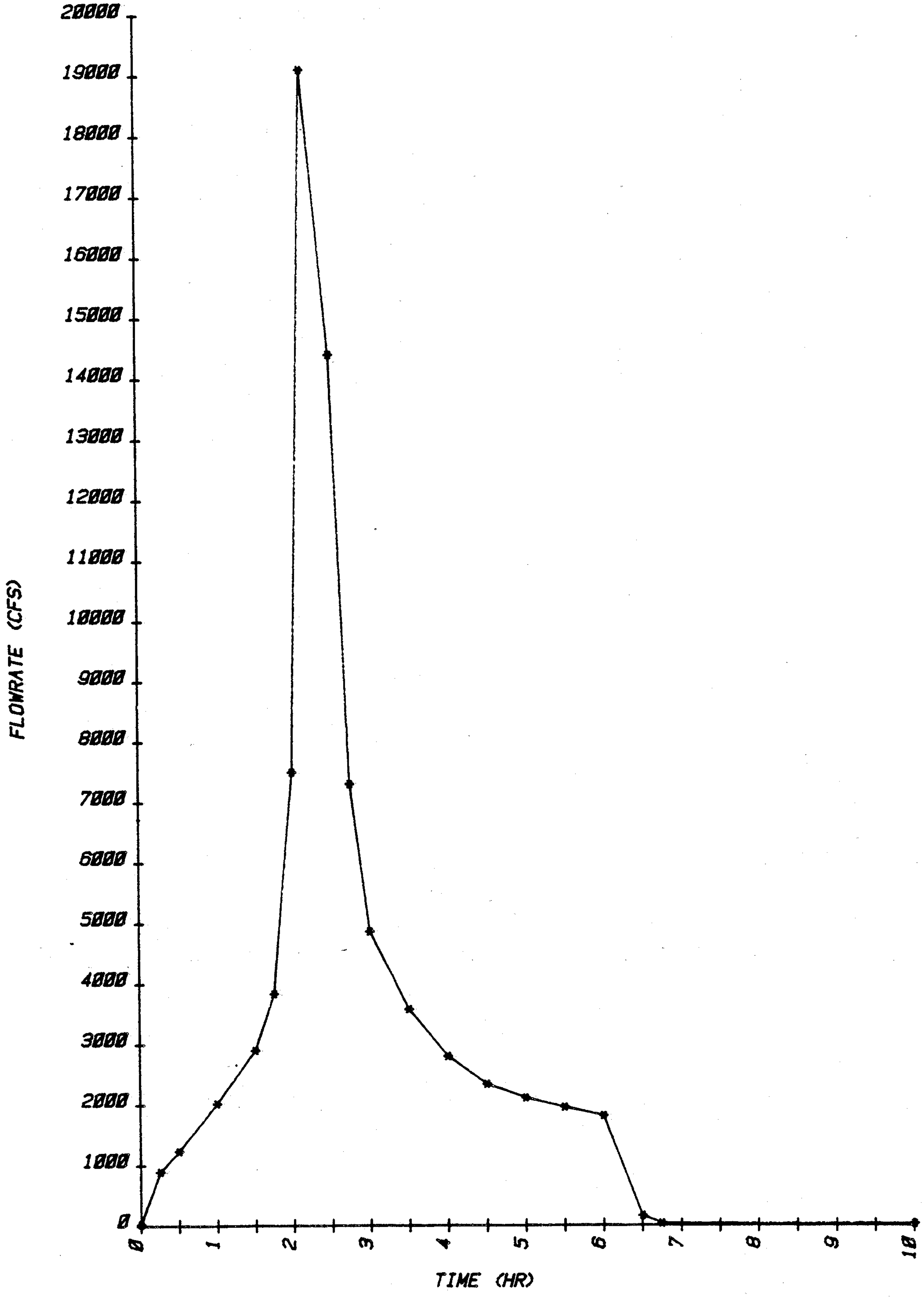


FIGURE 6.3 PMF INFLOW HYDROGRAPH
STINGY RUN FLYASH DAM - PHASE II

Option A - Flood routing of the PMF indicates the pool level will rise to a maximum elevation of 731.1 feet. The peak discharge from the principal spillway is 361 cfs and it will take approximately 7.8 days to release eighty per cent of the flood waters retained above the principal spillway.

Option B - Layout of the approach channel, control section, and outlet channel for the emergency spillway is shown on Drawing No. 12-3000D. The approach channel has a trapezoidal shape with a 100-foot bottom and 2H:1V side slopes, following a 2% adverse slope to the upstream edge of the dam crest. From the maximum normal pool level, the approach is 200 feet in length. The spillway control section has the same trapezoidal shape and a crest elevation of 730.0 feet, NGVD. A horizontal reach at this elevation extends for the width of the dam crest making the downstream edge the location of the control section.

From the control point, the outlet channel is also a 100-foot trapezoidal section at a 2% grade. The channel will be excavated for a length of approximately 550 feet until it intersects the existing topography. Due to topographic constraints, deviation from a straight alignment of the outlet channel is necessary. However, the curve is located at a safe distance downstream of the toe of the dam and will not affect the capacity of the channel because of the small depth of flow.

The entire emergency spillway system will be excavated in earth and then grassed to prevent erosion. In addition, the channel bottom will be sloped at a 1% grade towards the highwall side of the channel. This should encourage any erosion to take place away from the dam and towards the rock highwall.

During passage of the probable maximum flood, the pond level will rise to 731.3 feet corresponding to a peak discharge of 289 cfs, neglecting the discharges from the principal spillway. Considering those discharges, the pond rises to elevation 731.0 feet with a larger combined discharge of 541 cfs. Draining of the flood waters will take essentially the same length of time as in Option A since the emergency spillway is ineffective below a water level 730.0 feet.

Appendix C contains the computer listings of the flood routings. Also included are the calculations for the freeboard height which has been computed to be 3 feet.

6.6 SUMMARY AND CONCLUSIONS

The hydrologic studies for the raising of Stingy Run Fly Ash Dam included estimating the probable maximum flood (PMF) and designing the emergency spillway. The existing service spillway will continue to be used and requires no modifications. With the pond level at the maximum operating level of 726.0, the reservoir is capable of storing the 50-year flood volume without discharge through the emergency spillway.

Option A requires a permit variation from the ODNR. The main reason that a no emergency spillway request is made is because of the elevation of the coal in the present spillway channel area. The base coal seam elevations adjacent to the emergency spillway channel ranges from an elevation 726 ft. to 728 ft. This would require a new spillway channel for the proposed dam raising to pass through the mined out coal seam. To lower the emergency spillway channel below the coal seam and effected zones would reduce the overall useable capacity of the flyash reservoir. The alternatives to a new rock cut spillway channel are no emergency spillway, an emergency spillway channel on the abandoned strip bench adjacent to the proposed spillway dam, or an overflow spillway structure in the proposed spillway dam. The hydrologic feature that makes the no emergency spillway alternative viable is the fact that the reservoir will make up 40% of the reservoir at its maximum proposed operating pool. Therefore, the routing of a PMF event through the principal spillway will only increase the pool by 5.1 feet leaving 3.9 feet of freeboard for wave action and wind tide. Additionally, the maximum pool of the reservoir will only occur shortly before closure of the reservoir. Therefore, the average freeboard will be substantially greater than 3.9 feet.

It should be noted that for the first 18 years after the proposed raising is constructed the Option A conditions will be in effect even if the Option B spillway is constructed. The operating pool will have to be above elevation 724 feet to activate the Option B emergency spillway while routing a PMF event. After the proposed dam raising and the following 18 years a PMF event would be stored by the reservoir and passed by the principal spillway. Any emergency spillway constructed for the proposed raising would only function for the last 3 years ± of its operational life. Therefore, the probability of the emergency spillway being activated by a PMF event should be based on shorter time than the operational life of the proposed raising.

Option B proposes that a grass lined emergency spillway channel be constructed adjacent to the proposed spillway dam. A grass lined channel is proposed because the depth to competent rock is at least 15 feet below the crest of the proposed spillway dam. The grass lined channel invert will be 5 feet below the crest of the proposed spillway dam and dam raising. The slopes of the channel will be minimized to restrict flow velocities. The channel base will be sloped towards the highwall to promote any erosion to be directed away from the proposed spillway dam.

During the PMF, the combination of reservoir storage and spillway system has enough capacity to prevent overtopping of the dam regardless of option chosen. Freeboard during the flood is greater than the needed 3 feet for wave height and run-up for both design options. Table 6.1 gives a complete summary of the study.

TABLE 6.1 HYDROLOGIC/HYDRAULIC SUMMARY FOR RAISING OF STINGY RUN FLY ASH DAM

DRAINAGE AREA	1.48 SQ. MI.
DAM CREST; NGVD	735.0 FT.
DESIGN FLOODS (INFLOW)	
50-YR: RAINFALL	3.75 IN.
PEAK	NOT COMPUTED
PMF: RAINFALL	27.5 IN.
PEAK	19104 CFS.
PEAK DISCHARGE	
50-YR	NOT COMPUTED
PMF: OPTION A	361 CFS. PRINCIPAL SPILLWAY
OPTION B	541 CFS (WITH PRINCIPAL &EMERGENCY SPILLWAYS OPERATING)
MAXIMUM POOL ELEVATIONS, NGVD	
50-YR	726.8 FT.
PMF: OPTION A	731.1 FT.
OPTION B	731.0 FT.
PRINCIPAL SPILLWAY (EXISTING)	
CONCRETE SHAFT AND CONDUIT	
SHAFT OPENING 2 @ 4.0' X 4.0'	
CONDUIT 48-IN DIAMETER REINFORCED CONCRETE PRESSURE PIPE	
MAXIMUM OPERATING LEVEL, NGVD	726.0 FT.
EMERGENCY SPILLWAY	
OPTION A	NONE
OPTION B	GRASS LINED CHANNEL
CREST ELEVATION, NGVD	730.0 FT.
BOTTOM WIDTH,	100.0 FT.
SIDE SLOPES	2H:1V

7.0 PROPOSED DAM RAISING DESIGN

7.1 Design Parameters

The geotechnical parameters necessary to design the raising of fly ash dam are based upon the configuration, strength, compressibility, and permeability characteristics of the existing and proposed embankments. Some of these parameters can change with time, therefore, it is necessary to determine existing parameters and predict the effects of the increased loading of the dam raising on these parameters. It will also be necessary to predict the soil parameters for the materials used in the proposed dam raising.

The existing embankment and foundation materials have been evaluated with a drilling and testing program to determine the performance of the dam over the past 10 years. The existing dam has four basic soil zones: an upstream random fill, a cohesive clay core, a downstream random fill, and foundation soils.

All of the existing embankment materials were taken from essentially the same borrow sources. The cohesive clay core zone was to be constructed with the more cohesive materials. The upstream and downstream random fill zone were to be constructed with the sandy materials. The upstream random fill zone was not evaluated during the 1985 drilling and testing program.

The 1985 boring program was unable to delineate a clear distinction between the cohesive clay core and the downstream random fill. However, from the quality control reports on the compacted fill, it is apparent that the random fill was placed somewhat drier than the impervious fill. The impervious fill had allowable moisture limits of 1% wet and 2% dry of optimum. The majority of the impervious fill was placed in the night to avoid excessive drying of the soil. However, significant portions of the impervious fill were placed dry of optimum and some portions drier than 2% of optimum.

The existing foundation materials were found to consist of sands grading to clays and clays grading to sands. This report has divided the foundation soils into 5 basic zones: an upper brown clay, an upper brown sand, an intermediate gray clay, a lower brown-gray sand, and a lower red-brown clay.

There is no clear cut distinction between the zones; the sands are clayey sands with sand lenses, and the clays are sandy clays with clay lenses. In the 30 to 40 feet of original overburden there is less than 15 total feet of predominately clay material. The predominately clay seams are interbedded with sands seams. The intermediate gray clay zone has dessication cracks filled with sand and silt. These dessication cracks resemble slickensides and may have been previously recorded as a remolded material containing sliding surfaces. The fact that there is less than 15 total feet of clay and that it is interbedded with sand layers would tend to indicate why there was less settlement than originally anticipated.

The foundation materials controlled the stability analysis of the existing dam.

The borrow material for the clay core and downstream compacted shale shell is located immediately downstream of the existing embankment. The upper 8 to 15 feet of soil will be stripped for the cohesive clay core. The exposed soft shales and weathered sandstones will be used for the downstream shell. The materials found in the borrow site are geologically and geotechnically identical to the materials in the existing embankment. Therefore, behavior in the new borrow material is expected to be similar to the existing embankment. The new borrow site material has been tested for index properties, consolidation characteristics, compactability, permeability, and shear strength for various loading conditions. Because of the manner of reservoir useage the proposed dam raising is not being evaluated in the rapid drawdown loading condition.

7.2 Soil Testing Results

The soil testing results are divided into existing embankment and foundation soil materials and proposed dam raising and spillway dam construction materials.

7.2.1 Existing Embankment and Foundation Soil Testing Results

The existing embankment and foundation soils were probed with borings and performed standard penetration tests (herein SPT) and cone penetration tests (herein CPT). The split spoon samples obtained in conjunction with the SPTs were visually logged and reported in Appendix A-1. Shear strength correlations were made from the CPT and SPT and are found in Tables 7-1a and 7-1b. Shelby tube samples were also obtained in the existing dam and sub-soils. Soil specimens were obtained from Shelby tube samples in the impermeable clay core zone near the crest of the dam, the downstream random fill zone, and the foundation soil zone. The following procedures and tests were performed on the existing soils and can be found in Appendix B-2:

B-2.1	Visual Logging - Shelby Tubes
B-2.2	Grain Size Analyses, Atterberg Limits, and Natural Water Contents
B-2.3	Permeability Tests
B-2.4	Consolidation Tests
B-2.5	Unconsolidated Undrained Triaxial Compression Tests
B-2.6	Consolidated Undrained Triaxial Compression Tests
B-2.7	Direct Shear Tests

Table 7.1a

Undrained Shear Strength Parameters of Clays from CPT Data

$q_c = N_k \quad S_u + P$ $\qquad q_c$ = cone resistance

$S_u = \frac{q_c - P}{N_k}$ $\qquad N_k$ = bearing capacity number

P = total overburden pressure

for PI 30, N_k ranges from 11 to 19

...use 15 as avg. for N_k

Boring SR-2 (Elev. 585.78)

Elev.	Depth	q_c (avg)	S_u (avg)	Description
584.8	1-4'	34,000 psf	2,230 psf	M. Silty CLAY
580.3	4.5-7'	10,000 psf	620 psf	Soft Silty CLAY
577.8	7.-7.5'	30,000 psf	1,940 psf	M. CLAY

Elev.	Depth	q_c (avg)	S_u (avg)	Description
576.8	8-9.5'	30,000 psf	1,920 psf	M. CLAY
575.8	10-12'	40,670 psf	2,620 psf	M. Stiff CLAY
572.8	13-14.5'	48,000 psf	3,080 psf	M. Stiff CLAY
569.8	16-25'	64,000 psf	4,070 psf	M. Stiff to Stiff CLAY
560.8	25-26'	54,000 psf	3,400 psf	M. Stiff CLAY
556.8	29-30'	62,000 psf	3,900 psf	Stiff CLAY

Boring SR-5 (Elev. 592.10)

591.1	1'-6'	43,300 psf	2,840 psf	M. Silty CLAY
586.1	6'-8'	82,000 psf	5,400 psf	V. Stiff CLAY
584.1	8'-10'	47,200 psf	3,070 psf	M. Stiff CLAY
582.1	10'-11'	29,000 psf	1,850 psf	Silty CLAY
580.1	12-12.5'	26,000 psf	1,630 psf	Silty CLAY
572.1	20-20.5	46,000 psf	2,900 psf	M. Stiff CLAY
571.1	21-33'	54,000 psf	3,340 psf	M. Stiff to Stiff CLAY
558.6	33.5-36'	72,000 psf	4,510 psf	V. Stiff CLAY

Table 7.1b

Drained Shear Strength Parameters of SANDS
from SPT Data

$$N_{corr.} = N\ (.77\ \mathrm{Log}_{10}\ ({}^{20}/P))$$

Boring SR-2 (elev. 587.78)

Elevation	Depth	N	Vo	Ncorr	Ø	Description
581.8'	4'-4.5'	13	.25tsf	19.0	33.1°	Medium SAND
578.3'	7.5'-8'	8	.48tsf	10.0	30.1°	Loose SAND
573.8'	12'-13'	21	.78tsf	22.8	33.8°	Med. Dense SAND
559.8'	26'-29'	21	1.74tsf	17.2	32.1°	Medium SAND

Boring SR-5 (elev. 592.10)

Elevation	Depth	N	Vo	Ncorr	Ø	Description
581.1'	11'-12'	6	.72tsf	6.7	29.5°	Loose SAND
579.6'	12.5'-20'	10	1.2tsf	9.4	29.9°	Loose SAND
571.6'	20.5'-21'	10	1.26tsf	9.2	29.8°	Loose Silty SAND

The results of the existing embankment materials are summarized in Tables 7-2a and 7-2b. No major discrepancies were found in the parameters used in the original design and those obtained for the dam raising.

The foundation soils immediately below the embankment consist of sandy clays and clayey sands. Tables 7-2a and 7-2b also summarizes the parameters obtained for these soil materials.

GAVIN PLANT: STINGY RUN FLY ASH DAM: PHASE II RAISING

TABLE 7-2a

P.1 OF 2

DESCRIPTION	GRADATION				NAT'L WATER	ATTERBERG			STANDARD PROCTOR		PERMEABILITY	CONSOLIDATION		UNCONFINED COMPRESSION	TRIAXIAL UU		TRIAXIAL CU		TRIAXIAL CU		DIRECT SHEAR	
	D_{10}	D_{30}	D_{60}	D_{85}	w_n	PL	LL	PI	γ_d	w_{opt}	k	C_e	C_c	q_u	c	φ	c	φ	c'	φ'	c	φ
UNITS					%				pcf	%	cm/sec			tsf	tsf	°	tsf	°	tsf	°	tsf	°
EXISTING SOILS									γ_d													
SR-3/8 Brn-Gry Silty CLAY					24	22	40	18														
SR-4/5 Brn-Gry Silty CLAY					27	28	50	22														
SR-6/3 Red-Brn Silty CLAY					13	24	33	9														
SR-6/11 Brn-Gry Silty CLAY					18	20	35	15														
SR-6/15 Gry CLAY, f. SAND					31	27	42	15														
SR-7/2 Or-Brn Silty CLAY					18	23	45	22	112.						1.84	2°						
SR-8/2 Red-Brn Silty CLAY									123.								0	23°	0	27°		
SR-9/8 Brn Gry Silty CLAY					26	32	55	23	110.						1.84	2°						
SR-10/1 Or Brn Silty CLAY					23	28	56	28	105						.86	16°	.15	18°	0	23°		
SR-10/5 Red Brn Silty CLAY					14	20	33	13	121						.86	16°						
SR-11/3 Or-Brn Silty CLAY					14	17	35	18	118						.86	16°						
SR-13 Red Brn Silty CLAY					11	22	39	17	128						.86	16°						
SR-15/5 Brn Gry Silty CLAY					27	27	47	20	95						.72	0°						
SR-15/6 Brn Gry Silty CLAY					25	25	48	23	101						.72	0°						
SR-15/7 Brn Gry Silty CLAY					27	26	48	22	98						.72	0°						
SR-15/8 Gry-Brn Silty CLAY					26	26	44	18	99								0	18°	0	22°		
SR-16/9 Brn-Gry Silty CLAY					18	16	31	15	110								.43	16°	.43	16°		
SR-16/10 Brn-Gry Silty CLAY					19	20	33	13	111						.72	0°						
SR-16/13 Gry Silty CLAY					26	27	52	25													0	24°
SR-16/14 Gry-Rd Brn Silty CLAY					24	28	51	23	103						.70	0°						
SR-16/15 Gry-Rd Brn Silty CLAY					26	26	51	25	101						.70	0°	0	18°	0	22°		
SR-16/16 Gry Silty CLAY					25	24	47	23	102								0	18°	0	22°		
SR-16/18 Or Brn Silty CLAY																	.8	10°	.7	17°		
SR-16/19 Or-Brn Silty CLAY																	.8	10°	.7	17°		

GAVIN PLANT : STINGY RUN FLY ASH DAM : PHASE II RAISING

TABLE 7-2b

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DESCRIPTION	GRADATION				NAT'L WATER	ATTERBERG			STANDARD PROCTOR		PERMEABILITY	CONSOLIDATION		UNCONFINED COMPRESSION	TRIAXIAL UU		TRIAXIAL CU		TRIAXIAL CU		DIRECT SHEAR	
	D_{10}	D_{15}	D_{60}	D_{85}	w_n	PL	LL	PI	γ_d	w_{opt}	k	Ce	Cc	q_u	c	ϕ	c	ϕ	c'	ϕ'	c	ϕ
UNITS					%				pcf	%	cm/sec			tsf	tsf	°	tsf	°	tsf	°	tsf	°
EXISTING SOILS CONT'D																						
SR-6/15 Gry f. SAND																					0	34°
SR-10/23 Brn Gry Silty CLAY																					.58	24°
SR-15/4 Brn Gry Clayey SILT																					0	23°
SR-15/14 Brn Gry SAND																					0	23°
SR-16/13 Gry Silty CLAY																					0	24°

The testing of the clay with dessication cracks yielded inconsistent results. The orientation of the dessication cracks in the intermediate gray clay controls the shear strength of this material. It has been assumed that due to the length of the dam structure and the clay layers' proximity to bed rock, that the critical failure surface would be approximately horizontal. The triaxial shear strength tests were on specimens from vertical boreholes. In these tests all failures were along the dessication crack surfaces. The resulting shear strengths were inconsistant depending upon the random orienting of the up turned edges of the dessication cracks. Therefore, a program of direct shear tests that are oriented along the horizontal surfaces of the dessication cracks was performed to more realistically represent the available shear strengths of the intermediate clay zone.

7.2.2 Dam Raising and Spillway Dam Construction Material Testing

Test pits were excavated in the borrow site area and bag samples taken for laboratory testing. The field logs of these test pits can be found in Appendix B-1. The following procedures and tests were performed on the borrow site soil specimens and can be found in Appendix B-3.

B-3.1 Visual Logging - bag samples
B-3.2 Grain Size Analyses, Atterberg Limits, and Natural Water Contents
B-3.3 Standard Proctor Tests
B-3.4 Min-Max Density Tests (Bottom Ash)
B-3.5 Permeability Tests
B-3.6 Consolidation Tests
B-3.7 Unconfined Compression Tests
B-3.8 Unconsolidated Undrained Triaxial Compression Tests
B-3.9 Consolidated Undrained Triaxial Compression Tests
B-3.10 Consolidated Drain Triaxial Compression Tests (Bottom Ash only)

The borrow area has an upper layer that ranges from 5 to 10 feet thick that is generally more plastic than the lower materials. The upper materials are to be used for the clay core zone in the new construction. The clay core material was tested for shear strength, permeability, compressibility in the remolded state modeling - 1% dry and +3% wet of the standard proctor optimum moisture content. The +3% wet of optimum specimens yielded the low permeability and flexibility characteristics desired, but only marginal required strength. Unconfined compression tests were performed to supply additional shear strength results for the remolded specimens. These additional results serves to provide consistency in the test results of the remolded specimens.

The lower clay materials which will be used in the random fill zone was tested for shear strength permeability and compressibility in the remolded state modeling - 2% dry and +2% wet of the standard proctor optimum moisture content. No more than -2% dry of optimum is desired to prevent the silty soil fabric from collasing upon wetting and thereby reducing its shear strength.

A summary of the soil testing for the borrow soil materials is found in Tables 7-3a and 7-3b.

GAVIN PLANT: STINGY RUN FLY ASH DAM: PHASE II RAISING

TABLE 7-3a

p1 of 2

DESCRIPTION	GRADATION				NAT'L WATER	ATTERBERG			STANDARD PROCTOR		PERMEABILITY	CONSOLID.		UNCONFINED COMPRESSION	TRIAXIAL UU		TRIAXIAL CU		TRIAXIAL CU		DIRECT SHEAR	
	D_{10}	D_{15}	D_{60}	D_{85}	w_n	PL	LL	PI	γ_d	w_{opt}	k	Ce	Cc	q_u	c	ϕ	c	ϕ	c'	ϕ'	c	ϕ
units	mm	mm	mm	mm	%				pcf	%	cm/sec			tsf	tsf	°	tsf	°	tsf	°	tsf	°
BORROW SITE																						
TP-1 Brn-Gry CLAY	–	–	.014	.2	–	22	41	19	105.2	19.6												
-1 % opt														1.72	0.95	5°	0.5	10°	0.4	18°		
+3 % opt														1.17	0.6	0°	0.35	8°	0.25	14°		
TP-2 Brn-Gry CLAY			.0041	.02	26.6	28	47	19	102.6	24.1												
-1% opt											7.08×10^{-8}	.22	.012	1.60								
+3% opt											1.26×10^{-8}	.23	.016	0.73								
TP-2-1			.0042	.019	17.4	22	45	23														
TP-3 Red-Brn, Sft SHALE	.0021	.003	.025	5.6	9.2	23	34	11	120.6	14.4												
-2% opt															0.5	12°	0.15	21°	0.15	26°		
+2% opt															0.4	1°	0.2	18°	0.2	24°		
TP-3 Or-Brn CLAY	–	–	.0048	.038	24.3	25	48	23	105.3	18.8												
-1% opt												.20	.009	1.76								
+3% opt												.18	.015	1.60								
TP-4 Brn-Gry CLAY	–	–	.0033	.02	33.8	22	44	22	103.9	21.8												
-1% opt											3.17×10^{-9}			1.56	0.6	6°	0.3	12°	0.2	23°		
+3% opt											7.32×10^{-7}			0.95	0.5	0°	0.2	11°	0.35	16°		
TP-5 Gry to Red, Brn CLAY	–	.	.0069	.06	25.4	25	46	21	108.2	17.6												
-1% opt														1.29								
+3% opt														1.13								
TP-6 Red Brn Silty CLAY	.001	.002	.035	6.1	18.4	24	37	13	110.1	18.1												
-2% opt											1.73×10^{-8}			1.11	0.7	14°	.25	13°	.25	23°		
+2% opt											4.02×10^{-8}			1.28	0.5	1°	.15	16	.30	24°		
SRB-2	–	–	.009	.8	–	23	39	16	110.0	17.4												
-2% opt											6.25×10^{-9}			1.84								
+2% opt											1.65×10^{-5}			1.58								

GAVIN PLANT: STINGY RUN FLY ASH DAM: PHASE II RAISING

TABLE 7-3b

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DESCRIPTION	GRADATION				NAT'L WATER	ATTERBERG			STANDARD PROCTOR		PERMEABILITY	CONSOLID.		UNCONFINED COMPRESSION	TRIAXIAL UU		TRIAXIAL CU		TRIAXIAL CU		DIRECT SHEAR	
	D_{10}	D_{15}	D_{60}	D_{85}	w_n	PL	LL	PI	γ_d	W_{opt}	k	C_e	C_c	q_u	c	ϕ	c	ϕ	c'	ϕ'	c	ϕ
units	mm	mm	mm	mm	%				pcf	%	cm/sec			tsf	tsf	°	tsf	°	tsf	°	tsf	°
Borrow Site (Cont'd)																						
SRB-1&3	.001	.0024	.02	2.0	—	17	28	11	127.6	10.2												
−2% opt														0.60								
+2% opt														0.79								
							Max γ	Min γ	γ_d	Rel. Den.												
Bottom Ash							pcf	pcf	pcf	%							0	39°	0	39°		
S-2573 prior to tests	.11	.2	1.8	6.0			97.7	77.2	95.1	89.7	1.4×10^{-2}											
S-2573 after Max-Min	.17	.21	1.9	6.0																		
S-2573 after Permeability	.13	.17	1.4	4.9																		
S-2574 prior to Max-Min	.12	.18	1.6	5.6			99.2	79.4	95.3	83.6	1.8×10^{-2}											
S-2574 after Permeability	.17	.20	1.7	7.0																		
S-2575 prior to Max-Min	.13	.18	1.4	4.8			97.6	77.1	94.7	88.5	1.7×10^{-2}											
S-2575 after Max-Min	.13	.18	1.6	6.0																		
S-2576 prior to Max-Min	.11	.17	1.0	4.0			92.9	72.5	88.3	81.5	2.2×10^{-2}											
S-2577 after Max-Min	.12	.17	1.0	4.0			97.5	77.0	95.0	90.1	1.1×10^{-2}											
Note' S-2794 D-1 past comp. effort	.08	.10	.56	2.6					96.4													
D-2 " "	.08	.096	.60	2.5					96.8													
D-3 " "	.075	.092	.53	2.2					97.2													
D-4 " "	.075	.10	.59	2.4					96.9													
D-5 " "	.075	.095	.59	2.1					96.8													
W-1 " "	.074	.090	.50	2.5					88.5													
W-2 " "	.070	.090	.49	2.2					90.4													
W-3 " "	.075	.095	.60	2.6					90.1													
W-4 " "	.075	.095	.62	2.8					94.0													
W-5 " "	.075	.090	.60	2.4					92.7													

Note' Compacted with Modified Proctor to maximize Degredation, D = Dry; W = Wet

The test results of the Gavin Plant bottom ash indicate that the material is a strong light weight material that drains freely. Gradation tests were performed before and after max-min density tests and permeability tests to examine the degredation and filtering characteristics of the bottom ash. The bottom ash indicated no significant degredation during the max-min density tests and no significant washing of fines during the permeability tests.

7.3 Slope Stability:

All slope stability analyses are performed using Purdue University's STABL3 Slope Stability Program. The 2 methods of analysis include the wedge analysis for shallow and deep foundation failures and the modified Bishop method for circular failures through the embankment and foundation.

The design of the existing dam proposed raising the dam an additional 40 feet in 2 lifts. Each lift was approximately 20 feet in height and resulted in 5h:1V and 3h:1v downstream slopes respectively. In re-evaluating this configuration for a 40 foot dam raising it was found that the 3h:1v downstream concentrated a large additional driving force on the critical foundation layer. This driving force required a significant length of critical failure surface to develop sufficient cohesion to provide an acceptably stable configuration in the end-of-construction state. Therefore, the proposed dam raising has been configured with 2.5h:1v upstream and downstream slopes with a 30 foot wide berm to reduce the driving forces on the foundation soils. The interior zone of the dam will be compacted bottom ash. The bottom ash will serve as a filter and a drain for seepage penetrating the clay core. In addition, the bottom ash will drain seepage from the abutments. The bottom ash is a lighter material with a high friction angle. Both of these properties will serve to make the proposed raising a more stable configuration. The internal configuration of the bottom ash and the downstream random fill has been made to maximize the weight at the toe of the dam raising and minimize the weight near the crest. The lower crest weight reduces the driving forces and the higher toe weights increases the resisting forces. The location of the increased thickness of the downstream random fill is based upon the stability analyses' break point between driving and resisting masses. The downstream compacted shale zone will be used to protect the bottom ash from erosion and excessive precipitation infiltration. This material can be compacted sufficiently dry to provide adequate strength to safely maintain a 2.5h:1v downstream slope.

The following sub-appendices in Appendix B-4 contain the stability analyses performed for Gavin Fly Ash Dam Raising:

B-4.1 Downstream Wedge Analysis
min F.S. = 2.35 EOC (Shallow Slide)
min F.S. = 1.34 EOC (Deep Slide)

B-4.2 Downstream - Modified Bishop Analysis
min F.S. = 1.64 EOC

B-4.3 Downstream - Wedge Analysis
min. F.S. = 1.72 SSS (Full Pool, Shallow Slide)
min. F.S. = 1.46 SSS (Full Pool, Deep Slide)

B-4.4 Downstream - Modified Bishop Analysis
min. F.S. = 1.53 SSS

B-4.5 Upstream - Modified Bishop Analysis
min F.S. = 2.59 EOC

B-4.6 Upstream - Modified Bishop Analysis
min F.S. = 1.85 SSS (Full Pool, El. 726 Ft.)
min F.S. = 1.42 SSS (Partial Pool, El. 684 Ft.)

EOC = End of Construction Soil Parameters
SSS = Steady State Seepage Soil Parameters

The following shear strength parameters were used for the above analysis:

Material	End of Construction		Steady State	
	C(psf)	Ø	C(psf)	Ø
Fill Materials				
new u/s clay core	1200	0°	700	16°
new d/s random fill	1500	0°	800	16°
bottom ash zone	0	39°	0	39°
existing clay core	1720	0°	0	23°
existing d/s random fill	3680	0°	0	25°
u/s random fill	2000	0°	0	21°
u/s fly Ash	0	0°	-	-
Foundation Materials				
Upper Clay	1440	0°	860	21°
Upper Sand	0	30°	0	30°
Intermediate Clay	2160	0°	0	22°
Lower Sand	0	30°	0	30°
Lower Clay/Shale	3000	0°	0	22°

Intact Rock	5000	0°	5000	0°

The final Engineering Report will contain a pseudo-static seismic stability analysis. It is not anticipated that critical seismic conditions will be found. The granular materials are well graded and will be placed with a minimum relative density of 80% which would mitigate any liquification potential.

7.4 Settlements

The original embankment was designed for 8 feet of settlement. In 10+ years a maximum of 0.9 feet at the center of the dam has actually been recorded. The historical settlement data is shown on drawings 741028 and 791017 which are found in Appendix B-6. It is apparent from this data that the embankment has entered its secondary consolidation phase. The records began at the end of construction. Therefore, the consolidation that occurred during the 6 month construction period is not documented. It is doubtful if 7 feet of settlement occurred during construction. The 8 feet of predicted settlement reflects the conservatism of the analytical procedures used to predict settlements. Using these same procedures and conservative consolidation parameters a total of 2.8 feet of time dependent consolidation is predicted for the 40 foot dam raising. Half of this settlement is predicted to take place in the existing fill and half in the foundation soils. The calculations are found in Appendix B-6. The 1.4 feet of settlement predicted in the foundation materials was based upon the entire thickness being clay. In actuality the clay portion is at the most 60% to 70% of the foundation thickness and varies extensively. Therefore, the 1.4 feet of predicted settlement is high because of material variation. The presence of sand layers in the foundation clay will allow rapid drainage, therefore a portion of the consolidation will occur during construction. The combination of material variation and consolidation during construction should yield settlements less than 1 foot as a result of consolidation of the foundation soils. The existing embankment will also consolidate during construction. Therefore, the 2 foot construction crown of the dam raising should more than compensate for any post-construction settlements.

The initial elastic settlements were also evaluated. The total initial elastic compression of the soil particles due to the dam raising was estimated to be

0.25 feet. This settlement is not the same as the pore pressure dependent consolidation previously discussed. However, the distortion is small reflecting the limited compression impact of the loading of the dam raising.

7.5 Abutment Preparation

The abutments of the proposed dam raising were pressure tested to detect potential seepage problems. The results of the pressure tests are found in Appendix A-3. The abutments adjoining the proposed raising contains sandstone layers that are typically permeable at this site. The jointing in the sandstones has a set which aligns with the valley. This alignment would provide a direct path for seepage. Therefore, the existing grouting program will be extended on both abutments as shown on drawing 12-3000G and H. These drawings show the alignment of the primary grouting program. In areas of high takes a line of secondary grout holes will be performed in between the primary pattern. The existing dam abutments were grouted with a cement/fly ash grout. The additional grouting for the proposed raising will also be a stable cement/fly ash grout.

7.6 Internal Drainage and Filters

All seepage which originates from the embankment materials or the abutment will be collected in the bottom ash zone of the dam raising.

The existing downstream slope will have perimeter ditches excavated between the existing embankment and the abutments where the bottom ash will be placed. In doing this the collected seepage from the abutments can be kept separate from the embankment seepage for monitoring purposes. All seepage will then be collected in a toe ditch and discharge in the existing stilling basin for the principal spillway pipe discharge.

The bottom ash material has physical properties that make it a good filter as well as a drain. Appendix B-3.5 reports the permeability of the bottom ash material which ranges from $1.1*10^{-2}$ cm/sec to $2.2*10^{-2}$ cm/sec. The bottom ash also meets the following filter criteria suggested by J. L. Sherard and Corps of Engineer Criteria reported by H. R. Cedergren:

1. $\frac{D_{15} \text{ of filter}}{D_{85} \text{ of soil}}$ less than 5

   COE, piping ratio

2. Filter Uniformity Coefficient,Cu,$\frac{D_{60}}{D_{10}}$ less than 20

COE, grading criteria

3. $\frac{D_{15}\text{ of filter}}{D_{15}\text{ of soil}}$ greater than 4

COE, to insure sufficient permeability

The D_{85} of the borrow area tests ranges from 0.14 mm to .5 for the materials that will be used as a clay core. The bottom ash D_{15} historically ranges from 0.17 mm to 0.13 mm. The maximum uniformity coefficient for the bottom ash is 14. The bottom ash meets all of the above criteria for soils with D_{85} exceeding 0.025 mm. There are 4 of 11 specimens of clay which have D_{85} less than .025 mm. Of the 4, 3 specimen have .02 mm D_{85} which have max criteria 1 ratios of 6.5 and 1 specimen has a criteria 1 ratio exceeding 10. However, it is felt that the bottom ash is adequate to meet the filter criteria because of the sand content of the clays, the mixing of materials during construction, and the limited over run of the recommended criteria.

Where the bottom ash drain daylights, the toe drain will grade from a sand to a riprap to protect the exposed portion of the drain from erosion.

7.7 Principal Spillway Structure

The 48 inch reinforced concrete pressure pipe was originally designed to withstand the increased overburden of an additional 40 feet. In reviewing the design for the pipe, it was found that the pipe was designed for an overburden load, including the raising of 40 feet, to be placed all at once. By placing 40 feet less, allowing settlements to occur, and then to load the additional load even more capacity is available. This factor combined with the lighter loads and changed configuration insures the integrity of the pipeline.

7.8 Instrumentation

The proposed dam raising will be monitored with crest settlement indicators and piezometers will be located in critical areas. The existing observation wells will be extended in some fashion to continue the historical monitoring points of the existing dam. A detailed instrumentation plan will be submitted in the final engineering report.

8.0 PROPOSED SPILLWAY DAM DESIGN

8.1 Design Parameters

The proposed spillway dam will have 2.5h:1v upstream and downstream slopes with a 25 foot wide crest. The clay core will be the same material as in the proposed dam raising and will come from the same borrow source. The clay core will be 50 feet wide at the base and 25 feet wide at the crest. The upstream and downstream random fill will be the same compacted shale materials as used in the downstream shell of the proposed dam raising and will come from the same borrow area. The upstream face will be rip rapped the same as the proposed dam raising. The downstream face and crest will be top soiled and seeded. The chimney drain and blanket drain will be the bottom ash used in the proposed dam raising and serve as a filter and drain.

8.2 Soil Test Results

The design of the spillway dam will utilize the same test results as those obtained in the borrow area for the dam raising. These results are found in Appendix B-3. The blanket drain material may be constructed of bottom ash. Any additional testing performed on the spillway dam materials will be reported in the Final Engineering Report.

8.3 Slope Stability

Slope stability for the emergency spillway dam were performed with Purdue University's STABL3 program. Only modified bishop analyses were performed because the spillway dam is only 50 feet tall and founded directly on bedrock. The soil parameters used for the spillway dam are the same as those used for the same materials reported for the dam raising (see section 7.3).

The sub-appendices in Appendix B-5 contain the stability analyses performed for the spillway dam:

B-5.1	Downstream Modified Bishop Min. F.S. = 2.25 End of Construction
B-5.2	Downstream Modified Bishop Min. F.S. = 2.11 Steady State Full Pool

B-5.3 Upstream Modified Bishop
Min. F.S. = 1.78 End of Construction

B-5.4 Upstream Modified Bishop
Min. F.S. = 2.04 Steady State Full Pool

Note: During partial pool conditions for the upstream face the minimum F.S. varies between 2.04 and 2.18. Therefore, the minimum F.S. is 2.04 for this structure.

The high F.S. for this dam is directly related to the 2.5h:1v slopes and the rigid foundation. Technically, the slopes could be made steeper, however the 2.5h:1v slopes will not be as susceptible to surface sloughing. With effective friction angles ranging from 23° to 26° and no greater than 400 psf cohesion a surface slope of 21.8° (2.5h:1v) is required to maintain an infinite slope F.S. greater than 2.0 at a depth of 5 feet. This should minimize surface sloughing due to rain saturation of the slopes or seeps from the abutments. The embankment will be constructed on competent sandstone which makes the stability of the structure dependent upon the construction materials.

8.4 Settlements

The spillway dam is a 50 foot high embankment overlying a rigid base. Initial elastic settlements should not be distinguishable in the sandstone base. Time dependent settlements in the embankment will result from self weight. Assuming an apparent preconsolidation pressure of 4500 psf for the compacted fill, the upper 35 feet of the embankment should consolidate less than 0.03 feet. The lower 15 feet should consolidate less than 0.4 feet. These estimates are based upon the consolidation parameters given for the dam raising and shown in Appendix B-6. The resulting settlements in the spillway dam should be less than 0.5 feet.

8.5 Abutment Preparation

The abutments are in the same rock units as the proposed dam raising and will therefore, be grouted. These abutments were pressure tested and the results are given in Appendix A-6. The grouting program will involve similar spacing and the same grout materials as used in the dam raising. The dam is founded on a jointed sandstone. The abutments will be grouted, but the jointed sandstone foundation will not. The water take for the pressure test in the foundation sandstone was approximately 19 lugeons. The water take in the same sandstone unit in either abutment ranged from 1 to 8 lugeons. This indicates that the joints will probably tend to close up when the overburden from the dam is applied. There is no guaranty as to how much this permeability will decrease due to embankment loading. However, the permeability of the random fill and the clay core relative to the sandstone will cause the upstream portion of the dam to serve as a cutoff blanket. The downstream drainage blanket/filter will eliminate downstream pore pressures. The abutments will still need to be grouted due to short drainage paths and less overburden pressures.

The impervious clay core will extend from highwall to highwall on either abutment. The abutments will be keyed into competent rock on both abutment walls. The backfill material used to reclaim the surface mining is predominately sand, which necessitates constructing the impermeable cut off from high wall to high wall.

8.6 Internal Drainage and Filters

The spillway dam will be serviced by a blanket drain and chimney drain constructed of bottom ash. The bottom ash will meet the same 3 point filter/drain criteria as used for the dam raising. The drains will be placed directly on top of the bedrock after the weathered portion of the rock has been cleared and stripped. Where the blanket daylights, the drain will grade from a sand to a rip rap size material to protect the exposed portion of the drain from erosion.

8.7 Emergency Spillway Structure (Option B)

The emergency spillway channel will be excavated with a 2% upstream and downstream slope from the crest. The channel will have a 100 foot wide base which slopes at 1% towards the highwall. The side slope will have 2h:1v cut slopes. All surfaces within the spillway will be top soiled as necessary to establish a grass lined channel.

8.8 Instrumentation

The proposed spillway dam will be monitored with piezometers and crest settlement monuments. Seepage from the dam will be collected and routed through a control weir to monitor the flow through the dam and abutments.

9.0 PRELIMINARY COST ESTIMATES

Table 9.1 contains the preliminary cost estimates for raising the Stingy Run Fly Ash Dam and constructing the spillway dam.

Table 9.1

Preliminary Cost Estimates

I. Dam Raising

ITEM	COST EST
Mobilization	$ 300,000
Clearing and Grubbing	$ 124,000
Stripping	$ 183,000
Excavation	$4,500,000
Foundation Preparation	$ 150,000
Drilling and Grouting	$ 350,000
Fill	$1,050,000
Top Soil and Seeding	$ 100,000
Rip Rap	$ 420,000
Instrumentation	$ 25,000
	$7,202,000

II. Spillway Dam

ITEM	COST EST
Mobilization	$ 100,000
Clearing and Grubbing	$ 20,000
Stripping	$ 28,000
Excavation	$ 800,000
Foundation Preparation	$ 17,000
Drilling & Grouting	$ 124,000
Fill	$ 100,000
Top Soil and Seeding	$ 8,000
Rip Rap	$ 90,000
Instrumentation	$ 6,000
	$1,293,000

Total Cost $8,495,000

10.0 OTHER PERTINENT INFORMATION

10.1 Island Transmission Tower Access Road

A series of transmission towers will be effected by the increasing operating pool after the dam is raised. The towers immediately affected are the two towers approximately located at the coordinates N 352,450, E 2,099,050. Any further increases in pool elevation will make this peninsula an island within the reservoir. Access to the towers needs to be maintained, therefore, a 20 foot berm containing a roadway will be constructed during the dam raising project to maintain access to the towers for an additional 7 years.

10.2 Gas Well Activity

Gas wells have been and are currently being installed in the vicinity of the Stingy Run Fly Ash Dam and Reservoir. As of October 16, 1985, 30 wells have been drilled on property adjacent to the reservoir. All wells were drilled to the Berea Formation which is approximately 1700 feet deep in this area. The estimated life expectancy for these wells is 20-25 years with compression expected to be necessary after 10-15 years. The average daily production per well is 1 barrel of oil and 15-20 MCF's of gas with little or no salt water. The total lifetime production per well is 4,000 barrels of oil and 75,000 MCF's of gas. The depth of production and quantity of oil and gas is not expected to cause problems for the dam structures.

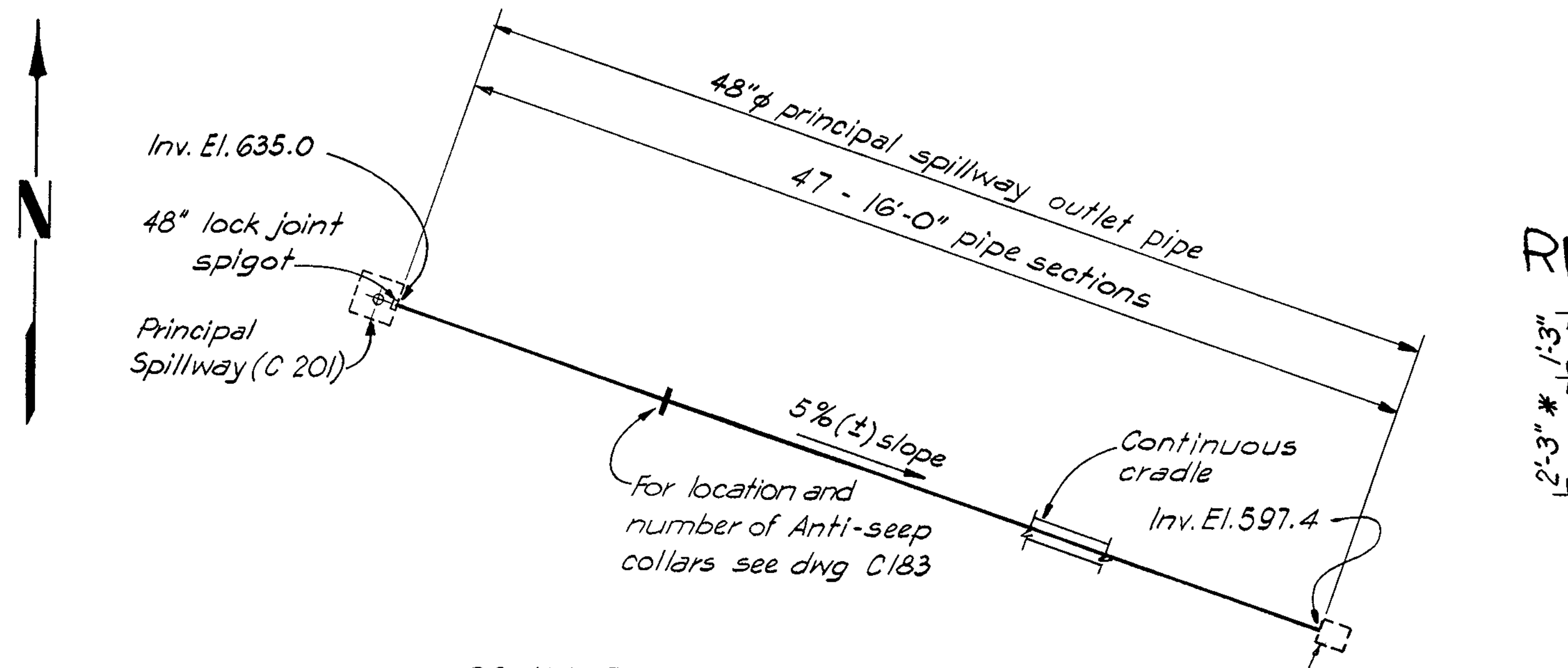
11.0 ADDITIONAL INFORMATION TO BE PROVIDED IN THE FINAL REPORT.

The following items will be addressed in the final report:

1. Responses to ODNR comments to the Preliminary Design Report.
2. Seismic Stability Analyses of both dam structures.
3. Bid Documents containing Construction Procedures.
4. Grouting Program for Emergency Spillway Dam.
5. Detailed Instrumentation plans for both dam structures.

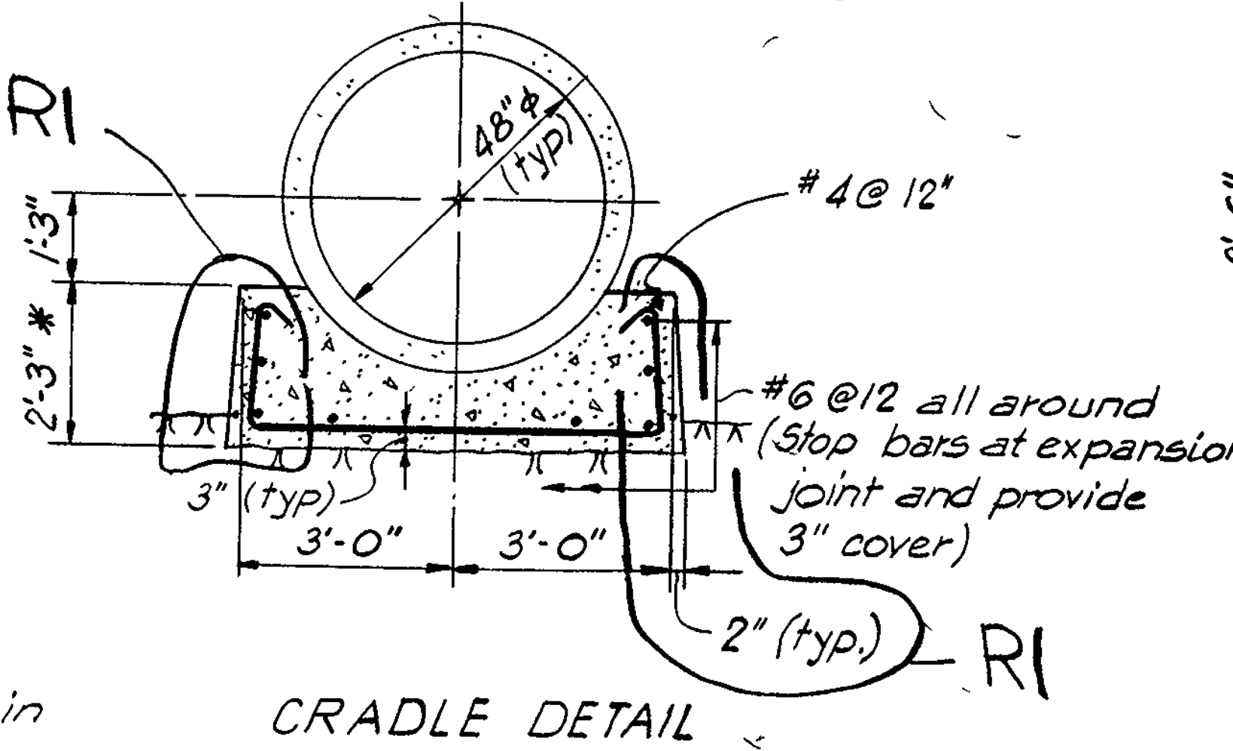
ATTACHMENT C

DESIGN DRAWINGS

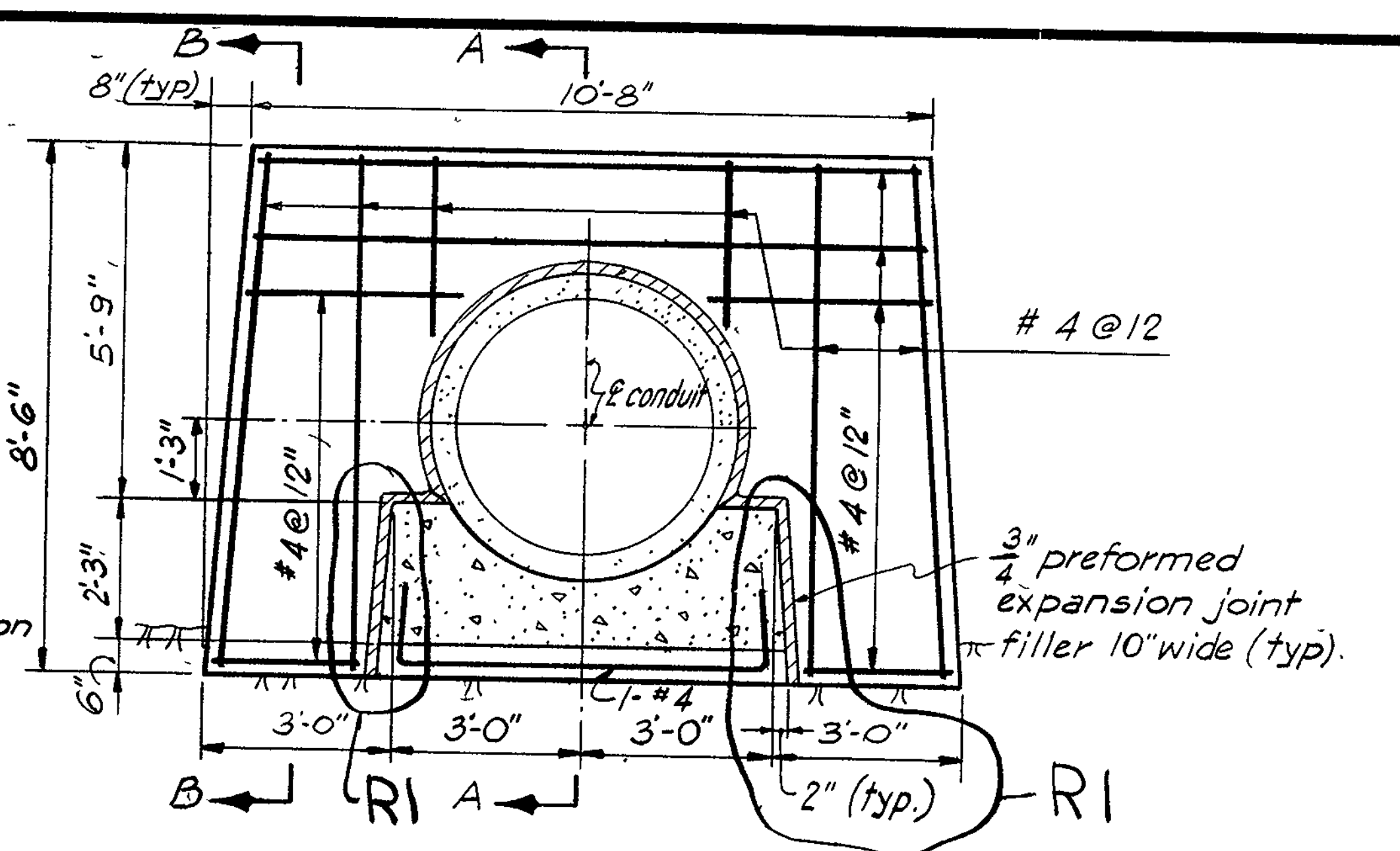


CONDUIT INSTALLATION LAYOUT
N. T. S.

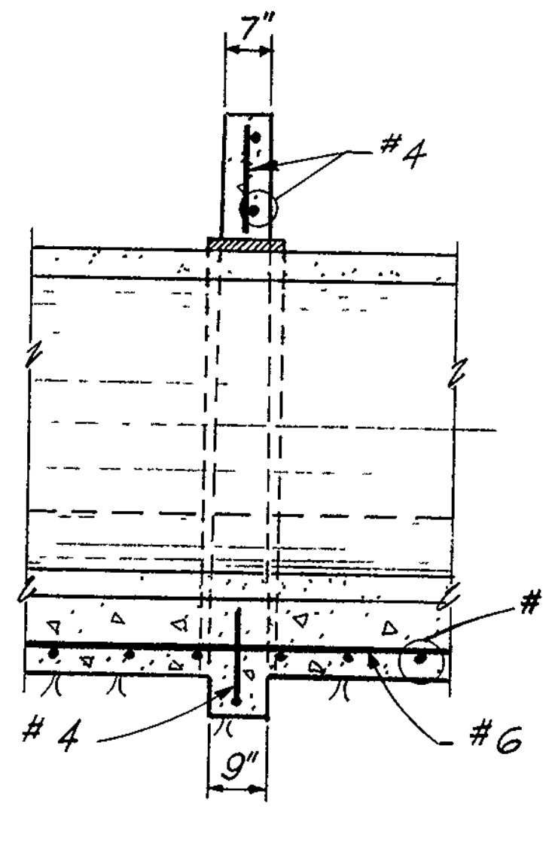
*Indicates minimum dimension. Cut rock to provide solid bearing or pour additional concrete to bear on rock surface.



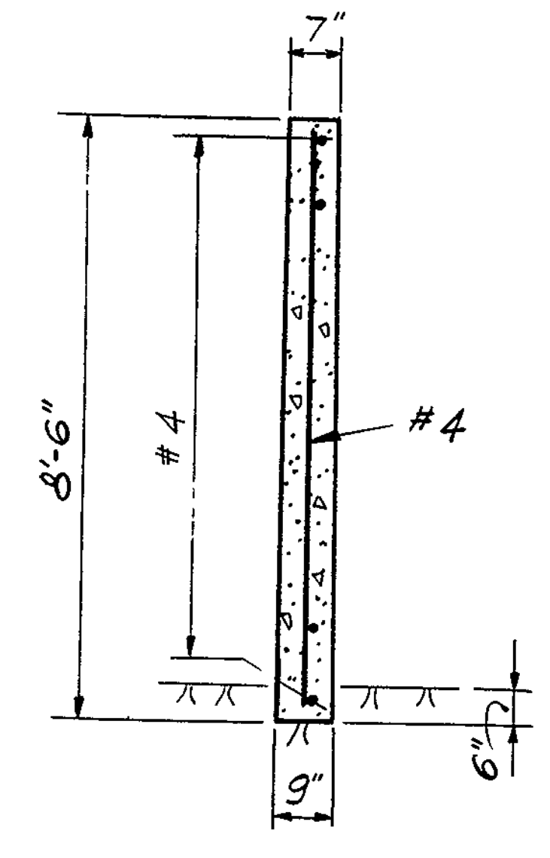
CRADLE DETAIL



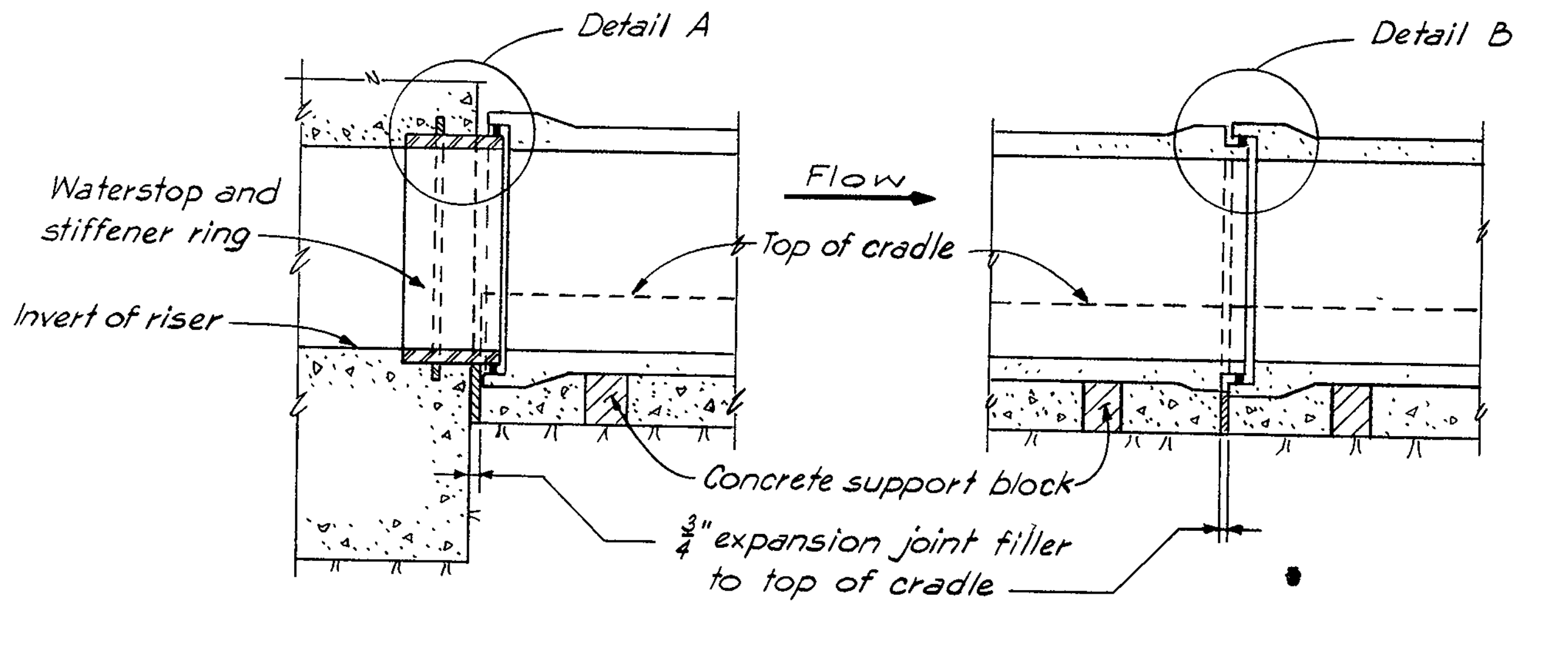
ANTI-SEEP COLLAR DETAIL



A - A

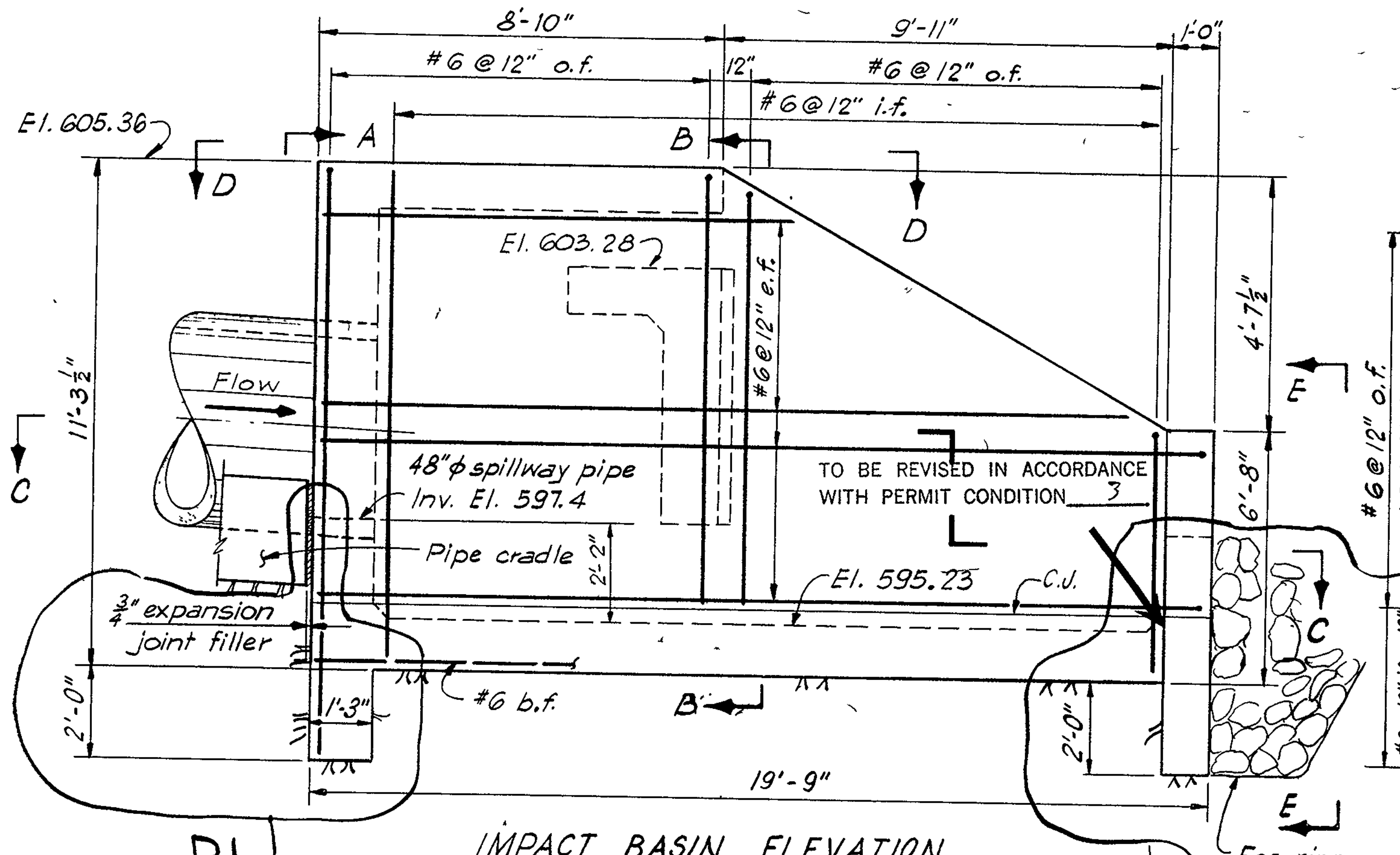


B - B



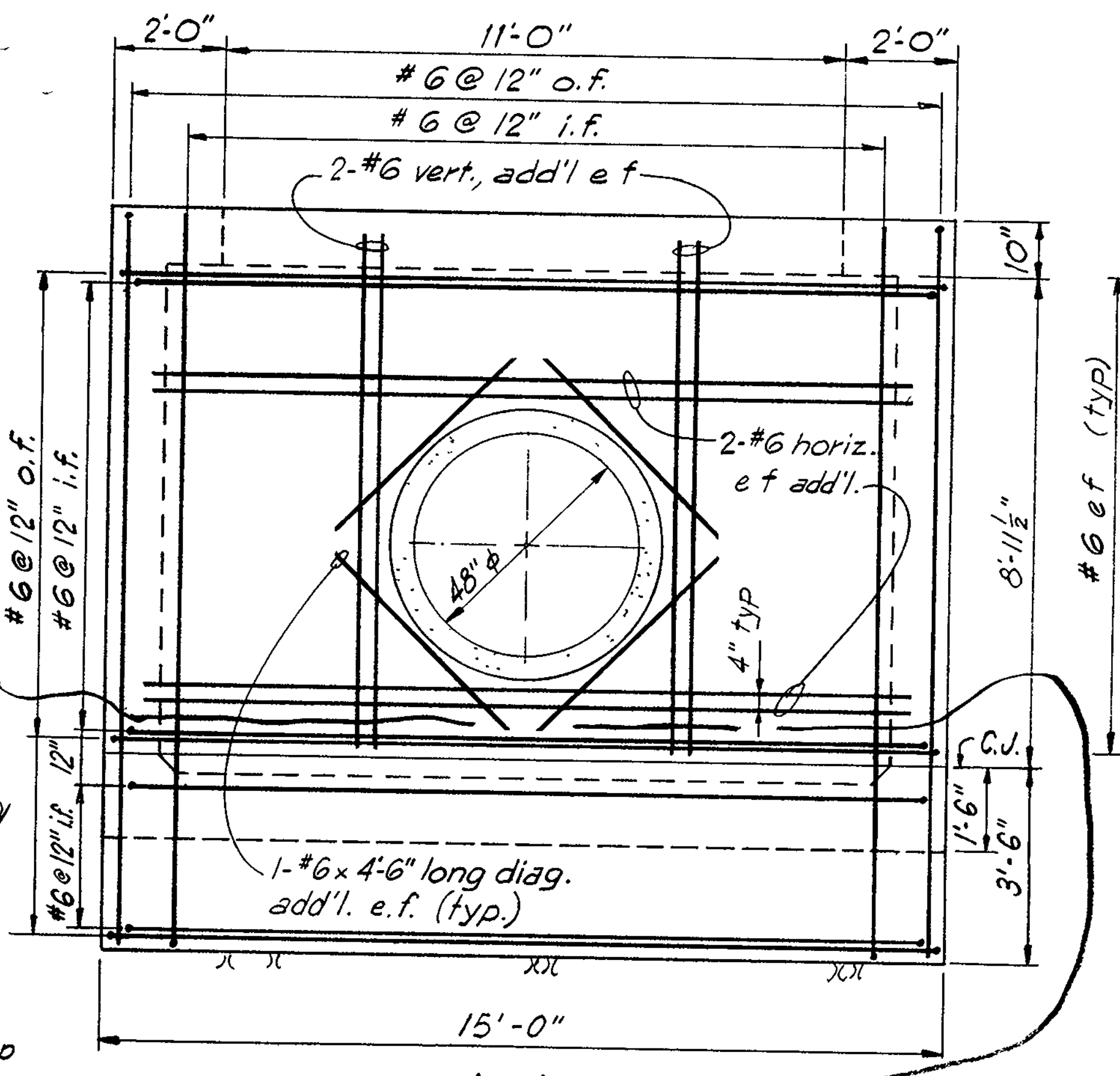
DETAIL OF SPIGOT WALL FITTING
(N.T.S.)

DETAIL OF PIPE JOINT
(N.T.S.)

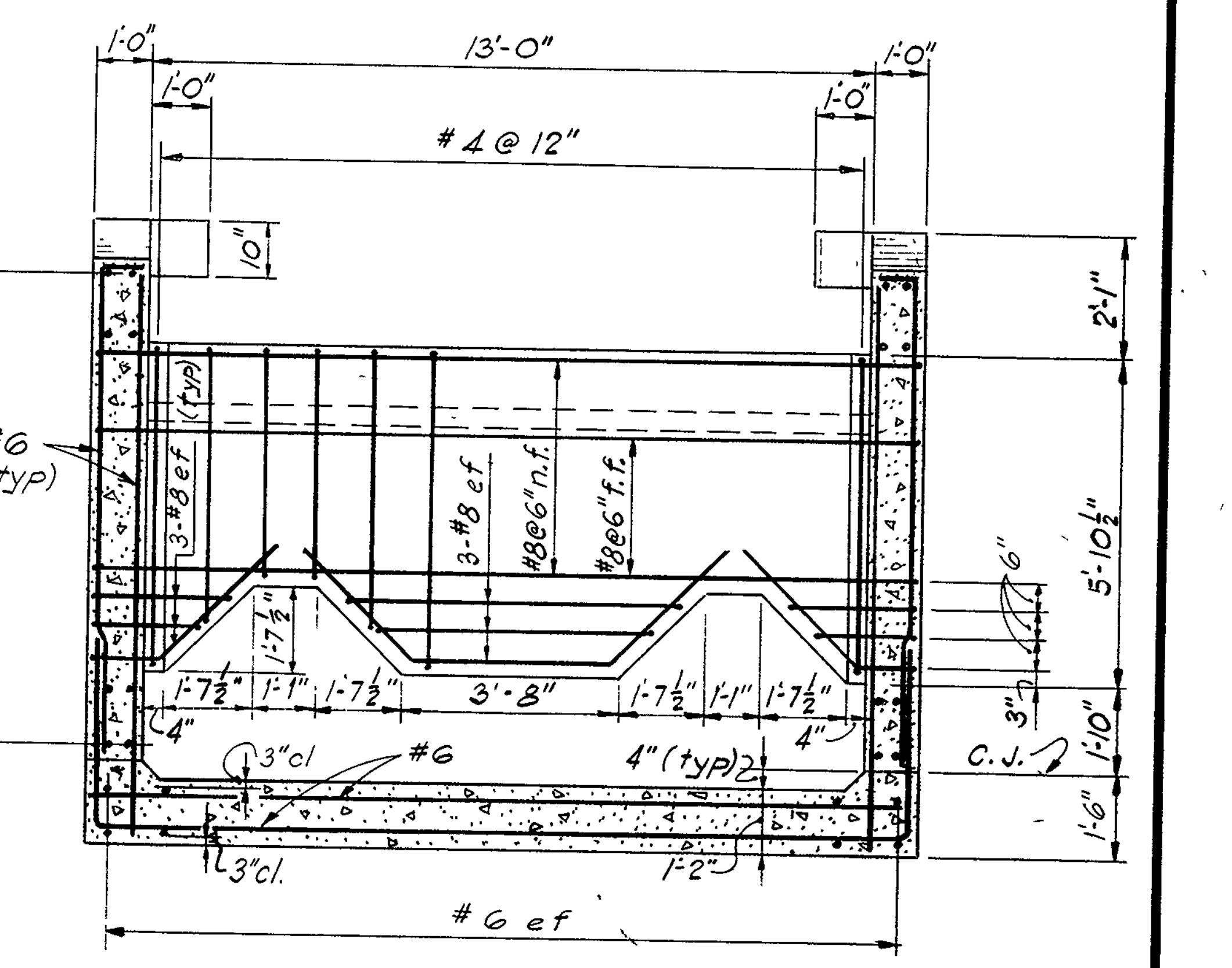


TO BE REVISED IN ACCORDANCE WITH PERMIT CONDITION 3

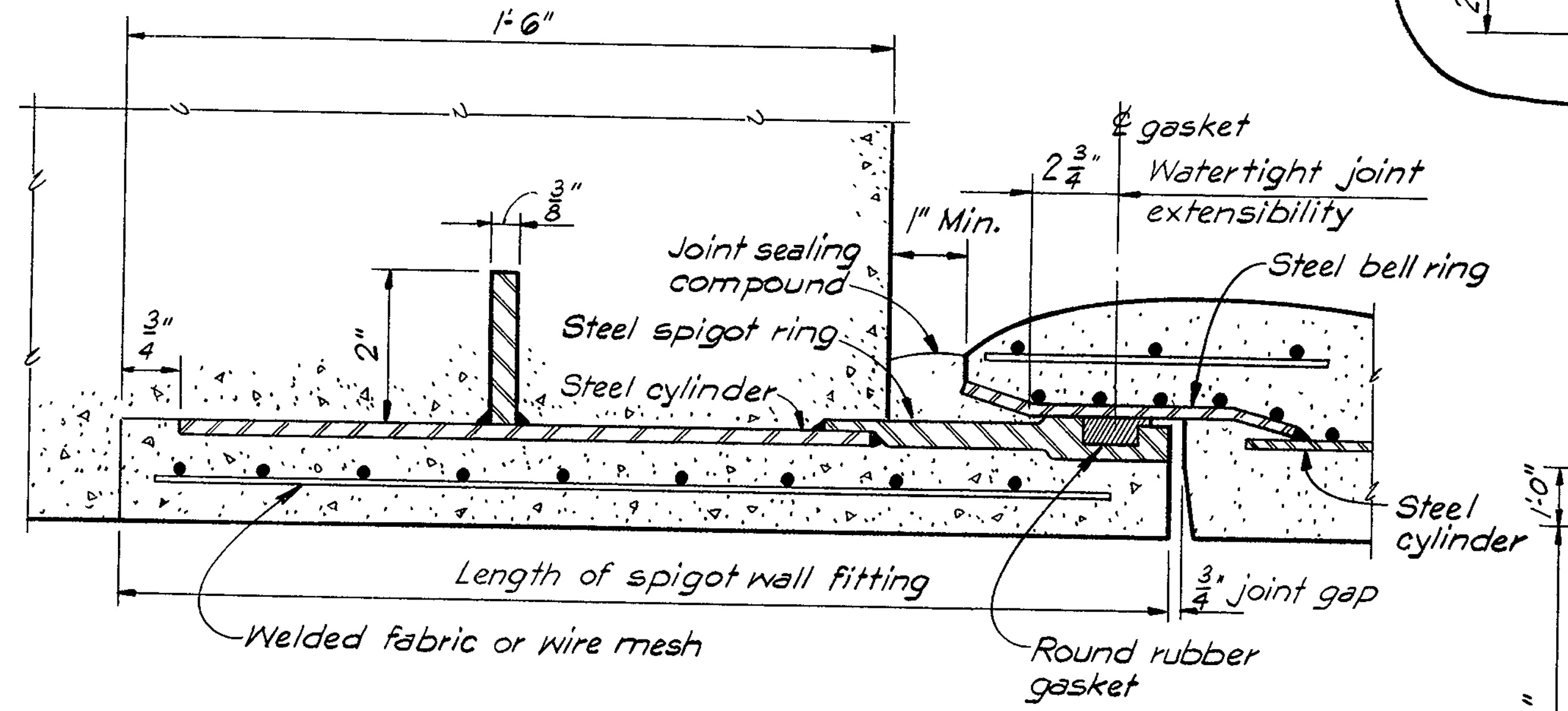
IMPACT BASIN ELEVATION



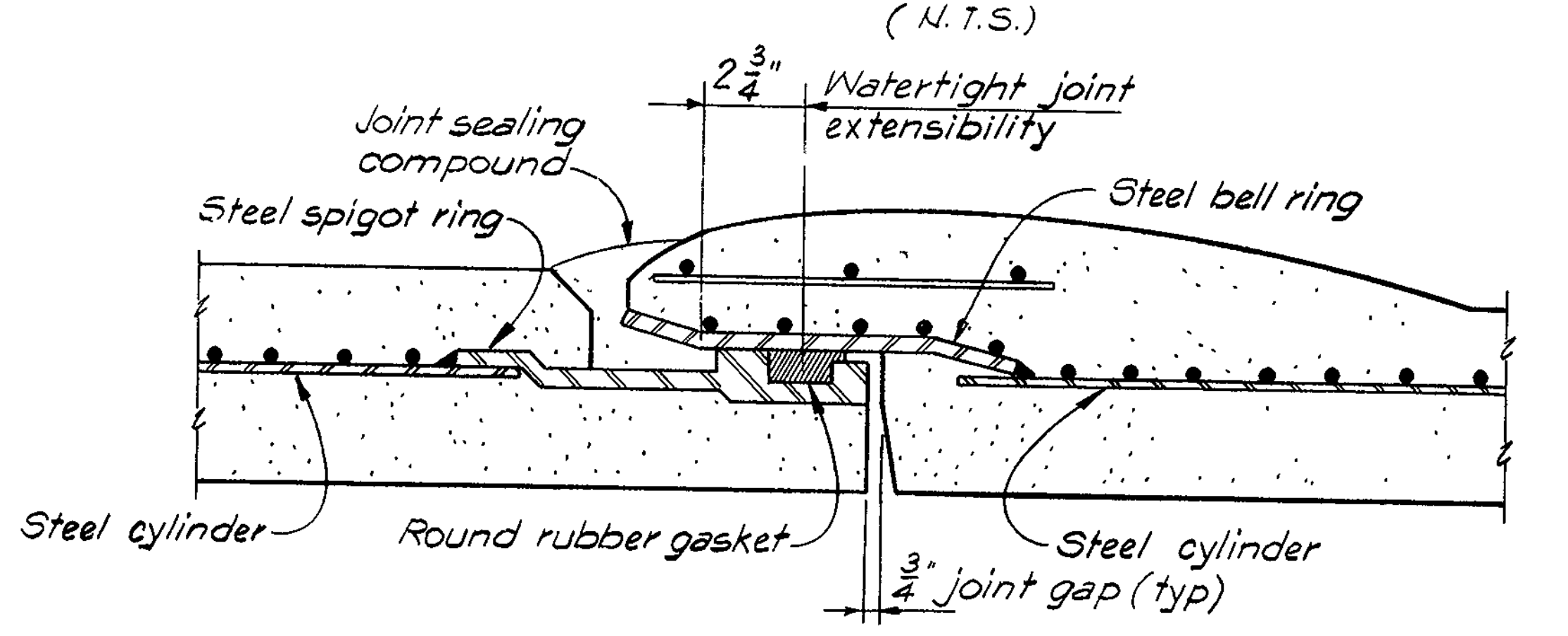
A - A



B - B

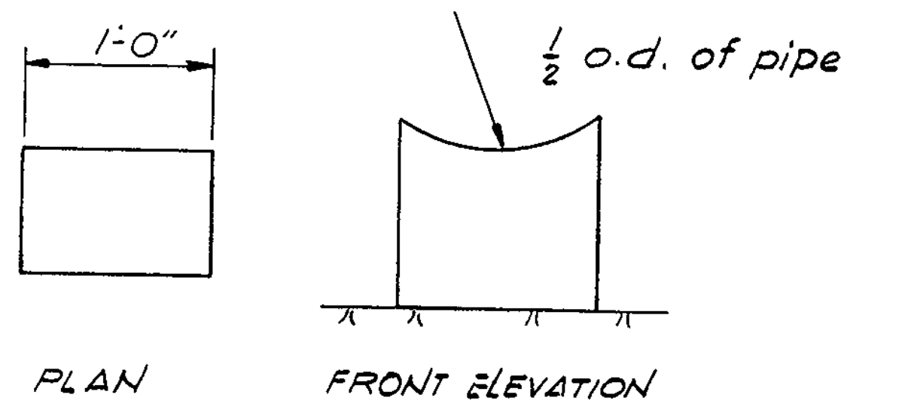


DETAIL A
(N.T.S.)



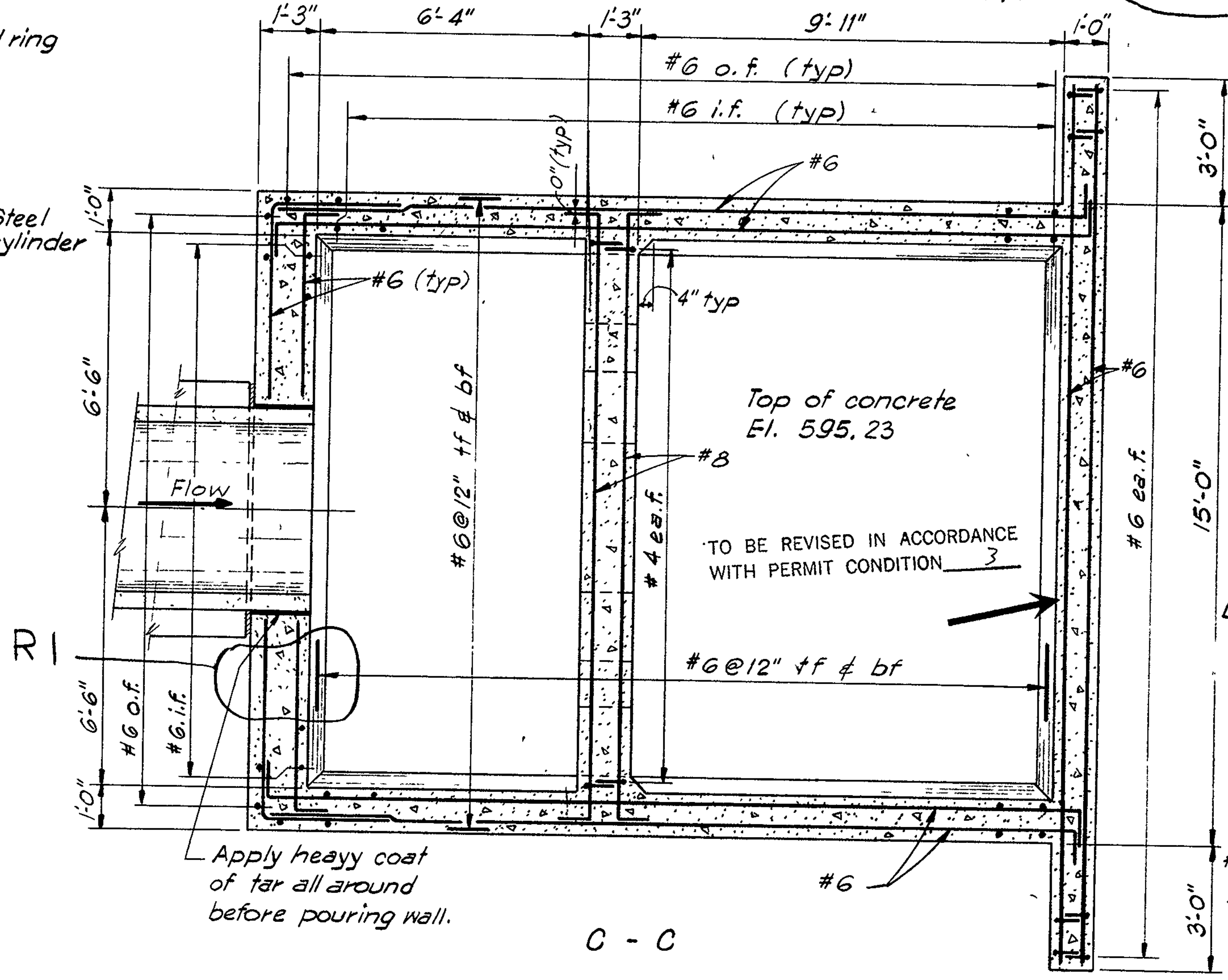
The pipe shall be drawn together so that the maximum joint gap does not exceed 3/4" for pipe laid on a straight line. (Typical unless noted otherwise)

DETAIL B
(N.T.S.)



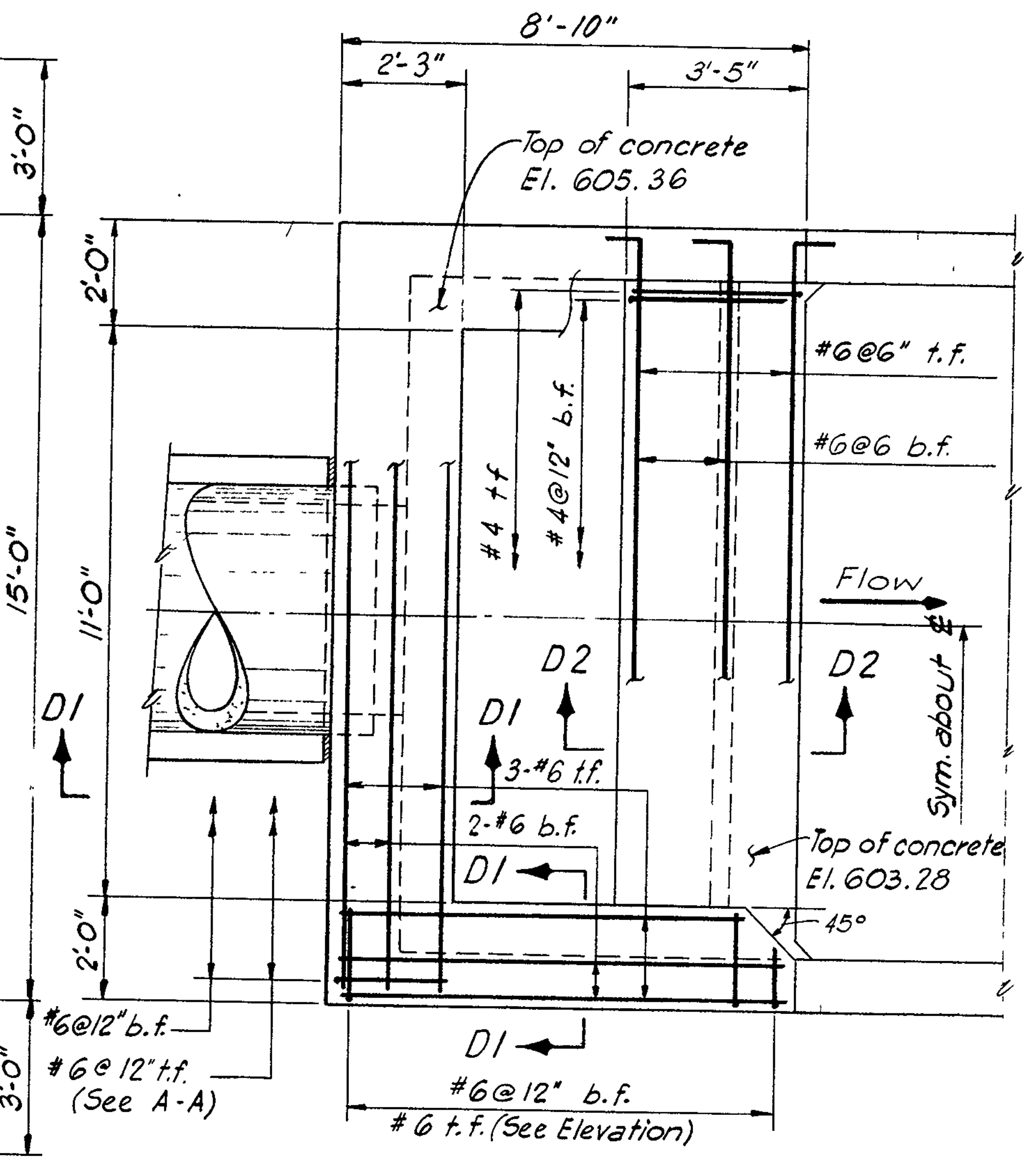
Sufficient blocks shall be provided to support the pipe to the required line and grade. The contractor shall determine the number and size of blocks required. Wedges may be used as an alternate.

SUGGESTED SUPPORT BLOCKS
(N.T.S.)

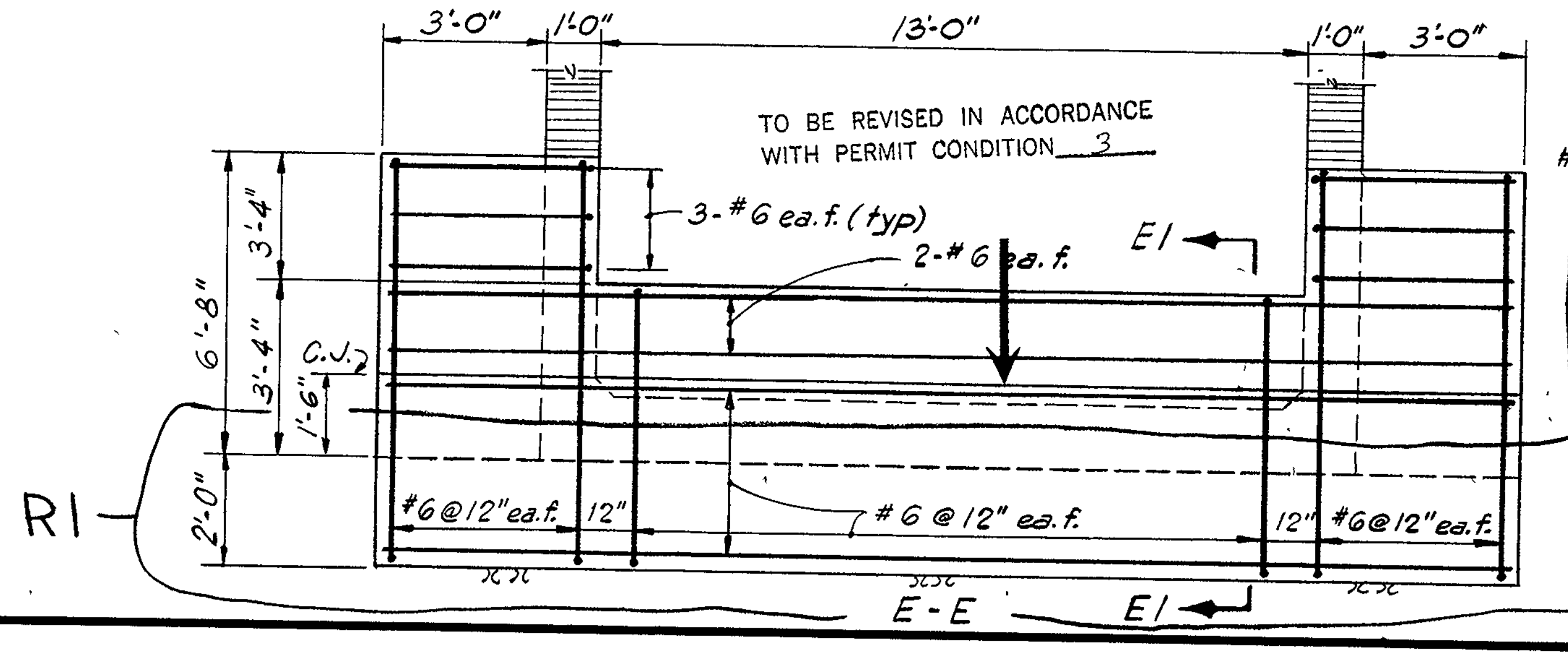


TO BE REVISED IN ACCORDANCE WITH PERMIT CONDITION 3

C - C

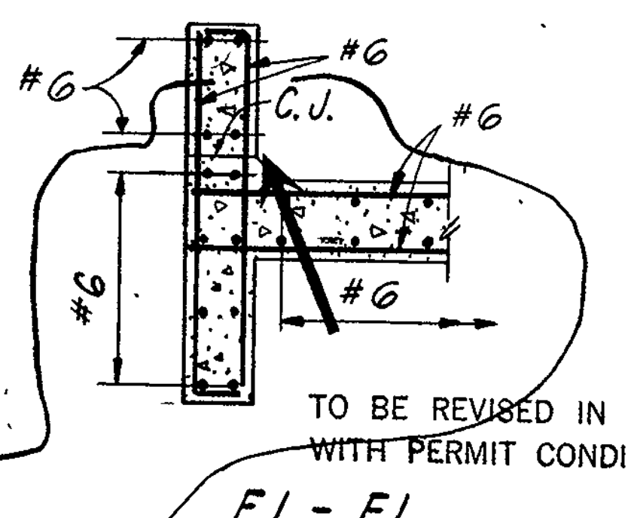


D - D



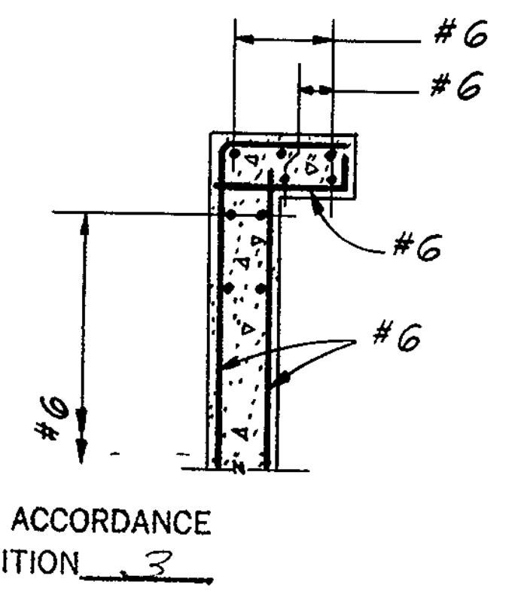
TO BE REVISED IN ACCORDANCE WITH PERMIT CONDITION 3

E - E

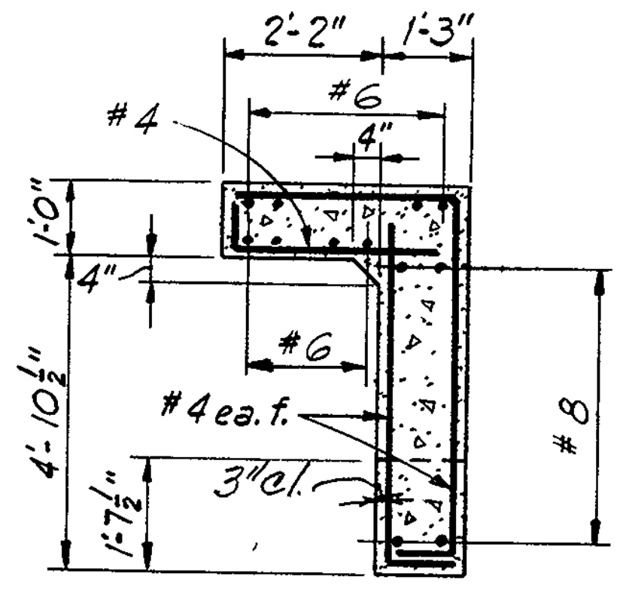


TO BE REVISED IN ACCORDANCE WITH PERMIT CONDITION 3

E1 - E1



D1 - D1



D2 - D2

REFERENCE DRAWINGS:

Work this dwg. with 670 C 201, C 202, C 203.

NOTES:

1. All exposed corners of structure shall have 3/4" chamfer.
2. For general notes see C 201.

STATE OF OHIO
DEPARTMENT OF NATURAL RESOURCES
DIVISION OF WATER
DAM PERMITS
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CONDITIONALLY
APPROVED
PERMIT NO. 7368
JUN 5 1973
Roy Winkle
CHIEF

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

DATE	LETTER NO.

DWG. TRANSMITTAL

	BY	DATE	CHKD.	DATE
DSGN.	M.J.	March 1973	V.J.Z.	March 1973
DWN.	P.F.	March 1973	M.J.	MARCH 1973
DIV.	GROUP LDR.	SECT. HEAD	DEPT. HEAD	DIV. HEAD
CIVIL		Hwc	M.Br.	RPW
MECH.				
ELECT.				
PLAN.				
STAFF	BR. HEAD	CH. ENGR.		

REV. NO.	DATE	NATURE OF REVISION	BY	CHKD.	APPD.
1	5-24-73	Revised Impact Basin Elevation, reinf. & Cradle Detail	M D	M.Br.	RPW

OHIO ELECTRIC COMPANY
GAVIN POWER PROJECT

STINGY RUN FLY ASH DAM
OUTLET WORKS

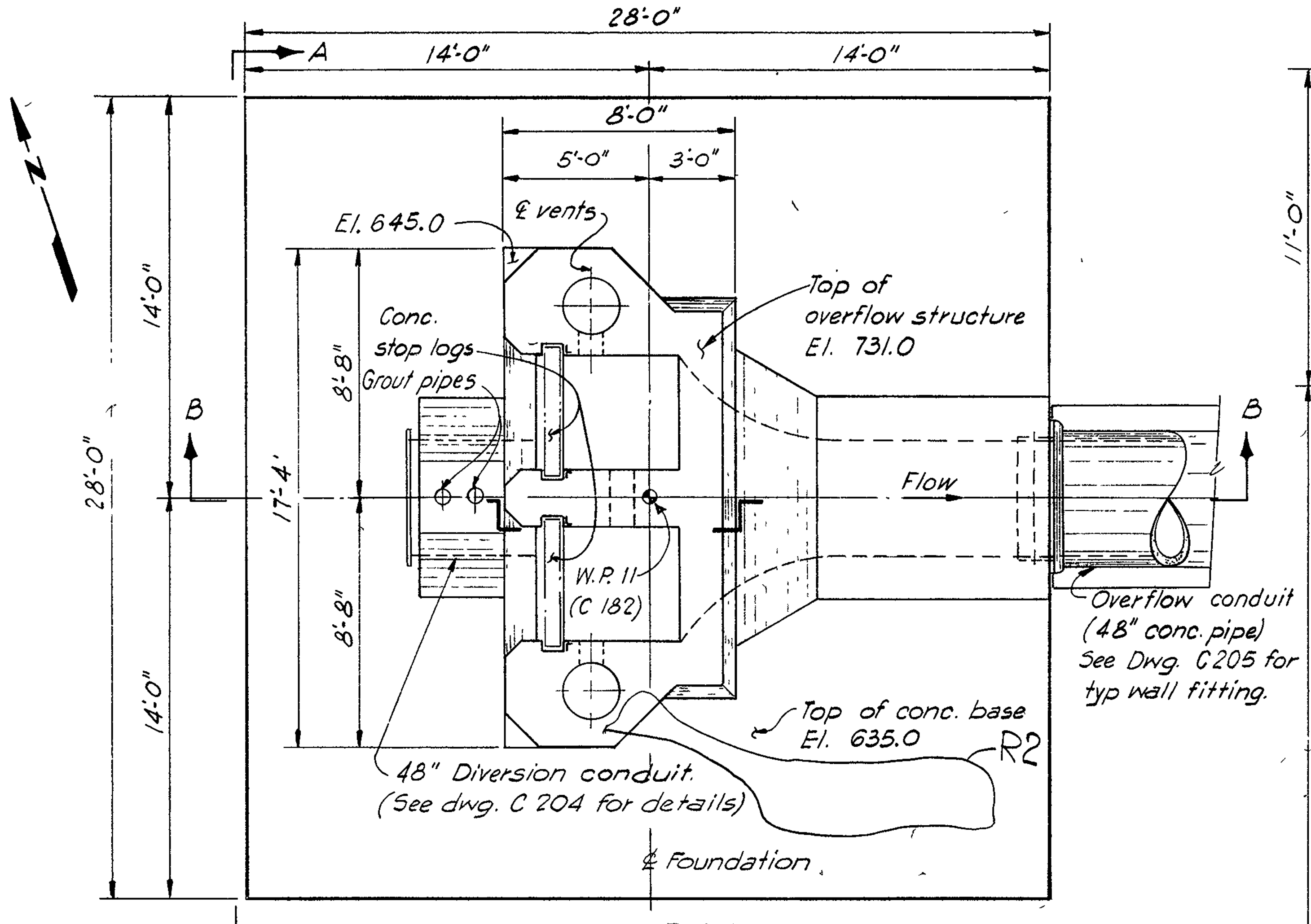
PRINCIPAL SPILLWAY
CONDUIT AND IMPACT BASIN

TO BE ~~REVISED~~ IN ACCORDANCE
~~WITH PERMIT CONDITION~~ 4

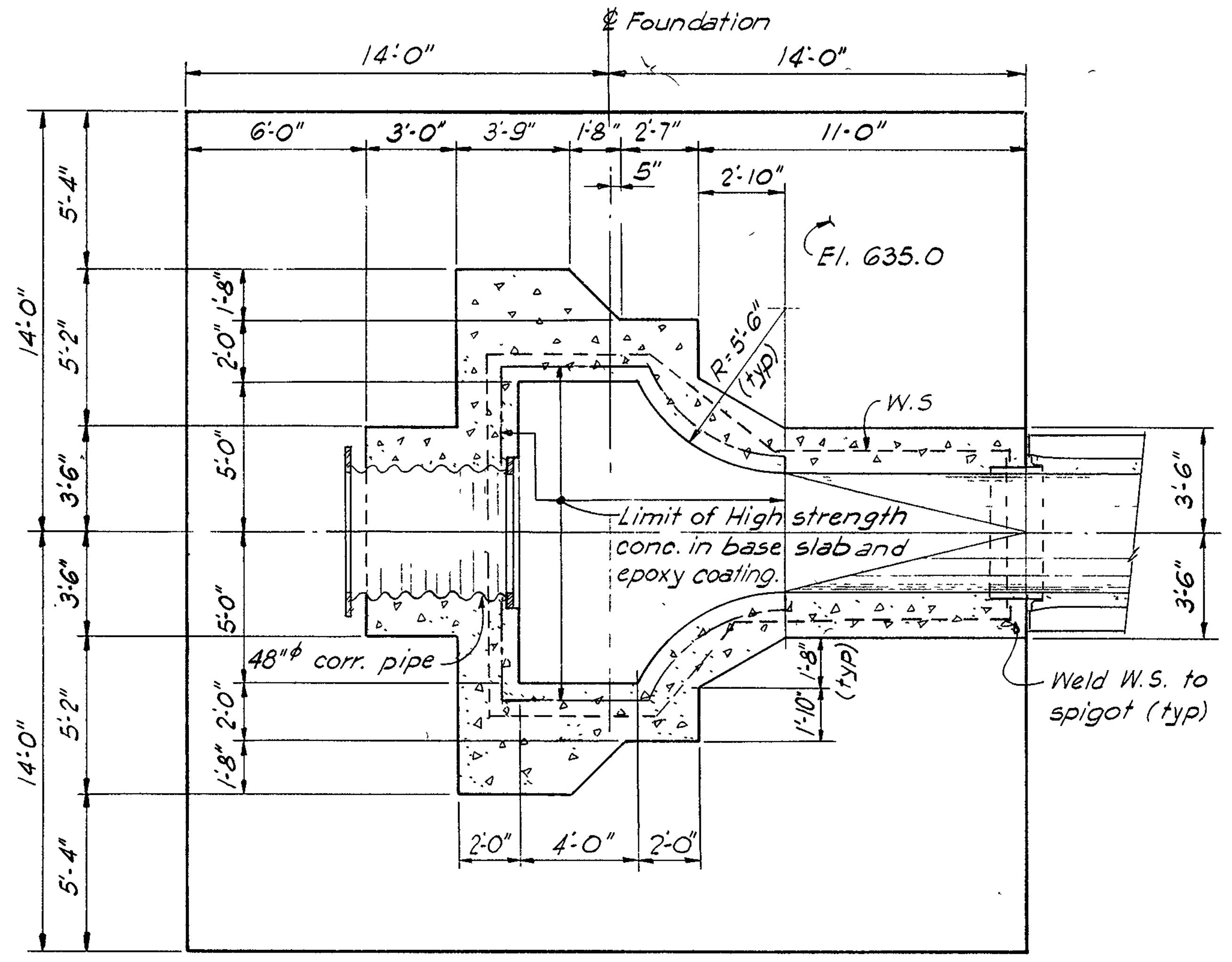
HARZA ENGINEERING COMPANY
APPROVED Richard D. Harza

CHICAGO, ILLINOIS
DATE MAR., 1973
DWG. NO. 670 C 205 R1

REVISED



PLAN



B1 - B1

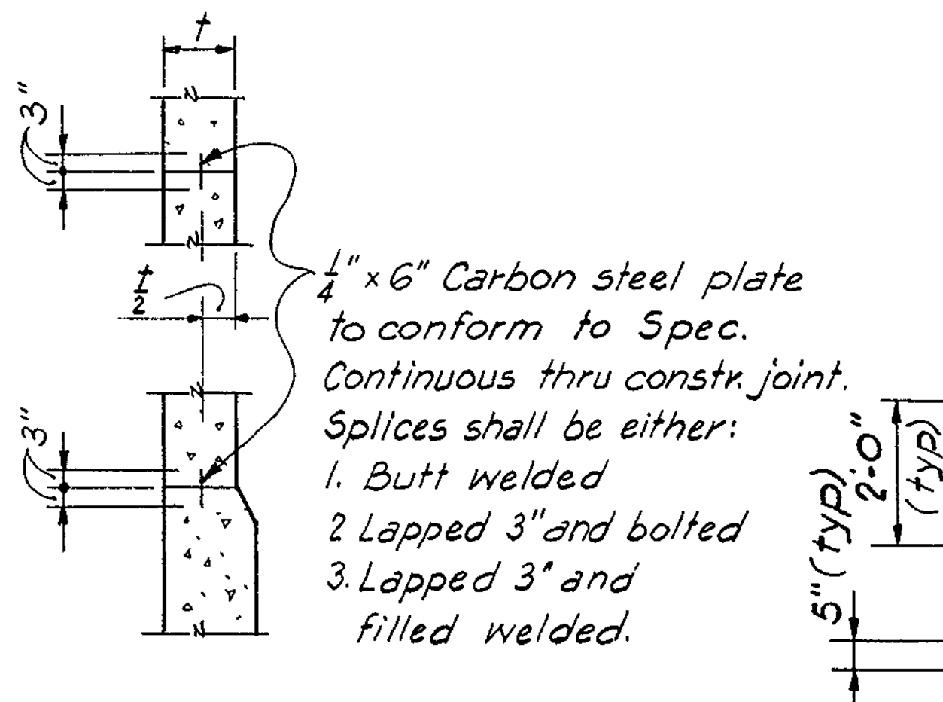
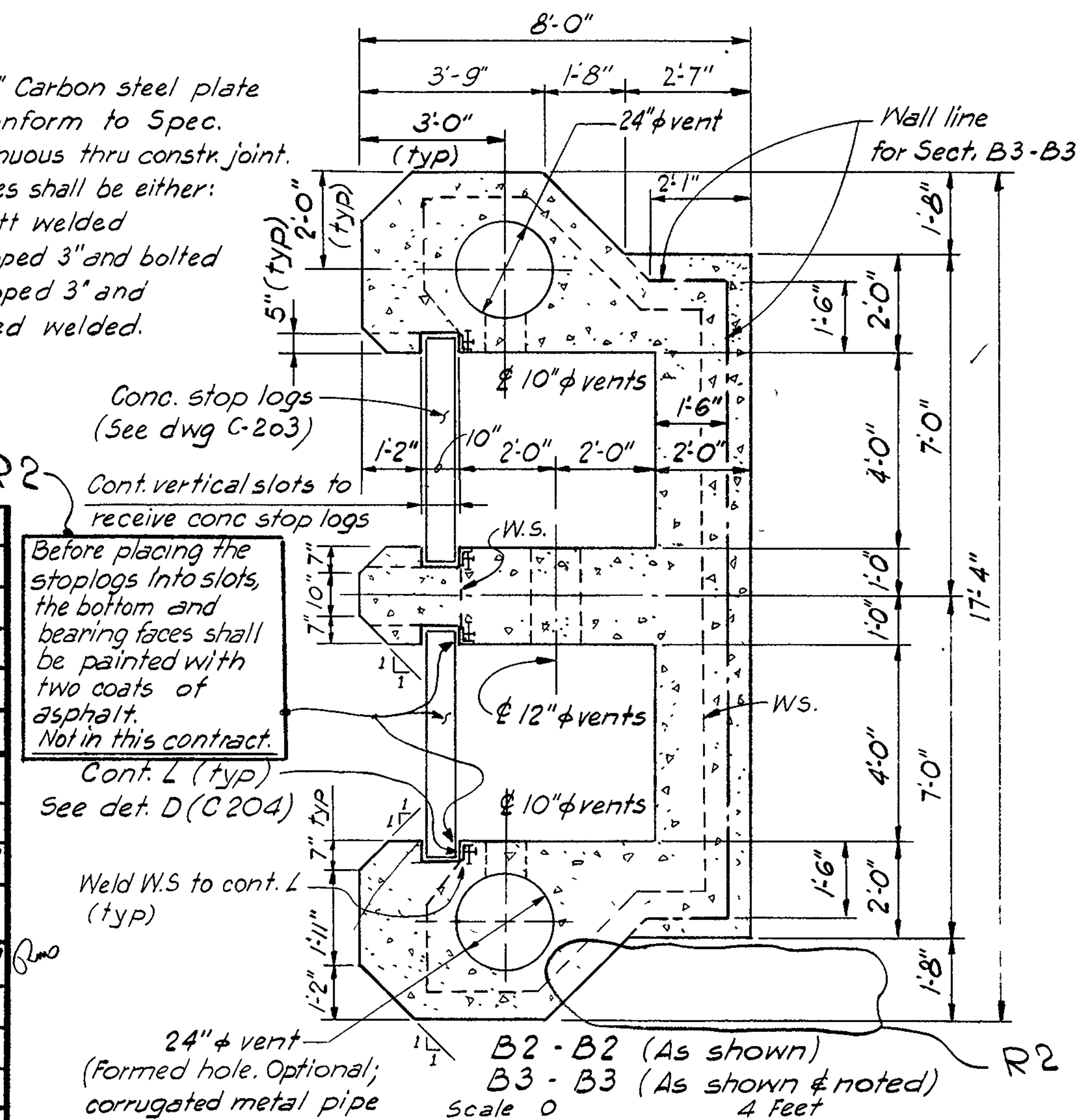
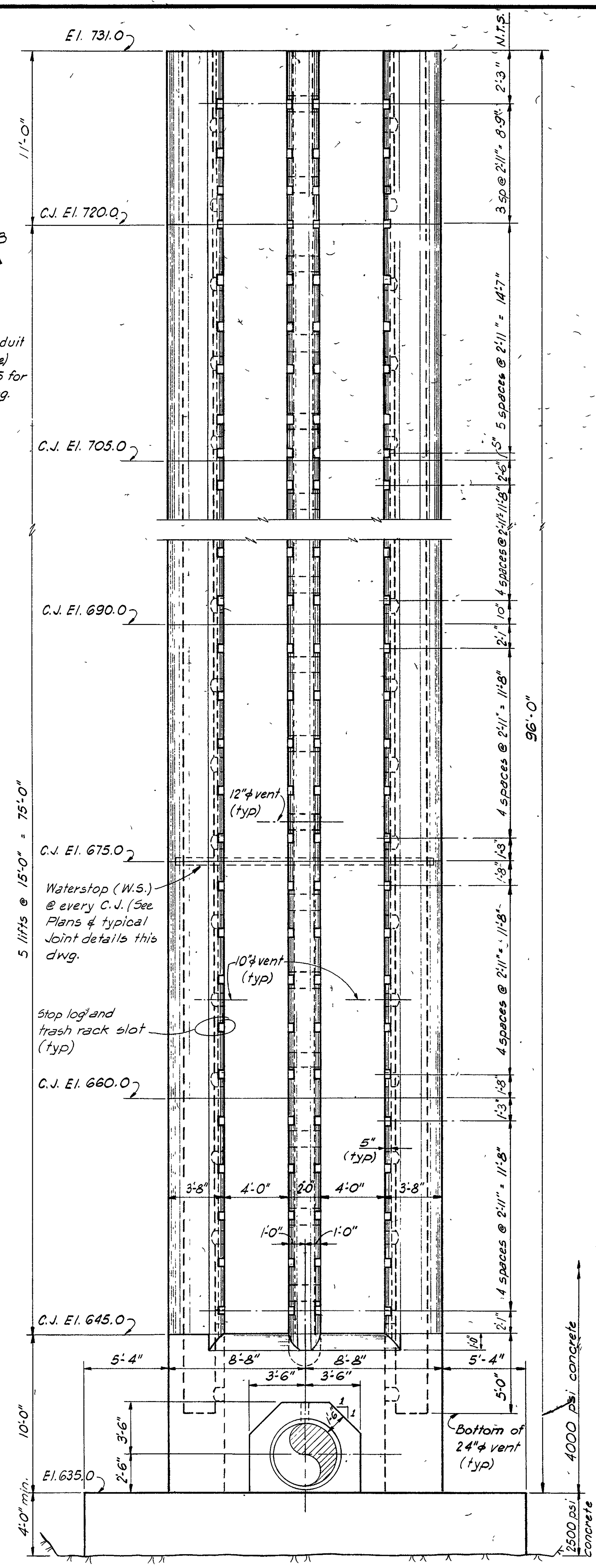


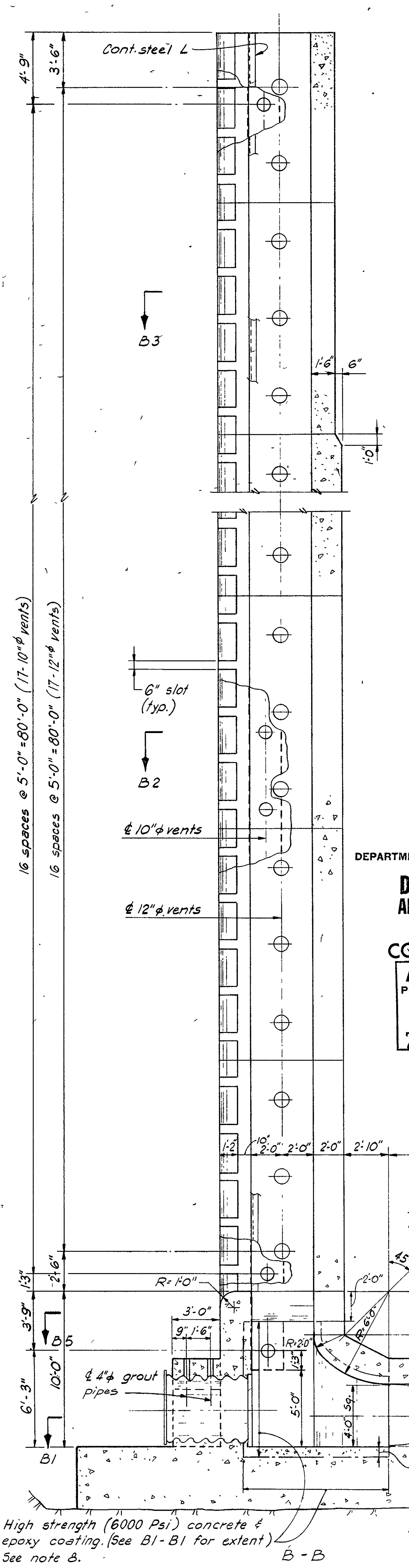
PLATE
CONSTR. JOINT
(N. T. S.)



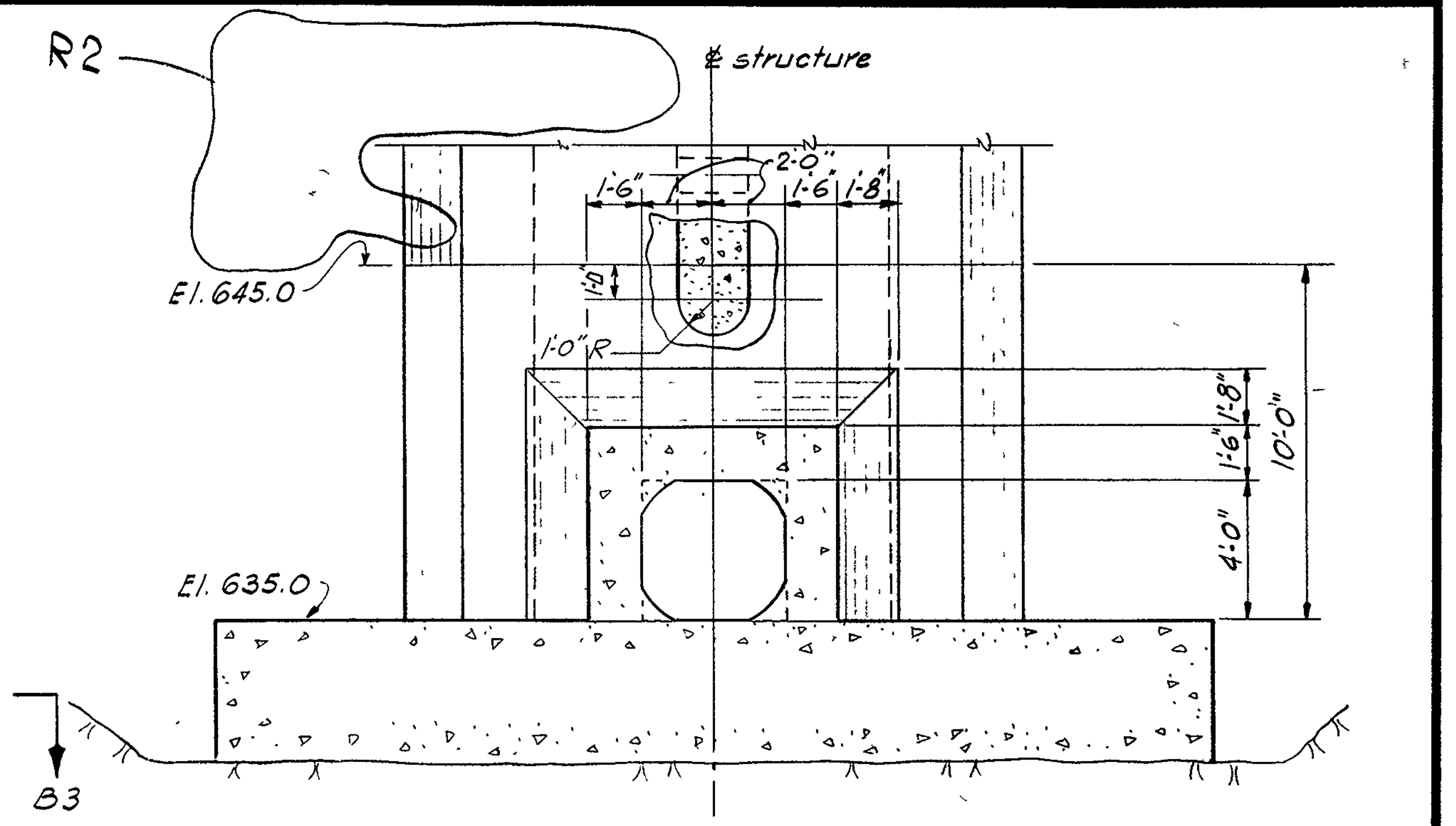
B2 - B2 (As shown)
B3 - B3 (As shown & noted)
Scale 0 4 Feet
3/8" = 1'-0"



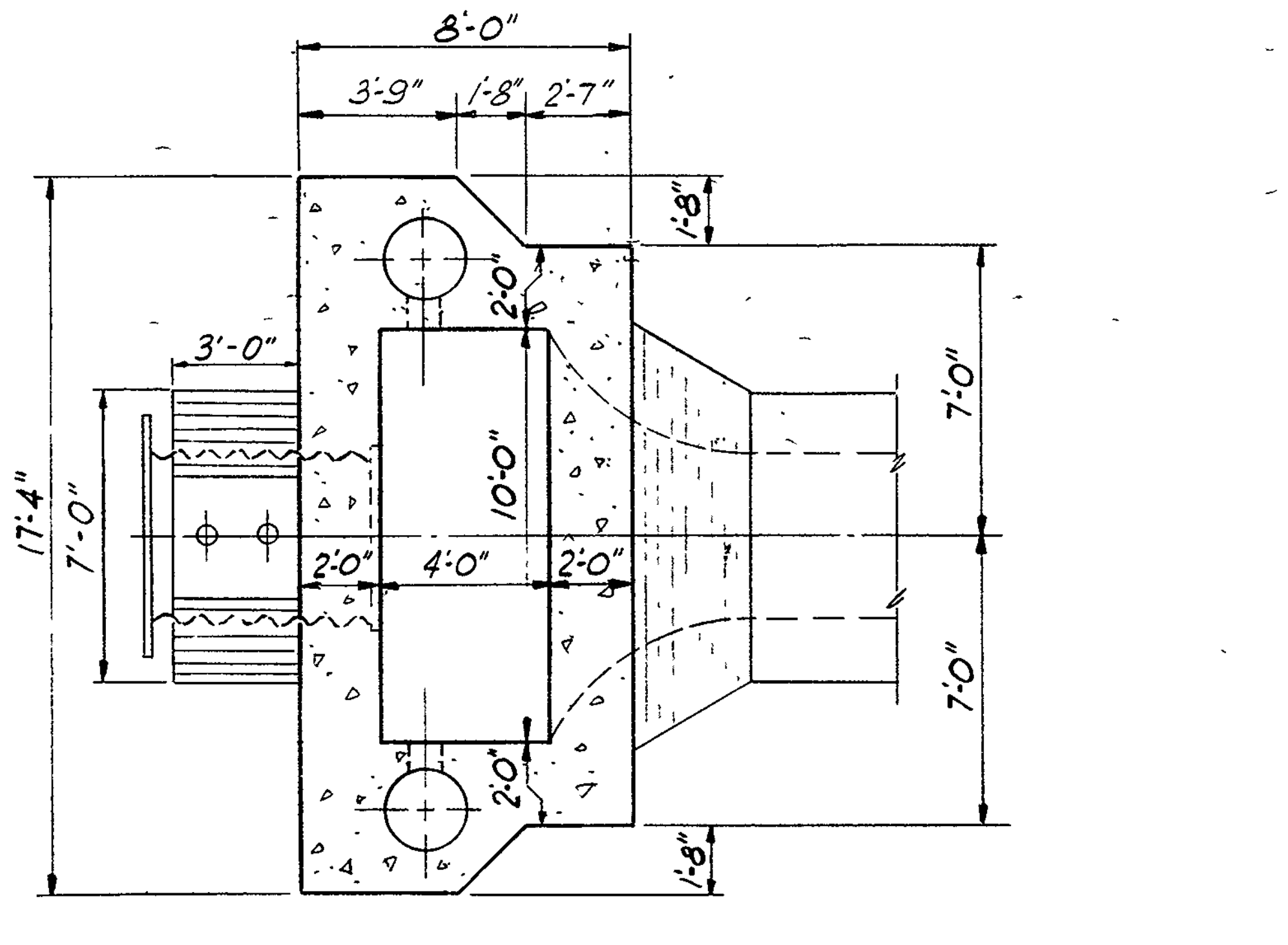
ELEVATION A - A



B - B



B4 - B4



B5 - B5

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

CONDITIONALLY
APPROVED
PERMIT NO. 7368
JUN 5 1973
Roy Winkle
CHIEF

REFERENCE DRAWINGS:

Work this dwg with 670 C 202, C 203, C 204, C 205.
Excavation and Foundation (Layout) = C 181
Sections and Details = C 183.

NOTES:

1. All concrete and grout shall be 4000 psi strength unless otherwise noted.
2. Reinforcing steel shall be deformed bars and shall conform to ASTM - A 615 - 71 Grade 60 Specifications.
3. Concrete cover for reinforcing bars shall be 2" or as indicated on drawings.
4. Bottom of foundation shall rest on solid rock.
5. All exposed corners of structure above El. 635.0 shall have 3/4" chamfer.
6. After completion of outlet works and dam, diversion conduit shall be completely filled with Class II concrete through grout pipes as shown (see C204)
7. For additional information see specifications and for additional notes see indvidual drawings.
8. The indicated concrete area shall be painted with "COLMA-DUR LV" epoxy resin or equal. Surface preparation and epoxy application shall be in accordance with manufacturers recommendations. Epoxy coats shall be applied as follows: 1. One primer coat of Epoxy Resin Binder, 2. Two finish coats, approx. 1/32 inch each.

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

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MAY 29 1973
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DIV. OF WATER

DATE	LETTER NO.

DWG. TRANSMITTAL

	BY	DATE	CHKD.	DATE
DSGN.	M.J.	Jan. 1973	V.J.Z.	Feb. 1973
DWN.	P.F.	Feb. 1973	M.J.	March 1973
DIV.	GROUP LDR.	SECT. HEAD	DEPT. HEAD	DIV. HEAD
CIVIL		Hwz	M.Br	RPW
MECH.				
ELECT.				
PLAN.				
STAFF		BR. HEAD	CH. ENGR.	

REV. NO.	DATE	NATURE OF REVISION	BY	CHKD.	APPD.
R2	5-17-73	Ladder rungs deleted & note added	P.F.	M.Br.	RPW
R1	4-2-73	Note added	P.F.	M.Br.	DmO

TO BE ~~REVISED~~ IN ACCORDANCE WITH PERMIT CONDITION 4

OHIO ELECTRIC COMPANY
GAVIN POWER PROJECT

STINGY RUN FLY ASH DAM
OUTLET WORKS

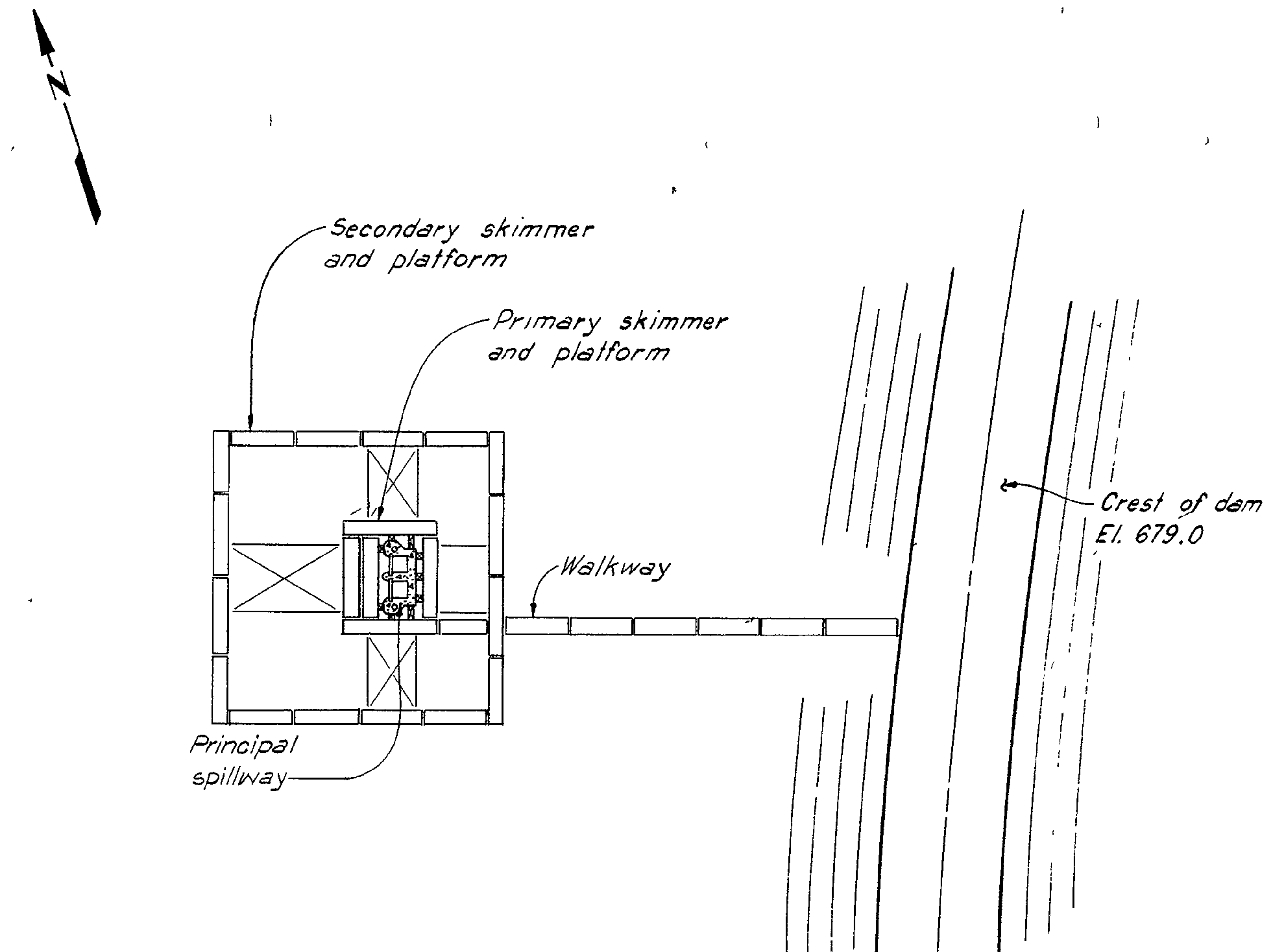
PRINCIPAL SPILLWAY PLAN AND SECTIONS

HARZA ENGINEERING COMPANY
APPROVED Richard D. Dozy

CHICAGO, ILLINOIS
DATE MAR., 1973
DWG. NO. 670 C 201 R2

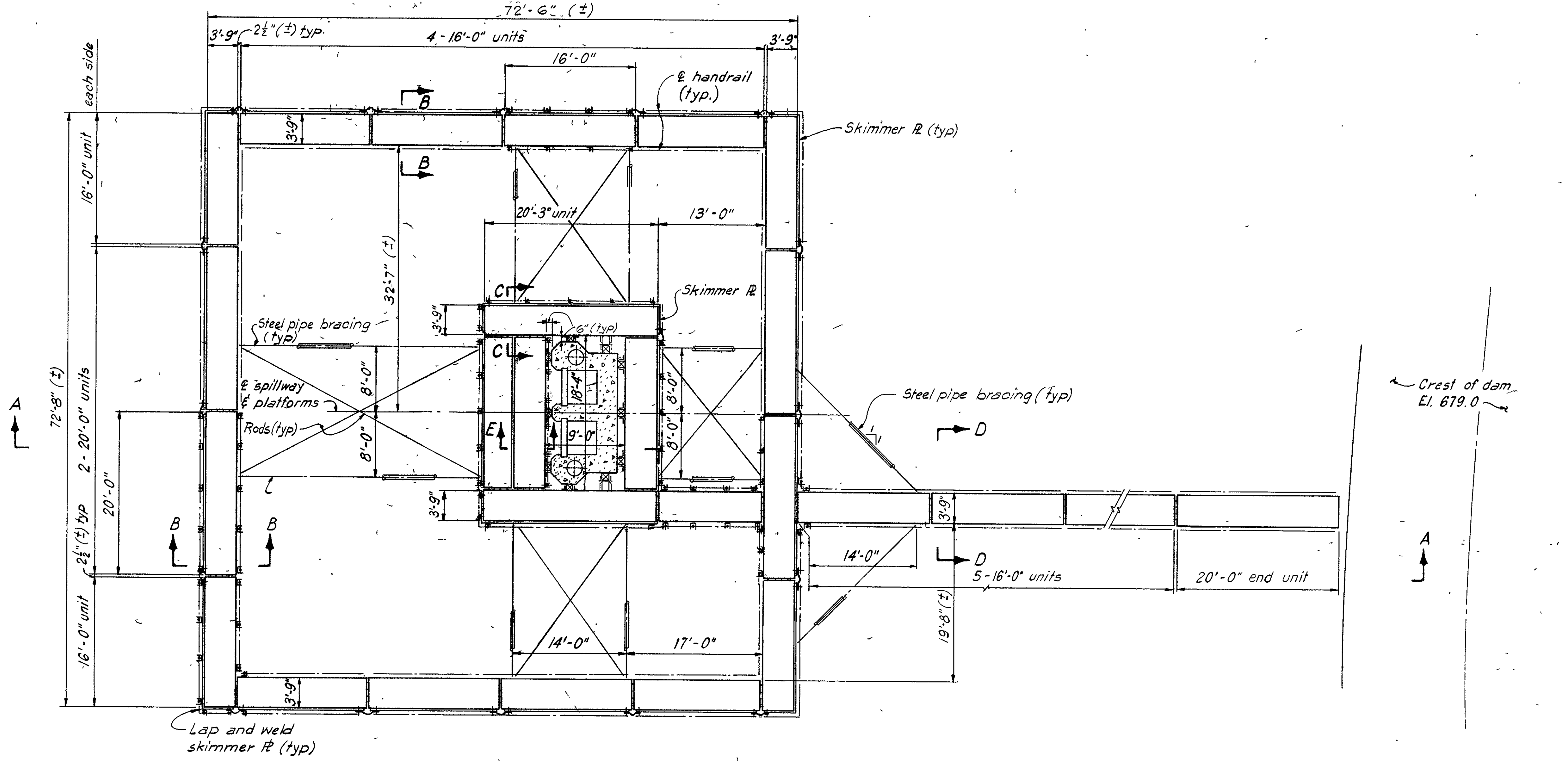
REVISED

K & E CO. 10-1253

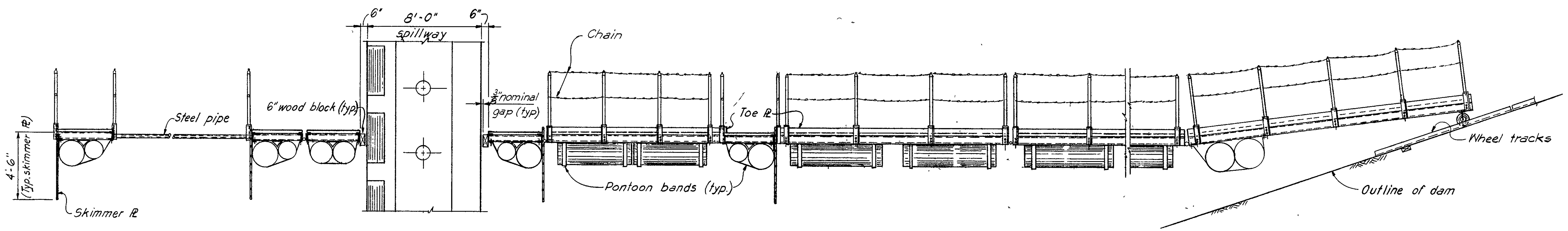


KEY PLAN

Scale 0 30 Feet
1" = 30'

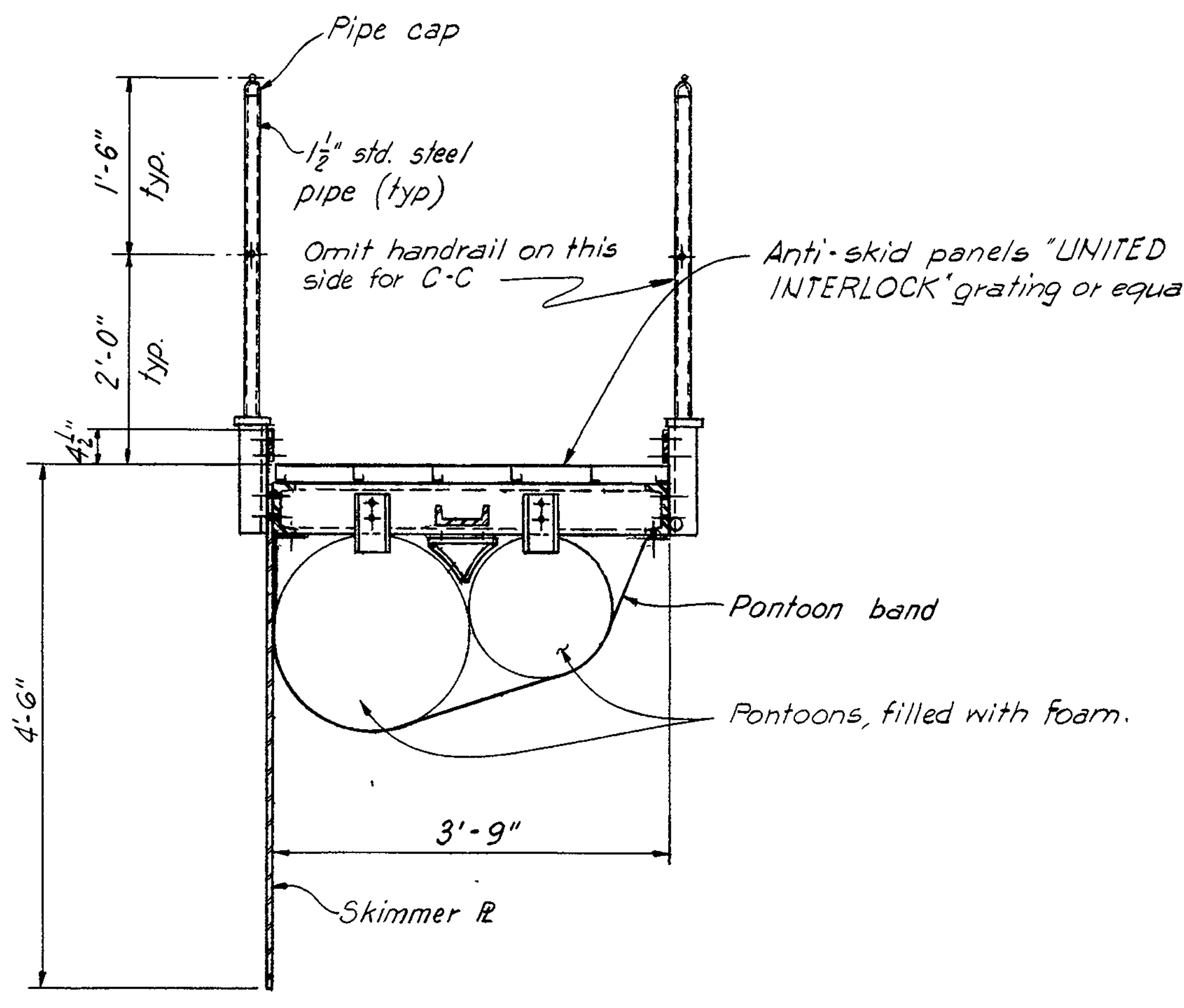


PLAN



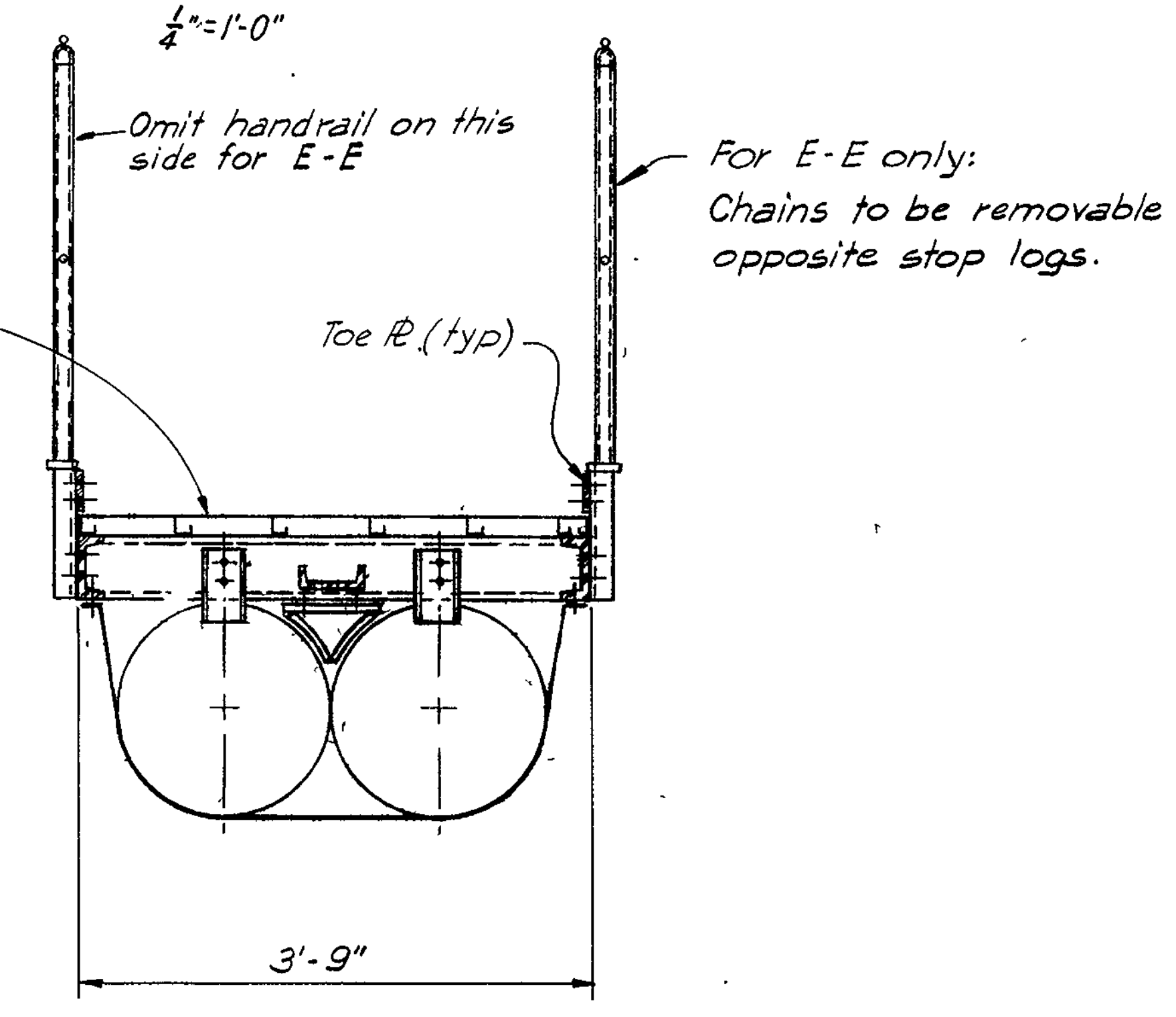
A - A

Scale 0 4 Feet
$\frac{1}{4}$" = 1'-0"



B - B (As shown)
C - C (Similar)

Scale 0 1 Foot
$\frac{3}{4}$" = 1'-0"



D - D (As shown)
E - E (As shown & noted)
$\frac{3}{4}$" = 1'-0"

REFERENCE DRAWINGS:

Principal Spillway, Plan and Sections 670 C 201

NOTES:

The Floating Platform and Skimmer will be furnished by Owner. For installation requirements see specifications.

Scale 0 8 16 Feet
$\frac{1}{8}$" = 1'-0"
Except as noted

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DIVISION OF WATER
DAM PERMITS
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CONDITIONALLY
APPROVED
PERMIT NO. 7368
JUN 5 1973
Roy Winkle
CHIEF

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MAY 29 1973
DEPT. OF NATURAL RESOURCES
DIV. OF WATER

DATE		LETTER NO.		
DWG. TRANSMITTAL				
	BY	DATE	CHKD.	DATE
DSGN.	M.J.	March 1973	L.P.	March 1973
DWN.	M.D.	March 1973	M.J.	March 1973
DIV.	GROUP LDR.	SECT. HEAD	DEPT. HEAD	DIV. HEAD
CIVIL			M.Ba.	RPW
MECH.				
ELECT.				
PLAN.				
STAFF		BR. HEAD	CH. ENGR.	

REV. NO.	DATE	NATURE OF REVISION	BY	CHKD.	APPD.

OHIO ELECTRIC COMPANY
GAVIN POWER PROJECT

STINGY RUN FLY ASH DAM | OUTLET WORKS

PRINCIPAL SPILLWAY
FLOATING PLATFORMS AND SKIMMER

HARZA ENGINEERING COMPANY
APPROVED Richard D. Harza
CHICAGO, ILLINOIS
DATE MARCH, 1973
DWG. NO. 670 C206

K&E CO. 19-1253

Exhibit 3

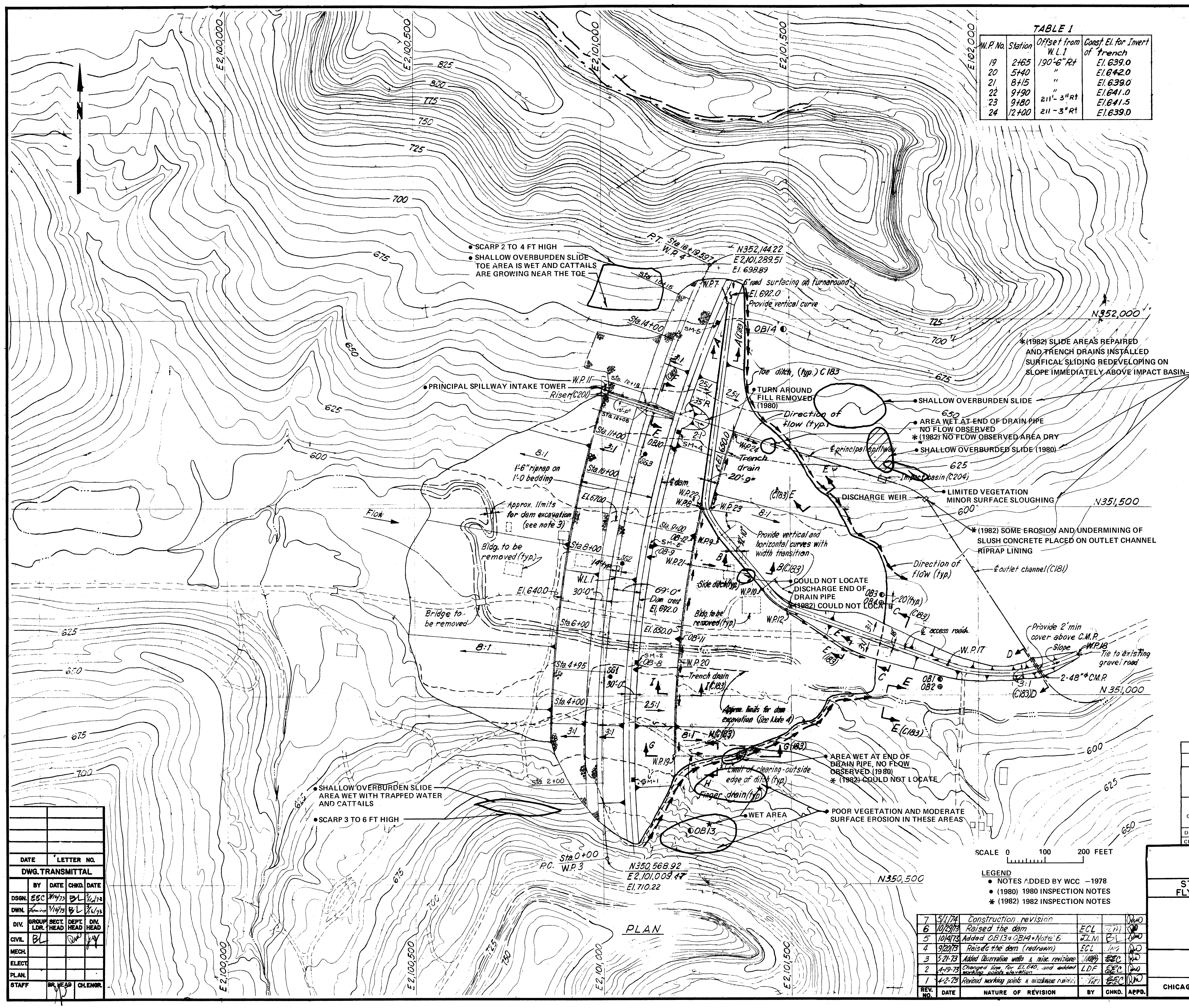


TABLE 1

W.P. No.	Station	Offset from W.L.1	Const. El. for Invert of trench
19	2+65	190'-6" Rt	El. 639.0
20	5+40	"	El. 642.0
21	8+15	"	El. 639.0
22	9+90	"	El. 641.0
23	9+80	211'- 3" Rt	El. 641.5
24	12+00	211 - 3" Rt	El. 639.0

CURVE DATA OF W.L. 1

R = 3,000.0'
Δ = 30°55'55.27"
T = 830.057'
L arc = 1,619.597'
D = 1°54'35.49"

W.P. No.	Sta.	Offset from W.L.1	Construction Elevation
7	15+45	84'-0" RT	691.50
8	9+80	189'-0" RT	654.26
9	9+09	253'-0" RT	646.20
10	7+78	390'-0" RT	627.67
11	12+18	140.0' LT	For horizontal control only
12	7+27	462.0 RT	617.27
17	6+08	909.0' RT	587.0
18	7+18	1252'± RT	582±

LEGEND

■ SM1 Surface settlement monument
● SG1 Settlement gage
◐ OB1 Observation well, Type I
⊕ OB2 Observation well, Type II
▲ OB8 - OB12 Observation well, Type II (see Note 6)

SURVEY CONTROL

Working points 3 and 4 were established by the Ohio Power Company. These points shall be used to establish construction survey by the Contractor.

REFERENCE DRAWINGS

Geology	670C101, C110 thru C112 C121 thru C138. C141 thru C143
Soil tests	670C151 thru C153, C156 C161 thru C164, C167
Outlet works	670C191 thru C193 C201 thru C206

NOTES

1. Work this drawing with 670C181, C183 thru C187
2. Topography on this drawing shall not be used for payment purposes. See note 3 on C184
3. Approximately 10' unsuitable material, mainly silt, along the stream bed is expected to be removed. Exact limits and depths may vary and will be determined by the Owner's Engineer.
4. Approximately 2' to 16' strip mine disposal material is distributed in this area. The upper 1'-6" shall be stripped and is to be defined as Stripping. The strip mine disposal material below 1'-6" is expected to remain, subject to approval by the Owner's Engineer. If removal is required, payment will be made as Dam Excavation.
5. All elevations shown are settled elevations except as noted. All slopes shown are settled slopes except as noted.
6. All observation wells will be installed by contractor.
7. All ponded water within strip mine areas within the reservoir to be drained.
8. All stationing along the dam refers to W.L.1
9. All P.V.C. pipe shall be Schedule 40 and conform to ASTM D1785

SCALE 0 100 200 FEET

LEGEND
- ● NOTES ADDED BY WCC —1978
- ● (1980) 1980 INSPECTION NOTES
- ✱ (1982) 1982 INSPECTION NOTES

REV. NO.	DATE	NATURE OF REVISION	BY	CHKD.	APPD.
7	5/1/74	Construction revision			RMO
6	10/29/73	Raised the dam	ECL	JLM	RO
5	10/4/73	Added OB13 & OB14 + Note 6	JLM	BL	RMO
4	9/20/73	Raised the dam (redrawn)	ECL	JWG	RO
3	5-21-73	Added Observation wells & misc. revisions	[illegible]	EEC	RMO
2	4-19-73	Changed line for El. 640, and added working points elevation	LDF	EEC	RMO
1	4-2-73	Revised working points & miscellaneous revisions	[illegible]	EEC	RMO

DATE	REVISION
1/7/83	ADDED 1982 INSPECTION NOTES

GAVIN FLY ASH DAM
FIELD INSPECTION NOTES

WOODWARD-CLYDE CONSULTANTS
CONSULTING ENGINEERS, GEOLOGISTS AND ENVIRONMENTAL SCIENTISTS
CLIFTON, NEW JERSEY

DR. BY:	DRS	SCALE:	AS SHOWN	PROJ. NO.:	77C11082L
CK'D. BY:	SAL	DATE:	21 DEC 1978	FIG. NO.:	2

OHIO ELECTRIC COMPANY
GAVIN POWER PROJECT

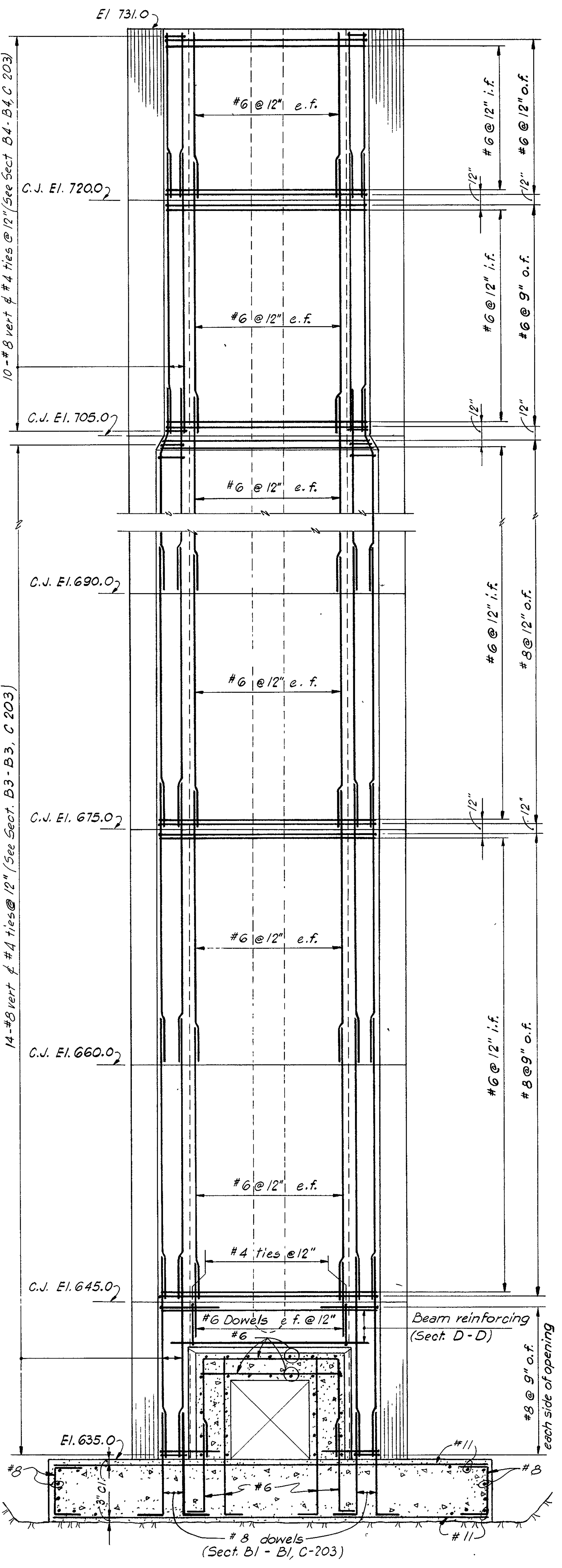
STINGY RUN FLY ASH DAM — DAM

PLAN

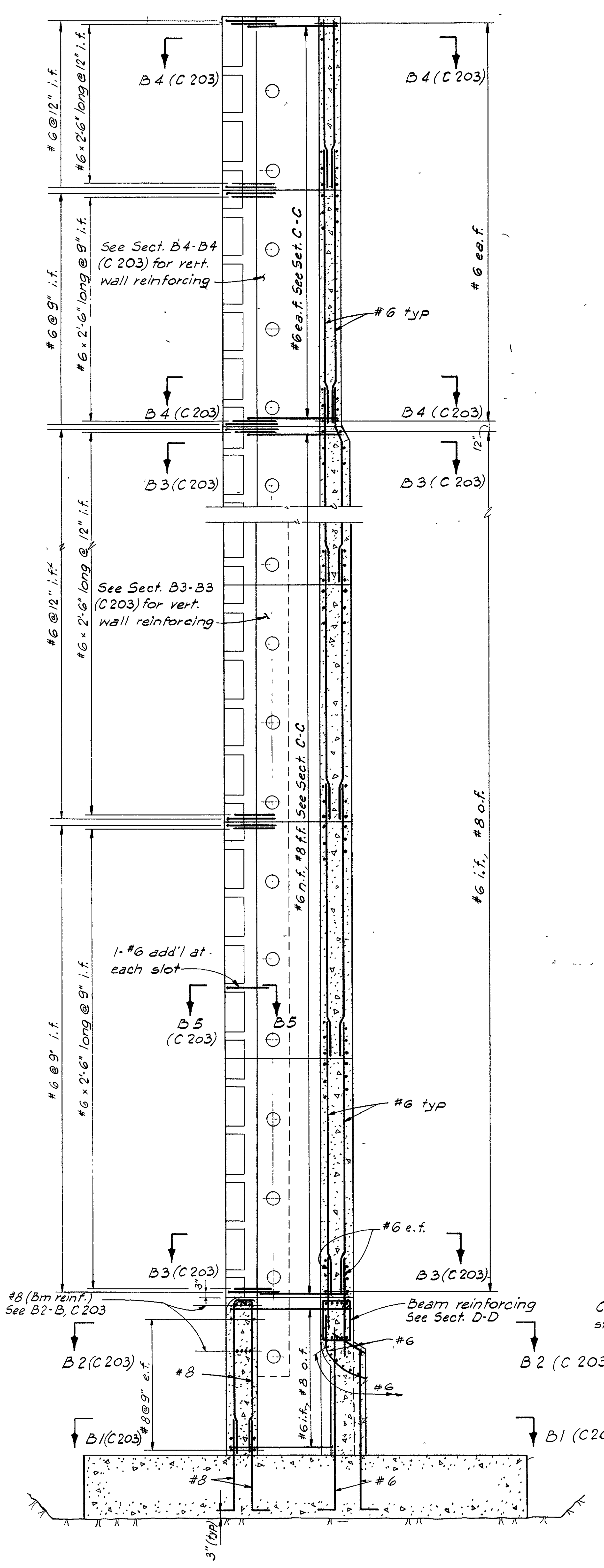
HARZA ENGINEERING COMPANY
APPROVED Richard D. Harza
CHICAGO, ILLINOIS
DATE MAR., 1973
DWG. NO. 670-C182 F7

DATE	LETTER NO.			
DWG. TRANSMITTAL				
	BY	DATE	CHKD.	DATE
DSGN.	EEC	3/16/73	BL	3/16/73
DWN.	Lemieux	3/16/73	BL	3/16/73
DIV.	GROUP LDR.	SECT. HEAD	DEPT. HEAD	DIV. HEAD
CIVIL	BL		RMO	JW
MECH.				
ELECT.				
PLAN.				
STAFF		BR. HEAD	CH. ENGR.	

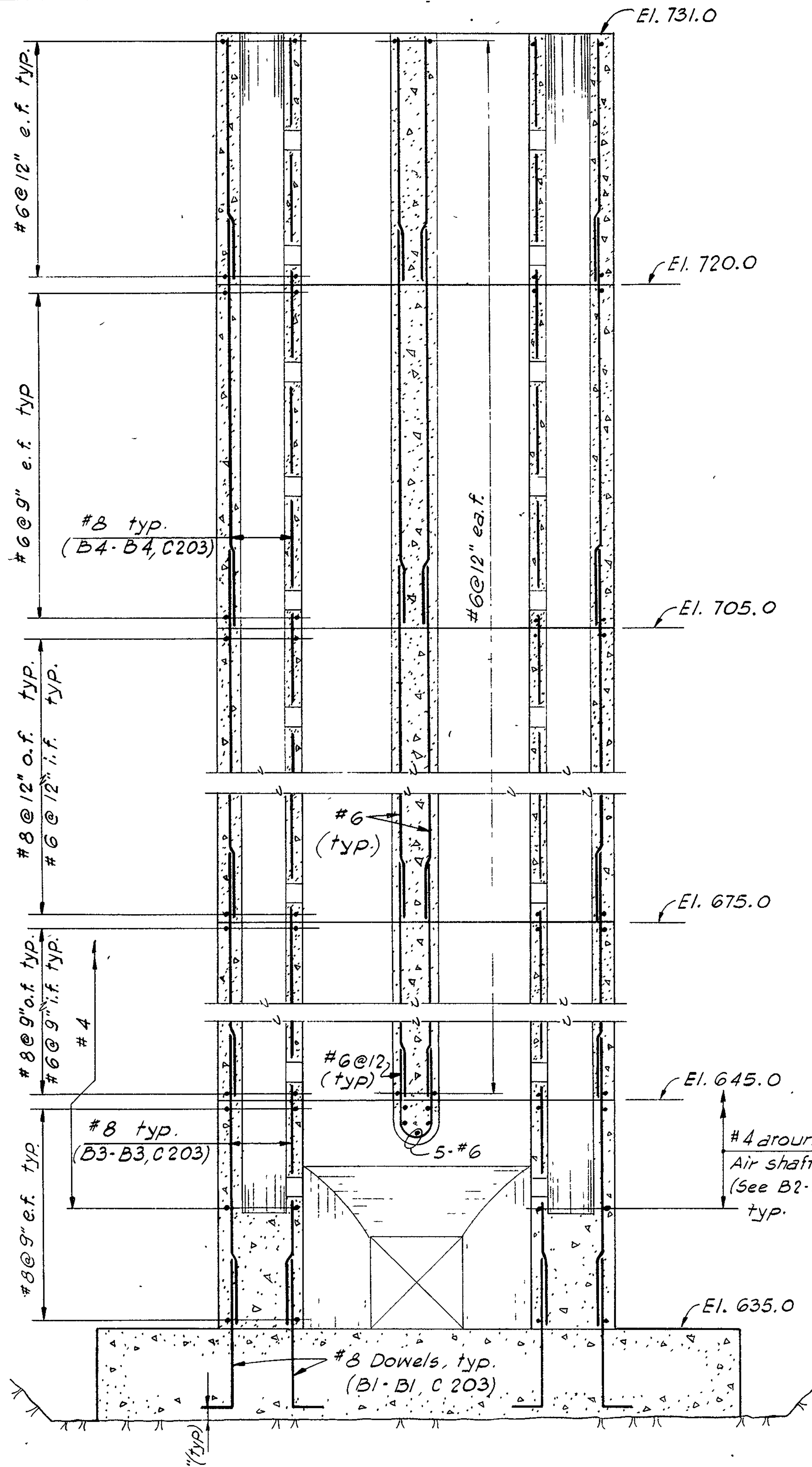
WCC FIG. 2



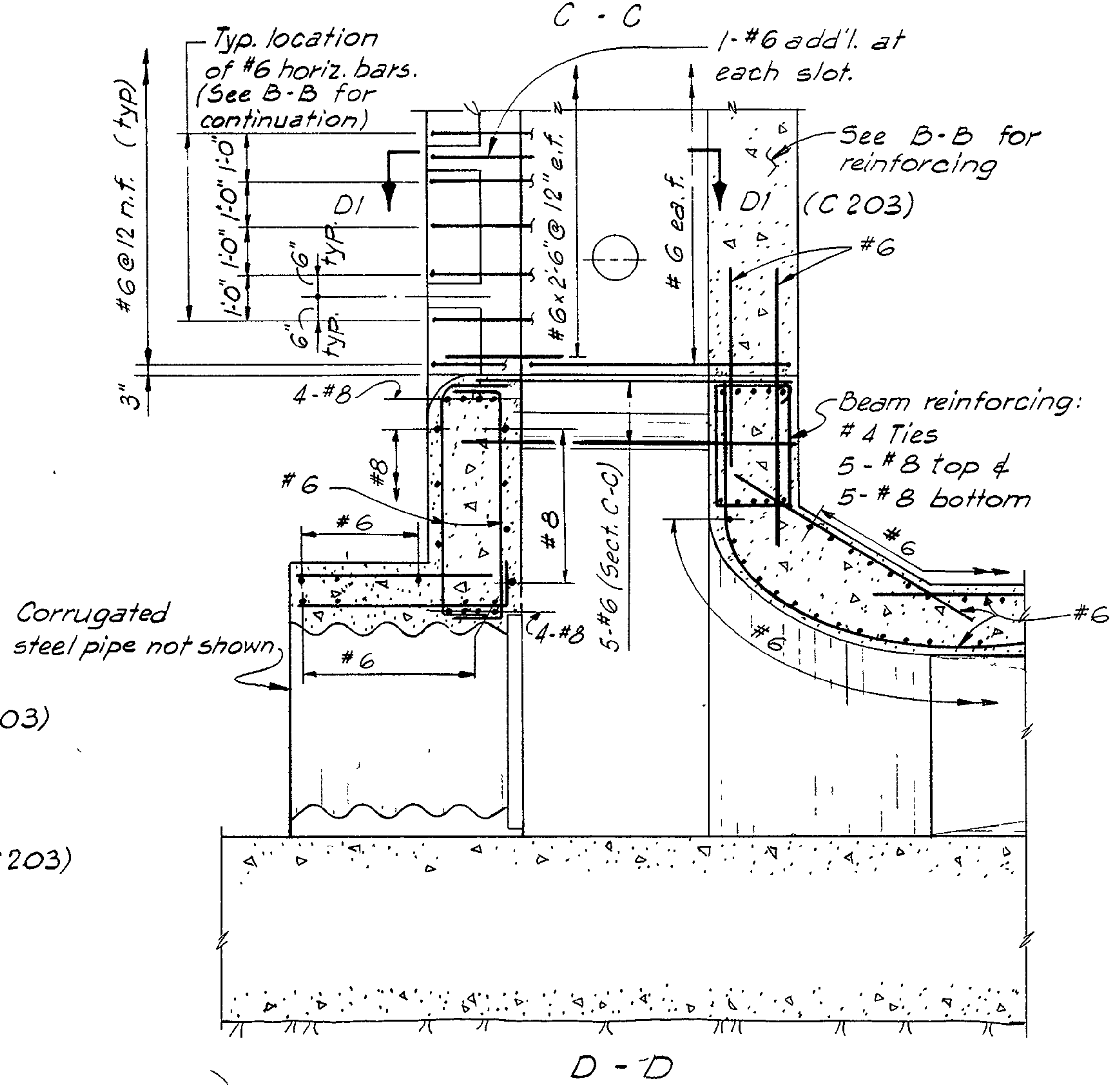
A - A



B - B

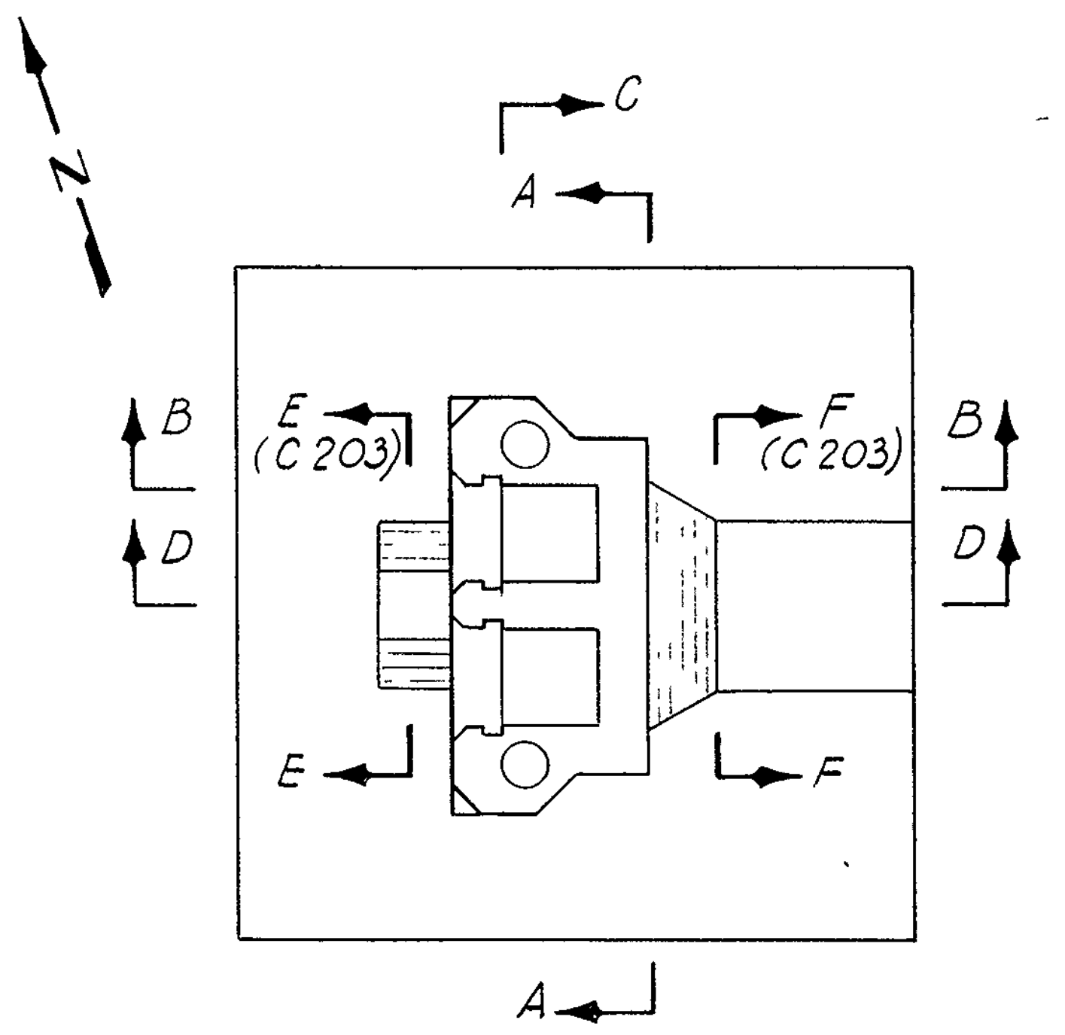


C - C



D - D

Scale 0 2 Feet
3/8" = 1'-0"



KEY PLAN

(N.T.S.)

ABBREVIATIONS:

i.f. = inside face
o.f. = outside face
e.f. = each face
t.f = top face
b.f. = bottom face
n.f. = near face
f.f. = far face
add'l = additional.

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

REFERENCE DRAWINGS:

Work this dwg. with 670 C 201, C 203, C 204

CONDITIONALLY
APPROVED
PERMIT NO. 7368
JUN 5 1973
Roy Winkle
CHIEF

NOTES:

1. All reinforcement placement shall conform to ACI 318-71 "Building and Detailing Code Requirements for Reinforced Concrete," unless otherwise noted.
2. Reinforcing bars shown thus ——⟩ indicate a continuation of these bars beyond the extent shown.
3. Lap length of smaller bar to be used where two different sizes of bars are lapped.
4. For general notes see C 201.

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

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	BY	DATE	CHKD.	DATE
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DWN.	P.F.	MARCH 1973	M.J	MARCH 1973
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OHIO ELECTRIC COMPANY
GAVIN POWER PROJECT

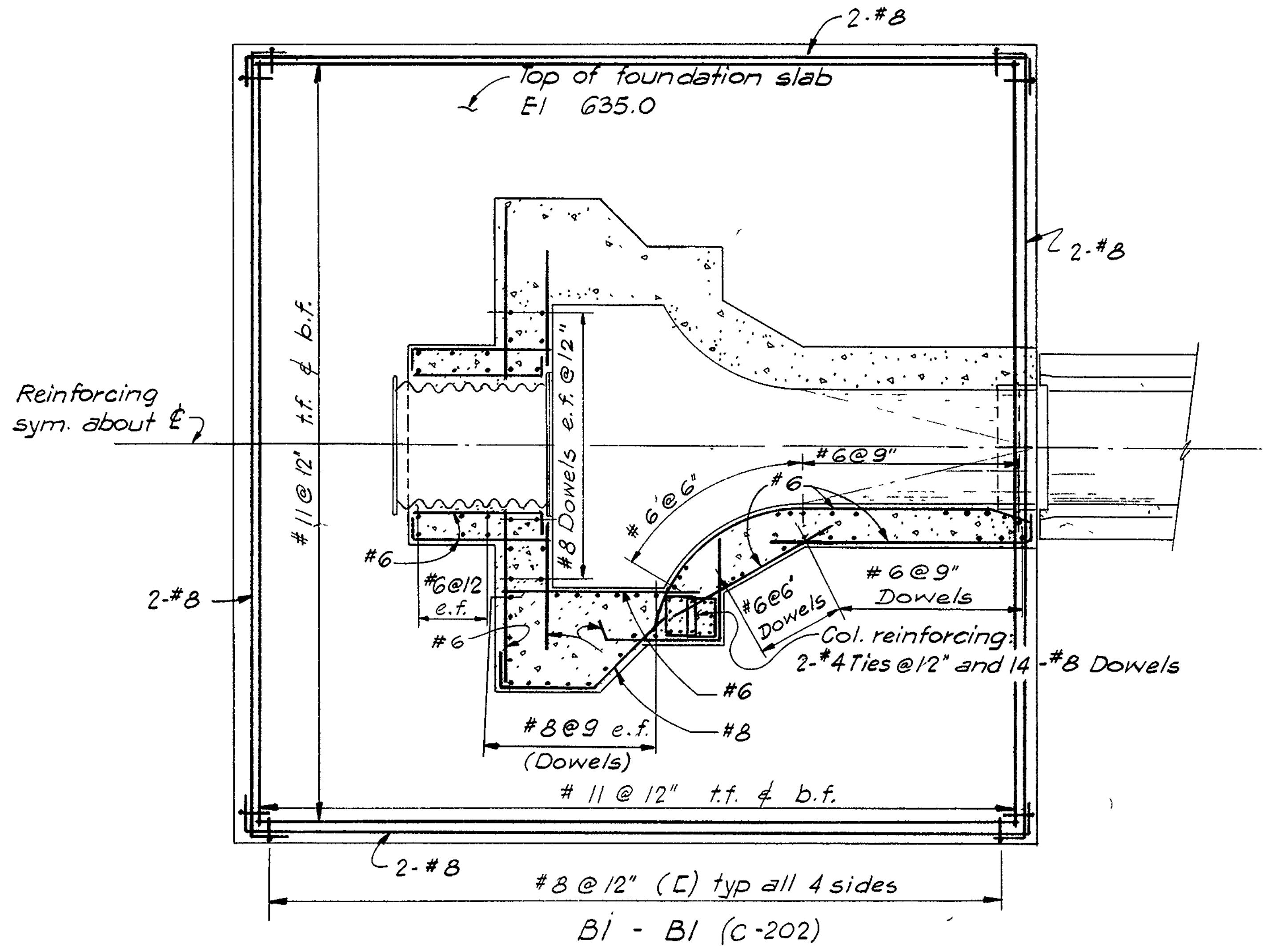
STINGY RUN
FLY ASH DAM

OUTLET WORKS

PRINCIPAL SPILLWAY REINFORCING DETAILS SHEET I

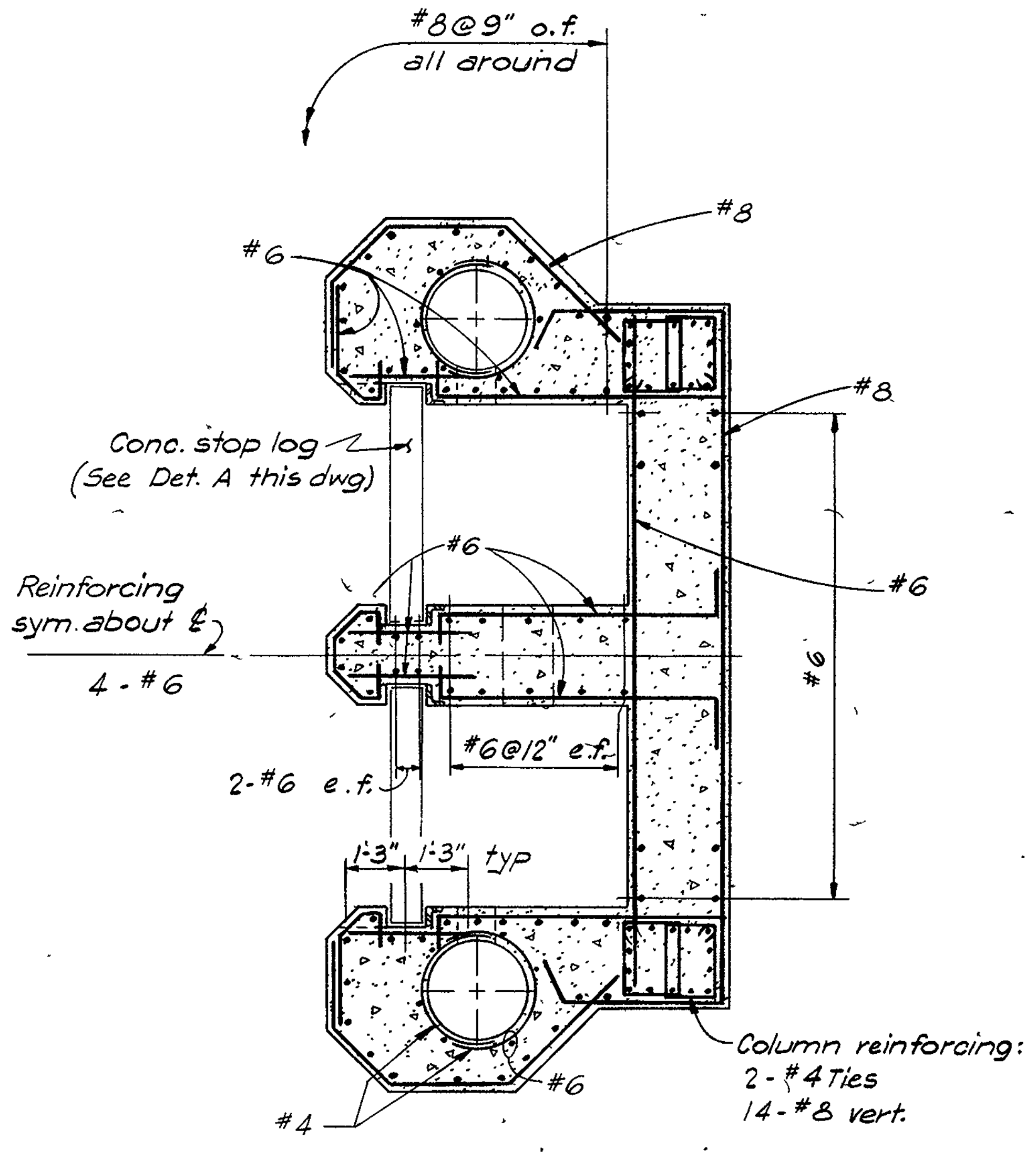
HARZA ENGINEERING COMPANY
APPROVED Richard D. Harza

CHICAGO, ILLINOIS
DATE MAR., 1973
DWG. NO. 670 C 202



B1 - B1 (C-202)

Scale, 0 4 Feet
1/4" = 1'-0"



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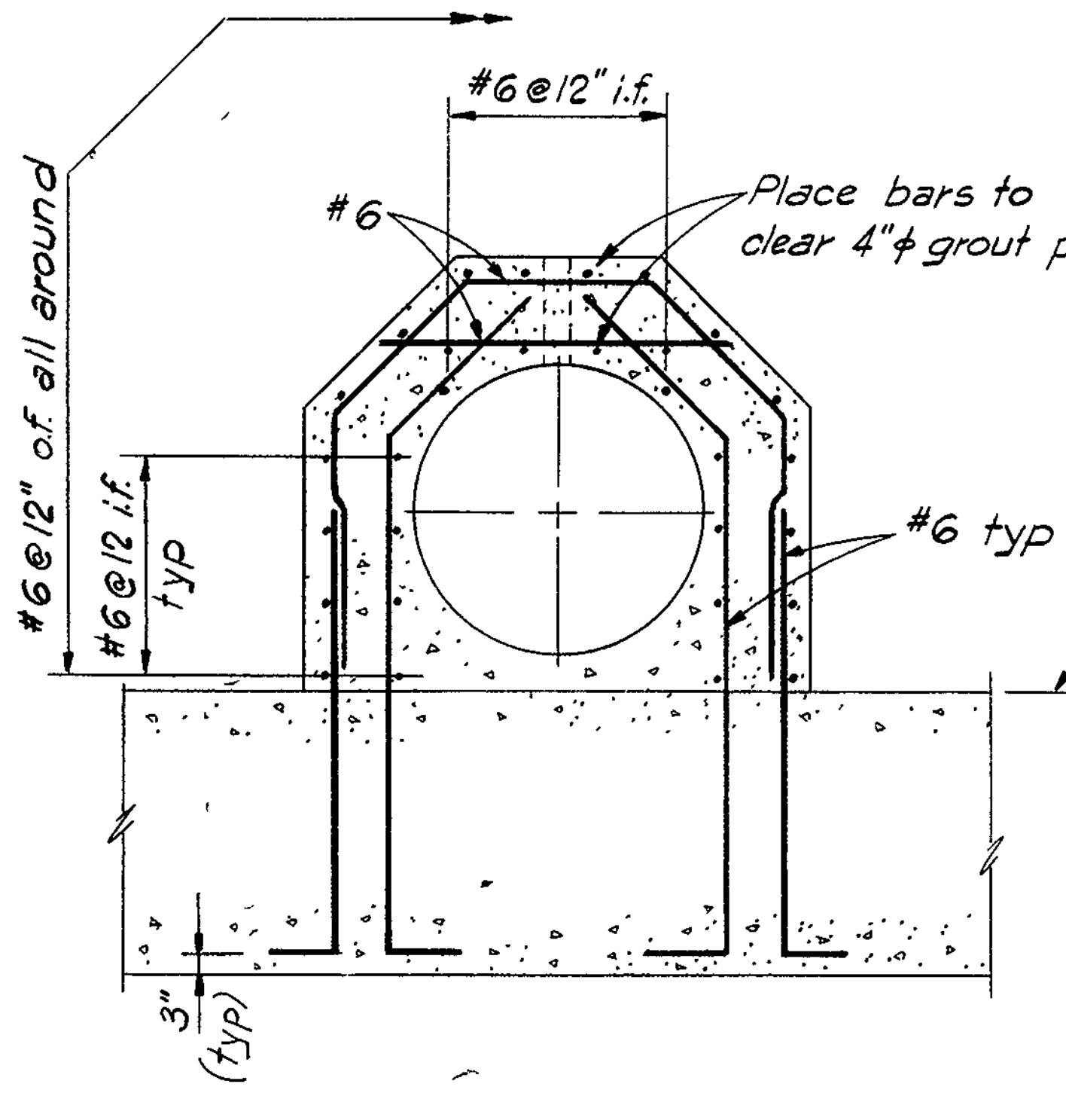
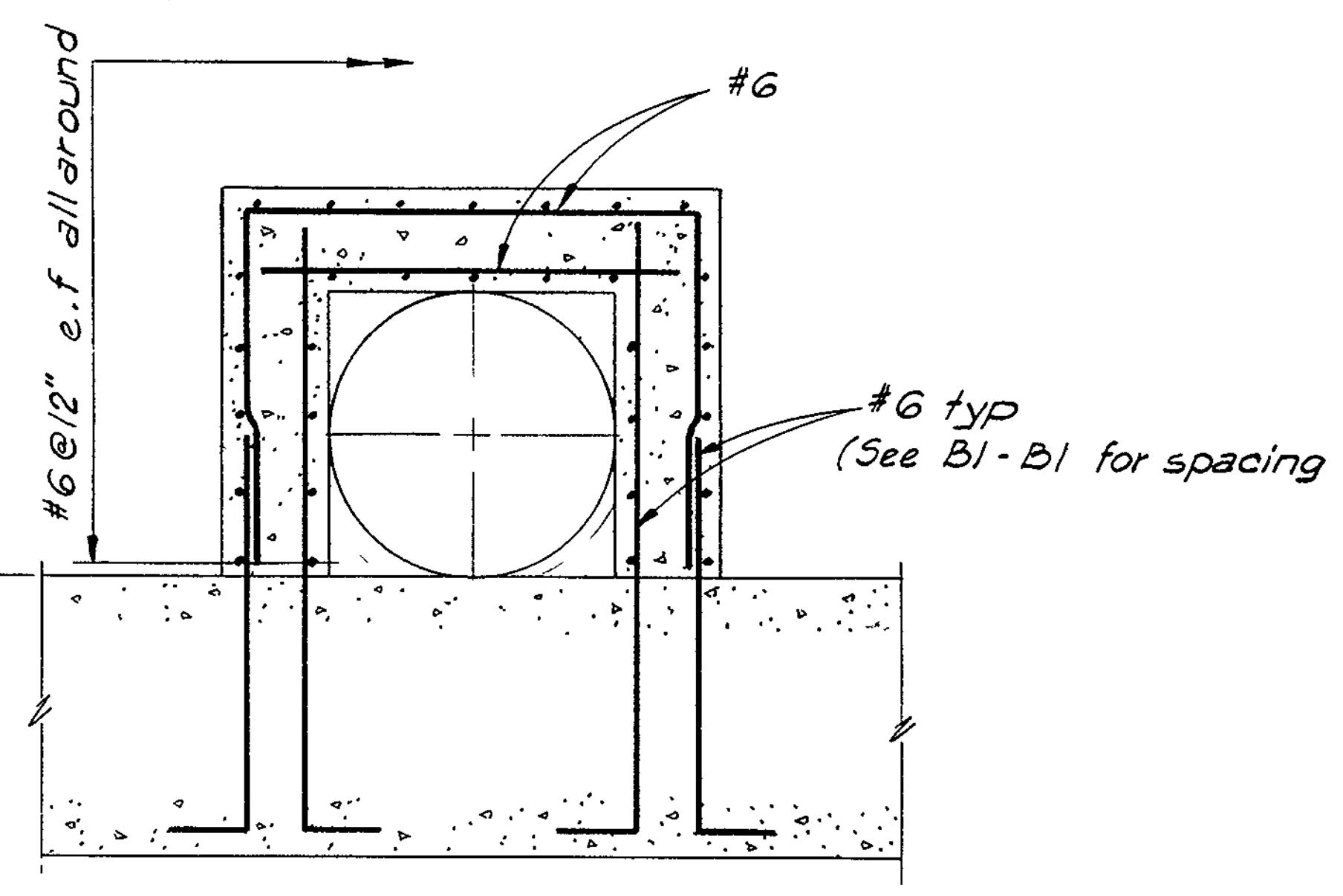
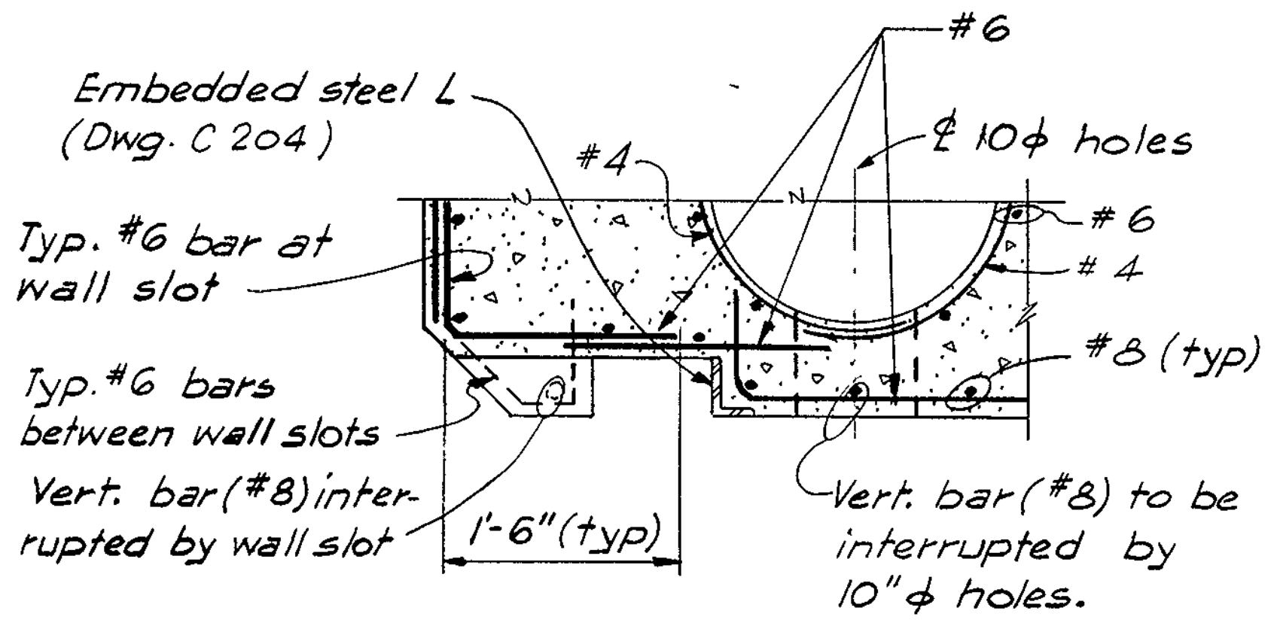


E - E (C-202)



F - F (C-202)



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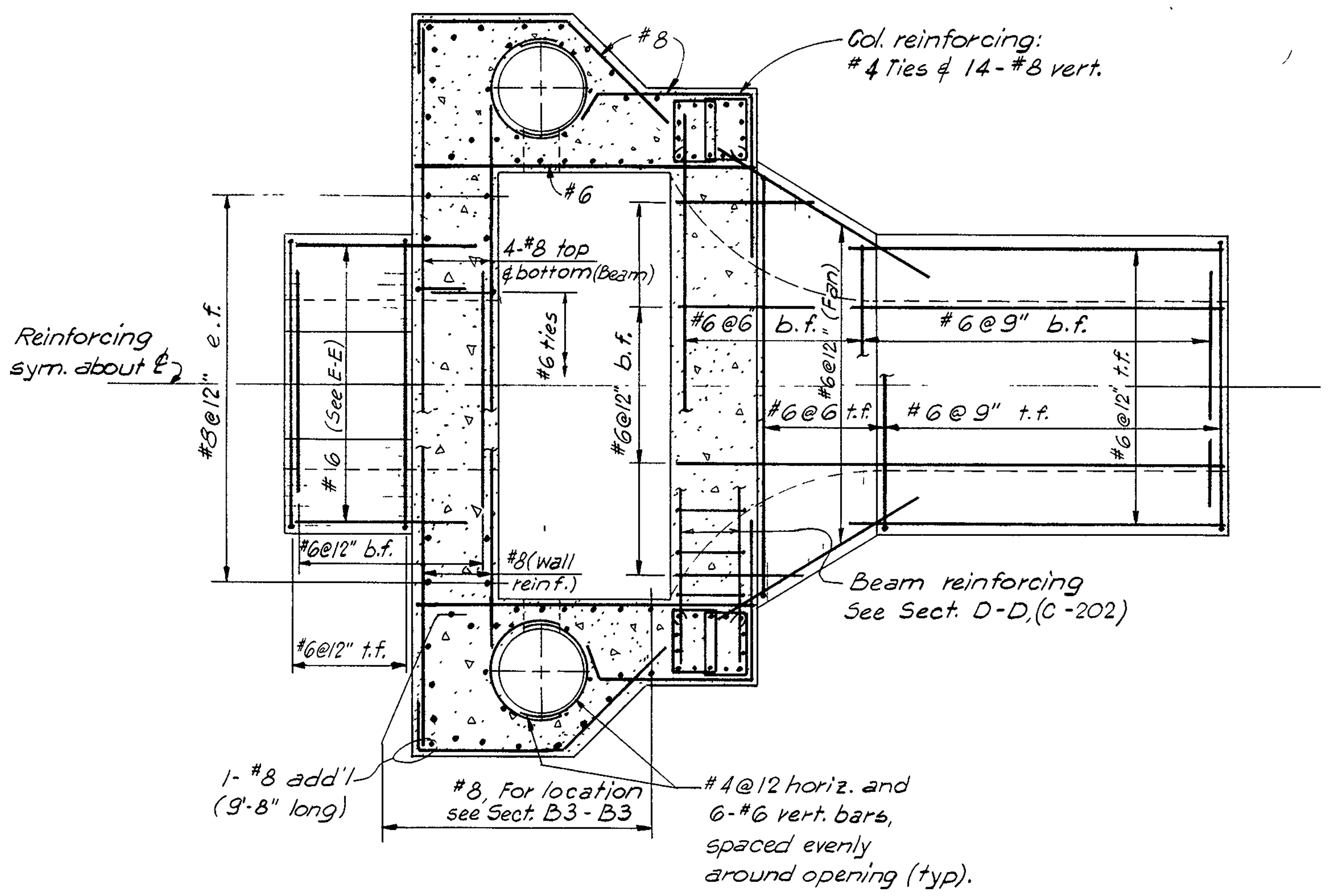

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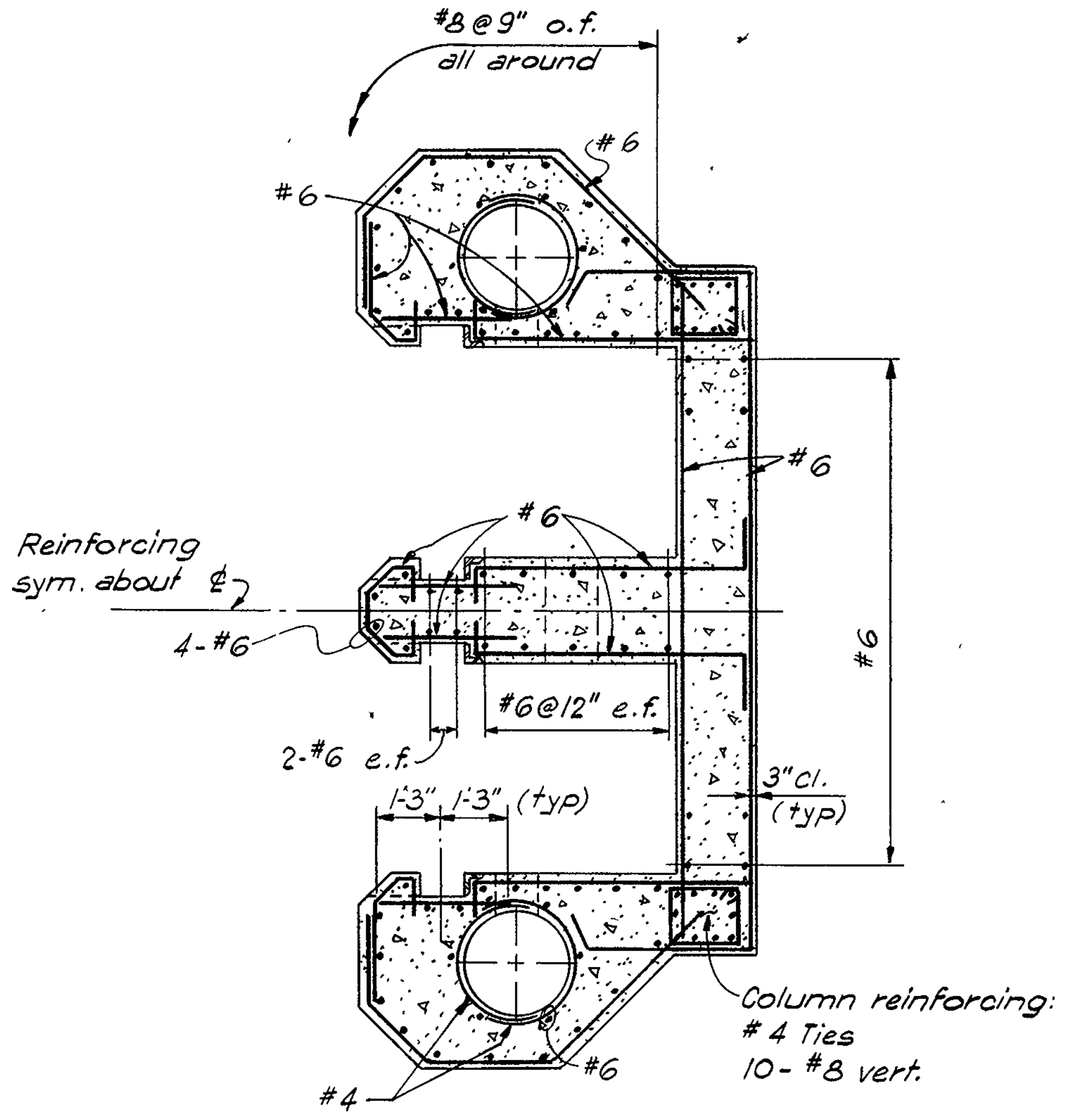

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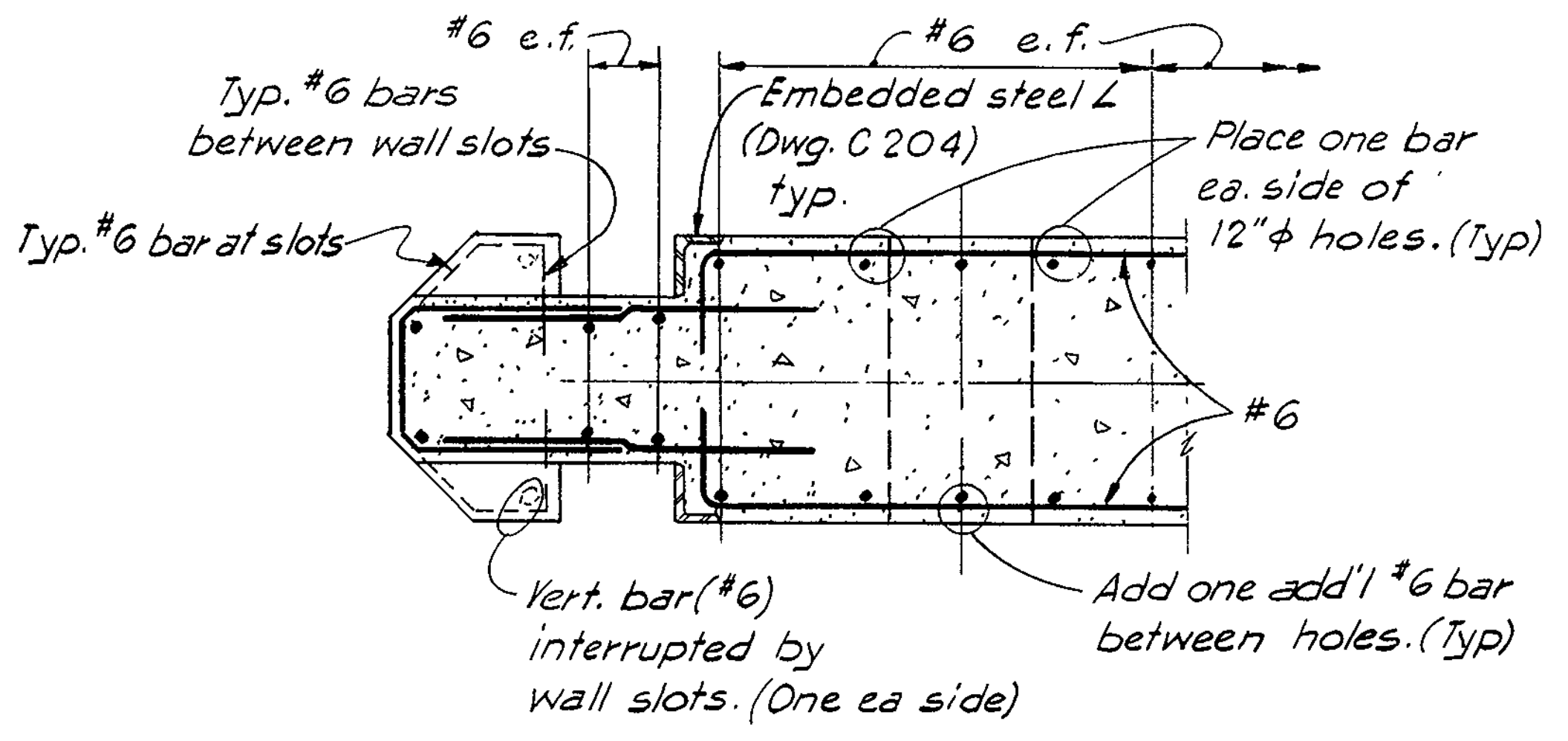

D1 - D1 (C-202)
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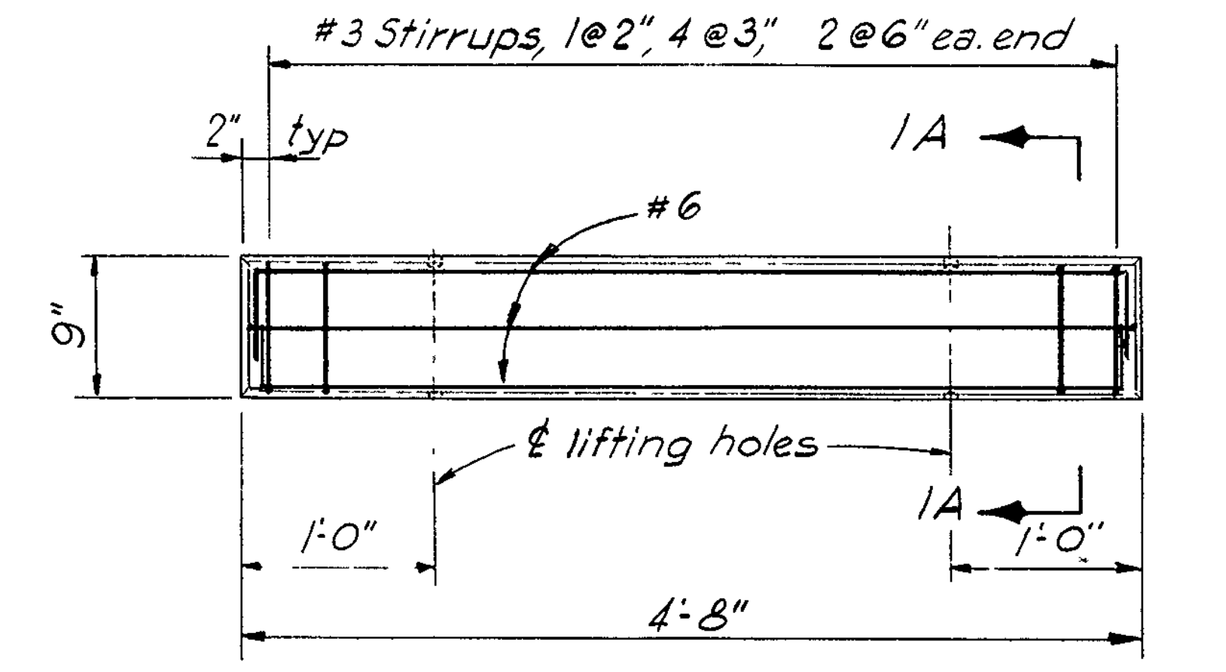
REFERENCE DRAWINGS:

Work this dwg with 670 C 201, C 202, C 204



DETAIL A
TYPICAL STOPLOG - 60 REQ'D - (FIRST STAGE)
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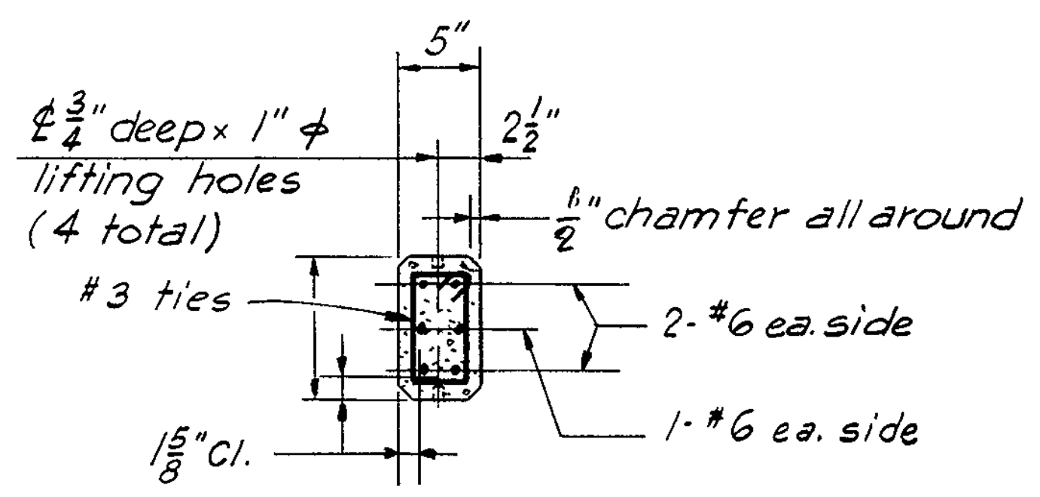
1" = 1'-0"



1A - 1A
1" = 1'-0"

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DIVISION OF WATER
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JUN 5 1973
Roy Winkle
CHIEF

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

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OHIO ELECTRIC COMPANY
GAVIN POWER PROJECT

STINGY RUN
FLY ASH DAM

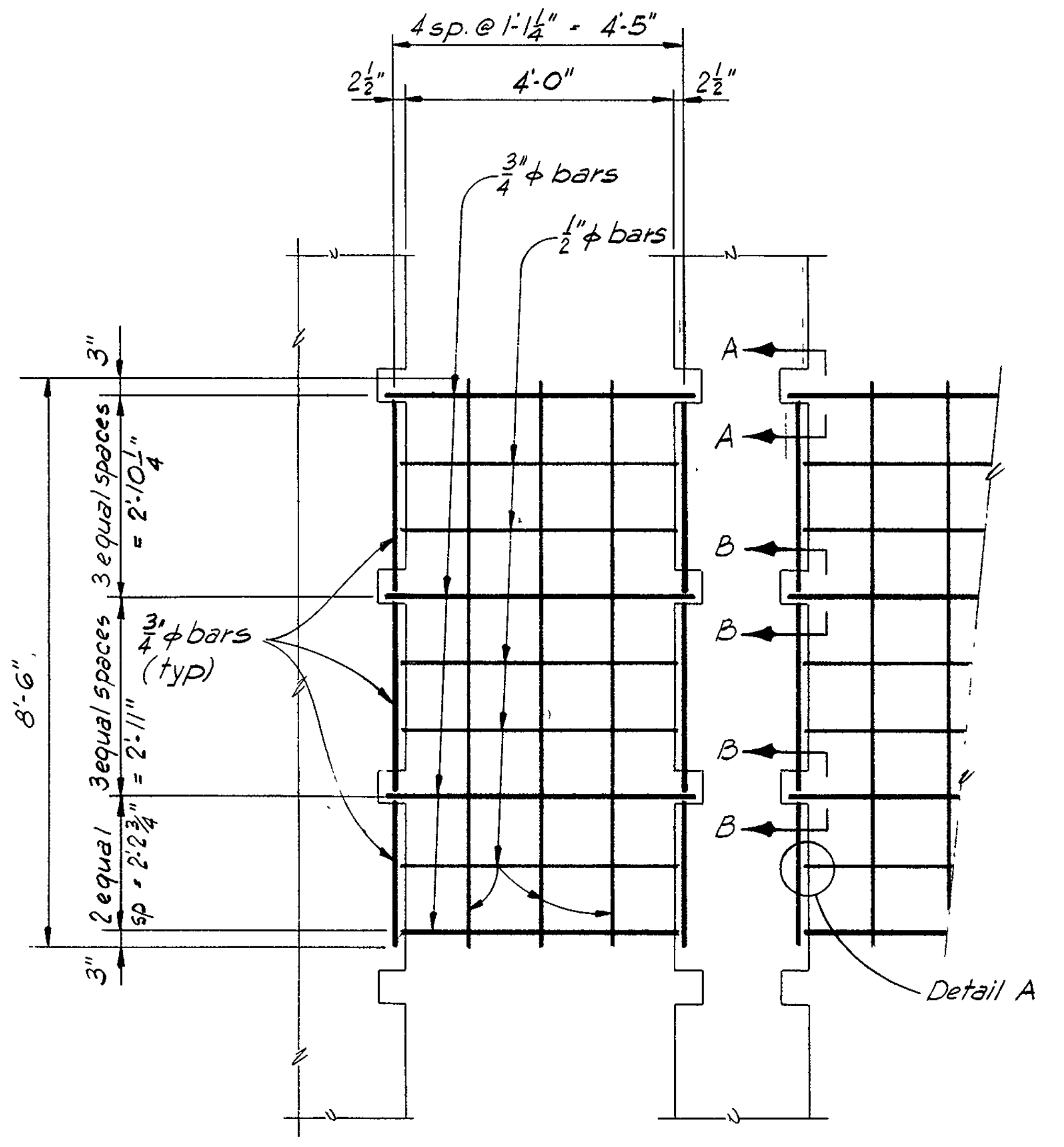
OUTLET WORKS

PRINCIPAL SPILLWAY REINFORCING DETAILS
SHEET 2

HARZA ENGINEERING COMPANY
APPROVED Richard D. Harza

CHICAGO, ILLINOIS
DATE MAR., 1973
DWG. NO. 670 C 203

K & E CO. 19-1253



ELEVATION OF TRASH RACKS
(2 REQUIRED)
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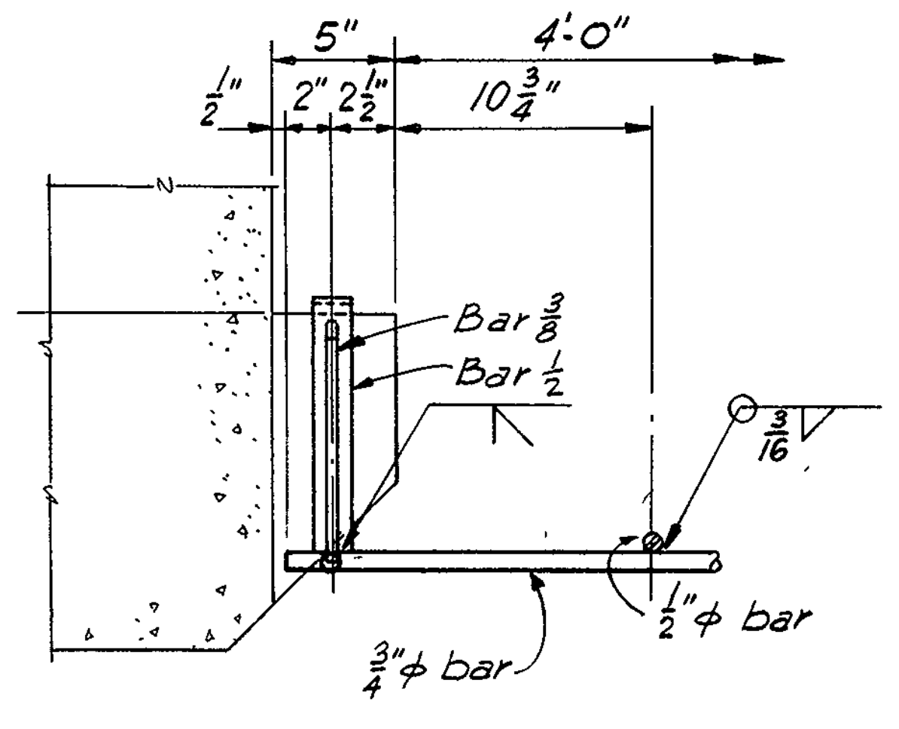
½" = 1'-0"



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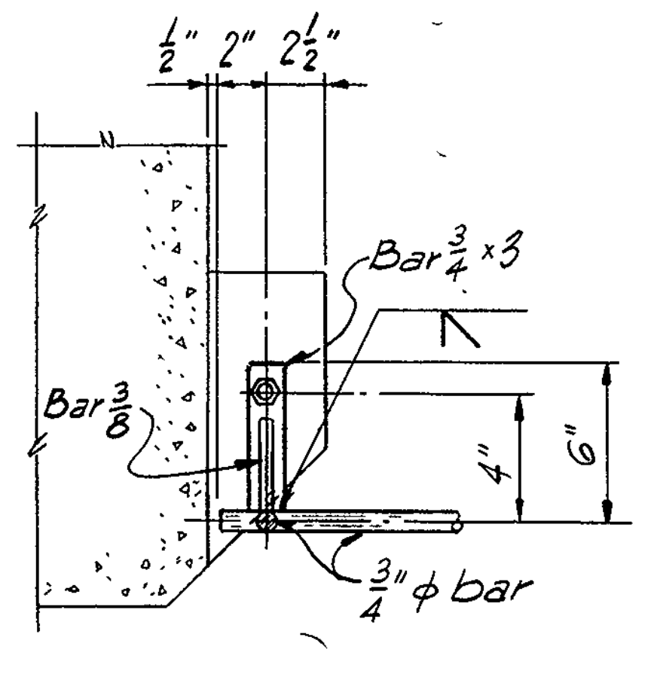

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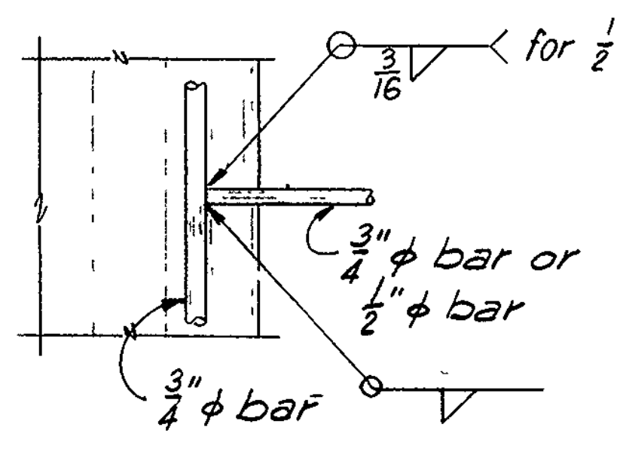

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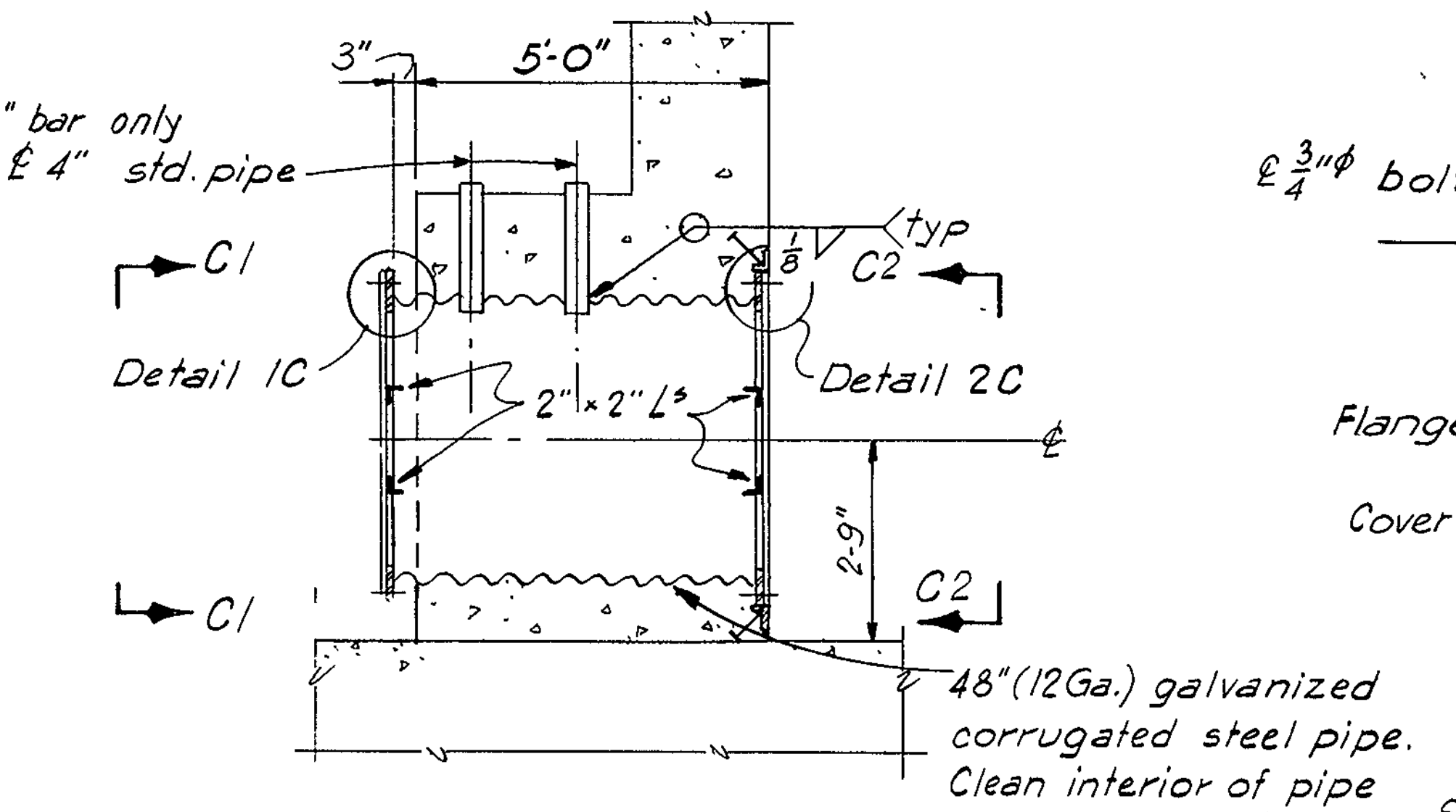
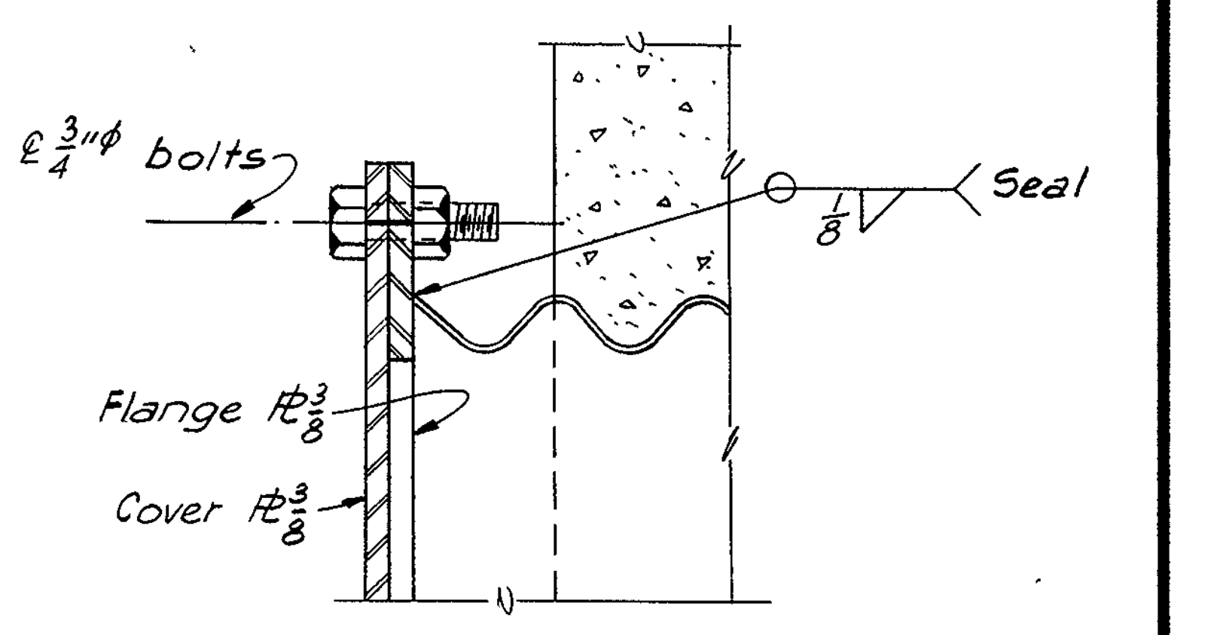


DETAIL C (C 201)
(DIVERSION CONDUIT)



DETAIL 1C
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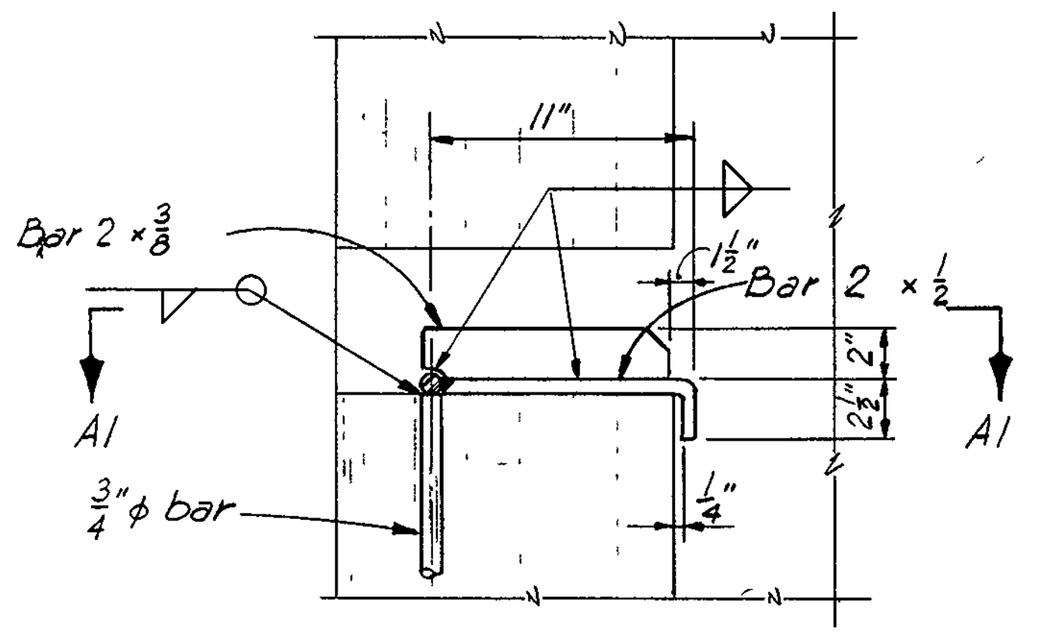

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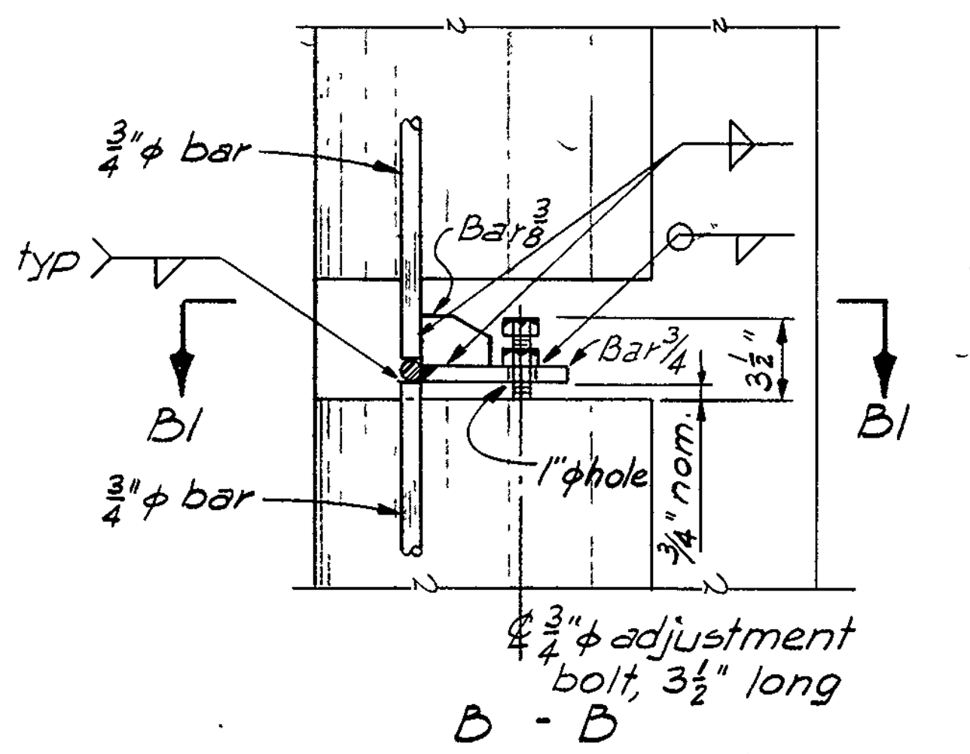

B - B
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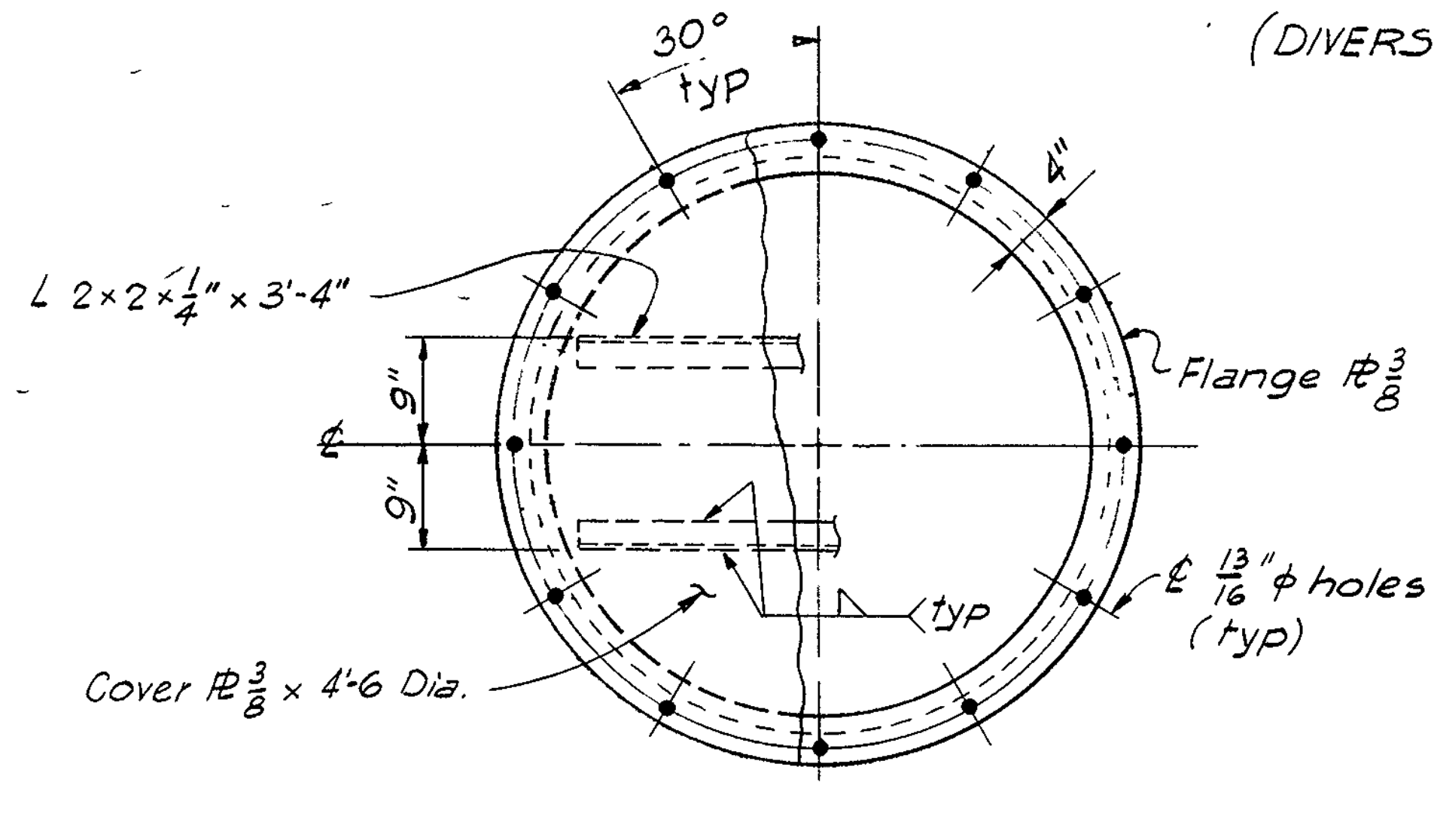
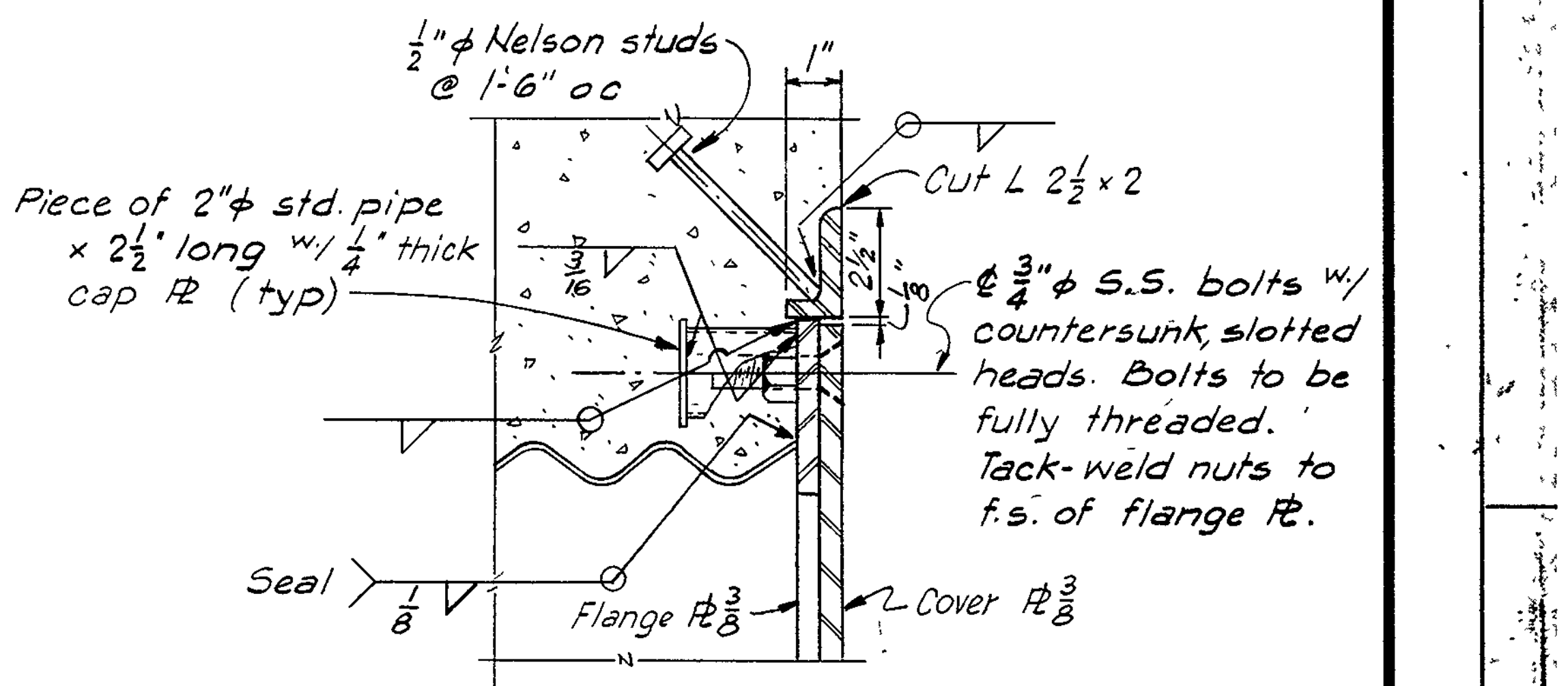
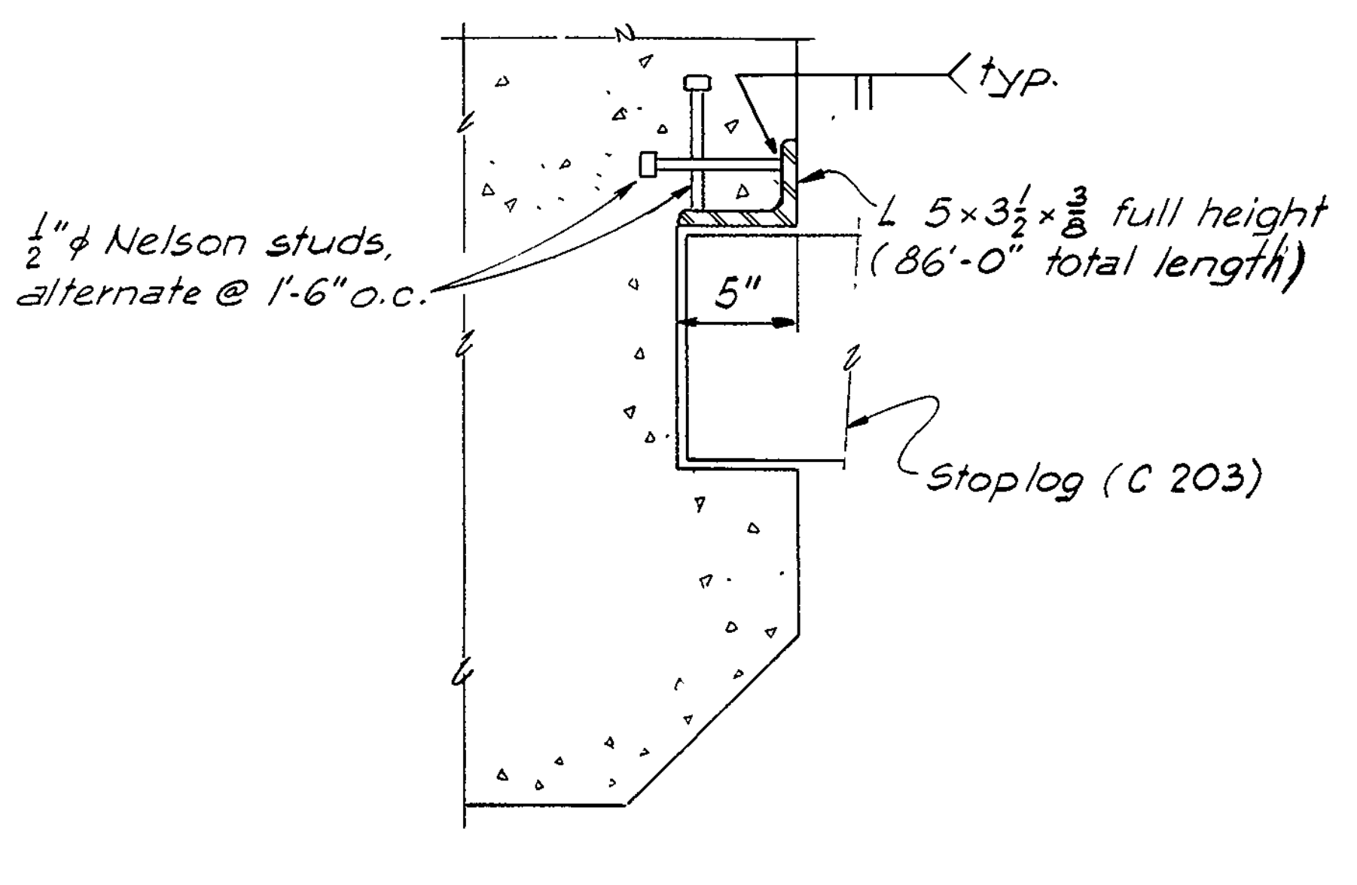


C1 - C1
Scale 0 2 Feet
¾" = 1'-0"



DETAIL 2C
3" = 1'-0"

R1



DETAIL D (C 201)
TYPICAL EMBEDDED L
(4 REQ'D)
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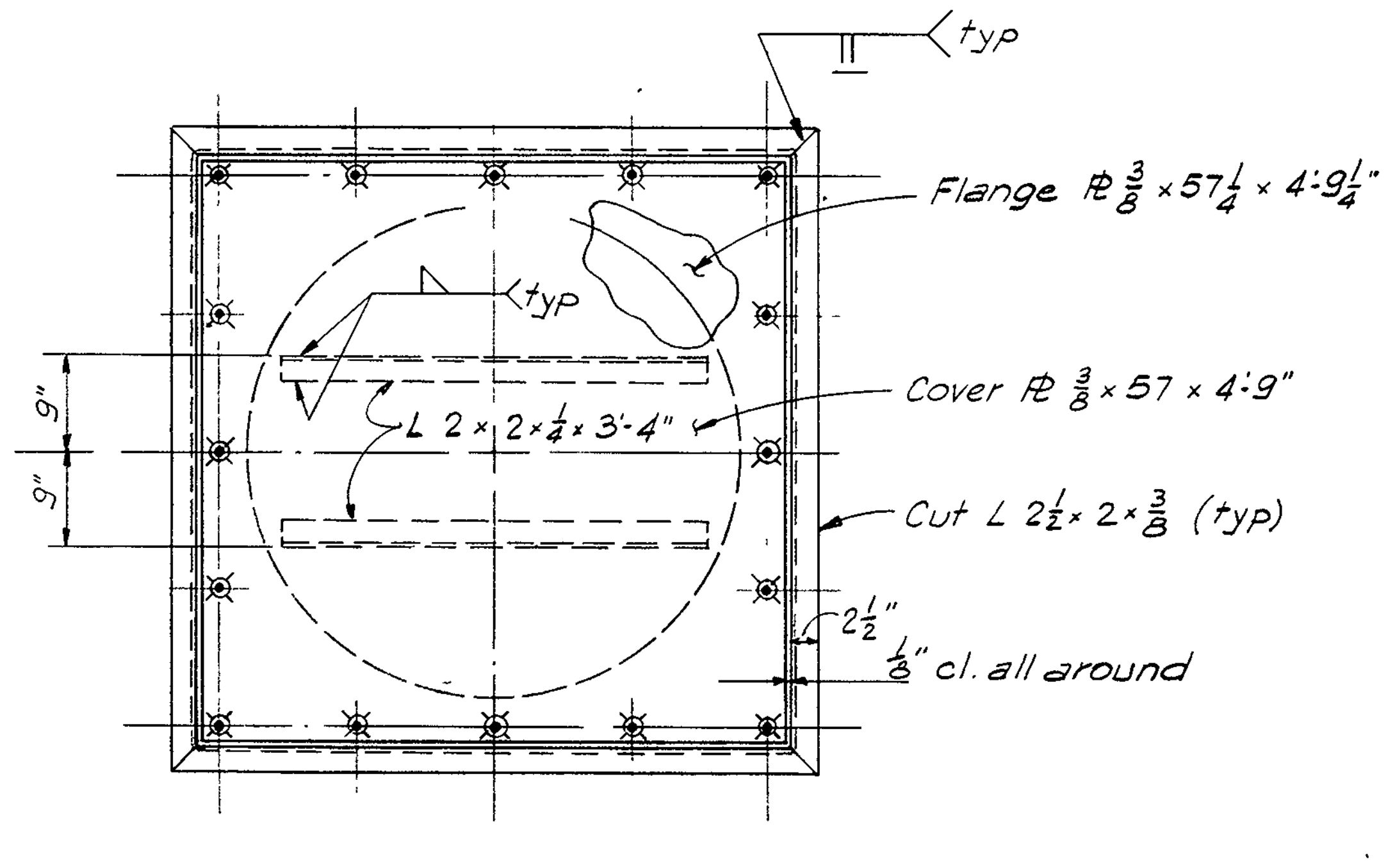

C2 - C2
¾" = 1'-0"

REFERECE DRAWINGS:
Work this dwg with 670 C 201, C 203.

NOTES:
1. For general notes see dwg. 670 C 201.
2. All material shall be galvanized structural steel ASTM - A36 except as noted.
3. All welds shall be continuous ¼" fillet welds except as noted.
4 For additional information see specifications.

Scale 0 3 6 Feet
⅜" = 1'-0"
Except as noted

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JUN 5 1973
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CHIEF

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PLAN.				
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R1	5-17-73	Ladder rungs deleted	P.F.		

OHIO ELECTRIC COMPANY
GAVIN POWER PROJECT
STINGY RUN FLY ASH DAM | OUTLET WORKS

PRINCIPAL SPILLWAY
STEEL DETAILS

HARZA ENGINEERING COMPANY
APPROVED Richard D. Harza
CHICAGO, ILLINOIS
DATE MAR., 1973
DWG. NO. 670 C 204R1

REVISED

K&E CO. 10-1253

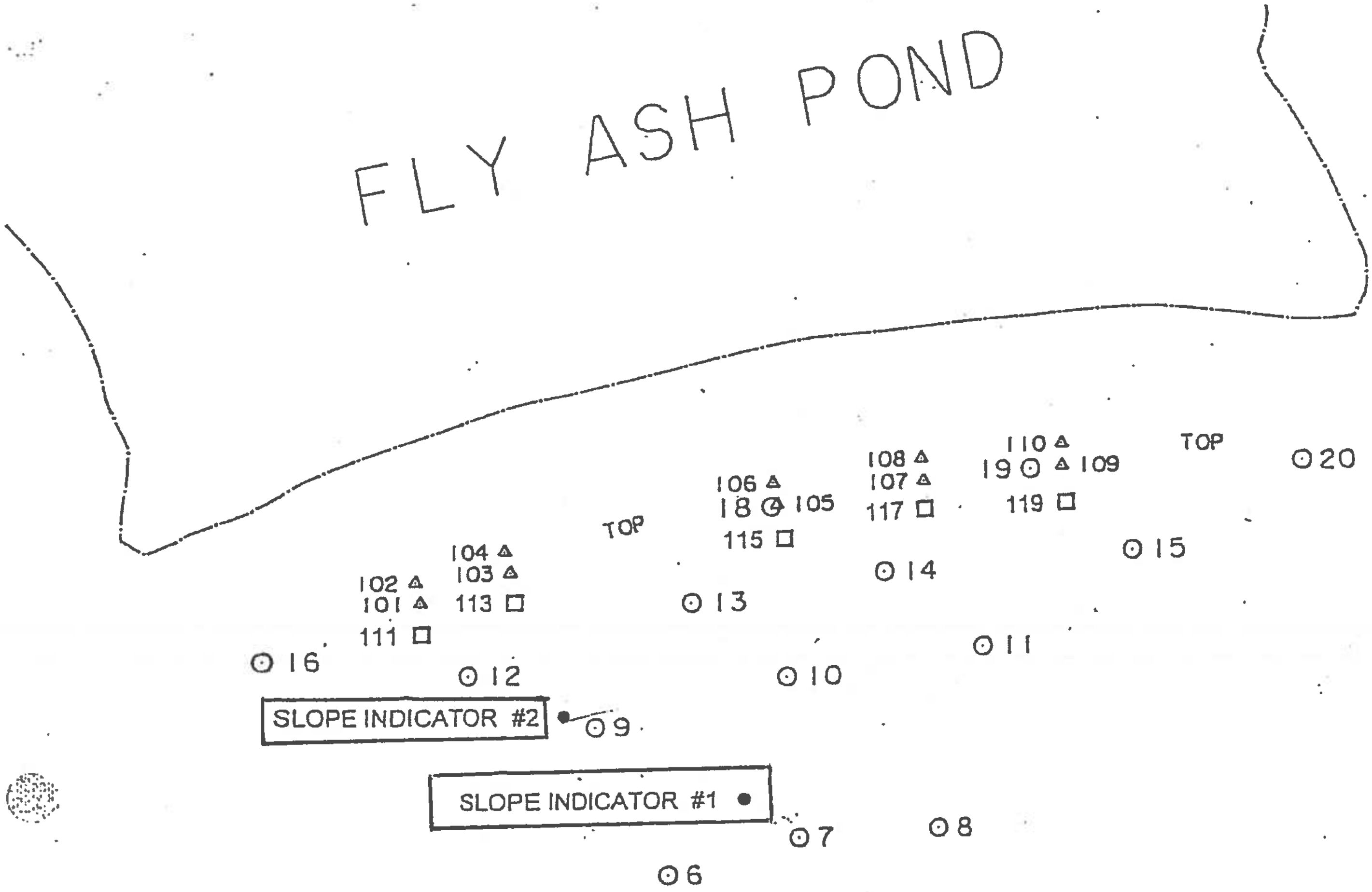
ATTACHMENT D

INSTRUMENTATION LOCATION MAP

Slope Inclinometers Location Map

Gen. James M. Gavin Plant

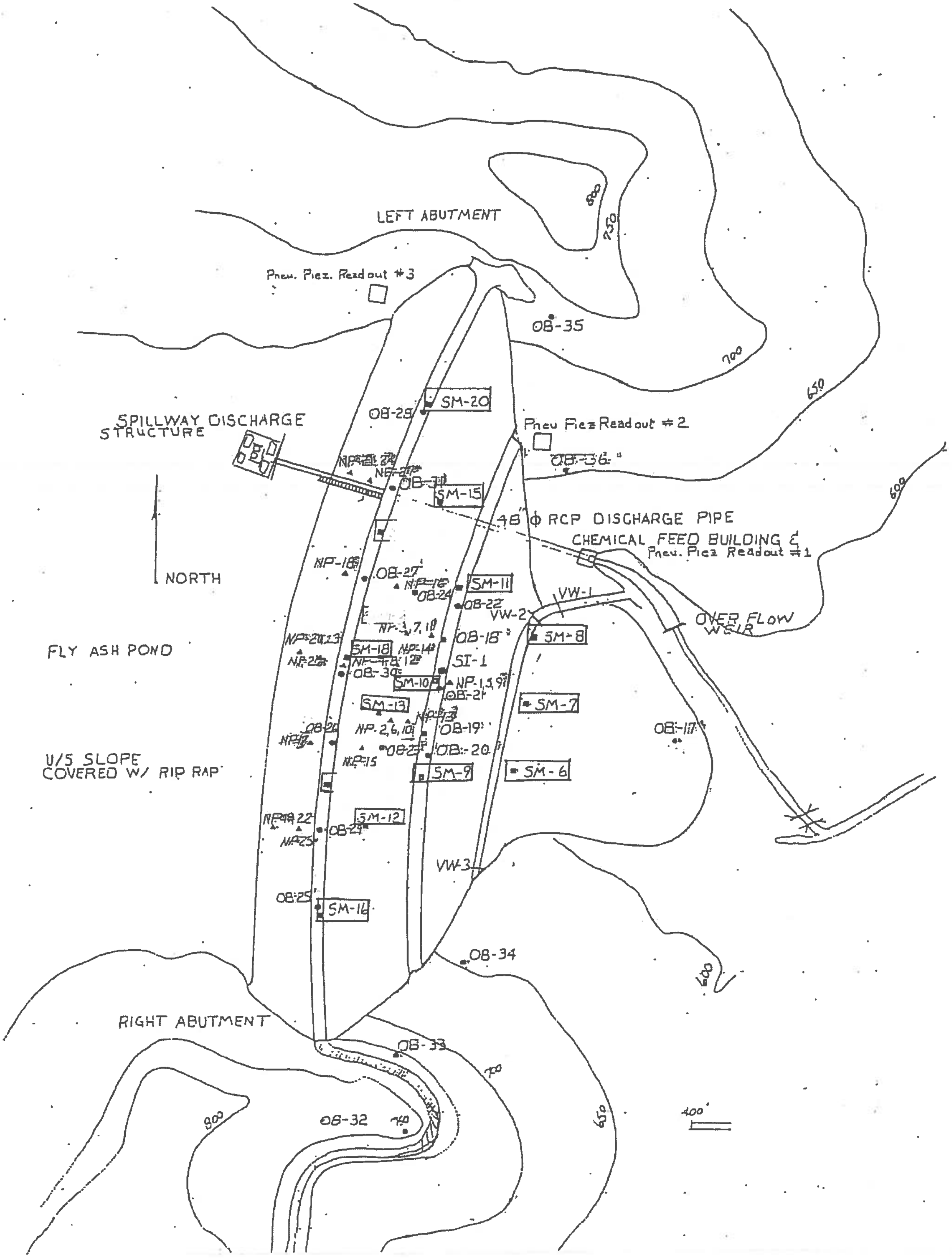
Stingy Run Flyash Dam



Deformation Monuments Location Map

Gen. James M. Gavin Plant

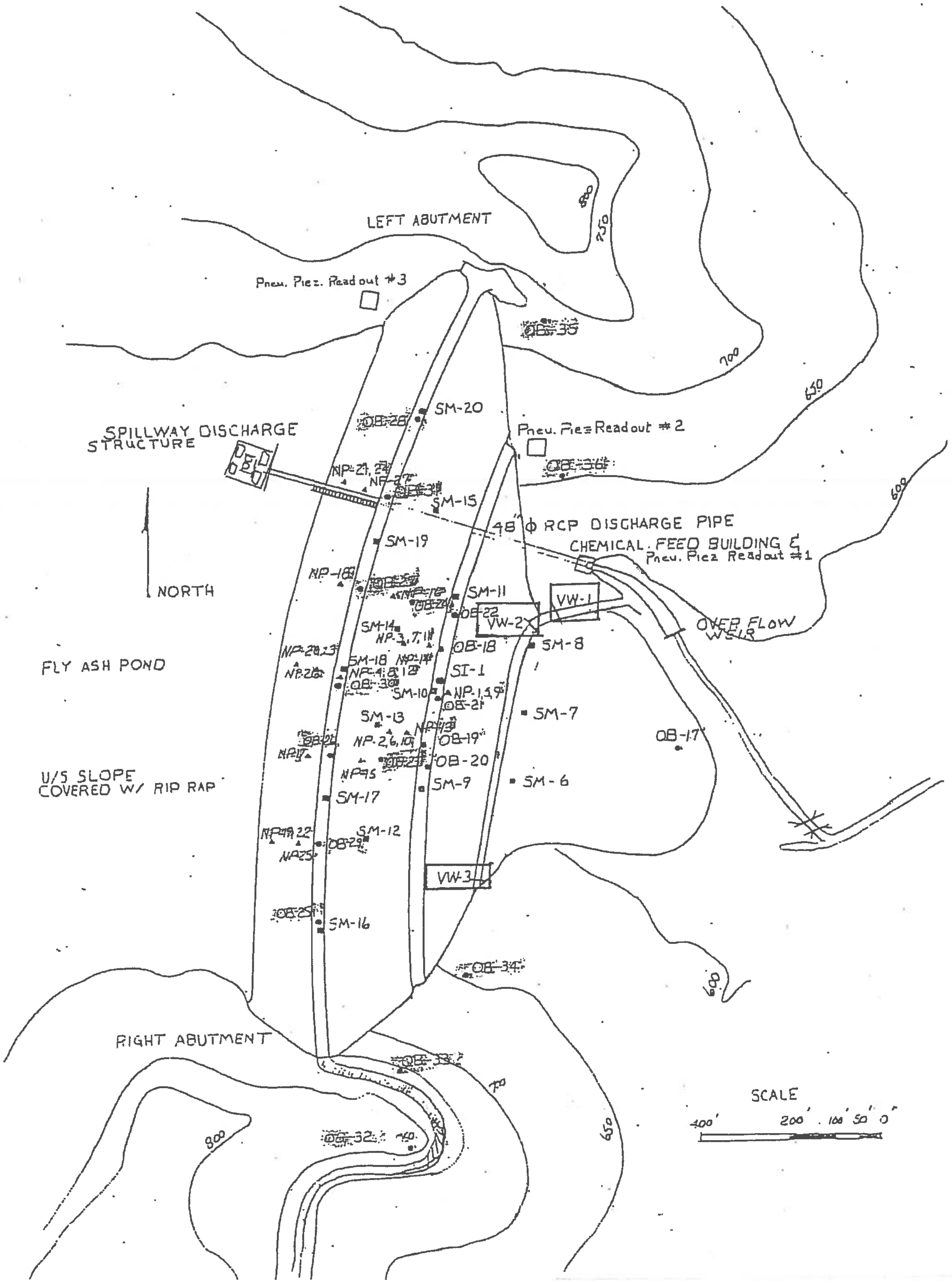
Stingy Run Flyash Dam



Weirs Location Map

Gen. James M. Gavin Plant

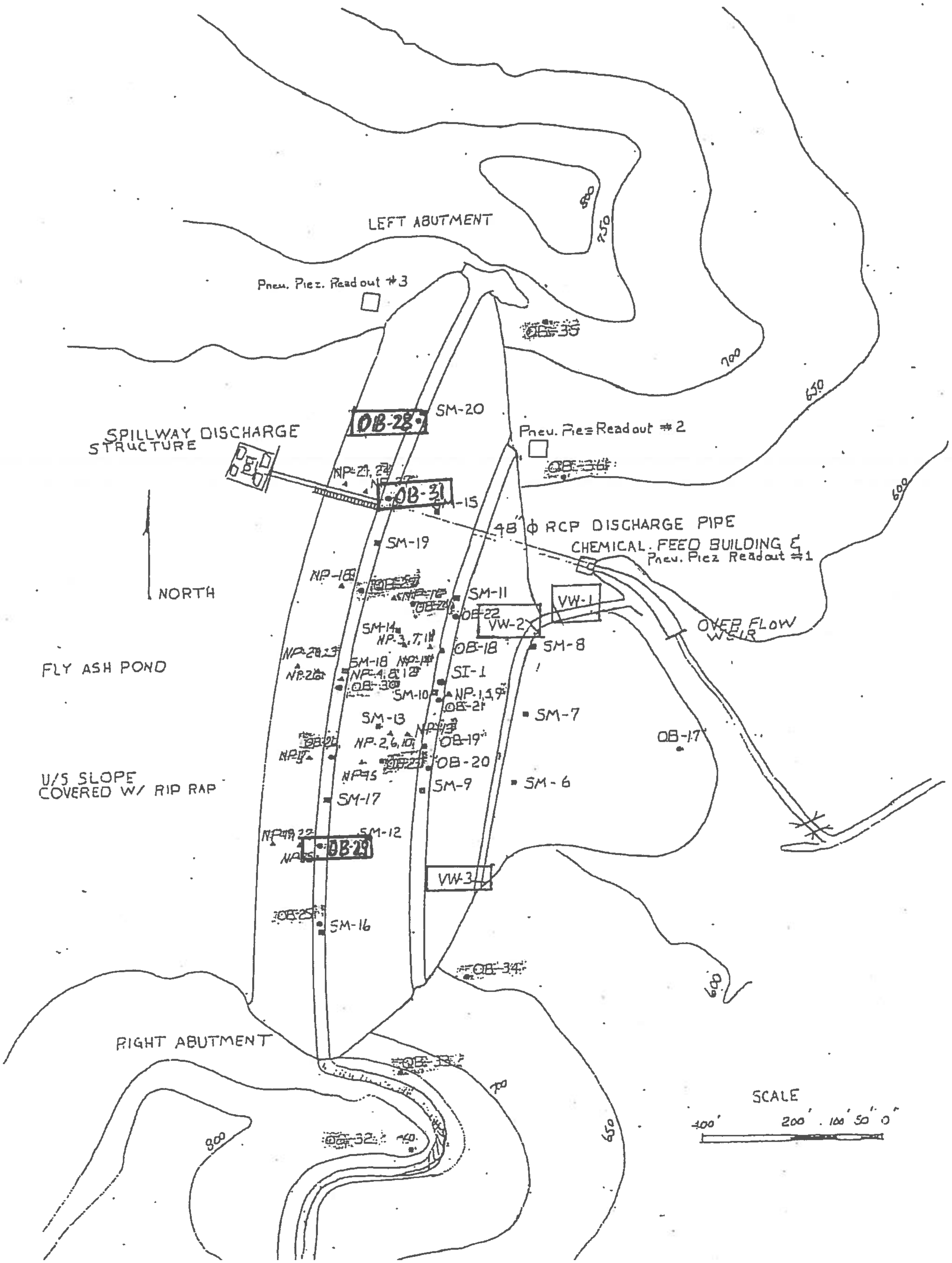
Stingy Run Flyash Dam



Observation Wells Location Map

Gen. James M. Gavin Plant

Stingy Run Flyash Dam



ATTACHMENT E

HYDROLOGY AND HYDROLOGIC REPORT

APPENDIX E

SPILLWAY SYSTEM DESIGN CALCULATIONS

APPENDIX E1: SPILLWAY/ENERGY DISSIPATER DESIGN FOR 100-YEAR EVENT

APPENDIX E2: SPILLWAY DESIGN FOR PROBABLE MAXIMUM FLOOD

APPENDIX E1

SPILLWAY/ENERGY DISSIPATER DESIGN FOR 100-YEAR EVENT

Written by: JNJ **Date:** 10/7/2015 Updated 12/28/2015

Reviewed by: RM **Date:** 11/6/2015

Client: AEP **Project:** FAR Closure **Project No.:** CHE8273 **Task No.:** 02/05.22

APPENDIX E1:

SPILLWAY/ENERGY DISSIPATOR DESIGN FOR THE 100 YEAR EVENT

PROJECT OVERVIEW

The Stingy Run Fly Ash Reservoir (FAR) is proposed to be closed by draining the reservoir, lowering the existing top of dam, and constructing a cover system. The Final Cover Stormwater Management Plan (Cover Plan), Drafted by Geosyntec in 2015, presented the design details of the final cover stormwater management system. The Cover Plan included a hydrologic analysis to determine peak flows at specific locations within the FAR and a hydraulic analysis of the proposed conveyance changes up to the proposed top of dam and spillway. The Cover Plan is included as part of the Dam Modification Report as Appendix D.

As presented in the Cover Plan, the three main conveyance channels on the proposed cover will converge and the combined flow will be conveyed within a single channel to the modified dam and proposed spillway. The proposed cover conveyance swales consist of a two stage channel design. The low flow channel has been designed to convey the 2 year – 24 hour storm event, which was selected due to the frequency of this potential flow. The second state was designed to safely convey the 100 year- 24 hour storm event to protect the adjacent cover system. The intent is to minimize flow across the cover cap by concentrating the flows within stabilized channels that will safely convey the 100 year design storm directly to the proposed spillway and over the modified dam.

The design for the cover system presented in the Cover Plan extends to the upstream end of the proposed spillway at the face of the dam. This calculation package details the conveyance across the dam crest and the system downstream of the spillway. Per the hydrologic calculations shown in the Cover Plan, flows in the spillway during the 100 year – 24 hour storm event will reach approximately 1,918 cfs. Conveying these flows from the top of the spillway to Stingy Creek in a safe and efficient manner will require significant energy dissipation.

This calculation package describes the design of the conveyance system (spillway over dam, energy dissipator at the toe of the dam, and reinforced channel downstream of the energy dissipator) for the 100-year flood event. Subsequent to the development of this design, Ohio DNR indicated that the dam spillway component of the system should also be design for the Probable Maximum Flood (PMF). Other parts of the conveyance system described in this chapter (energy dissipator and reinforced channel downstream of the energy dissipator) were sized for the 100-year event. The PMF spillway was developed by designing an additional overbank spillway centered on the 100-year concrete spillway described in this section. Appendix E2 describes the hydrologic/hydraulic analysis and design modifications for the PMF spillway.

DESIGN METHODLOGY

The spillway design is based on guidance provided under USBR's Design of Small Dams manual (USBR 1987). The intent of the FAD spillway is to safely pass the 100 year – 24 hour peak event down the face of the dam to Stingy Run without resulting in permanent ponding of surface water on the FAR cover after dam closure. Therefore the spillway design will consist of an open channel chute without use of a control structure to allow for free-flow discharge. Geometry of the spillway system was selected based on physical, topographical and hydraulic constraints within the system. Initial channel widths, side slopes, channel bed slopes and bed material where analyzed using Manning's equation through an iterative process and were refined through hydraulic modeling and input from AEP and ODNR. The spillway inlet elevation and profile was set to match the conveyance system of the final FAR cover to allow for a smooth transition over the dam crest.

Supercritical flow conditions are anticipated to occur along the proposed spillway due to peak flows reaching nearly 2,000 cfs and a spillway slope of 13%. Energy dissipation measures are necessary at the base of the spillway to protect the downstream channel and surrounding area from the potential significant erosion resulting from high flow velocities exiting the spillway. The effectiveness of the dissipator is dependent on the tailwater conditions. Therefore, the design of both an efficient energy dissipating system and the downstream stream channel were evaluated as one complete system within the model. The spillway length extends until the downstream slope of the channel at the exit of the dissipator, which provides adequate tailwater under subcritical conditions in order to provide system stability.

The energy dissipation design was carried out in accordance with guidance from the U.S. Bureau of Reclamation (USBR, 1987) and the U.S. Army Corp of Engineers (USACE, 1990). Based on the spillway flow having a calculated velocity of 38.4 ft/sec and a Froude number of 7.04 immediately upstream of the energy dissipator, a stilling basin was designed in accordance with the Type III Basin Characteristics recommended by the USBR guidance for Small Dams (USBR 1987). See attachment B for a schematic of this basin design. This type of energy dissipator is known as a stilling basin. Dimensions and design parameters from the final design are described in details in the Hydraulic Analysis Results section below.

The downstream channel design was first carried out based on topographical conditions and necessary hydraulic capacity. The downstream channel slope is approximately 0.5%. A trapezoidal channel design with 3H:1V side slopes was assumed. Scour protection was analyzed in accordance with U.S. Department of Transportation Highway Administration's (FHA) guidance set forth in Hydraulic Engineering Circular No. 11 (FHA, 1989), as well as a more recent article published by Craig Fischenich at the Ecosystem Management & Restoration Research Program (Fischenich, 2001). Both of these specifications were in agreement on suggesting channel stability measures based on flow velocities and scour potential. Details of this design can be found in the Hydraulic Analysis Results below. See Attachment C for excerpts from the Hydraulic Engineering Circular No. 11 paper that was used for the selection of rip rap armament.

HYDRAULIC METHOD OF ANALYSIS

Due to the need to analyze the spillway transition, spillway chute, stilling basin and reinforced channel as a single system, as well as the many dependent variables involved in the analysis, computer modeling using advanced software was the method of choice to performing the primary hydraulic analysis. This analysis was completed using the computer program U.S. Army Corps of Engineers (USACE) Hydrologic Engineering Center-River Analysis System V4.1.0 (HEC-RAS). HEC-RAS allows for the analysis of one-dimensional steady flows through complex open channel systems and is capable of analyzing mixed flow regimes as well as hydraulic conditions where the water surface profile varies rapidly, such as during a hydraulic jump. These capabilities provided an efficient method for performing the analysis of the spillway, energy dissipator and open channel design parameters. Output from HEC-RAS includes detailed analytical parameters such as energy grade changes, flow velocities, water depths, and shear stresses that will occur within the various components of the spillway. The model also predicts whether the flow is sub-critical or super-critical within the system, which is important for understanding hydraulic stability at various locations within the system. All of these parameters are essential to the efficient design of the conveyance system downstream of the spillway.

HYDRAULIC ANALYTICAL PARAMETERS

The hydrologic calculations used to develop the design flow in this package are detailed in the Cover Plan (Geosyntec Consultants, 2015). The peak 100 year – 24 hour event flow was determined as 1,918 cfs. This design flow was used to evaluate both the proposed conveyance system upstream of the dam as well as the spillway system and was conservatively rounded to 2,000 cfs. The 2 year – 24 hour peak event, estimated in the Cover Plan as 425 cfs, was also simulated in order to analyzed the system performance under more typical hydrologic conditions, and to analyze the design performance of a downstream culvert at Stingy Creek Road.

The spillway extends from the upstream (west) edge of the crest of the dam to the downstream (east) toe of the dam, where the energy dissipator begins. The length of this reach is approximately 750 feet, including a dam crest width of approximately 50 feet and a spillway length along the downstream face of 700 feet. Spillway invert elevations range from 661 at the upper crest of the dam to 586.5 at the end of the spillway/beginning of the disspator. The longitudinal bottom slope of the spillway is approximately 1.5% for the first 200 feet of the spillway (through the crest of the dam and at the top area of the dam face). The first 200 feet of the spillway was designed to aid in the transition of the Type 1 FAR conveyance channel geometry (see Appendix D) to the rectangular spillway chute geometry described below.

A significant convergence length was required to transition the FAR and FAD components due to the differences in channel geometry. The channel transitions from a 12 foot wide tiered-trapezoidal channel at the FAR cover to a 55 foot wide rectangular channel at the crest of the spillway. USBR guidance recommends that this transition be made gradually to reduce the potential formation of standing waves or flow run-up on the spillway walls. Equation 21 from Chapter 9 of the USBR guidance manual(USBR 1987) was used to estimate the acceptable flare angle between the cover channel and the spillway channel:

$$\text{Tan}\alpha = \frac{1}{3F}$$

The parameter F in the above equation represents the Froude number of the flow based on average values of the velocities and depths of flow at the beginning and end of contraction (Khatsuria, 2004). A flare angle of approximately 4 degrees was calculated which results in a 1:12 (horizontal to longitudinal) transition. Approximately 515 feet will be required to achieve an appropriate transition between the cover and spillway channel geometry.

The 515 feet convergence will include two transitional areas. The initial 315 feet of transition will occur on the FAR cover and will allow the tiered 12 foot wide trapezoidal channel to transition to a wider interim non-tiered trapezoidal channel at the dam crest. The required channel slope for the 315 foot transition is approximately 1.0%.

The interim trapezoidal spillway channel will have a bottom width of 35 feet which will taper both upstream and downstream to match the cover and spillway channel geometry. The interim trapezoidal geometry allows for vehicle access across the dam crest for future maintenance and will have side slopes 6H:1V. The interim channel geometry will only extend for approximately 70 feet from the face of the dam and then begin to transition to the 55 foot wide rectangular spillway channel geometry. This second transition area will take place over 130 feet. The longitudinal slope of this 200 foot transitional area will be 1.5% to keep the flow within this area at critical depth. The spillway transitional area will consist of concrete

Once the spillway completes the transition from trapezoid to a rectangular channel the spillway chute steepens to a 13% slope until the toe of the spillway is reached. The spillway chute will consist of a 55 foot wide rectangular channel with 5 foot vertical walls and will be constructed entirely of concrete. The geometry was selected so that the channel will contain the 100-year peak event with approximately 2.5 feet of minimum freeboard (does not include the PMF portion of the spillway design) calculated from Equation 22 of the USBR guidance manual (USBR 1987).

$$Freeboard(ft) = 2.0 + 0.025v\sqrt[3]{d}$$

The freeboard for the spillway has been added to the channel geometry to account for wave action, air bulking, surface roughness and splashing that may be encountered under supercritical flow conditions. The spillway chute extends approximately 550 feet from the face of the dam and discharges into a stilling basin to force the flow to transition from the turbulent, supercritical flow observed in the spillway to subcritical flow conditions as it flows back into the natural channel.

In order to avoid projection of the discharge off of the end of the steep spillway, the floor of the exit of the spillway was designed to follow the approximate curvature shape as recommended by USBR (USBR 1987 until it tied into a flatter entrance slope of 4:1 (horizontal:vertical). The curvature was approximated by the equation:

$$-y = x tan\theta + x^2/K[4(d+h)\cos^2\theta]$$

Using this approach, the length of the stilling basin entrance is approximately 33 feet as it extends from the bottom of the spillway at an elevation of 586.5 feet to the top of the bottom of the stilling basin at an elevation of 577.6 feet. Curvature calculations are provided in Attachment B.

As stated above, the stilling basin was designed in accordance with the Type III Basin characteristics recommended by USBR (USBR, 1987) due to the hydraulic conditions observed at the outlet of the spillway. A summary of the final design parameters is detailed in the Hydraulic Analysis Results section below. The basin was initially assumed to have a rectangular shape with a bottom width equivalent to the total width of the spillway (55 feet). The depth, length and other design parameters of the basin were calculated from the USBR Small Dams design guidance. The still height and transition apron elevation at the downstream end of the stilling basin were adjusted between model runs until an appropriate sill height was achieved to maintain the necessary tailwater and tie into the existing channel grade downstream of the spillway system. The stilling basin was assumed to be constructed of concrete with a Manning's roughness coefficient of 0.013 and a flat basin bottom.

The reinforced channel located downstream of the stilling basin was assumed to be trapezoidal with 3H:1V side slopes. Consistent channel geometry was a target of the design to simplify the constructability. The bottom slope of this channel was assumed to be constant from the end of the stilling basin to the downstream culvert at the maintenance drive. The resulting slope is approximately 0.55%. The channel was assumed to have a gradual curve from the end of the stilling basin to the maintenance drive (See Drawing 8273-121-21 of the Modified Fly Ash Dam Plan Set). The excavated depth of the channel is dependent on the surrounding topography and the depth of flow determined to occur in the channel; however at no time should the channel be less than 6 feet deep.

Downstream of the proposed improvements near the property limits, Stingy Creek passes through an existing double box culvert at Stingy Creek Road. The existing double box culvert has the hydraulic capacity to only convey a flow equivalent to the 2 year- 24 hour storm event. The proposed culvert system for the maintenance drive was designed to have a similar capacity to the Stingy Creek Road crossing. The proposed culvert system consists of three 60" culverts. Downstream of the new culverts, the proposed design conveyance system will transition to the natural existing creek geometry.

The details of these structures were determined by performing numerous iterations of HEC-RAS, altering parameters as required until an efficient, feasible design was achieved. The model simulated 3,650 ft of channel length, from approximately 1,000 feet upstream of the spillway, where the Type 1 Channel is planned to be constructed to approximately 1,400 feet beyond the new culvert system, where Stingy Creek flows adjacent to Stingy Creek Road before turning south and heading offsite. The distance from the bottom of the design spillway to the new culvert system at Stingy Creek Road was approximately 370 ft. This was assumed to be the total length of the conveyance system design as part of this package.

Input and output summaries from the HEC-RAS model can be found in Table 1A and Table 2 respectively. Additionally, Attachment A presents cross-sections and model schematics for both the 2 year- 24 hour and 100 year – 24 hour peak event simulations.

HYDRAULIC ANALYSIS RESULTS

A written summary of the results of the hydraulic analysis using HEC-RAS is provided below. Results are presented based on notable transitions and components of the conveyance system, beginning at the upstream end of the system and working downstream. Table 2A and 2B presents tabulated results from both the 100 year and 2 year storm event model simulations respectively. Other supporting figures, tables, and attachments are noted.

FAD Transition: The FAD transition area will start at the face of the dam and extend approximately 200 feet downstream. The channel will have a 35 foot bottom width and side walls that slope at a 6:1 (horizontal: vertical) grade. The intent of the FAD interim geometry is to transition the Type I FAR conveyance channels to the rectangular spillway chute.

Spillway Chute: The spillway chute will extend for a length of 550 feet at a 13% slope down the face of the dam. The spillway will consist of a rectangular, concrete channel with a bottom width of 55 feet. The sides of the channel will extend to a height of 5 feet in order to convey the 100 year flow while providing a 2.5 foot minimum freeboard throughout the extent.

Spillway Transition: From the end of the rectangular spillway chute, the entry transition to the stilling basin will consist of a concrete channel which follows the recommended USBR curvature until it transitions to a 4:1 (H:V) slope. This transition will be approximately 36.6 ft in length, extending from elevation 586.5 feet to 577.6 feet. The transition will have a base width of 55 feet with vertical side walls which will be constructed at a steady elevation 591.5 feet to provide adequate freeboard (minimum of 2 feet) for the stilling basin. Chute blocks measuring 1.0 foot high by 1 foot wide will be equally spaced across the bottom of the transition channel, where the transition meets the stilling basin. A total of 22 chute blocks will be constructed along the base of the spillway transition. See Attachment B for schematic showing a general design of the chute blocks.

Stilling Basin: The stilling basin was designed in accordance with the Type III Basin characteristics recommended by USBR (USBR, 1987). See Attachment B for a schematic of the stilling basin design parameters. The concrete basin will be 55 feet wide and will extend downstream from the spillway with a flat bottom. The hydraulic jump required out of the spillway was calculated using the USBR methods at approximately 8.9 feet. The floor of the stilling basin will be installed at an elevation of 577.6 feet giving the basin a total depth of approximately 13.9 feet including 5 feet of freeboard. Baffle blocks will be constructed 7.1 feet from the beginning of the stilling basins. The baffle blocks will be 2.0 feet wide and 1.7 feet high with a vertical face. The tops of the blocks will be 0.3 feet long. The downstream side of each baffle will be sloped 1H;1V. A total of 14 blocks will be constructed in place and will be equally spaced across the stilling basin. At the downstream end of the basin, a 2H:1V sloped sill will be constructed. The bottom of the sill will begin 11.25 feet from the beginning of the stilling basin, and will slope upwards for a vertical height of 5 feet to an elevation of 582.6 feet. The sill height was a parameter that was adjusted through trial in error to achieve appropriate tailwater depth as well as allow the system to have an adequate channel slope downstream of the dissipator. The total length of the basin will be 23 feet from the entry transition to the end of the sill. The downstream side of the sill will drop vertically 1 foot to the basin exit transition channel (elevation 581.6 ft in order to allow for an adequate downstream channel slope in order to tie-into the natural channel while maintaining the necessary tailwater depth. A summary of design parameters for the stilling basin is provided in Table 1B.

Basin Exit Transition: From the base of the sill at an elevation of 581.6 feet the conveyance system will be graded at a steady slope of 0.5% until the channel reaches the new culvert system at Stingy Creek Road. Exiting the stilling basin, a 50 foot long concrete transition channel will be built to transition the conveyance system from the stilling basin to the reinforced channel. The channel bottom will be 55 ft wide at the base of the stilling basin exit and will taper to a 25 foot wide bottom approximately 50 feet downstream of dissipator to an elevation of 581.3 feet. The side slopes will also taper over this transition from vertical side walls at the stilling basin exit to 3H:1V sloped walls 50 feet downstream. The depth of the channel will be determined by the grading in the surrounding area for the PMF spillway exit transition, but at no point should the minimum channel depth within the transition be less than 9 feet. A summary of design parameters for the dissipator basin exit transition is provided in Table 1C

Reinforced Channel: The basin exit transition will tie into a trapezoidal channel constructed to be consistent with the bed slope to Stingy Creek Road and the newly designed culvert system. The invert culvert elevation (upstream side) at this point will be approximately 579.6 feet based on current topographic data. The channel will have a 25 foot wide base with 3H:1V side slopes. The depth of the channel will be dependent on the surrounding grading, but at no point shall the depth be less than 6 feet. The channel will be reinforced with hard armoring for its entire length to prevent scour and erosion during the 100 year event. Rip-rap will be used to armor the entire bottom of the channel as well as the sidewalls to a depth of 3 feet. Above a depth of 3 feet, armoring of the channel is not necessary. Rip-rap sizing was based on FHA Hydraulic Engineering Circular No. 11 (FHA, 1989) using shear stresses and velocities provided by the HEC-RAS model. Ohio Type C Rip Rap with a D_{50} (1.0 foot) will be used to armor the downstream channel. Table 1D presents the design parameters for the reinforced channel.

Culverts at Outfall 001: Currently, twin 48" corrugated metal culverts are installed at the Stingy Creek Road crossing (Outfall 001); however, based on the HEC-RAS model results, the existing culverts do not have the capacity to pass the 2 year peak event. Therefore, the existing culverts will be replaced with three 60" diameter culverts. Culverts will be ADS N-12 or approved equivalent, which has a smooth interior with corrugated outer shell. The new culverts can sufficiently handle the 2 year storm event. The upstream invert of the culverts will be at an elevation 579.6 feet and the downstream invert will be at an elevation 578.8 feet.

SUMMARY AND CONCLUSIONS

The spillway was designed pass the 100 year- 24 hour storm. A conveyance system downstream of the Stingy Run FAR spillway was designed to handle flows from the 100 year- 24 hour storm event. To convey these flows from the base of the spillway to Stingy Creek in a safe and efficient manner, a stilling basin and reinforced channel was designed using HEC-RAS. The stilling basin was designed in accordance with U.S. Bureau of Reclamation Standards (USBR, 1987), complete with chute blocks at the entrance, baffle blocks and a 5 foot exit still to create a hydraulic jump within the basin. The downstream conveyance channel will be armored to protect the channel from scour and erosion during large flow events. Three 60" diameter culverts were designed at the Stingy Creek Road crossing to effectively convey the 2 year – 24 hour storm event.

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Written by: JNJ **Date:** 10/7/2015 Updated 12/28/2015

Reviewed by: RM **Date:** 11/6/2015

Client: AEP **Project:** FAR Closure **Project No.:** CHE8273 **Task No.:** 02/05.22

FIGURES AND TABLES

TABLES

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Table 1A

JOB CHE8273-FAR Closure
SHEET NO. OF
CALCULATED BY Rishab Mahajan DATE 10/1/2015
CHECKED BY Jennifer Jenkins DATE 10/1/2015
DESCRIPTION Stilling Basin and Conveyance Channel Sizing Summary
Inputs from Previous Sources and Analyses

Inputs taken from previous sources	Value	Units	Source
Distance between spillway begin and edge of dam	350	ft	scaled off of Figure 2 of downstream analysis
Distance between edge of dam and tie in to natural channel	630	ft	scaled off of Figure 2 of downstream analysis
Elevation/slopes/lengths/cross sections of upstream channels and spillway	Various	-	Read off of Figure 8273-126 (Details of dam configuration) and 8273-123 (Details-Cover System)
Sump elevation at tie in to natural channel	577.07	-	Taken from Figure 3 of downstream analysis
X-sections of natural channel	Various	-	Taken from Figure 3 and 4 of downstream analysis and estimated from topography where necessary
Length, exit slope, n of downstream channel	Various	-	Taken from SWMM analysis of downstream analysis
Manning's n (Riprap d50=1.0')	0.0395	-	From HEC-11 equation 20 n=0.0395*D^0.167
Manning's n (Earthen)	0.025	-	From SWMM model for natural area
Manning's n (Turf protection grass)	0.035	-	From cover swale design
Manning's n (Concrete)	0.013	-	From cover swale design
Manning's n fabriform mat	0.028	-	Manufacturer's recommendation:
Flow Rate from spillway	2000	cfs	From Cover swale design spreadsheet
Depth of flow in spillway	0.94	ft	From spreadsheet used for sizing spillway
Velocity in spillway	38.6	ft/sec	From spreadsheet used for sizing spillway
Froude number in spillway	7.04	-	From spreadsheet used for sizing spillway and checked by calculation

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Table 1B

JOB CHE8273-FAR Closure
SHEET NO. OF
CALCULATED BY Rishab Mahajan DATE 10/1/2015
CHECKED BY Jennifer Jenkins DATE 10/1/2015
DESCRIPTION Stilling Basin and Conveyance Channel Sizing Summary
Stilling Basin Design

Stilling Basin Design	Value	Units	Source
Max allowable velocity	60	ft/sec	USBR, 1987
Horizontal offset from bottom of spillway to transition to stilling basin. Continue spillway slope of 13%	20	ft	Set design parameter
Slope of transition from spillway to stilling basin	4:1	H:V	USBR, 1987
Elevation drop from end of extended spillway to bottom of basin	12	ft	Design parameter
Depth of hydraulic jump (and depth of stilling basin), d2	8.9	ft	d2=d1*0.5*((1+8*Fr^2)^0.5-1)
Width of basin, W	55	ft	Design parameter
Length of basin, L	22.9	ft	Read from chart in Fig 9-41 (L/d2=2.75, d2=15.6)
chute block offset from wall	0.47	ft	From diagram (d1/2)
Chute block height,width, and space between chute blocks	0.94	ft	From diagram (d1)
Distance to row of baffles	7.12	ft	From diagram (0.8*d2)
Baffle height (h3)	1.7	ft	(From chart in Fig 9-41 (h3/d1=2.2)
Baffle distance from wall	0.64	ft	From diagram (0.375*h3)
Baffle width and space between baffles	1.275	ft	From diagram (0.75*h3)
Baffle top length	0.34	ft	From diagram (0.2*h3)
Baffle length	1.7	ft	From diagram 1:1
distance to front of sill	11.108	0	Based on 2:1 sill slope and total length of
Sill height (h4)	1.316	ft	Analyzed Parameter
Elevation drop to channel	1	ft	Based on riprap d50=2', USACE energy dissipator manual recommends 0.25=0.5 of d100. Assume d100 is about 3', so 1' is between 0.25'-0.5'

Notes:
Stilling Basin Material: concrete
Stilling basin side walls are vertical
Using Type III stilling basin design see figure, From USBR, 1987

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Table 1C

JOB CHE8273-FAR Closure
SHEET NO. OF
CALCULATED BY Rishab Mahajan DATE 10/1/2015
CHECKED BY Jennifer Jenkins DATE 10/1/2015
DESCRIPTION Stilling Basin and Conveyance Channel Sizing Summary
Stilling Basin Exit Transition

Stilling Basin Exit Transition	**Value**	**Units**
Flare width after basin	5	ft
Transition apron length	50	ft
Transition apron top width at sill	65	ft
Transition apron bottom width at sill	55	ft
Side slopes at downstream end	3:1	H:V
Transition Apron Shape	Trapezoidal	-
Transition apron material	Concrete	-

Note:

This area gradually and continuously transitions to a 25ft bottom width trapezoidal channel with 3:1 side slopes

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Table 1D

JOB CHE8273-FAR Closure
SHEET NO. OF
CALCULATED BY Rishab Mahajan DATE 10/1/2015
CHECKED BY Jennifer Jenkins DATE 10/1/2015
DESCRIPTION Stilling Basin and Conveyance Channel Sizing Summary
Reinforced Channel

Riprap Exit Channel Design	Value	Units	Source
Channel bottom width	25	ft	Design Parameter
Channel side slope	3:1	H:V	Assumed
Max velocity in channel at exit	6.4	ft/sec	HECRAS Output
Max hydrualic depth in channel at exit	4.53	ft	HECRAS Output
Max shear stress in channel at exit	0.12	lb/sq ft	HECRAS Output
Shield's equation constant, tc*	0.054	-	From Fischenrich, 2001
Max allowable shear stress based on Sheild's equation for 1' d50	5.54	lb/sq ft	Shear stresses acceptable throughout for 1.0' d50; sheer stress acceptable for 0.5' but velocity is not acceptable
Specific gravity of riprap	2.64	g/cm3	Assumed
Calculated d50 according to HEC-11	0.67	ft	d50: 0.001 *V^3 / (d^0.5 * K1^0.5)
Calculated d50 according to HEC-14	0.27	ft	d50: 0.692/(S-1) * (V^2/2g)
Ohio Type C Rip Rap/AASTO Light Facing	1	ft	HEC-11 Table 3 which references AASHTO gradations

Notes:

Riprap provides higher roughness than concrete to help with velocity, and size is set to protect against scour

Trapezoidal channel from check structure to culvert, at gradual, constant slope and gradual curve, as necessary to match downstream conditions

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Table 2

JOB CHE8273-FAR Closure
SHEET NO. OF
CALCULATED BY Rishab Mahajan DATE 10/1/2015
CHECKED BY Jennifer Jenkins DATE 10/1/2015
SCALE
DESCRIPTION HEC-RAS Outputs, 100-yr Flow

Cross-Description	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	Hydr Depth C	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl	Shear Chan
			(cfs)	(ft)	(ft)	(ft)	(ft)	ft	(ft/ft)	(ft/sec)	(sq ft)	(ft)		(lb/sq ft)
Type 1 channel	2230	100 year	2000	670.11	676.3131	675.61	4.32	677.22	0.006484	7.94	275.92	90.4	0.67	1.75
Begin Type 1 to non-tier trapezoid	1230	100 year	2000	665.65	671.153	671.15	3.64	672.59	0.012557	9.84	216.95	80	0.9	2.85
Transition spillway to non-tier trapezoid	1170	100 year	2000	661	664.9119	664.73	2.77	666.1	0.014245	8.74	228.73	81.94	0.92	2.46
Transition non-tier trapezoid to rectangular spillway	1120	100 year	2000	660.25	664.0381	663.98	2.7	665.34	0.016146	9.15	218.68	80.46	0.98	2.72
Begin Spillway Chute	970	100 year	2000	658	661.3904	661.44	3.02	663.18	0.002019	10.73	186.47	55	1.03	0.38
100' Down Spillway	870.*	100 year	2000	645	646.2606	648.44	1.21	659.18	0.049656	28.85	69.33	55	4.53	3.74
200' Down Spillway	770.*	100 year	2000	632	633.0664	635.44	1.03	651.12	0.085938	34.1	58.65	55	5.82	5.51
300' Down Spillway	670.*	100 year	2000	619	619.9924	622.44	0.96	640.84	0.108832	36.64	54.58	55	6.48	6.51
400' Down Spillway	570.*	100 year	2000	606	606.9602	609.44	0.93	629.23	0.121309	37.87	52.81	55	6.81	7.03
500' Down Spillway	470.*	100 year	2000	593	593.9524	596.44	0.92	616.59	0.124612	38.18	52.38	55	6.89	7.16
Begin transitioning walls for basin	420	100 year	2000	586.5	587.4524	589.94	0.92	610.09	0.124612	38.18	52.38	55	6.89	7.16
Start Entrance Curve	419.9	100 year	2000	586.36	587.312	589.8	0.92	609.97	0.12477	38.2	52.36	55	6.9	7.17
1' Down Dissipator Entrance Curve	419.8	100 year	2000	586.21	587.1615	589.65	0.92	609.84	0.125007	38.22	52.33	55	6.9	7.18
2' Down Dissipator Entrance Curve	419.7	100 year	2000	586.05	587.0009	589.49	0.92	609.71	0.125271	38.24	52.3	55	6.91	7.19
3' Down Dissipator Entrance Curve	419.6	100 year	2000	585.87	586.8197	589.31	0.92	609.58	0.125774	38.29	52.23	55	6.92	7.21
4' Down Dissipator Entrance Curve	419.5	100 year	2000	585.67	586.6182	589.11	0.92	609.45	0.126415	38.35	52.15	55	6.94	7.23
5' Down Dissipator Entrance Curve	419.4	100 year	2000	585.47	586.4167	588.91	0.92	609.32	0.127059	38.41	52.07	55	6.96	7.26
6' Down Dissipator Entrance Curve	419.3	100 year	2000	585.25	586.1949	588.69	0.91	609.19	0.127871	38.48	51.97	55	6.98	7.29
7' Down Dissipator Entrance Curve	419.2	100 year	2000	585.01	585.9526	588.45	0.91	609.06	0.128908	38.58	51.84	55	7	7.33
End Curve transition to 4H:1V	419.1	100 year	2000	584.76	585.7001	588.2	0.91	608.93	0.13004	38.68	51.71	55	7.03	7.38
Front of chute block	414.69	100 year	2000	578.54	590.7823		8.47	590.92	0.000039	2.97	673.2	55	0.15	0.02
Back of chute block (bottom elev)	413.46	100 year	2000	577.6	590.7894		5.15	590.92	0.00007	2.86	699.64	55	0.14	0.02
Just after chute block	413.45	100 year	2000	577.6	590.7957		8.92	590.91	0.000031	2.76	725.62	55	0.13	0.02
Just before baffles	400.97	100 year	2000	577.6	590.7957		8.92	590.91	0.000031	2.76	725.62	55	0.13	0.02
Top edge of baffles	400.96	100 year	2000	577.6	590.7591		5.15	590.9	0.000078	3.01	664.18	55	0.15	0.03
Back top of baffles	400.46	100 year	2000	577.6	590.7591		5.15	590.9	0.000078	3.01	664.18	55	0.15	0.03
Back of baffles	387.754	100 year	2000	577.6	590.7634		8.9	590.88	0.000032	2.76	723.85	55	0.13	0.02
Front edge of sill	380.55	100 year	2000	577.6	590.7634		8.9	590.88	0.000032	2.76	723.85	55	0.13	0.02
Top of sill	370.55	100 year	2000	582.6	590.4637		6.12	590.8	0.000146	4.63	432.42	55	0.29	0.06
Drop after sill and flare out to begin transition	370.54	100 year	2000	581.6	590.4781		6.71	590.74	0.000101	4.1	488.3	55	0.24	0.04
End concrete transition	320.54	100 year	2000	581.3	590.4468		6.07	590.7	0.000116	4.09	575.44	226	0.29	0.04
1' Down Exit Channel	316.53	100 year	2000	581.3	590.4534		6.69	590.66	0.000783	3.69	619.62	225	0.25	0.33
2' Down Exit Channel	316.52	100 year	2000	581.3	590.3868		6.03	590.64	0.00111	4.1	557.47	225	0.29	0.42
4' Down Exit Channel	314.52*	100 year	2000	581.29	590.3845		6.03	590.64	0.001108	4.1	557.45	225	0.29	0.42
6' Down Exit Channel	312.52*	100 year	2000	581.28	590.3822		6.03	590.63	0.001106	4.09	557.42	225	0.29	0.42
8' Down Exit Channel	310.52*	100 year	2000	581.27	590.3799		6.04	590.63	0.001104	4.09	557.4	225	0.29	0.42
10' Down Exit Channel	308.52*	100 year	2000	581.26	590.3776		6.04	590.63	0.001103	4.09	557.38	225	0.29	0.42
15' Down Exit Channel	303.52*	100 year	2000	581.23	590.3719		6.05	590.62	0.001096	4.09	557.62	225	0.29	0.41
20' Down Exit Channel	298.52*	100 year	2000	581.2	590.3663		6.07	590.62	0.00109	4.08	557.86	225	0.29	0.41
25' Down Exit Channel	291.52	100 year	2000	581.16	590.3585		6.08	590.61	0.001082	4.07	558.09	225	0.29	0.41
50' Down Exit Channel	266.949*	100 year	2000	580.99	590.3541		6.52	590.57	0.000855	3.79	570.9	123.62	0.26	0.35
100' Down Exit Channel	216.805*	100 year	2000	580.65	590.3206		6.78	590.53	0.000766	3.68	579.09	115.7	0.25	0.32
200' Down Exit Channel	114.471*	100 year	2000	579.94	590.2653		7.34	590.45	0.000608	3.46	600.07	99.54	0.22	0.28
250' Down Exit Channel	63.3039*	100 year	2000	579.59	590.243		7.61	590.42	0.000542	3.35	611.67	91.45	0.21	0.26
Cross section just upstream of culvert	50	100 year	2000	579.5	590.2375	586.17	7.68	590.41	0.000528	3.32	614.53	89.35	0.21	0.25
Cross section downstream culvert	3	100 year	2000	578.8	585.4701	585.47	6.67	588.79	0.004823	14.63	136.74	65.04	1	2.01
Begin Natural Channel	2.94974*	100 year	2000	577.07	586.2943	585.38	5.66	587.13	0.001739	7.87	301.52	79	0.56	0.61
Cross Section 2 in Natural Channel	2	100 year	2000	576.17	584.4875	584.47	5.98	586.4	0.003842	12.14	200.16	50	0.85	1.43
Cross Section 1 in Natural Channel	1	100 year	2000	573.02	580.4928	580.23	3.64	581.9	0.00477	9.71	214.35	59.99	0.87	1.08
Cross section just upstream of final culvert	0	100 year	2000	569.87	575.5477	575.55	4.57	577.35	0.005045	11.63	194.99	52.8	0.94	1.44

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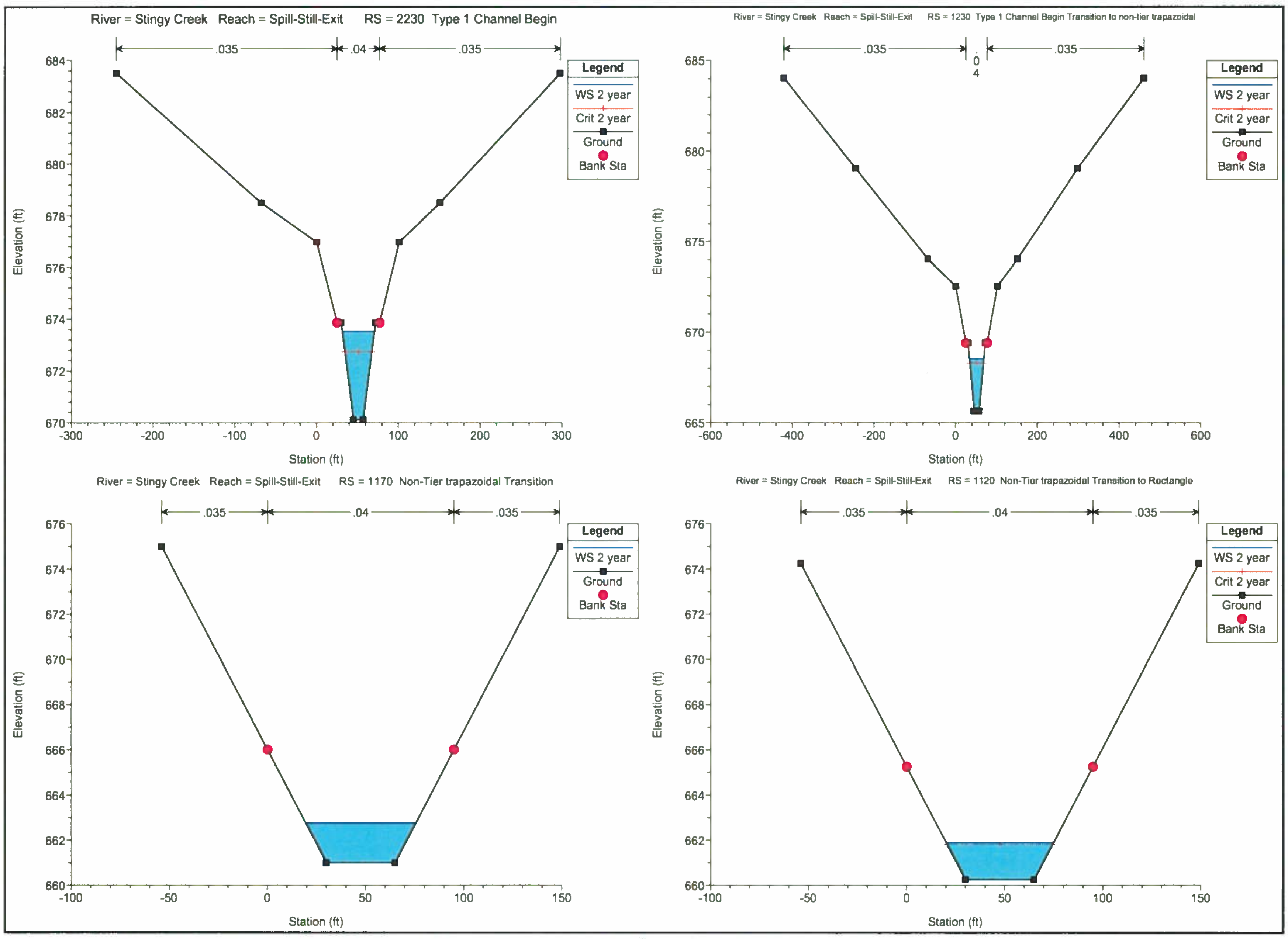
JOB CHE8273-FAR Closure
SHEET NO. ____ OF ____
CALCULATED BY Rishab Mahajan DATE 10/1/2015
CHECKED BY Jennifer Jenkins DATE 10/1/2015
SCALE ____
DESCRIPTION HEC-RAS Outputs, 2-yr Flow

Cross-Description	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	Hydr Depth C	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl	Shear Chan
			(cfs)	(ft)	(ft)	(ft)	(ft)	ft	(ft/ft)	(ft/sec)	(sq ft)	(ft)		(lb/sq ft)
Type 1 channel	2230	2 year	450	670.11	673.5187	672.73	2.18	673.93	0.006804	5.15	87.38	39.27	0.61	0.93
Begin Type 1 to non-tier trapezoid	1230	2 year	450	665.65	668.5063	668.27	1.88	669.21	0.014108	6.73	66.91	34.85	0.86	1.66
Transition spillway to non-tier trapezoid	1170	2 year	450	661	662.7672		1.43	663.25	0.014071	5.58	80.59	56.21	0.82	1.25
Transition non-tier trapezoid to rectangular spillway	1120	2 year	450	660.25	661.8923	661.81	1.34	662.47	0.018309	6.11	73.66	54.71	0.93	1.53
Begin Spillway Chute	970	2 year	450	658	658.9584	659.27	0.93	660.09	0.006179	8.54	52.71	55	1.54	0.36
100' Down Spillway	870.*	2 year	450	645	645.397	646.27	0.39	651.99	0.113537	20.61	21.84	55	5.76	2.77
200' Down Spillway	770.*	2 year	450	632	632.3817	633.27	0.38	639.52	0.129424	21.44	20.99	55	6.12	3.04
300' Down Spillway	670.*	2 year	450	619	619.3818	620.27	0.38	626.51	0.129287	21.43	21	55	6.11	3.04
400' Down Spillway	570.*	2 year	450	606	606.3815	607.27	0.38	613.52	0.12963	21.45	20.98	55	6.12	3.04
500' Down Spillway	470.*	2 year	450	593	593.3817	594.27	0.38	600.52	0.129424	21.44	20.99	55	6.12	3.04
Begin transitioning walls for basin	420	2 year	450	586.5	586.8815	587.77	0.38	594.02	0.12963	21.45	20.98	55	6.12	3.04
Start Entrance Curve	419.9	2 year	450	586.36	586.7411	587.63	0.38	593.9	0.129975	21.47	20.96	55	6.13	3.05
1' Down Dissipator Entrance Curve	419.8	2 year	450	586.21	586.5908	587.48	0.38	593.76	0.130459	21.49	20.94	55	6.14	3.06
2' Down Dissipator Entrance Curve	419.7	2 year	450	586.05	586.43	587.32	0.37	593.63	0.131294	21.53	20.9	55	6.16	3.07
3' Down Dissipator Entrance Curve	419.6	2 year	450	585.87	586.2488	587.14	0.37	593.49	0.132631	21.6	20.84	55	6.18	3.09
4' Down Dissipator Entrance Curve	419.5	2 year	450	585.67	586.0471	586.94	0.37	593.36	0.134707	21.7	20.74	55	6.23	3.13
5' Down Dissipator Entrance Curve	419.4	2 year	450	585.47	585.8454	586.74	0.37	593.22	0.136677	21.79	20.65	55	6.27	3.16
6' Down Dissipator Entrance Curve	419.3	2 year	450	585.25	585.6234	586.52	0.37	593.08	0.139212	21.91	20.53	55	6.32	3.2
7' Down Dissipator Entrance Curve	419.2	2 year	450	585.01	585.3809	586.28	0.37	592.94	0.142273	22.06	20.4	55	6.38	3.25
End Curve transition to 4H:1V	419.1	2 year	450	584.76	585.1284	586.03	0.36	592.79	0.145503	22.21	20.26	55	6.45	3.3
Front of chute block	414.69	2 year	450	578.54	585.7689		5.72	585.79	0.00001	1.13	397.49	54.99	0.07	0
Back of chute block (bottom elev)	413.46	2 year	450	577.6	585.7706		3.37	585.79	0.000017	1.06	423.63	54.99	0.07	0
Just after chute block	413.45	2 year	450	577.6	585.7719		6.3	585.79	0.000007	1	449.34	54.99	0.06	0
Just before baffles	400.97	2 year	450	577.6	585.7719		6.3	585.79	0.000007	1	449.34	54.99	0.06	0
Top edge of baffles	400.96	2 year	450	577.6	585.7637		3.27	585.78	0.000021	1.16	389.45	55	0.08	0
Back top of baffles	400.46	2 year	450	577.6	585.7637		3.27	585.78	0.000021	1.16	389.45	55	0.08	0
Back of baffles	387.754	2 year	450	577.6	585.7646		6.3	585.78	0.000007	1	448.94	54.99	0.06	0
Front edge of sill	380.55	2 year	450	577.6	585.7646		6.3	585.78	0.000007	1	448.94	54.99	0.06	0
Top of sill	370.55	2 year	450	582.6	585.6274		2.73	585.74	0.000147	2.7	166.46	54.99	0.27	0.02
Drop after sill and flare out to begin transition	370.54	2 year	450	581.6	585.6373		3.52	585.7	0.000059	2.03	222.05	55	0.18	0.01
End concrete transition	320.54	2 year	450	581.3	585.5385		3.09	585.66	0.000135	2.82	159.86	50.43	0.28	0.03
1' Down Exit Channel	316.53	2 year	450	581.3	585.5435		3.42	585.62	0.000738	2.29	196.41	55.57	0.21	0.16
2' Down Exit Channel	316.52	2 year	450	581.3	585.4762		3.06	585.61	0.001364	2.89	155.8	49.62	0.29	0.26
4' Down Exit Channel	314.52*	2 year	450	581.29	585.4736		3.06	585.6	0.001355	2.88	156.11	49.63	0.29	0.26
6' Down Exit Channel	312.52*	2 year	450	581.28	585.4709		3.06	585.6	0.001347	2.88	156.41	49.64	0.29	0.26
8' Down Exit Channel	310.52*	2 year	450	581.27	585.4684		3.07	585.6	0.001339	2.87	156.73	49.66	0.28	0.26
10' Down Exit Channel	308.52*	2 year	450	581.26	585.4659		3.07	585.59	0.001331	2.87	157.04	49.67	0.28	0.26
15' Down Exit Channel	303.52*	2 year	450	581.23	585.4599		3.09	585.59	0.001305	2.85	158.05	49.73	0.28	0.25
20' Down Exit Channel	298.52*	2 year	450	581.2	585.4539		3.11	585.58	0.00128	2.83	159.06	49.78	0.28	0.25
25' Down Exit Channel	291.52	2 year	450	581.16	585.4457		3.13	585.57	0.001247	2.81	160.4	49.85	0.28	0.24
50' Down Exit Channel	266.949*	2 year	450	580.99	585.4288		3.18	585.53	0.001043	2.6	173.4	53.13	0.25	0.21
100' Down Exit Channel	216.805*	2 year	450	580.65	585.3939		3.37	585.48	0.000814	2.38	189.01	54.69	0.23	0.17
200' Down Exit Channel	114.470*	2 year	450	579.94	585.3467		3.76	585.41	0.000499	2.01	224.29	57.97	0.18	0.12
250' Down Exit Channel	63.3035*	2 year	450	579.59	585.3306		3.95	585.38	0.000399	1.85	242.77	59.58	0.16	0.1
Cross section just upstream of culvert	50	2 year	450	579.5	585.1228	581.97	5.62	585.36	0.001105	3.9	115.27	58.75	0.29	0.39
Cross section downstream culvert	3.1	2 year	450	578.8	582.5476	581.26	3.75	583.08	0.001668	5.86	76.83	47.49	0.53	0.39
Begin Natural Channel	3	2 year	450	577.07	582.4916	581.19	2.8	582.93	0.002037	5.33	84.4	27.91	0.54	0.36
Cross Section 2 in Natural Channel	2	2 year	450	576.17	580.6865	580.61	2.43	581.83	0.006409	8.6	53.21	23.43	0.94	0.97
Cross Section 1 in Natural Channel	1	2 year	450	573.02	577.6138	577.61	1.71	578.51	0.008028	7.6	59.18	32.72	1	0.86
Cross section just upstream of final culvert	0	2 year	450	569.87	572.8568	572.84	2.08	573.75	0.006301	7.69	61.58	37.25	0.92	0.82

ATTACHMENT A

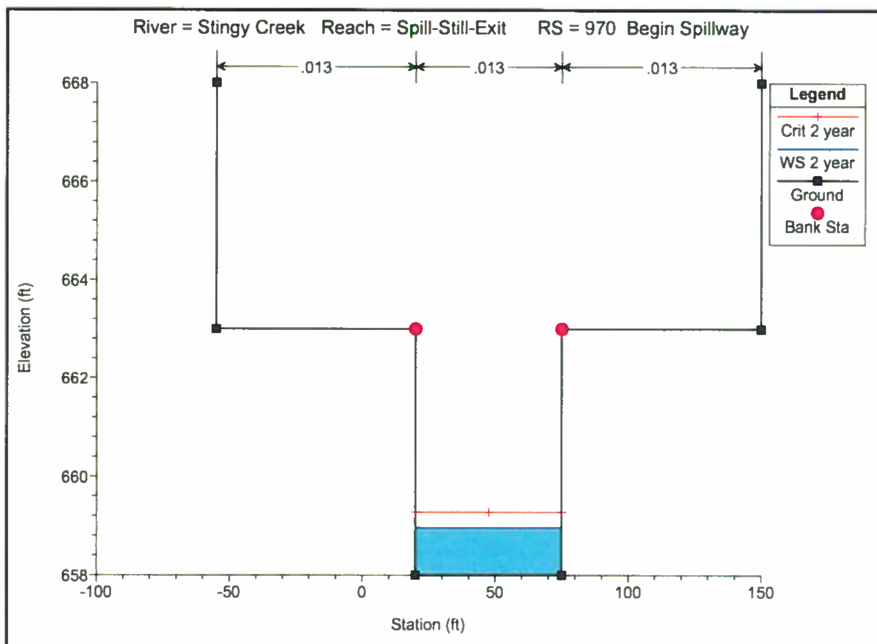
HEC-RAS Outputs and Cross Sections for the 2-yr and 100-yr Storms

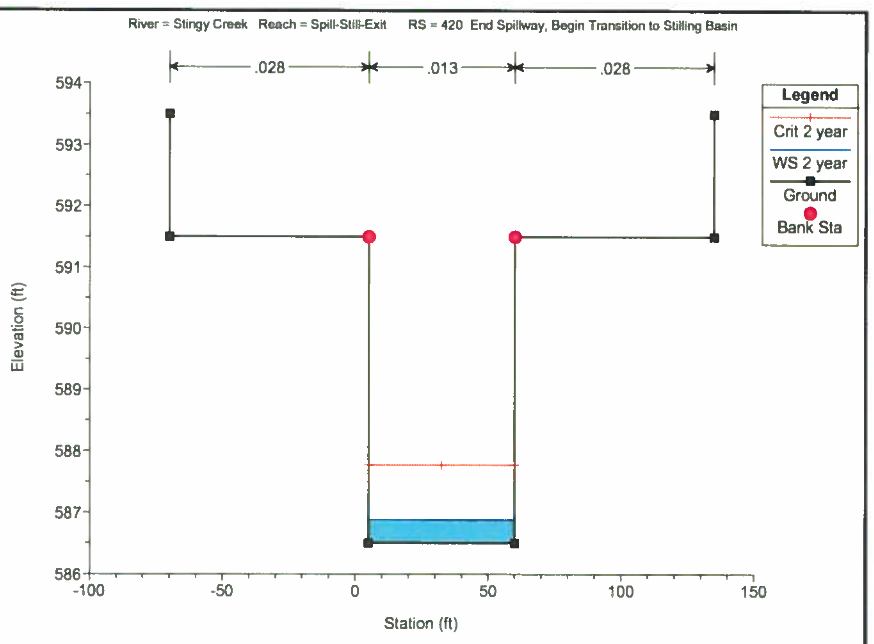
 River = Stingy Creek Reach = Spill-Still-Exit RS = 2230 Type 1 Channel Begin

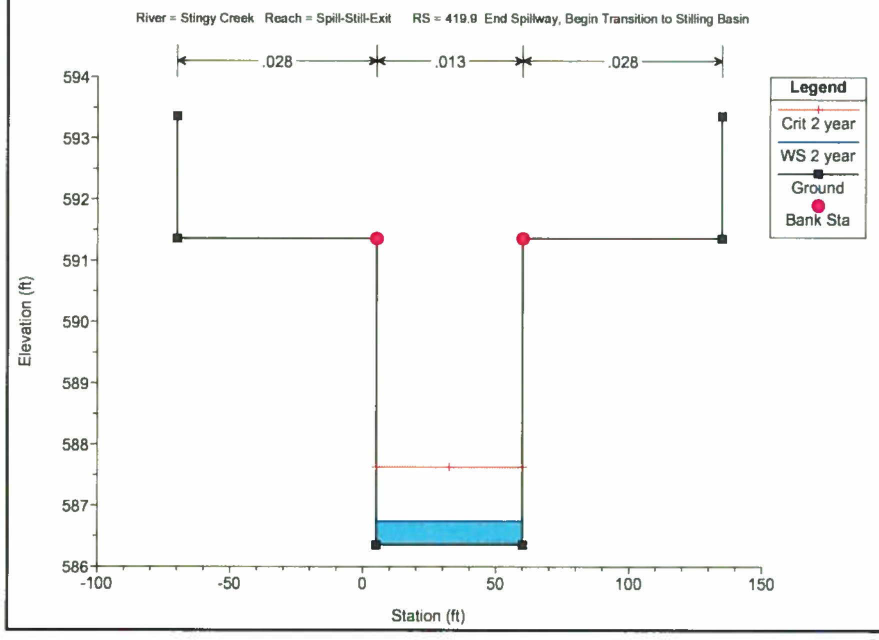
River = Stingy Creek Reach = Spill-Still-Exit RS = 1230 Type 1 Channel Begin Transition to non-tier trapazoidal

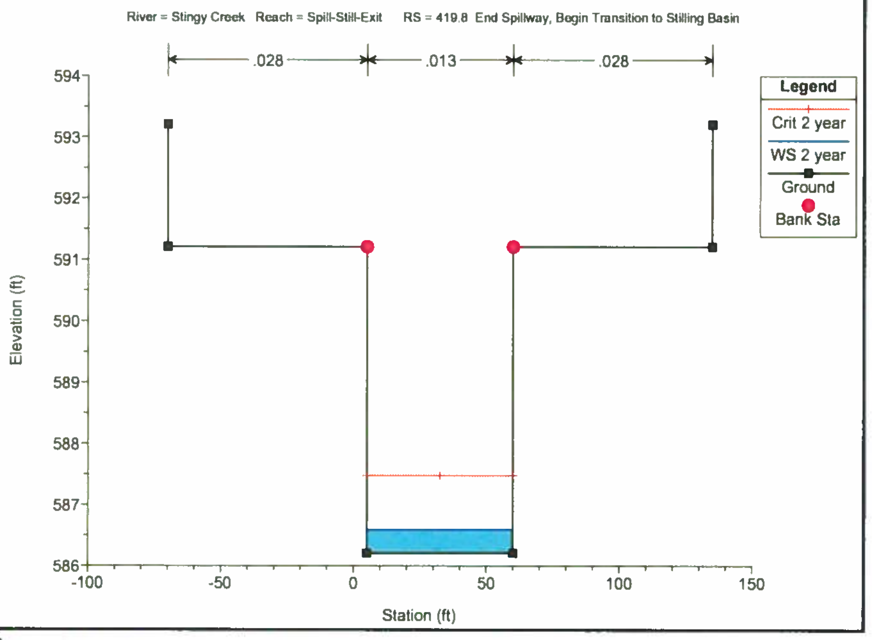
River = Stingy Creek Reach = Spill-Still-Exit RS = 1170 Non-Tier trapazoidal Transition

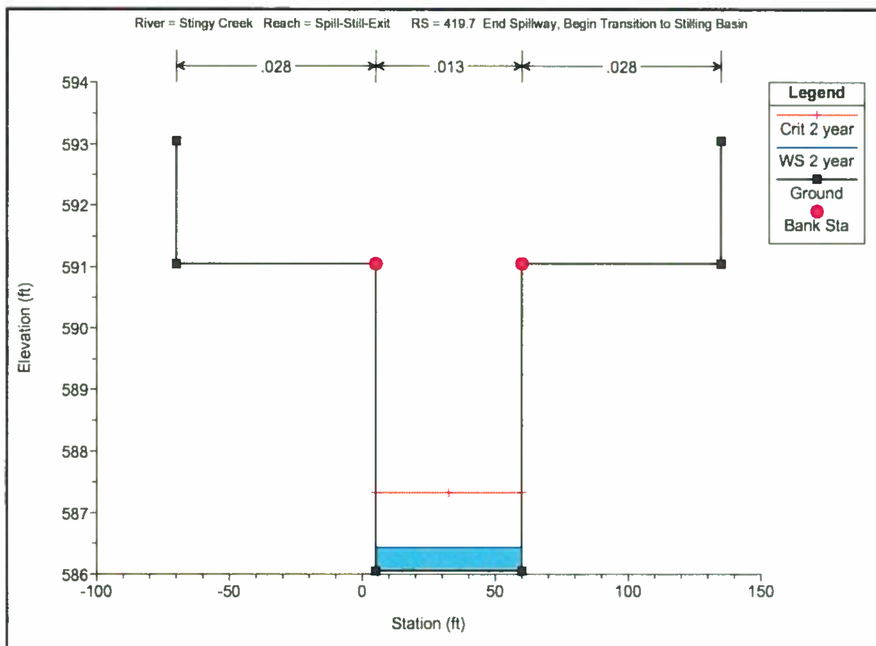
River = Stingy Creek Reach = Spill-Still-Exit RS = 1120 Non-Tier trapazoidal Transition to Rectangle


River = Stingy Creek Reach = Spill-Still-Exit RS = 970 Begin Spillway

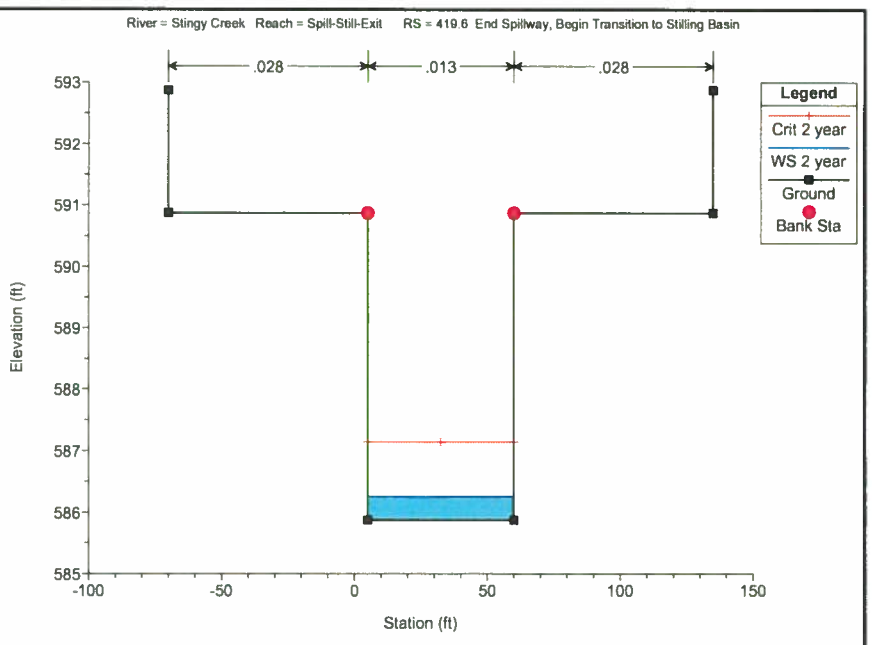

River = Stingy Creek Reach = Spill-Still-Exit RS = 420 End Spillway, Begin Transition to Stilling Basin


River = Stingy Creek Reach = Spill-Still-Exit RS = 419.9 End Spillway, Begin Transition to Stilling Basin

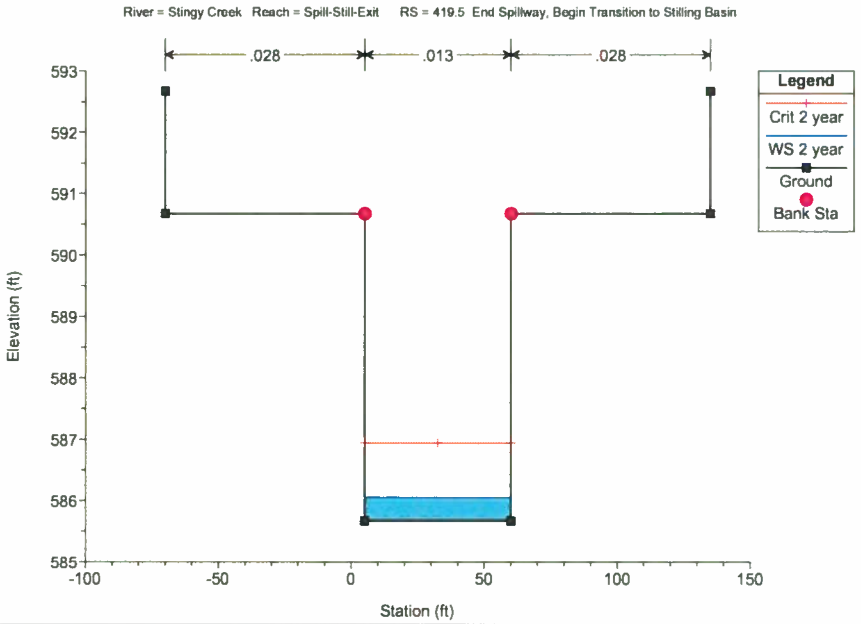

River = Stingy Creek Reach = Spill-Still-Exit RS = 419.8 End Spillway, Begin Transition to Stilling Basin



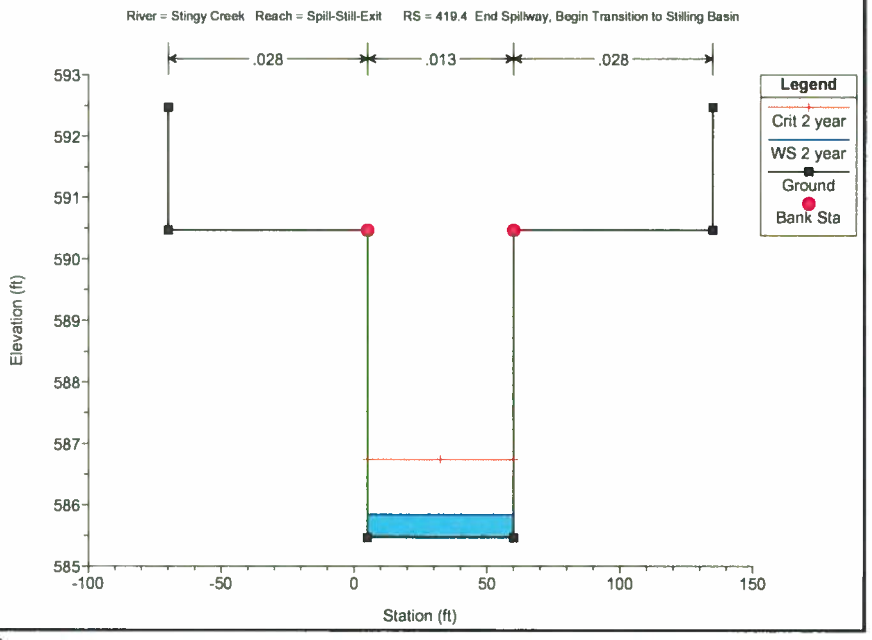
River = Stingy Creek Reach = Spill-Still-Exit RS = 419.7 End Spillway, Begin Transition to Stilling Basin



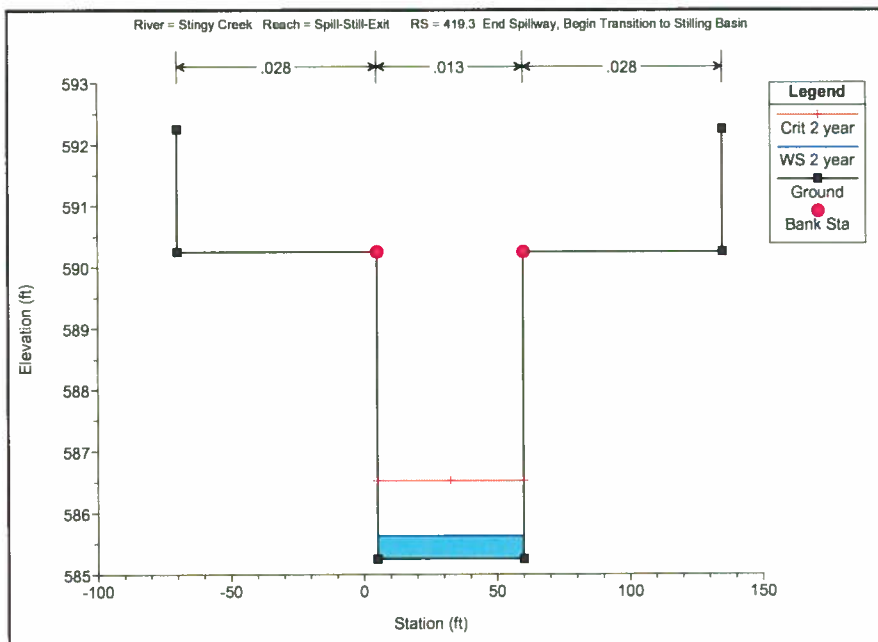
River = Stingy Creek Reach = Spill-Still-Exit RS = 419.6 End Spillway, Begin Transition to Stilling Basin

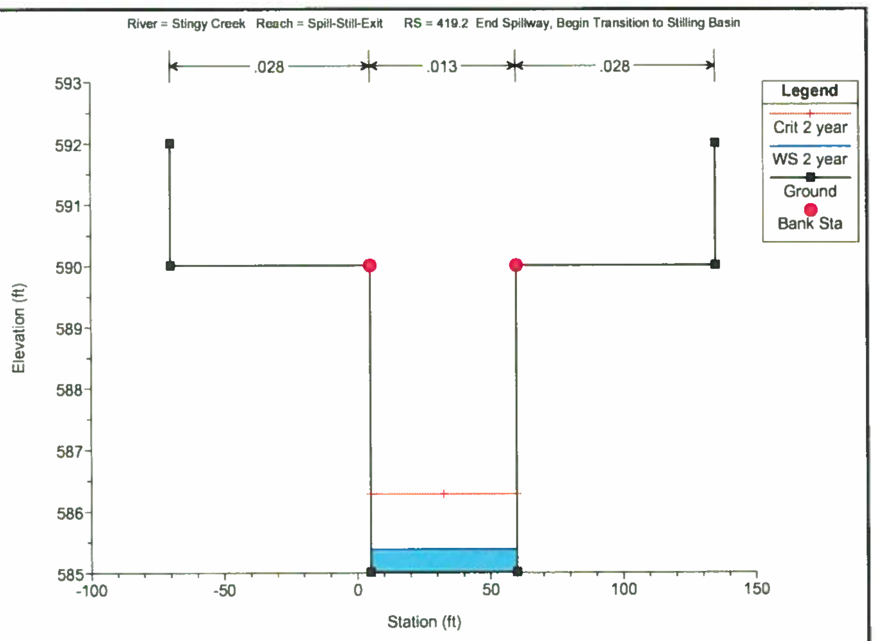


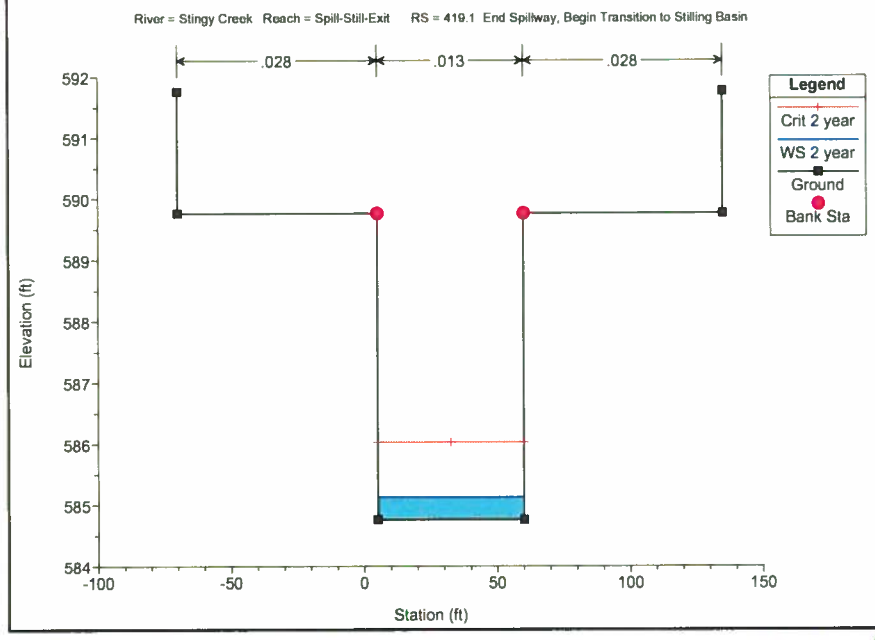
River = Stingy Creek Reach = Spill-Still-Exit RS = 419.5 End Spillway, Begin Transition to Stilling Basin

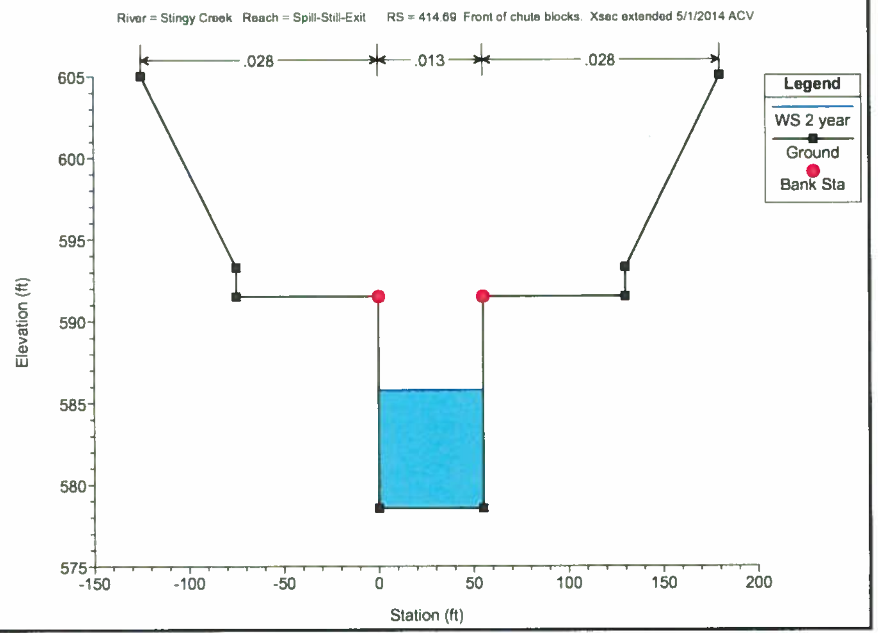


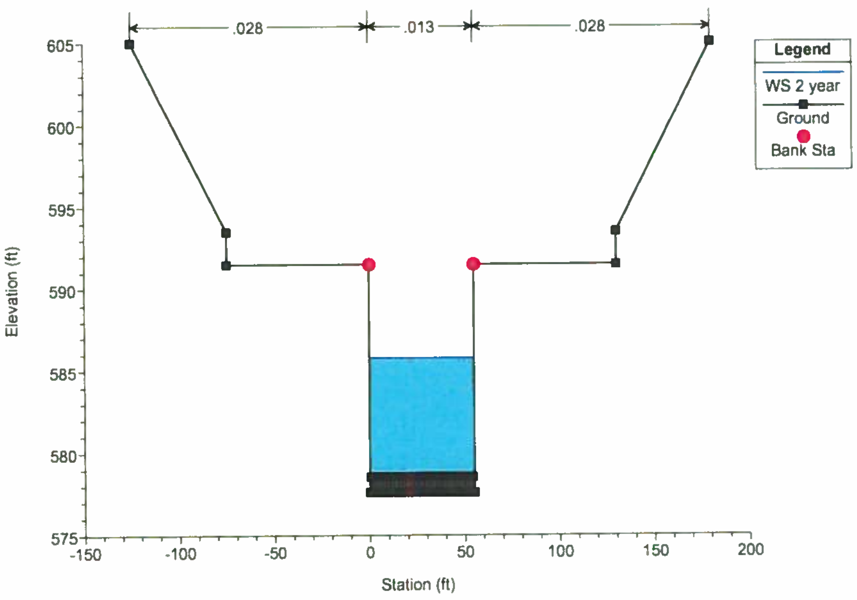
River = Stingy Creek Reach = Spill-Still-Exit RS = 419.4 End Spillway, Begin Transition to Stilling Basin


River = Stingy Creek Reach = Spill-Still-Exit RS = 419.3 End Spillway, Begin Transition to Stilling Basin

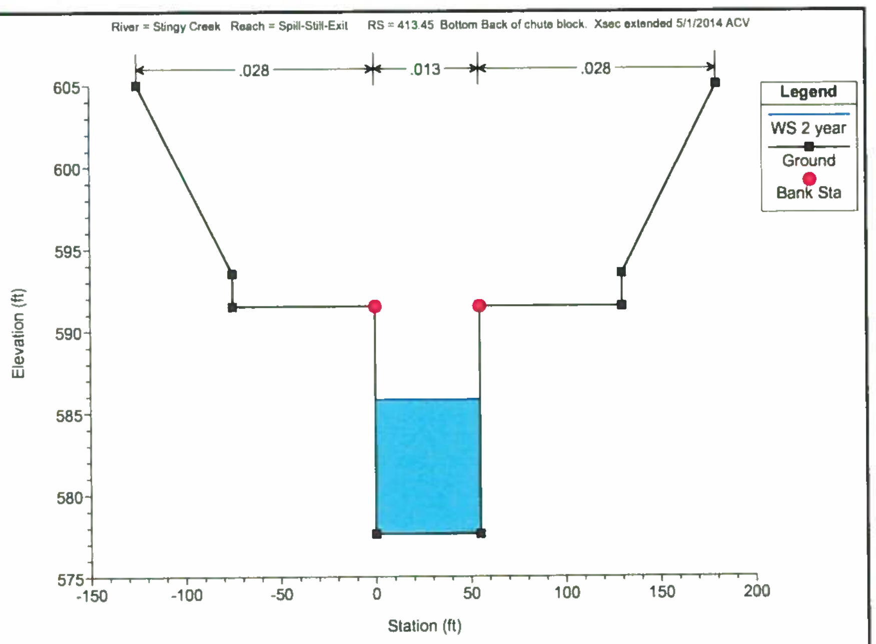

River = Stingy Creek Reach = Spill-Still-Exit RS = 419.2 End Spillway, Begin Transition to Stilling Basin


River = Stingy Creek Reach = Spill-Still-Exit RS = 419.1 End Spillway, Begin Transition to Stilling Basin

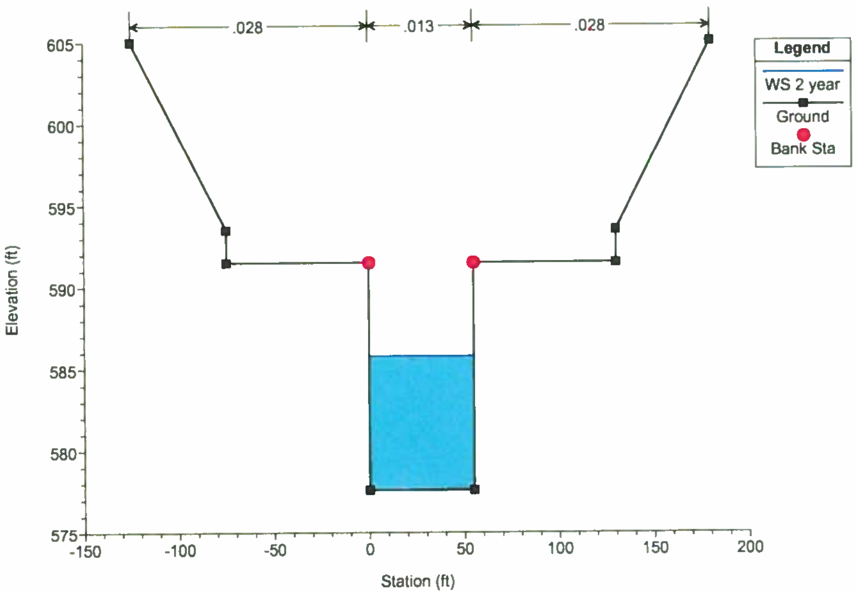

River = Stingy Creek Reach = Spill-Still-Exit RS = 414.69 Front of chute blocks. Xsec extended 5/1/2014 ACV



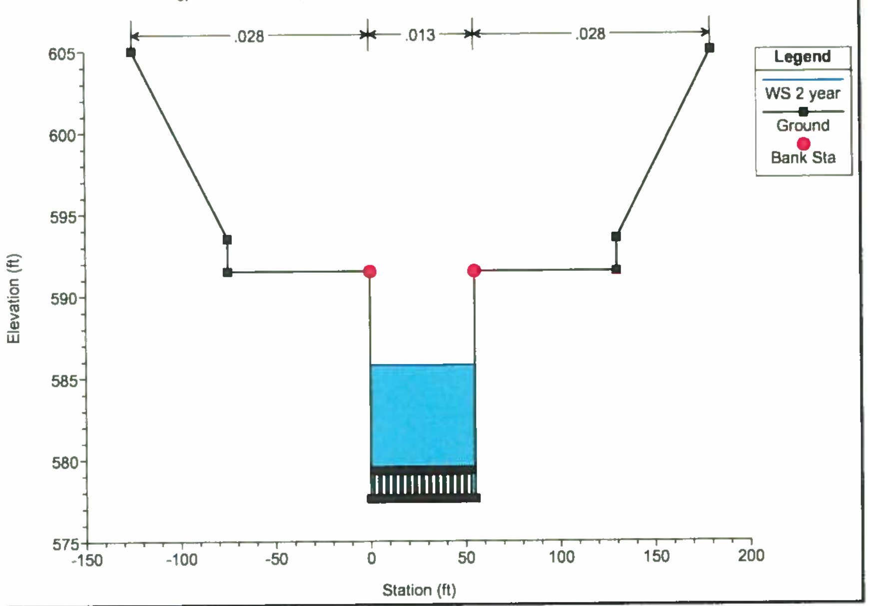
River = Stingy Creek   Reach = Spill-Still-Exit   RS = 413.46 Top back of chute blocks. Xsec extended 5/1/2014 ACV



River = Stingy Creek   Reach = Spill-Still-Exit   RS = 413.45 Bottom Back of chute block. Xsec extended 5/1/2014 ACV

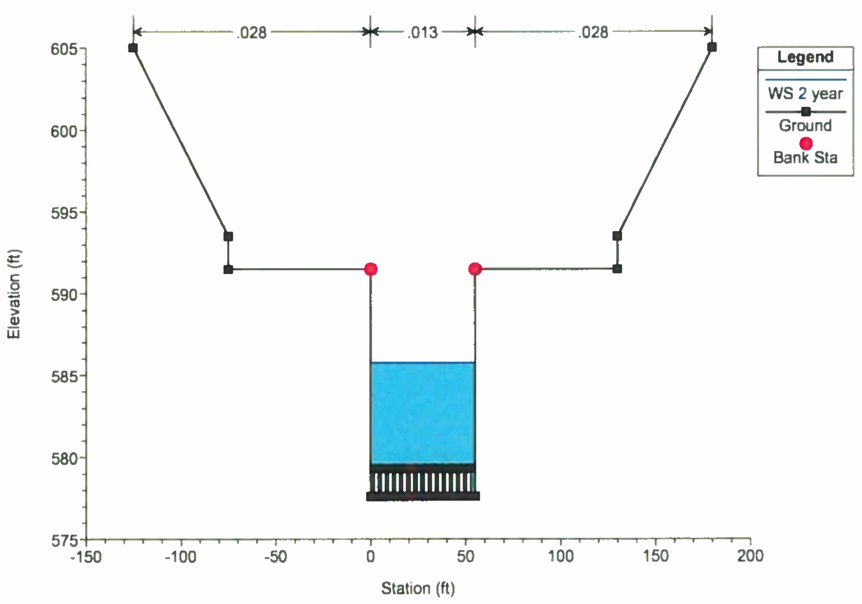


River = Stingy Creek   Reach = Spill-Still-Exit   RS = 400.97 Bottom front of baffles. xsec extended ACV 5/1/2014

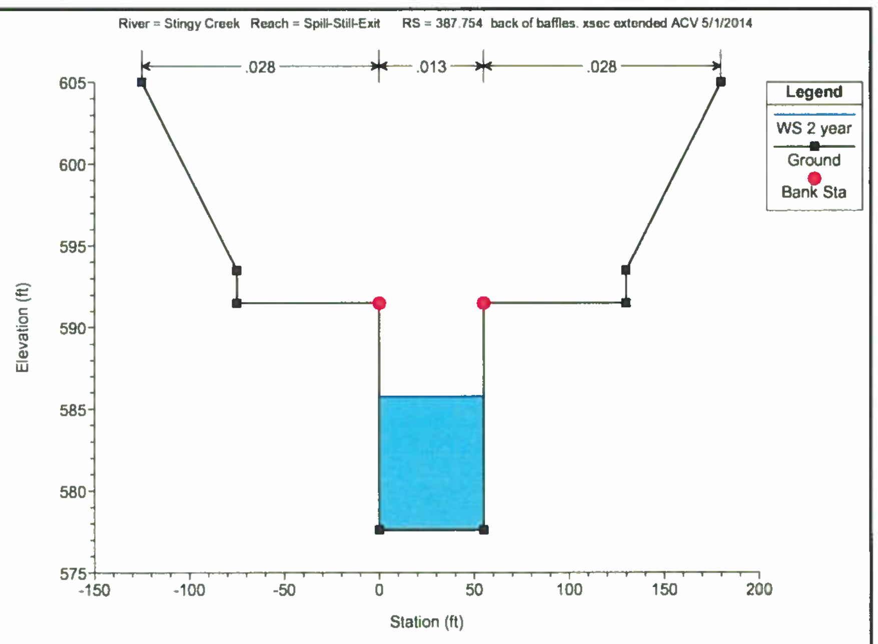


River = Stingy Creek   Reach = Spill-Still-Exit   RS = 400.96 Top front of baffles. Xsec extended 5/1/014 ACV

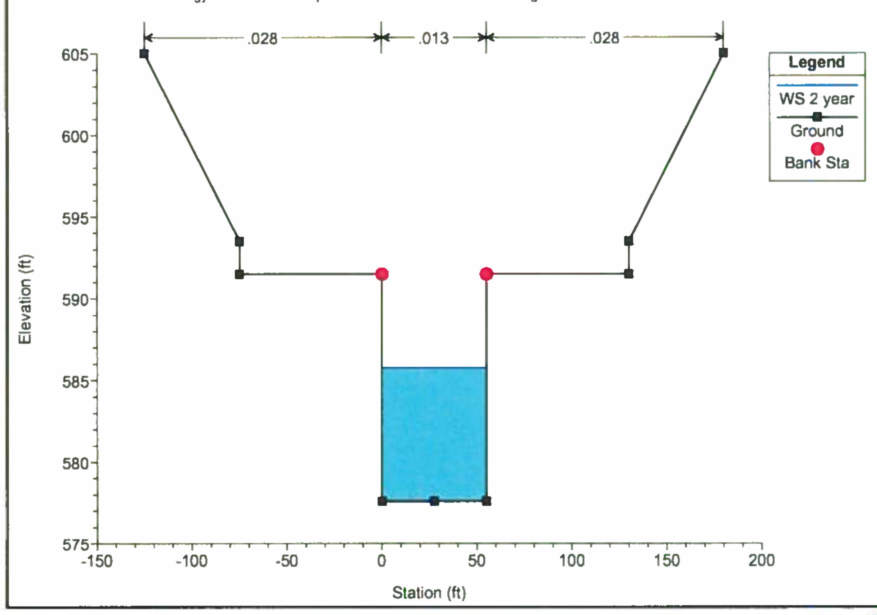
River = Stingy Creek Reach = Spill-Still-Exit RS = 400.46 top back of baffles. xsec extended 5/1/2014 ACV



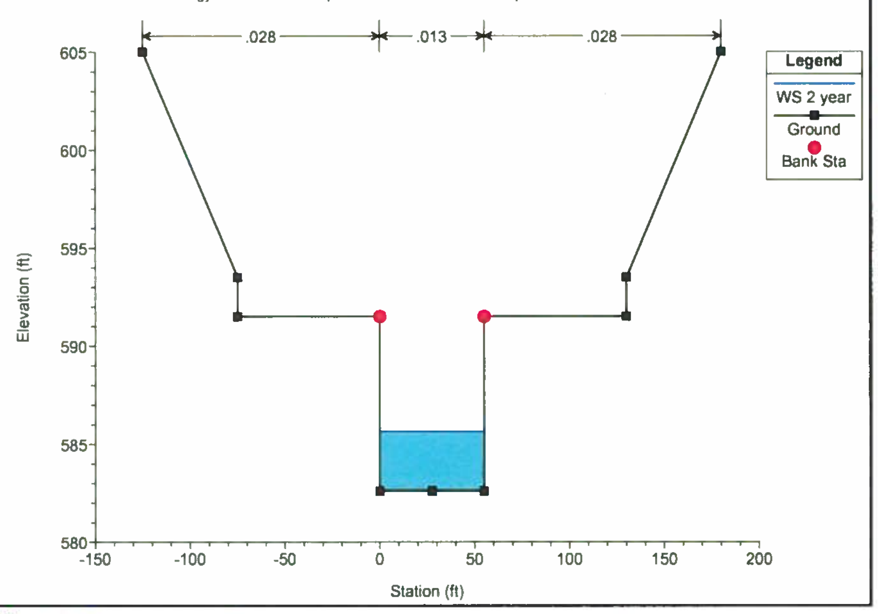
River = Stingy Creek Reach = Spill-Still-Exit RS = 387.754 back of baffles. xsec extended ACV 5/1/2014

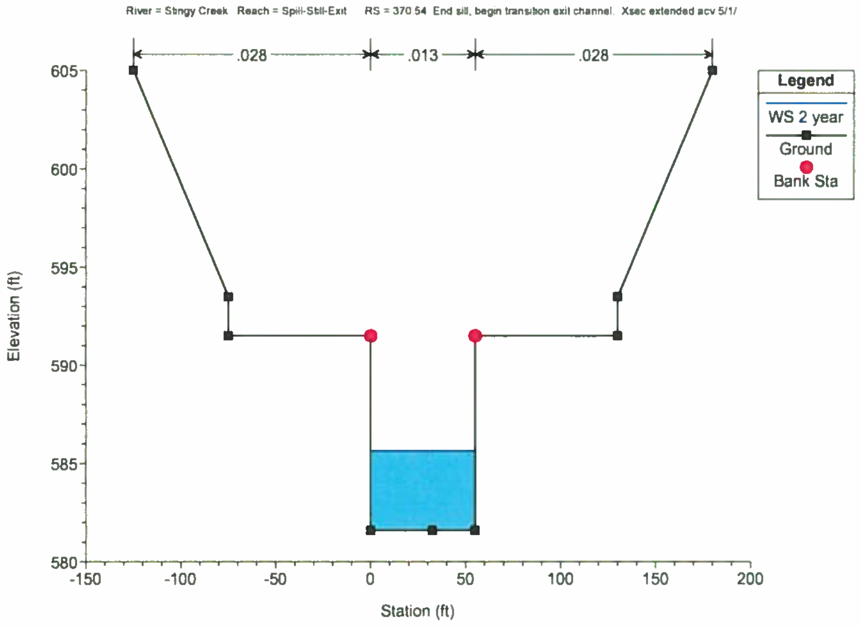


River = Stingy Creek Reach = Spill-Still-Exit RS = 380.55 Front edge of sill. xsec extended 5/1/2014 ACV

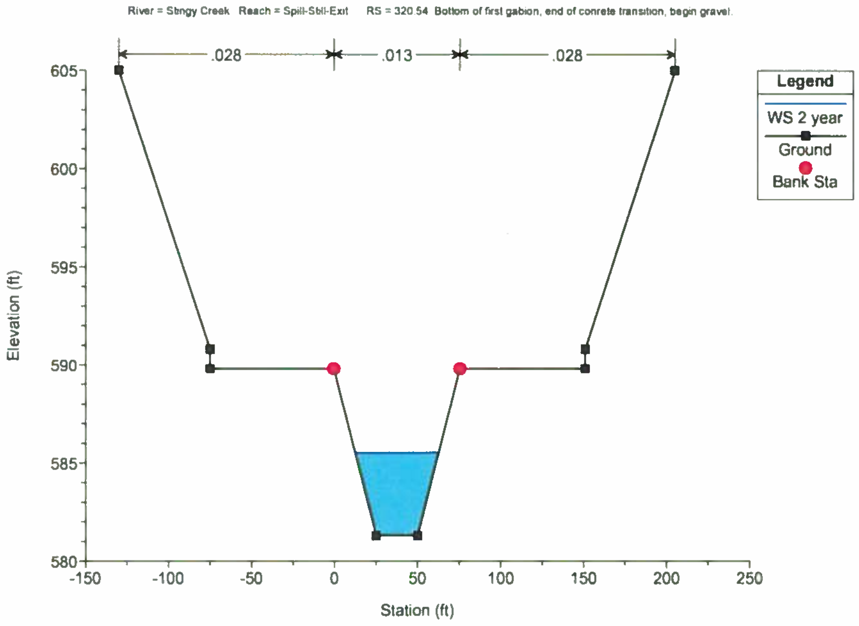


River = Stingy Creek Reach = Spill-Still-Exit RS = 370.55 top of sill. Xsec extended 5/1/2014 ACV

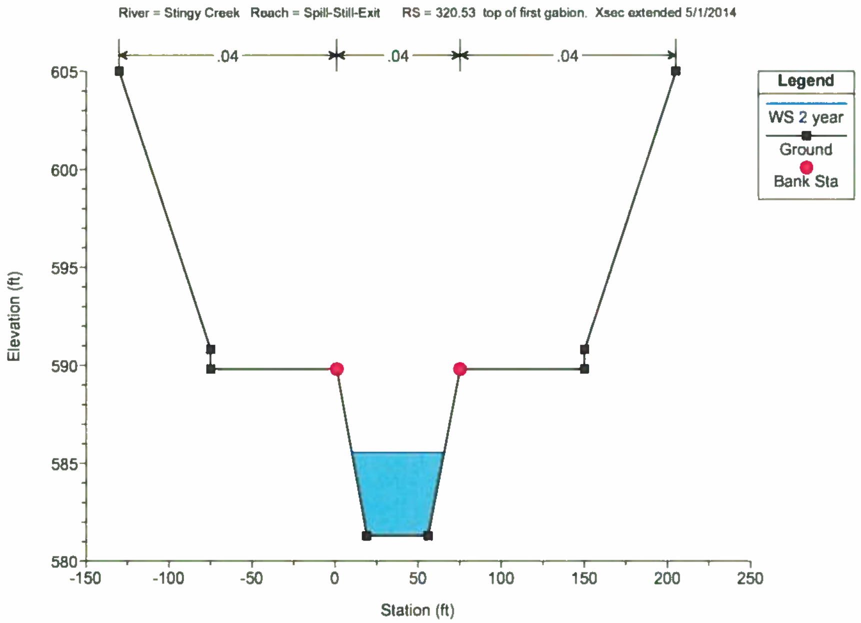




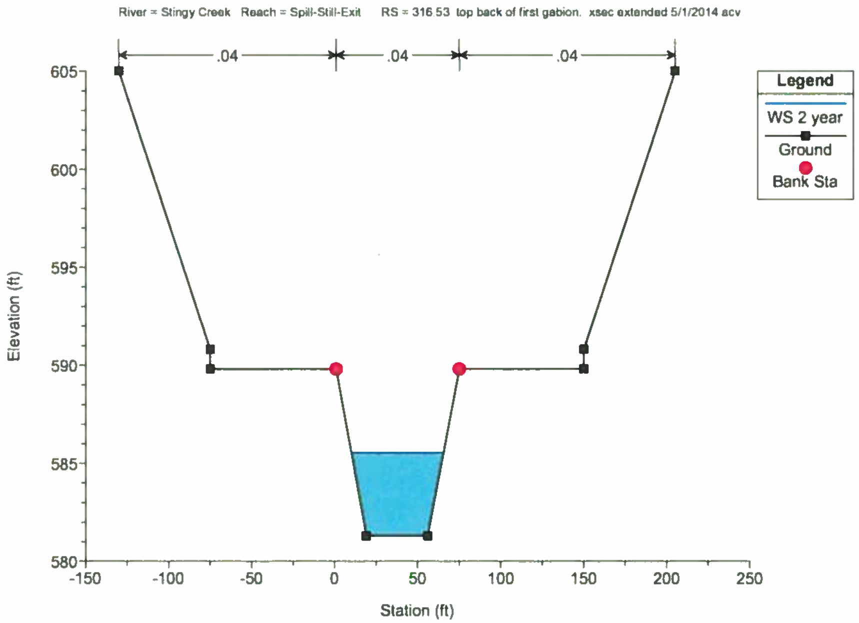
River = Stingy Creek Reach = Spill-Still-Exit RS = 370.54 End sill, begin transition exit channel. Xsec extended acv 5/1/



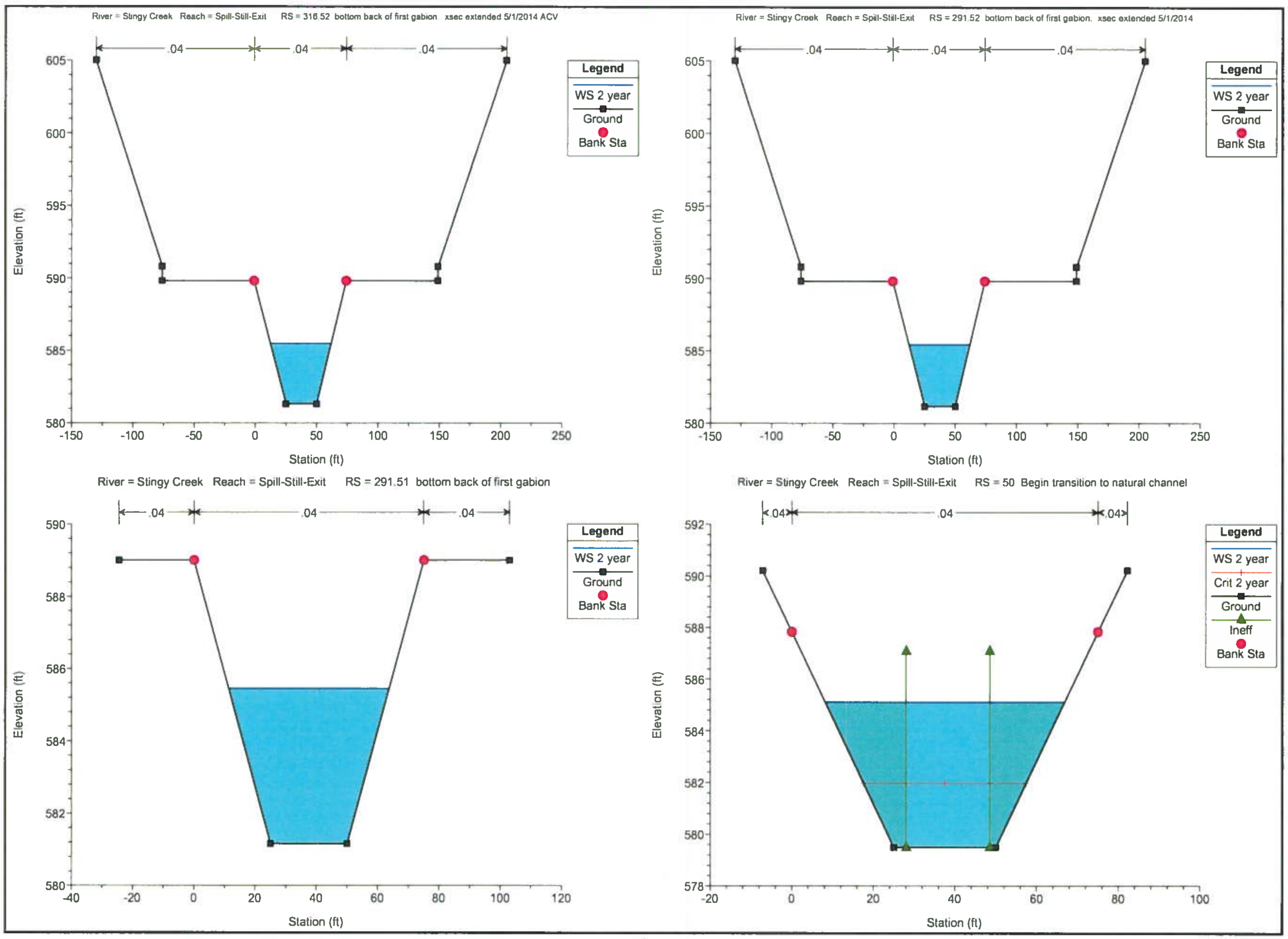
River = Stingy Creek Reach = Spill-Still-Exit RS = 320.54 Bottom of first gabion, end of conrete transition, begin gravel



River = Stingy Creek Reach = Spill-Still-Exit RS = 320.53 top of first gabion. Xsec extended 5/1/2014



River = Stingy Creek Reach = Spill-Still-Exit RS = 316.53 top back of first gabion. xsec extended 5/1/2014 acv

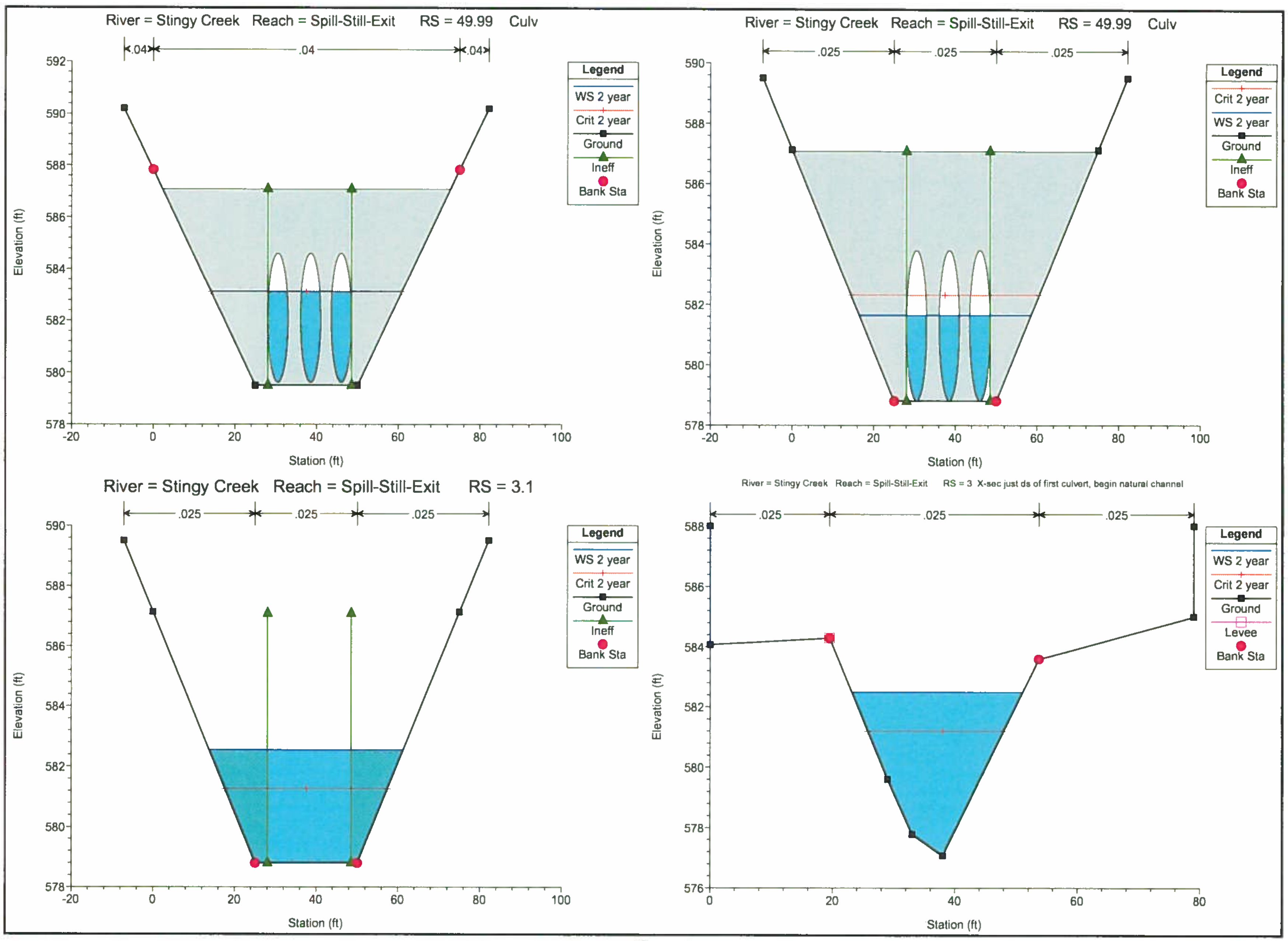


River = Stingy Creek   Reach = Spill-Still-Exit   RS = 316.52 bottom back of first gabion   xsec extended 5/1/2014 ACV

River = Stingy Creek   Reach = Spill-Still-Exit   RS = 291.52 bottom back of first gabion. xsec extended 5/1/2014

River = Stingy Creek   Reach = Spill-Still-Exit   RS = 291.51 bottom back of first gabion

River = Stingy Creek   Reach = Spill-Still-Exit   RS = 50 Begin transition to natural channel



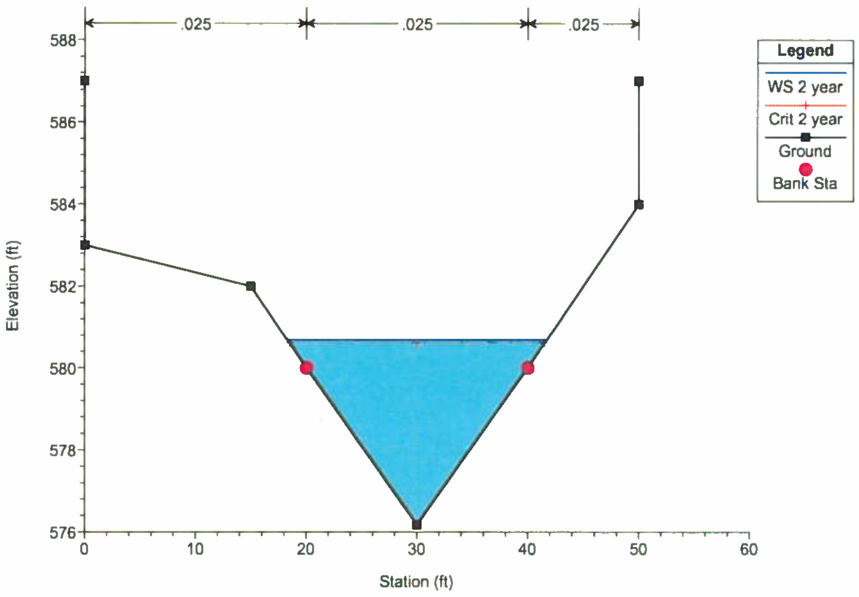
River = Stingy Creek Reach = Spill-Still-Exit RS = 49.99 Culv

River = Stingy Creek Reach = Spill-Still-Exit RS = 49.99 Culv

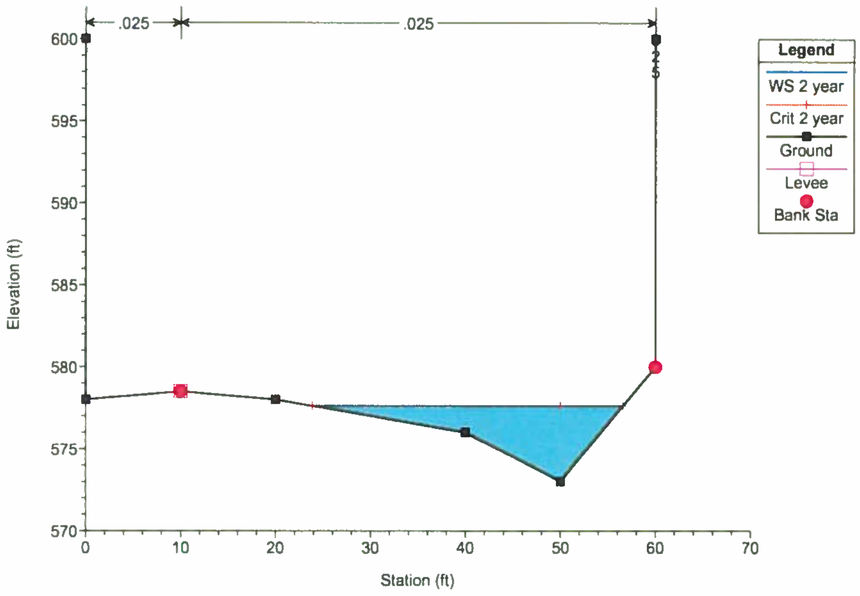
River = Stingy Creek Reach = Spill-Still-Exit RS = 3.1

River = Stingy Creek Reach = Spill-Still-Exit RS = 3 X-sec just ds of first culvert, begin natural channel

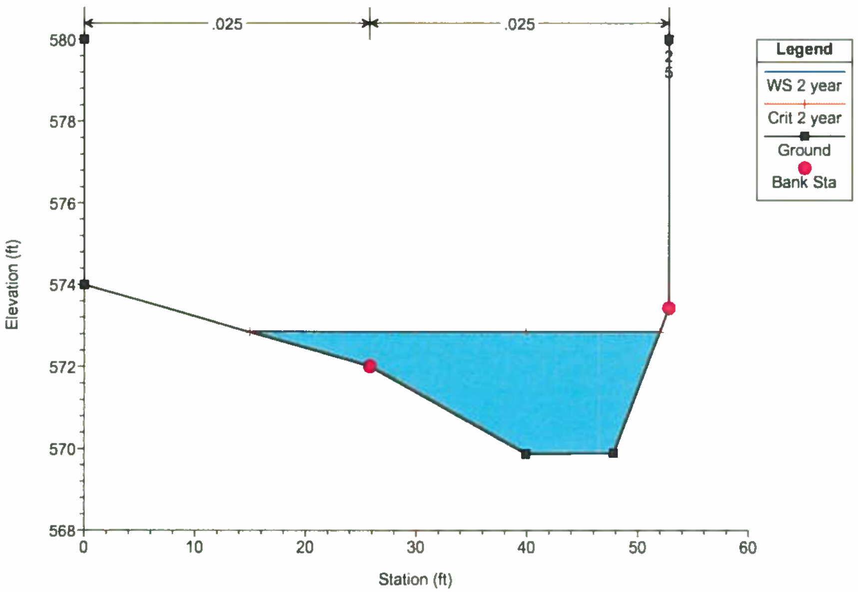
River = Slingy Creek   Reach = Spill-Still-Exit   RS = 2 X-sec just after curve in natural channel-~1000' upstream of lar

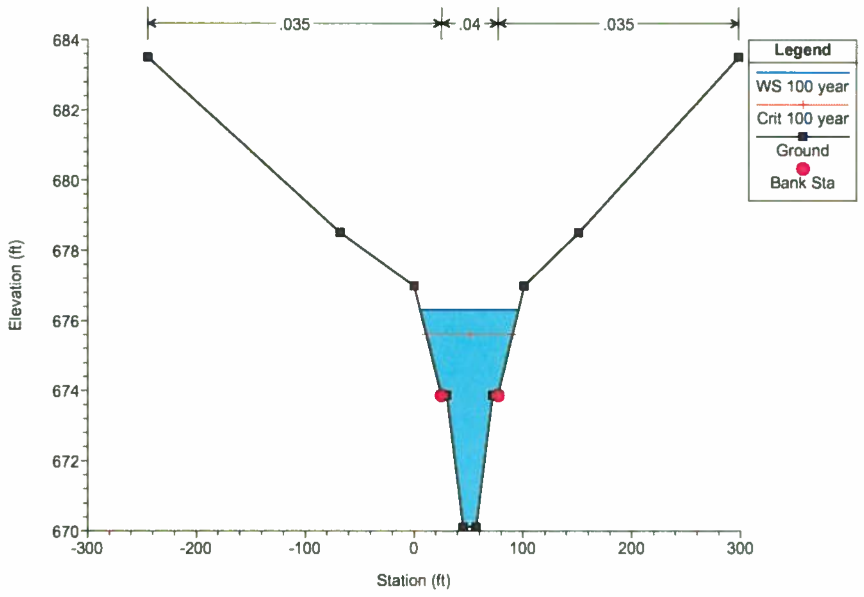


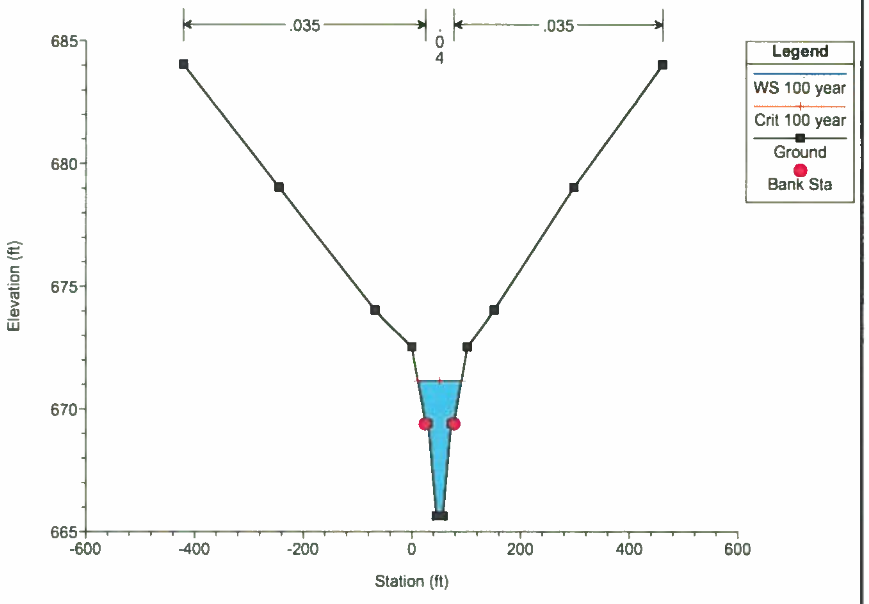
River = Stingy Creek   Reach = Spill-Still-Exit   RS = 1

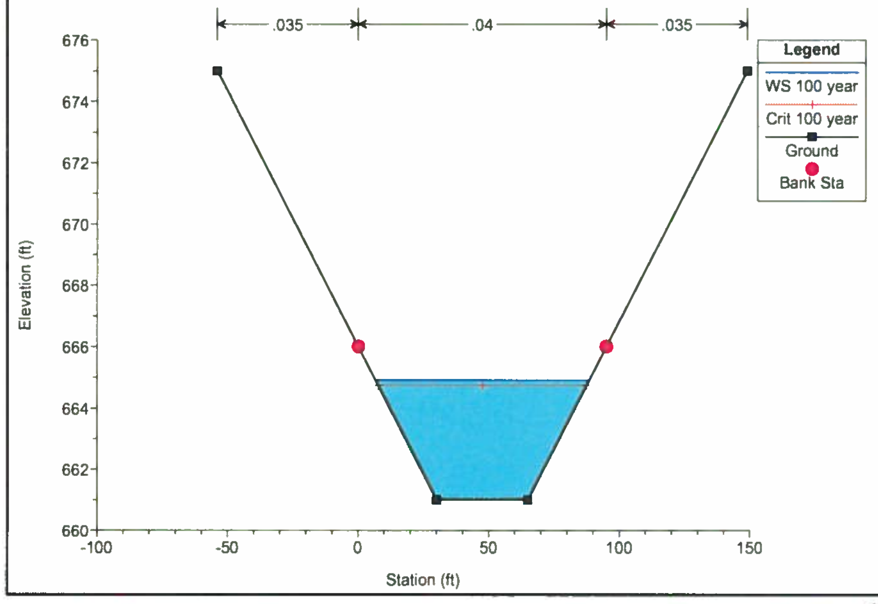


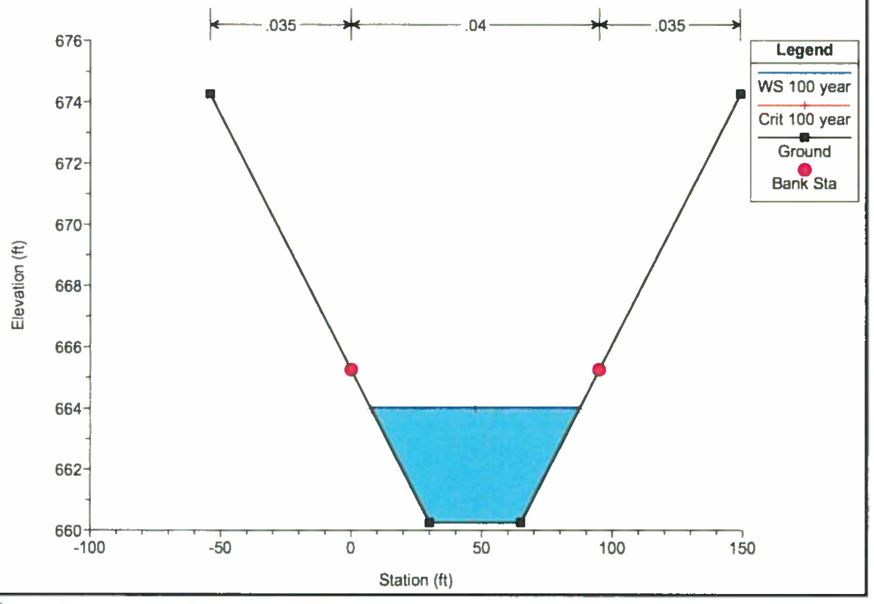
River = Stingy Creek   Reach = Spill-Still-Exit   RS = 0 Final x-sec just upstream of last culvert

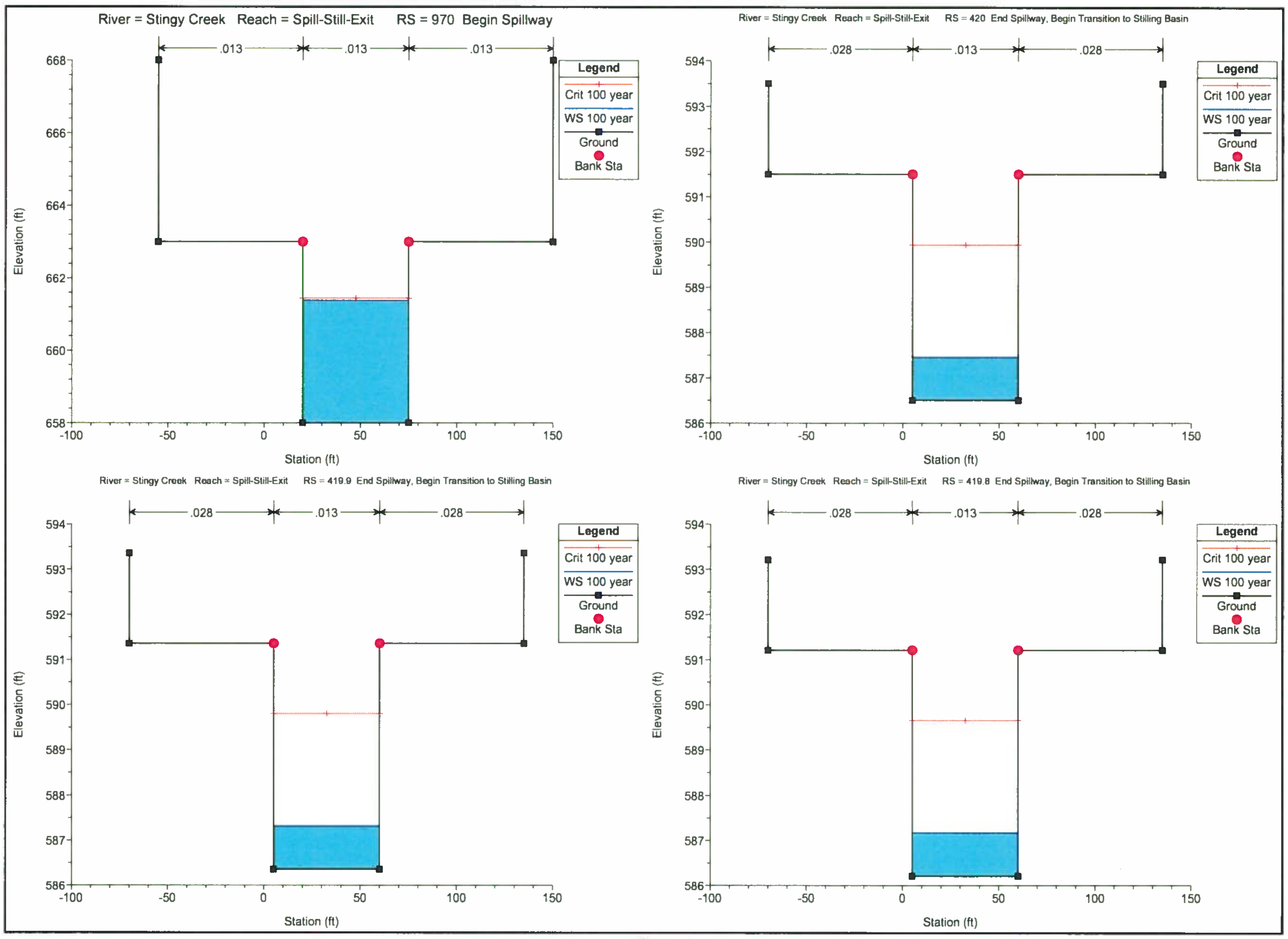



River = Stingy Creek Reach = Spill-Still-Exit RS = 2230 Type 1 Channel Begin


River = Stingy Creek Reach = Spill-Still-Exit RS = 1230 Type 1 Channel Begin Transition to non-tier trapazoidal


River = Stingy Creek Reach = Spill-Still-Exit RS = 1170 Non-Tier trapazoidal Transition


River = Stingy Creek Reach = Spill-Still-Exit RS = 1120 Non-Tier trapazoidal Transition to Rectangle

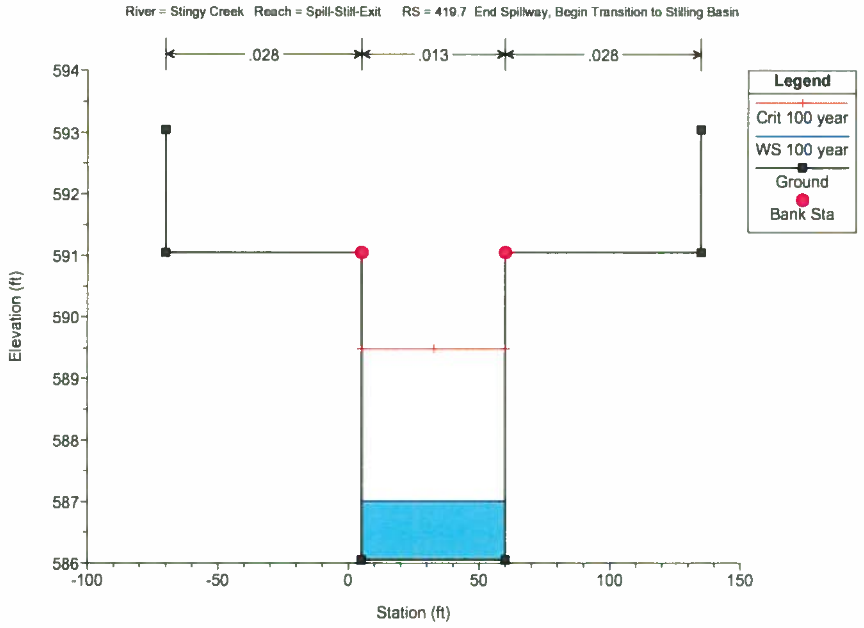


River = Stingy Creek Reach = Spill-Still-Exit RS = 970 Begin Spillway

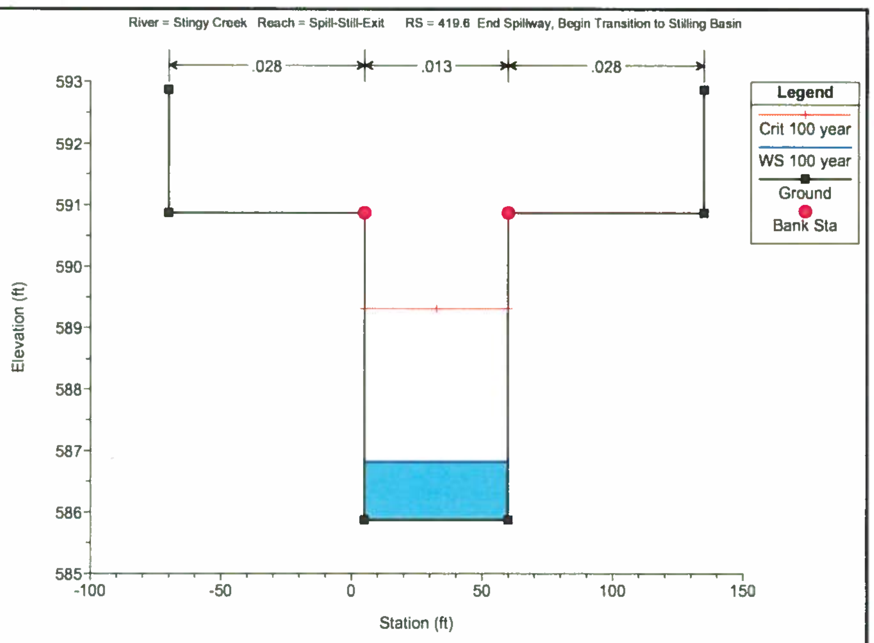
River = Stingy Creek Reach = Spill-Still-Exit RS = 420 End Spillway, Begin Transition to Stilling Basin

River = Stingy Creek Reach = Spill-Still-Exit RS = 419.9 End Spillway, Begin Transition to Stilling Basin

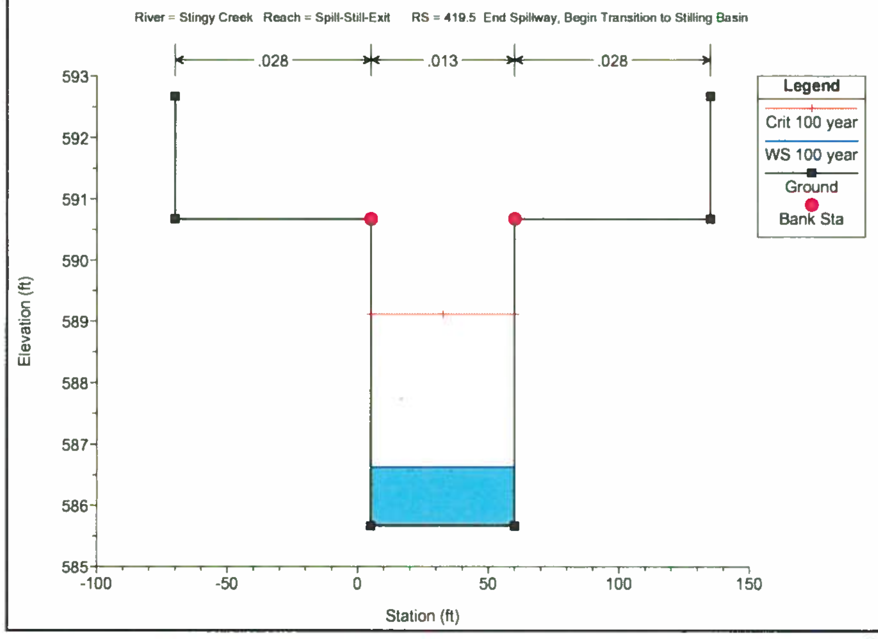
River = Stingy Creek Reach = Spill-Still-Exit RS = 419.8 End Spillway, Begin Transition to Stilling Basin



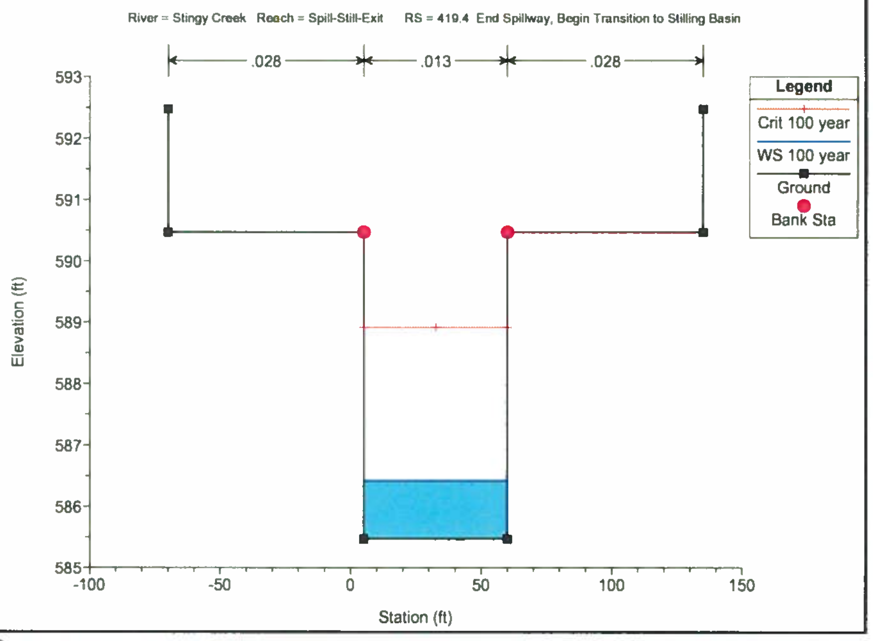
River = Stingy Creek Reach = Spill-Still-Exit RS = 419.7 End Spillway, Begin Transition to Stilling Basin



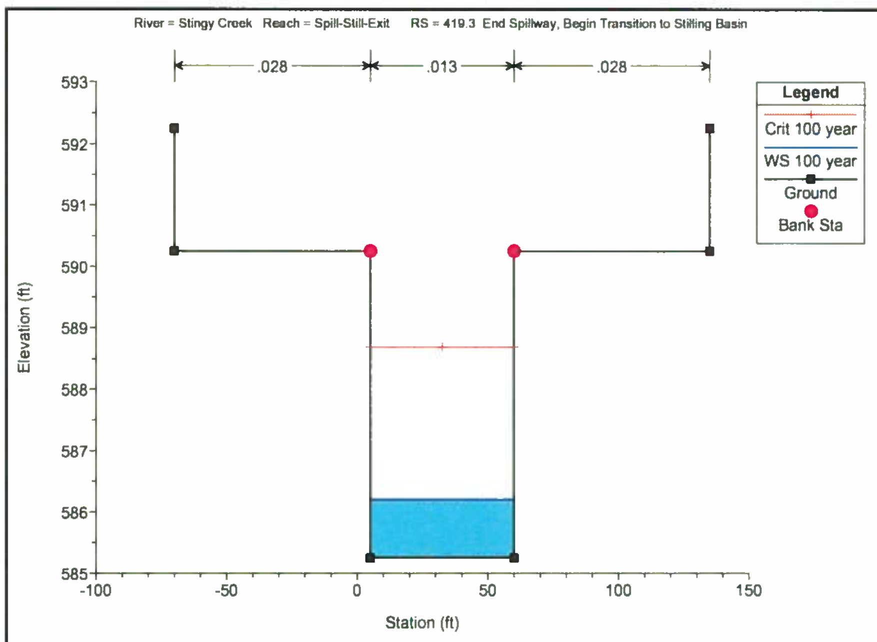
River = Stingy Creek Reach = Spill-Still-Exit RS = 419.6 End Spillway, Begin Transition to Stilling Basin

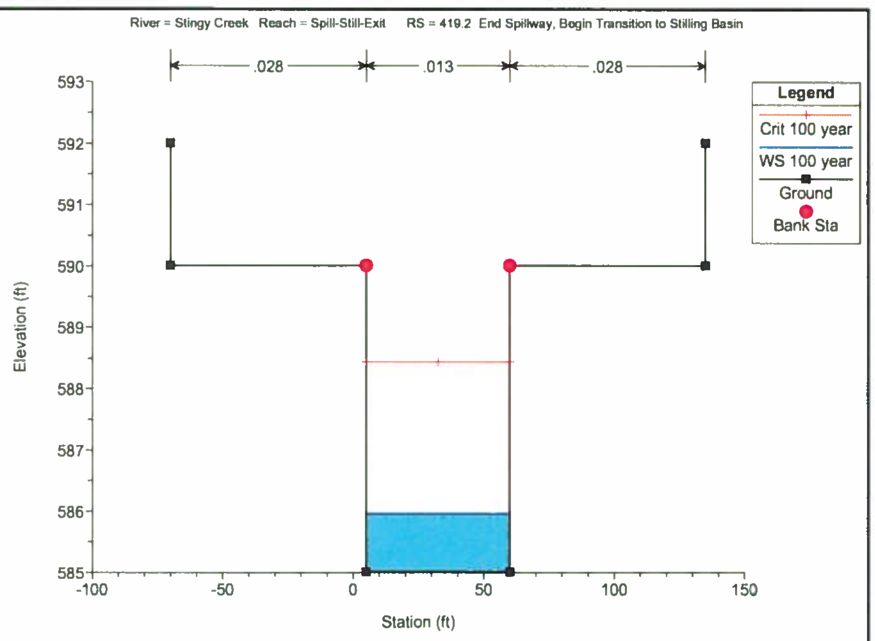


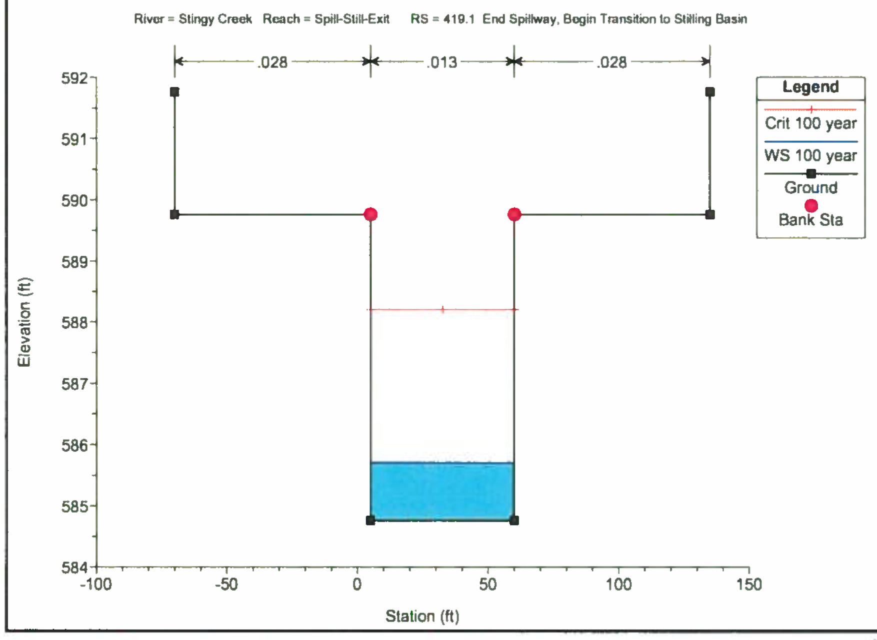
River = Stingy Creek Reach = Spill-Still-Exit RS = 419.5 End Spillway, Begin Transition to Stilling Basin

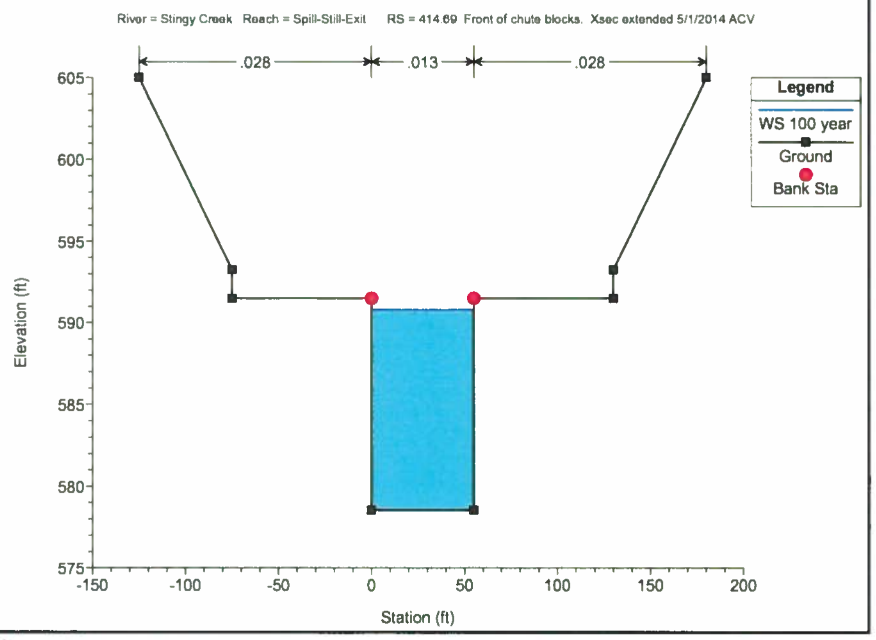


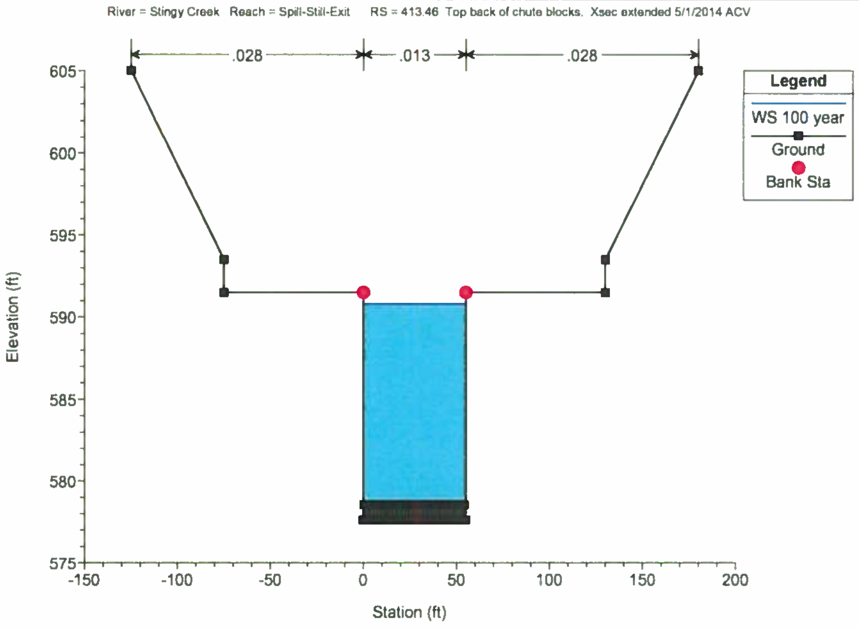
River = Stingy Creek Reach = Spill-Still-Exit RS = 419.4 End Spillway, Begin Transition to Stilling Basin

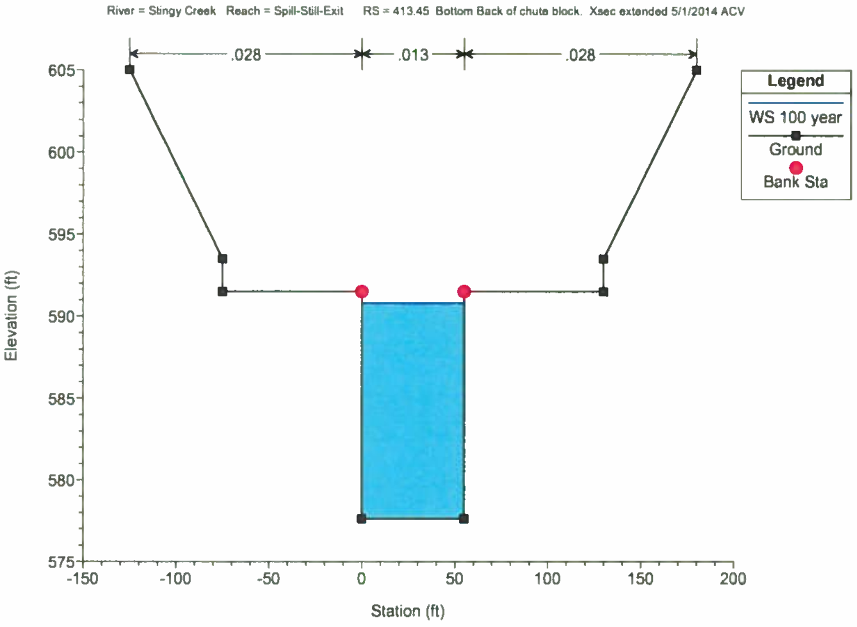

River = Stingy Creek Reach = Spill-Still-Exit RS = 419.3 End Spillway, Begin Transition to Stilling Basin
.028
.013
.028
Legend
Crit 100 year
WS 100 year
Ground
Bank Sta
Elevation (ft)
593
592
591
590
589
588
587
586
585
-100
-50
0
50
100
150
Station (ft)

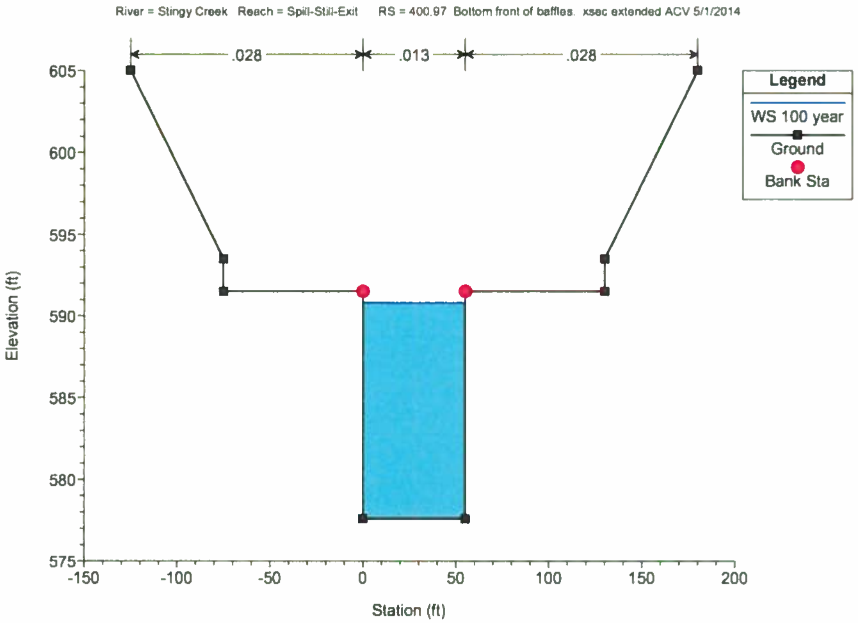

River = Stingy Creek Reach = Spill-Still-Exit RS = 419.2 End Spillway, Begin Transition to Stilling Basin
.028
.013
.028
Legend
Crit 100 year
WS 100 year
Ground
Bank Sta
Elevation (ft)
593
592
591
590
589
588
587
586
585
-100
-50
0
50
100
150
Station (ft)

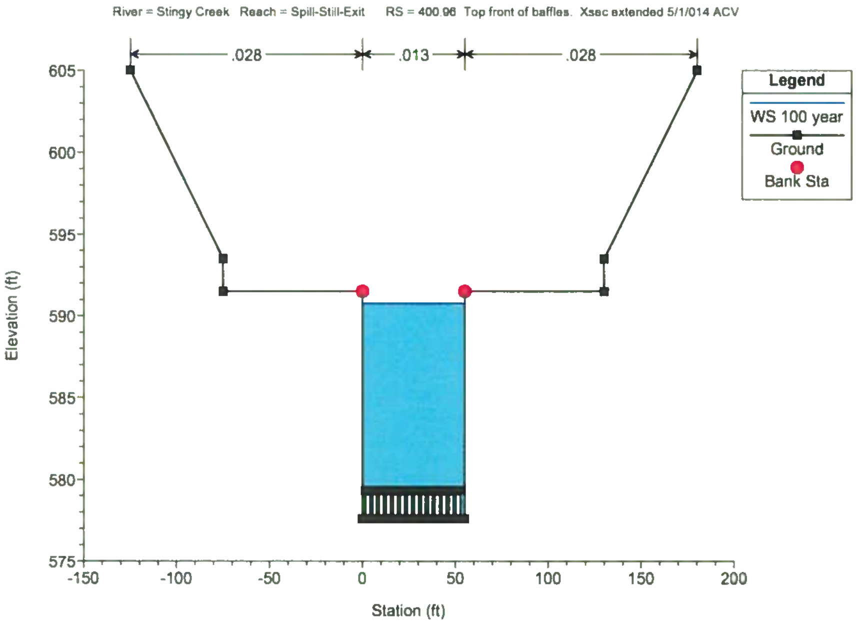

River = Stingy Creek Reach = Spill-Still-Exit RS = 419.1 End Spillway, Begin Transition to Stilling Basin
.028
.013
.028
Legend
Crit 100 year
WS 100 year
Ground
Bank Sta
Elevation (ft)
592
591
590
589
588
587
586
585
584
-100
-50
0
50
100
150
Station (ft)

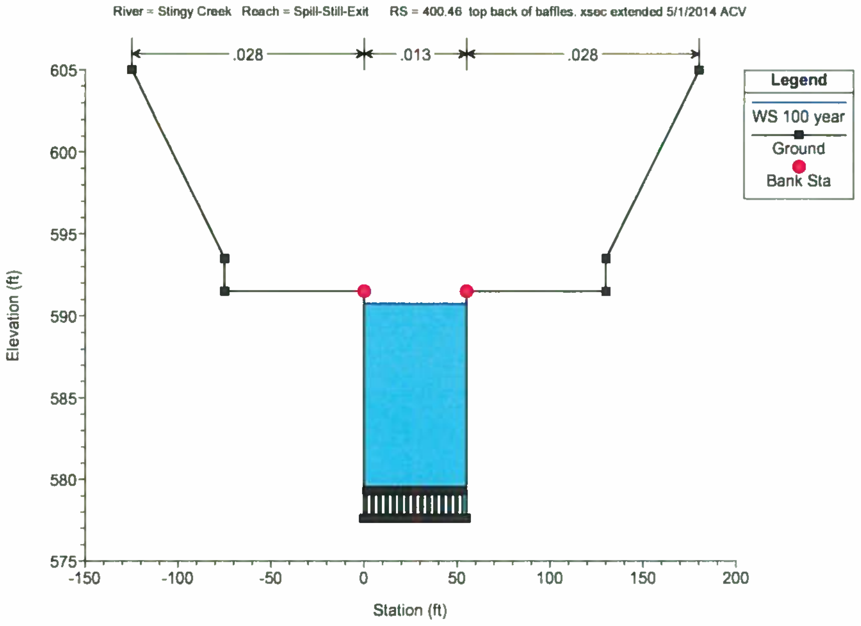

River = Stingy Creek Reach = Spill-Still-Exit RS = 414.69 Front of chute blocks. Xsec extended 5/1/2014 ACV
.028
.013
.028
Legend
WS 100 year
Ground
Bank Sta
Elevation (ft)
605
600
595
590
585
580
575
-150
-100
-50
0
50
100
150
200
Station (ft)

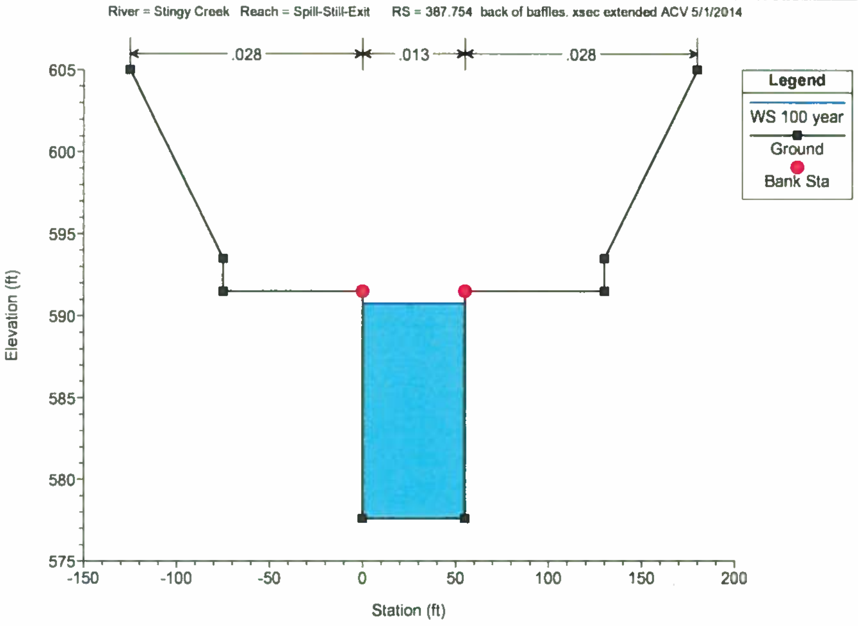

River = Stingy Creek   Reach = Spill-Still-Exit   RS = 413.46 Top back of chute blocks. Xsec extended 5/1/2014 ACV
.028
.013
.028
Legend
WS 100 year
Ground
Bank Sta
Elevation (ft)
605
600
595
590
585
580
575
-150
-100
-50
0
50
100
150
200
Station (ft)

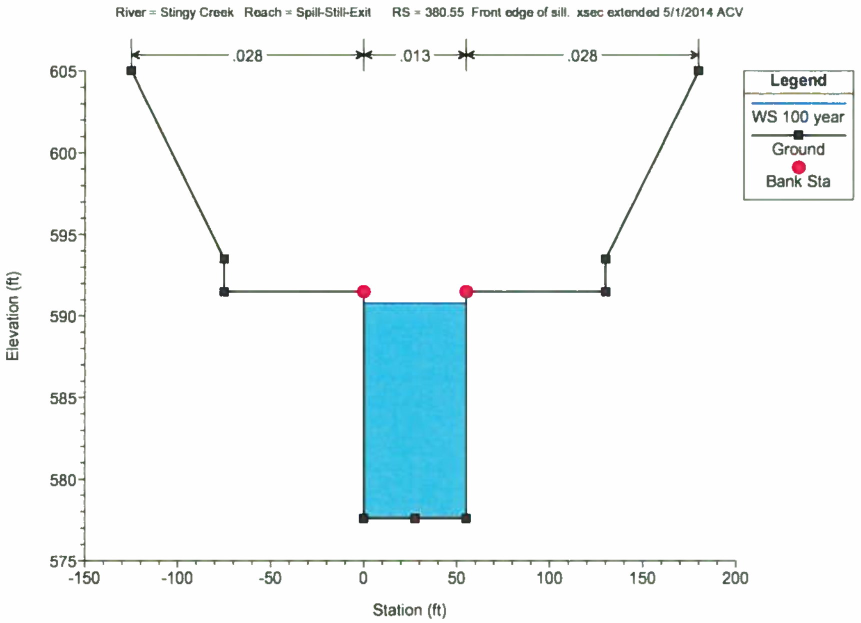

River = Stingy Creek   Reach = Spill-Still-Exit   RS = 413.45 Bottom Back of chute block. Xsec extended 5/1/2014 ACV
.028
.013
.028
Legend
WS 100 year
Ground
Bank Sta
Elevation (ft)
605
600
595
590
585
580
575
-150
-100
-50
0
50
100
150
200
Station (ft)

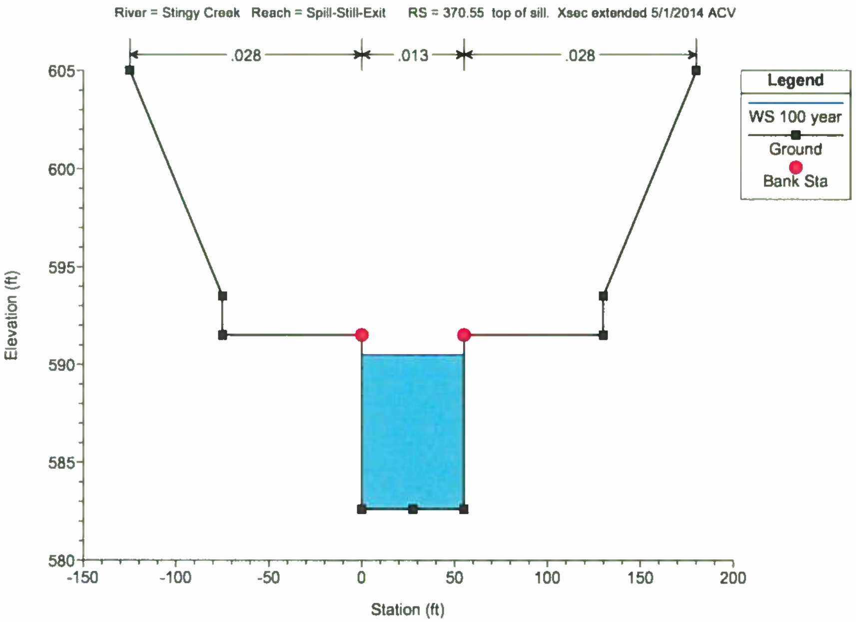

River = Stingy Creek   Reach = Spill-Still-Exit   RS = 400.97 Bottom front of baffles. xsec extended ACV 5/1/2014
.028
.013
.028
Legend
WS 100 year
Ground
Bank Sta
Elevation (ft)
605
600
595
590
585
580
575
-150
-100
-50
0
50
100
150
200
Station (ft)

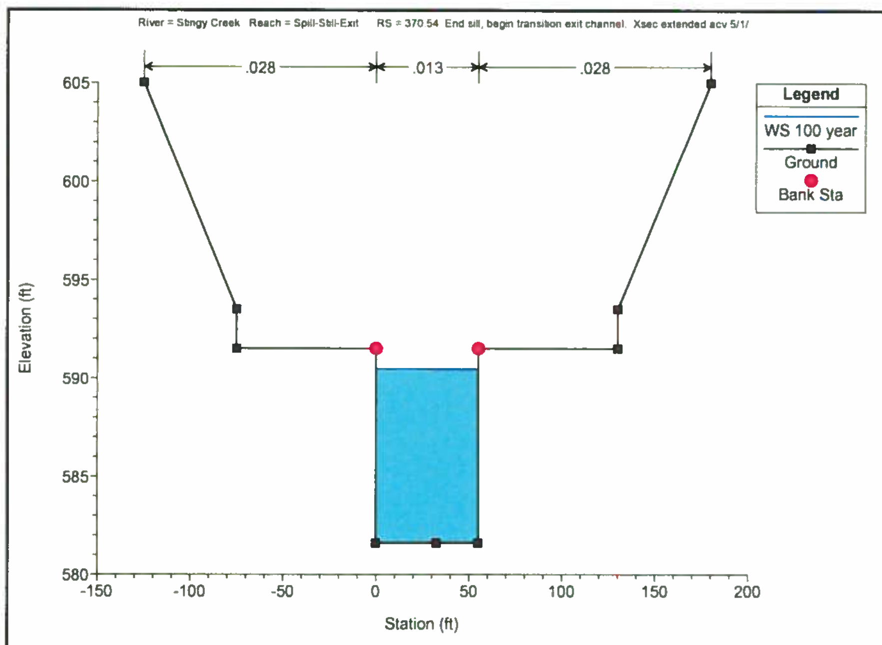

River = Stingy Creek   Reach = Spill-Still-Exit   RS = 400.96 Top front of baffles. Xsec extended 5/1/014 ACV
.028
.013
.028
Legend
WS 100 year
Ground
Bank Sta
Elevation (ft)
605
600
595
590
585
580
575
-150
-100
-50
0
50
100
150
200
Station (ft)


River = Stingy Creek Reach = Spill-Still-Exit RS = 400.46 top back of baffles. xsec extended 5/1/2014 ACV
.028
.013
.028
Legend
WS 100 year
Ground
Bank Sta
Elevation (ft)
605
600
595
590
585
580
575
-150
-100
-50
0
50
100
150
200
Station (ft)

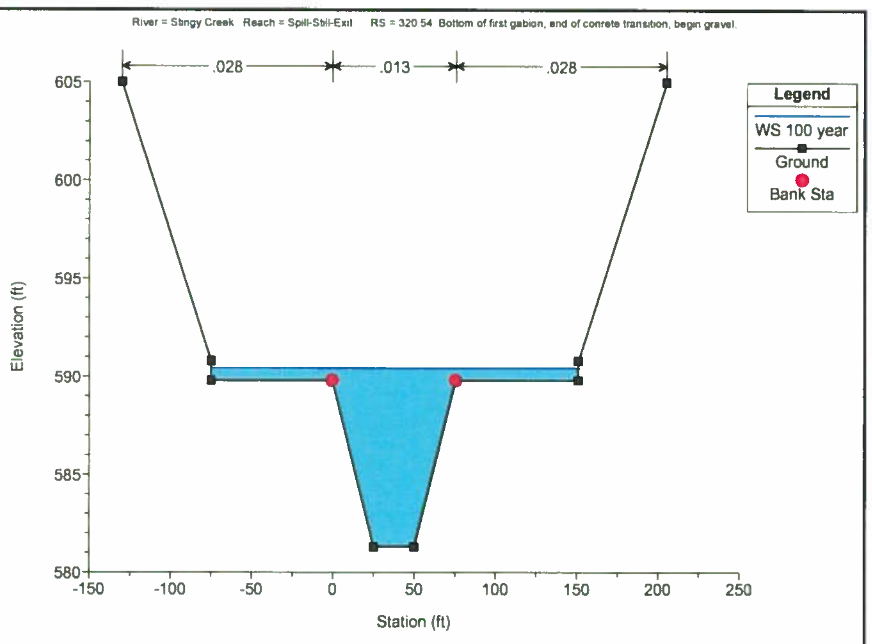

River = Stingy Creek Reach = Spill-Still-Exit RS = 387.754 back of baffles. xsec extended ACV 5/1/2014
.028
.013
.028
Legend
WS 100 year
Ground
Bank Sta
Elevation (ft)
605
600
595
590
585
580
575
-150
-100
-50
0
50
100
150
200
Station (ft)


River = Stingy Creek Reach = Spill-Still-Exit RS = 380.55 Front edge of sill. xsec extended 5/1/2014 ACV
.028
.013
.028
Legend
WS 100 year
Ground
Bank Sta
Elevation (ft)
605
600
595
590
585
580
575
-150
-100
-50
0
50
100
150
200
Station (ft)

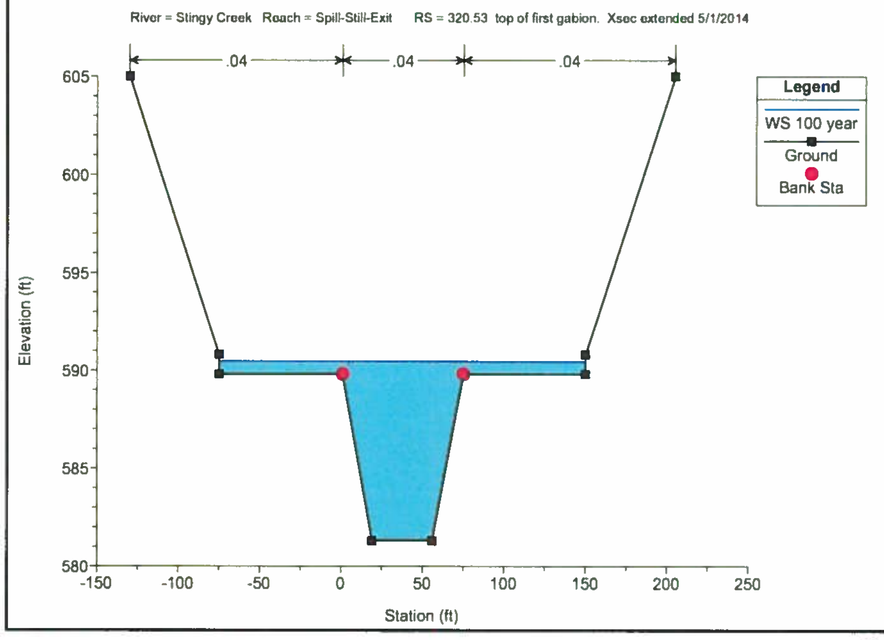

River = Stingy Creek Reach = Spill-Still-Exit RS = 370.55 top of sill. Xsec extended 5/1/2014 ACV
.028
.013
.028
Legend
WS 100 year
Ground
Bank Sta
Elevation (ft)
605
600
595
590
585
580
-150
-100
-50
0
50
100
150
200
Station (ft)



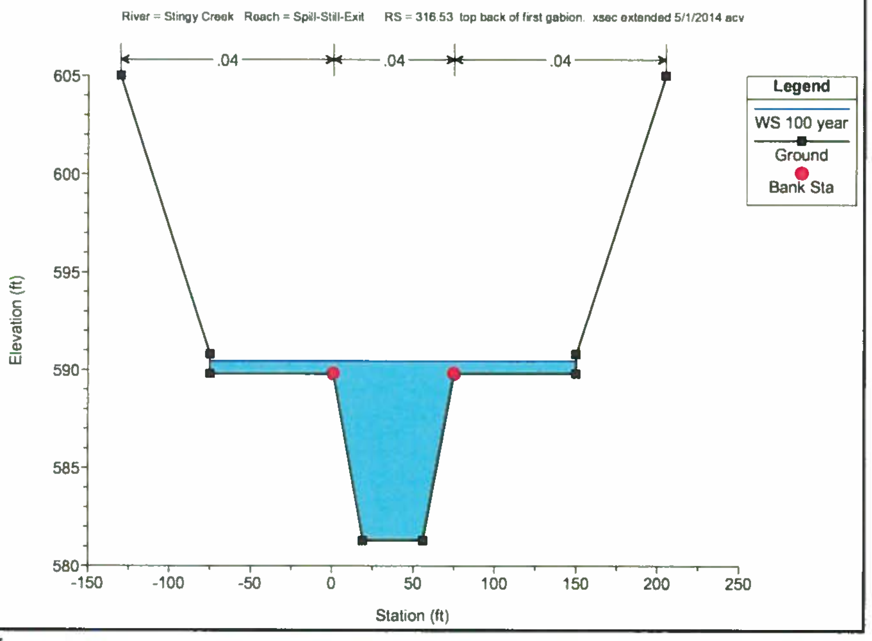
River = Stingy Creek Reach = Spill-Still-Exit RS = 370.54 End sill, begin transition exit channel. Xsec extended acv 5/1/



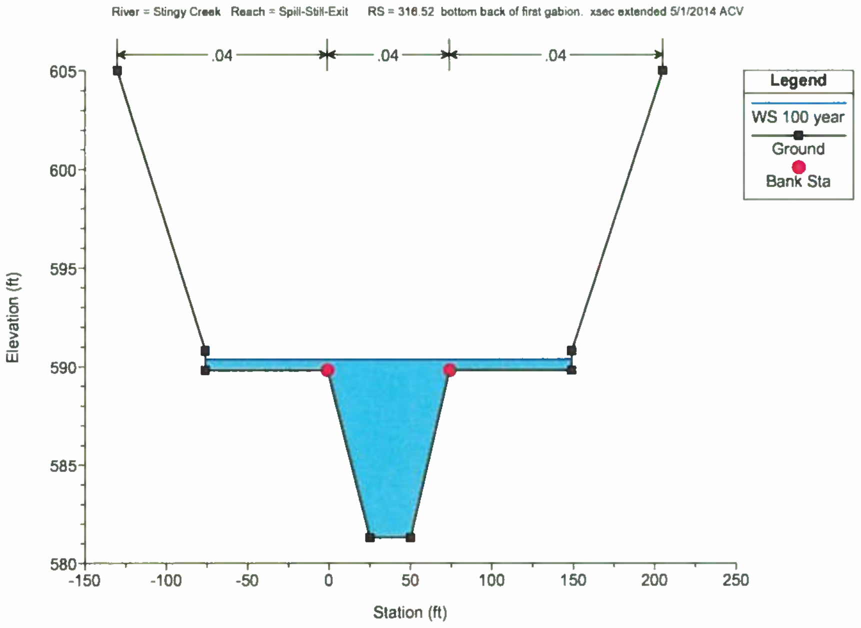
River = Stingy Creek Reach = Spill-Still-Exit RS = 320.54 Bottom of first gabion, end of conrete transition, begin gravel.

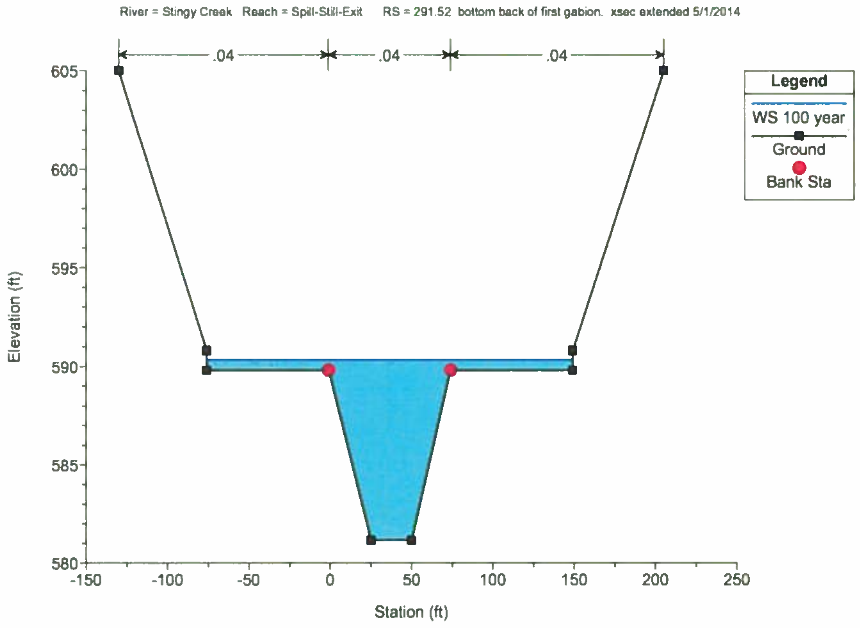


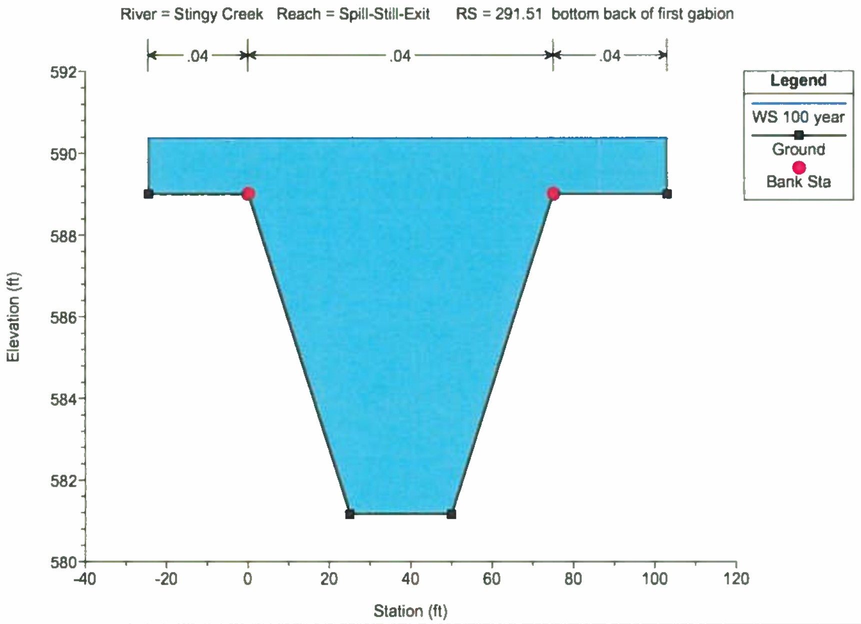
River = Stingy Creek Reach = Spill-Still-Exit RS = 320.53 top of first gabion. Xsec extended 5/1/2014

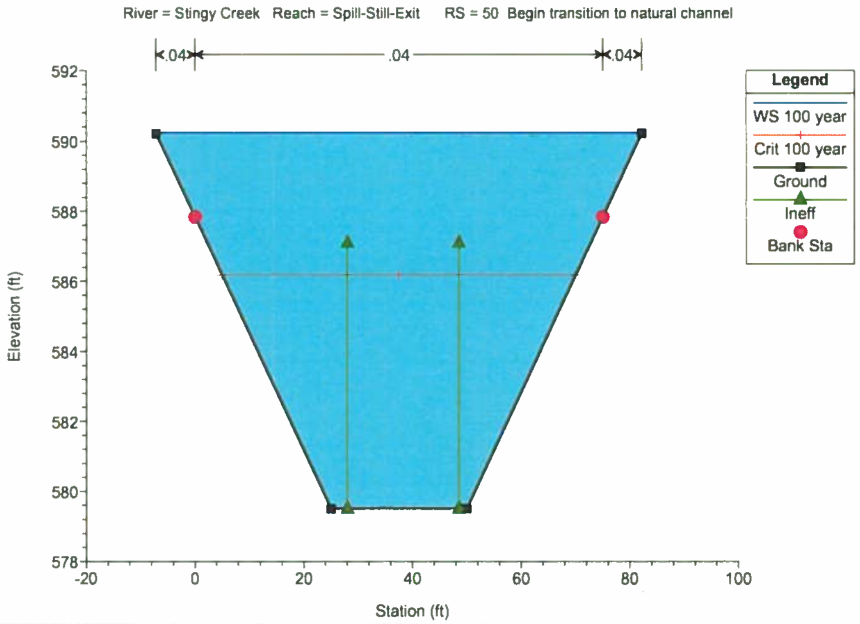


River = Stingy Creek Reach = Spill-Still-Exit RS = 316.53 top back of first gabion. xsec extended 5/1/2014 acv

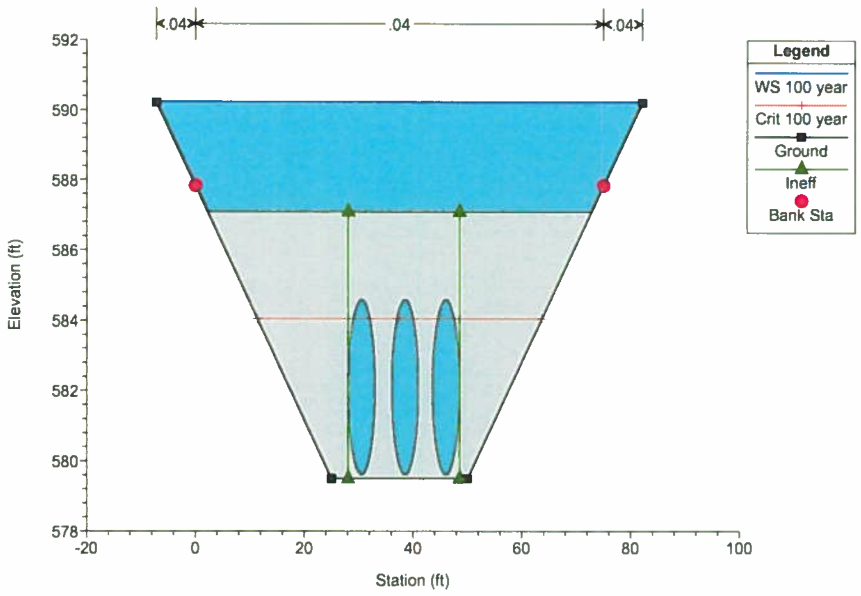

River = Stingy Creek Reach = Spill-Still-Exit RS = 316.52 bottom back of first gabion. xsec extended 5/1/2014 ACV


River = Stingy Creek Reach = Spill-Still-Exit RS = 291.52 bottom back of first gabion. xsec extended 5/1/2014

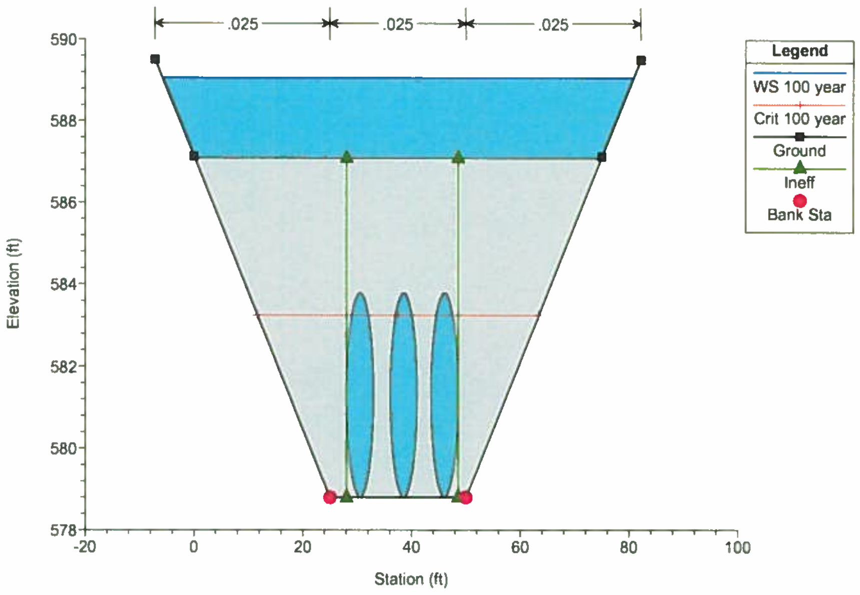

River = Stingy Creek Reach = Spill-Still-Exit RS = 291.51 bottom back of first gabion


River = Stingy Creek Reach = Spill-Still-Exit RS = 50 Begin transition to natural channel

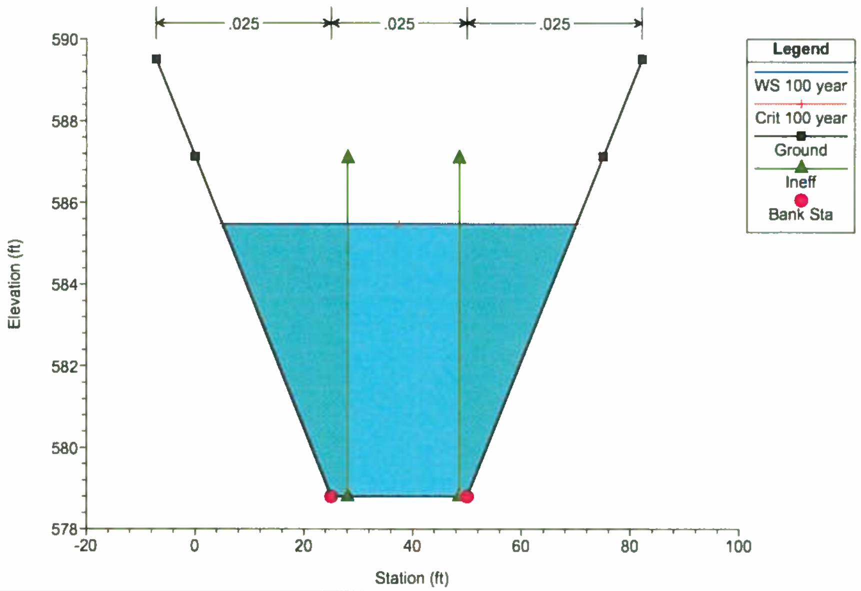
River = Stingy Creek   Reach = Spill-Still-Exit   RS = 49.99   Culv



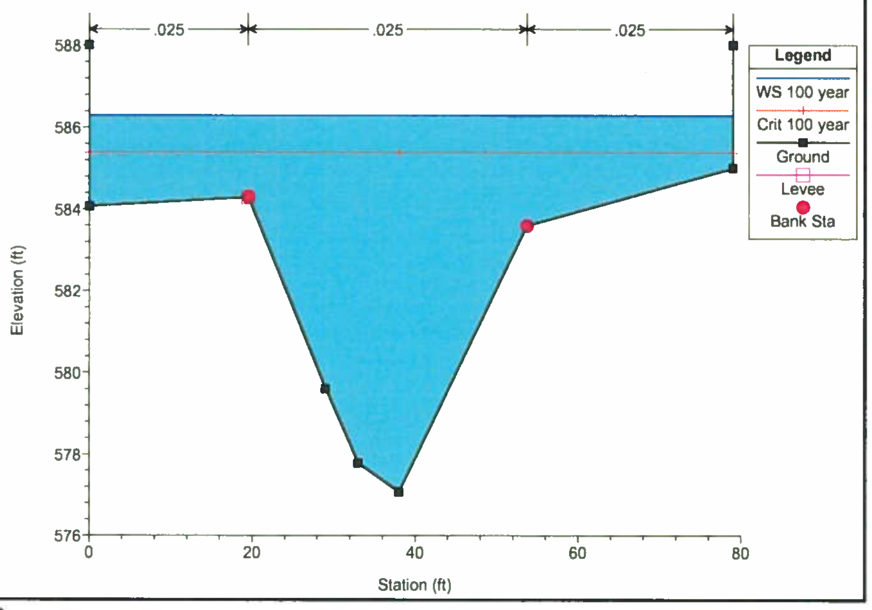
River = Stingy Creek   Reach = Spill-Still-Exit   RS = 49.99   Culv

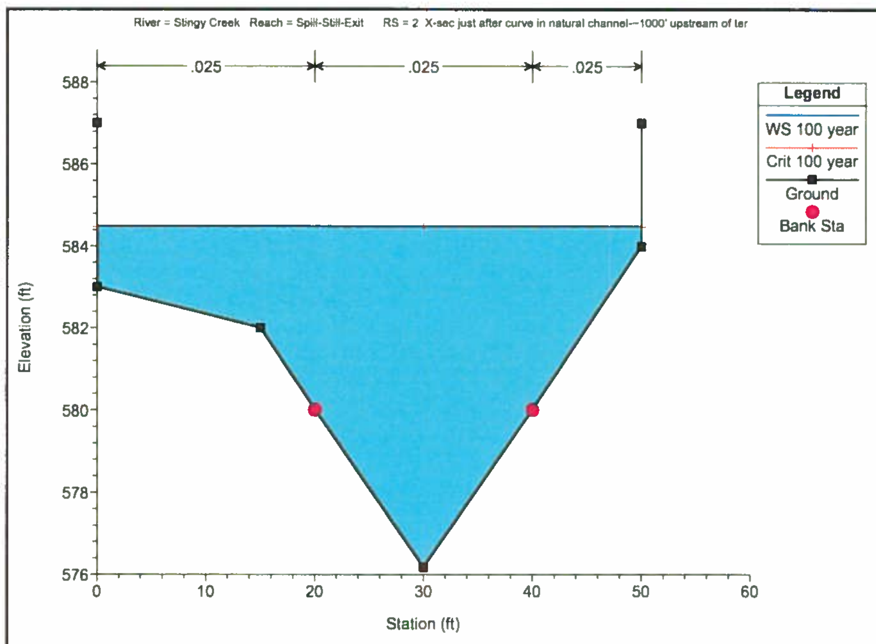


River = Stingy Creek   Reach = Spill-Still-Exit   RS = 3.1

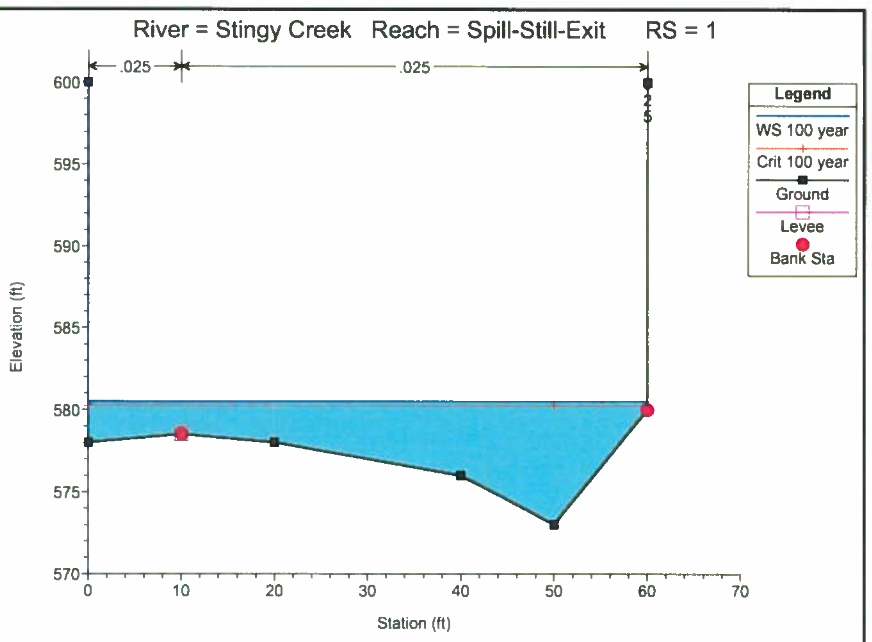


River = Stingy Creek   Reach = Spill-Still-Exit   RS = 3   X-sec just ds of first culvert, begin natural channel

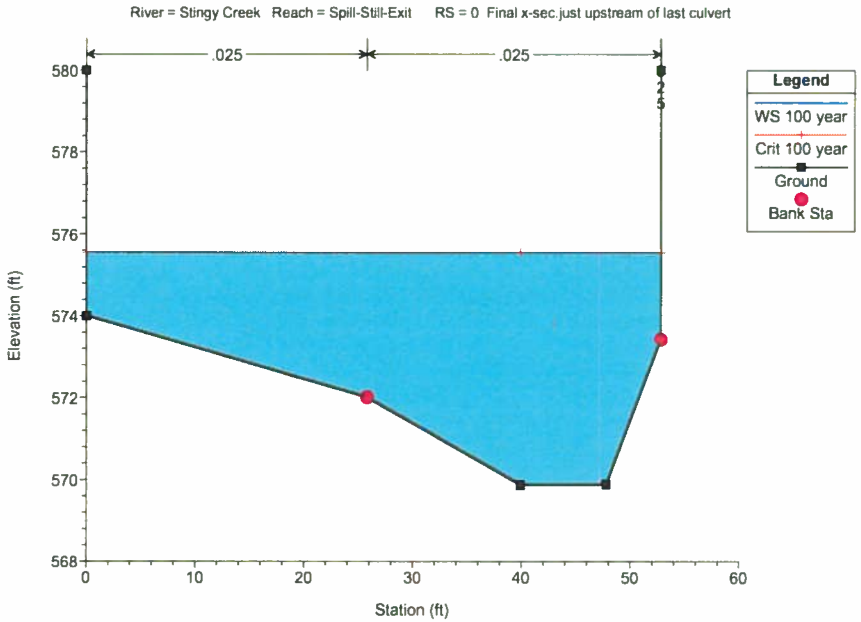




River = Stingy Creek   Reach = Spill-Still-Exit   RS = 2 X-sec just after curve in natural channel--1000' upstream of ter



River = Stingy Creek   Reach = Spill-Still-Exit   RS = 1



River = Stingy Creek   Reach = Spill-Still-Exit   RS = 0 Final x-sec just upstream of last culvert

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Reviewed by: RM **Date:** 11/6/2015

Client: AEP **Project:** FAR Closure **Project No.:** CHE8273 **Task No.:** 02/05.22

ATTACHMENT B

Stilling Basin Type III Design Calculations and Schematic, USBR, 1987

Geosyntec consultants

Attachment B

1420 Kensington Road, Suite 103
Oak Brook, IL
TELEPHONE (630) 203 3340
FAX (630) 203 3341

JOB CHE8273-FAR Closure
SHEET NO 1 OF 2
CALCULATED BY Rishab Mahajan DATE 10/1/2015
CHECKED BY Jennifer Jenkins DATE 10/1/2015
DESCRIPTION Stilling Basin and Conveyance Channel Calculation Sheet

Inputs taken from previous sources	Value	units	Source
Distance between spillway begin and edge of dam	350	ft	scaled off of Figure 2 of downstream analysis
Distance between edge of dam and tie in to natural channel	630	ft	scaled off of Figure 2 of downstream analysis
Elevation/slopes/lengths/cross sections of upstream channels and spillway			Read off of Figure 8273-126 (Details of dam configuration) and 8273-123 (Details Cover System)
Sump elevation at tie in to natural channel	577.07		Taken from Figure 3 of downstream analysis
X-sections of natural channel			Taken from Figure 3 and 4 of downstream analysis and estimated from topography where necessary
Length, exit slope, n of downstream channel			Taken from SWMM analysis of downstream analysis
Manning's n (Riprap d50=1.0')	0.040		From HEC-11 equation 20 n=0.0395*D^0.167
Manning's n (Earthen)	0.025		From SWMM model for natural area
Manning's n (Turf protection grass)	0.035		From cover swale design
Manning's n (Concrete)	0.013		From cover swale design
Manning's n fabriform mat	0.028		Manufacturer's recommendation: fabriform1.com/assets/fabriform.pdf
Flow Rate from spillway	2000	cfs	From Cover swale design spreadsheet
Depth of flow in spillway	0.94	ft	From spreadsheet used for sizing spillway
Velocity in spillway	38.6	ft/sec	From spreadsheet used for sizing spillway
Froude number in spillway	7.04		From spreadsheet used for sizing spillway and checked by calculation

Stilling Basin Design

Using Type III stilling basin design see figure			From USBR, 1987 see figure
Max allowable velocity	60	ft/sec	USBR, 1987
Horizontal offset from bottom of spillway to transition to stilling basin. Continue	20	ft	Set design parameter
Slope of transition from spillway to stilling basin	4:1	H:V	USBR, 1987
Elevation drop from end of extended spillway to bottom of basin	12	ft	Design parameter
Depth of hydraulic jump (and depth of stilling basin), d2	8.9	ft	d2=d1*0.5*((1+8*Fr^2)^0.5-1)
Width of basin, W	55	ft	Design parameter/Match Spillway Width for Consistancy
Length of basin, L	22.9	ft	Read from chart in Fig 9-41 (L/d2=2.75, d2=15.6)
chute block offset from wall	0.47	ft	From diagram (d1/2)
Chute block height,width, and space between chute blocks	0.94	ft	From diagram (d1)
Distance to row of baffles	7.12	ft	From diagram (0.8*d2)
Baffle height (h3)	1.7	ft	(From chart in Fig 9-41 (h3/d1=2.2)
Baffle distance from wall	0.6375	ft	From diagram (0.375*h3)
Baffle width and space between baffles	1.275	ft	From diagram (0.75*h3)
Baffle top length	0.34	ft	From diagram (0.2*h3)
Baffle length	1.7	ft	From diagram 1:1
distance to front of sill	11.108		Based on 2:1 sill slope and total length of basin
Sill height (h4)	1.316	ft	Analyzed Parameter
Elevation drop to channel	1	ft	Based on riprap d50=2', USACE energy dissipator manual recommends 0.25-0.5 of d100. Assume d100 is about 3', so 1' is between 0.25'-0.5'
Flare width after basin	5	ft	USACE energy dissipator manual recommends minimum of 5' flare width

Stilling Basin Exist Transition		
Flare width after basin, each side	5	ft
Transition apron length	50	ft
Transition apron top width at sill	65	ft
Transition apron bottom width at sill	55	ft
Transition apron bottom width at gabion check structure (downstream end)	25	ft
Side slopes at downstream end	3:1	H:V
Transition Apron Shape	Trapezoidal	
Transition apron material	Concrete	
Note, this area gradually and continuously transitions to a 25ft bottom width		

Riprap Exit Channel Design

Max velocity in channel at exit	6.4	ft/sec	HECRAS Output
Max hydraulic depth in channel at exit	4.53	ft	HECRAS Output
Max shear stress in channel at exit	0.12	lb/sq ft	HECRAS Output
Angle of repose	42	degrees	From Chart 4, for d50 of 12" or 24"
angle with 3:1 side slopes	18.435	degrees	tan^-1 (1/3)*180/pi
Angle of repose	0.733038286	radians	From Chart 4, for d50 of 12" or 24"
angle with 3:1 side slopes	0.321751448	radians	tan^-1 (1/3)
K1 constant for equation 6 for d50 in HEC-11	0.881279063		From equation 7 in HEC-11
Shield's equation constant, tc*	0.054		From Fischenrich, 2001
Max allowable shear stress based on Shield's equation for 1' d50	5.5404	lb/sq ft	Shield Equation
Specific gravity of riprap	2.64	g/cm3	Assumed
Calculated d50 according to HEC-11	0.67	ft	d50: 0.001 *V^3 / (d^0.5 * K1^0.5)
Calculated d50 according to HEC-14	0.27	ft	d50: 0.692/(S-1) * (V^2/2g)
Ohio Type C Rip Rap/AASTO Light Facing	1	ft	HEC-11 Table 3 which references AASHTO gradations

* Noted: Shear stresses acceptable throughout for 1.0' d50; sheer stress acceptable for 0.5' but velocity is not acceptable

4	ft/2	Max velocity for 2.5" stone based on Isbach Curve
6.25	ft/s	Max velocity for 6" stone based on Isbach Curve
10	ft/s	Max velocity for 12" stone based on Isbach Curve

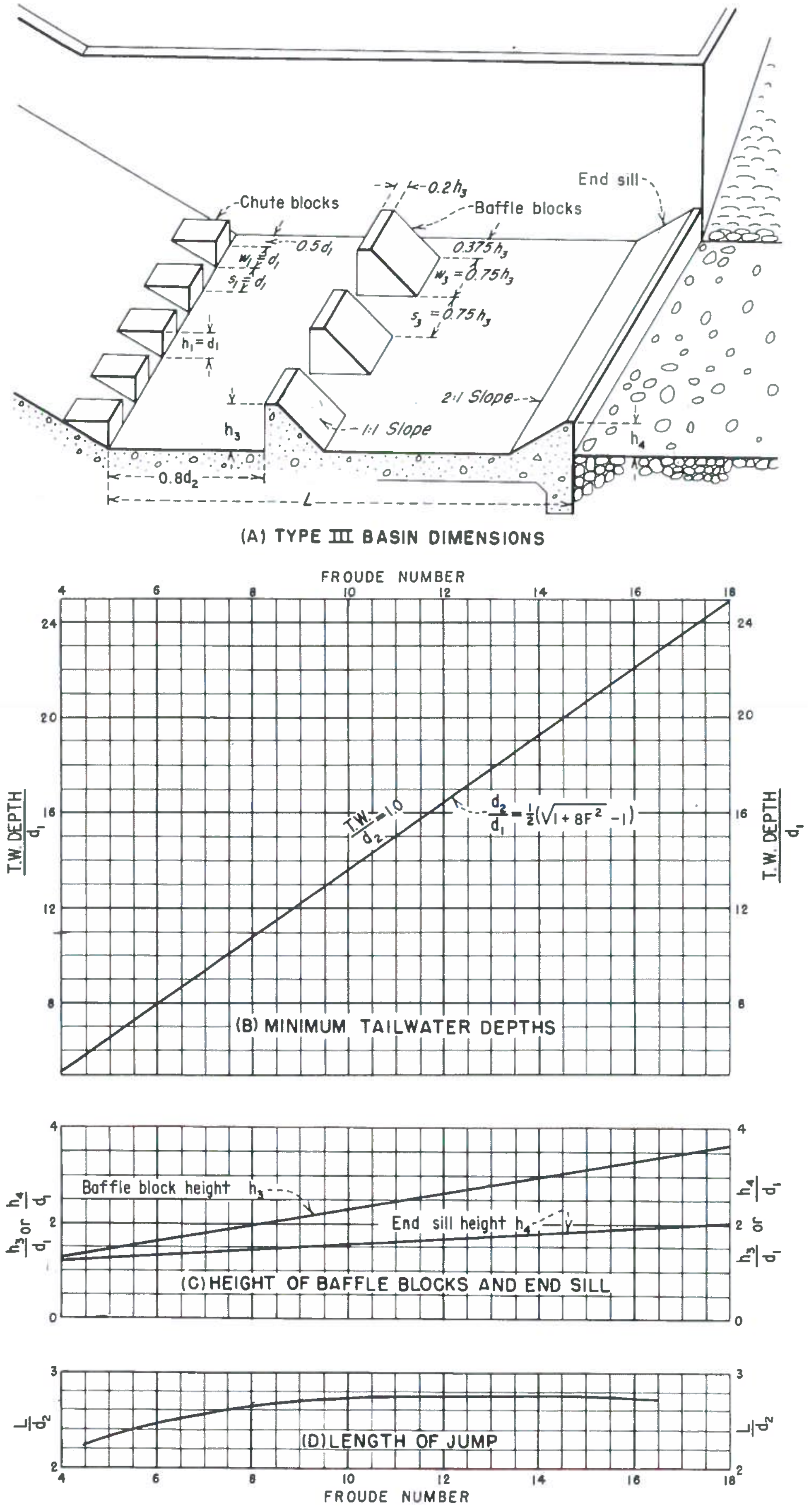

Figure 9-41.—Stilling basin characteristics for Froude numbers above 4.5 where incoming velocity, $V_1 \leq$ 60 ft/s. 288–D–2426.

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Reviewed by: RM **Date:** 11/6/2015

Client: AEP **Project:** FAR Closure **Project No.:** CHE8273 **Task No.:** 02/05.22

ATTACHMENT C

Excerpts from HEC-11 Rip-Rap Sizing Guidance

4. DESIGN GUIDELINES FOR ROCK RIPRAP

As defined in chapter 2, rock riprap consists of a well graded mixture of rock, broken concrete, or other material, dumped or hand placed to prevent erosion, scour, or sloughing of a structure or embankment. In the context of this chapter, the term rock riprap is used to refer to both rock and rubble riprap.

Rock riprap is the most widely used and desirable type of revetment in the United States. The term "riprap" connotes rock riprap. The effectiveness of rock riprap has been well established where it is properly installed, of adequate size and suitable gradation. Riprap materials include quarry-run rock, rubble, or other locally available materials. Performance characteristics of rock and rubble riprap are reviewed in section 2.1.1.

This chapter contains design guidelines for the design of rock riprap. Guidelines are provided for rock size, rock gradation, riprap layer thickness, filter design, material quality, edge treatment, and construction considerations. In addition, typical construction details are illustrated. In most cases, the guidelines presented apply equally to rock and rubble riprap. Sample specifications for rock riprap are included in appendix A.

4.1 ROCK SIZE

The stability of a particular riprap particle is a function of its size, expressed either in terms of its weight or equivalent diameter. In the following sections, relationships are presented for evaluating the riprap size required to resist particle and wave erosion forces.

4.1.1 Particle Erosion

In chapter 1, riprap failure modes were identified as particle erosion, translational slide, modified slump, and slump. Translational slide, modified slump, and slump are slope or soils processes. Particle erosion is a hydraulic phenomenon which results when the tractive force exerted by the flowing water exceeds the riprap materials ability to resist motion. It is this process that the riprap design relationships presented in this section were developed for.

Two methods or approaches have been used historically to evaluate a materials resistance to particle erosion. These methods are the permissible velocity approach and the permissible tractive force (shear stress) approach. Under the permissible velocity approach the channel is assumed stable if the computed mean velocity is lower than the maximum permissible velocity. The tractive force (boundary shear stress) approach focuses on stresses developed at the interface between flowing water and materials forming the channel boundary. By Chow's definition, permissible tractive force is the maximum unit tractive force that will not cause serious erosion of channel bed material from a level channel bed (9). Permissible tractive force methods are generally considered to be more academically correct; however, critical velocity approaches are more readily embraced by the engineering community.

4.1.1.1 Design Relationship

A riprap design relationship that is based on tractive force theory yet has velocity as its primary design parameter is presented in equation 6. The design relationship in equation 6 is based on the assumption of uniform, gradually varying flow. The derivation of equation 6 along with a comparison with other methods is presented in appendix D. Chart 1 in appendix C presents a graphical solution to equation 6. Equation 7 can be solved using charts 3 and 4 of appendix C.

$$D_{50} = 0.001\ V_a^3 / (d_{avg}^{0.5}\ K_1^{1.5}) \qquad (6)$$

where

D_{50} = the median riprap particle size;
C = correction factor (described below);
V_a = the average velocity in the main channel (ft/s (m/s));
d_{avg} = the average flow depth in the main flow channel (ft (m)); and
K_1 is defined as:

$$K_1 = [1-(\sin^2\theta/\sin^2\phi)]^{0.5} \qquad (7)$$

where

θ = the bank angle with the horizontal; and
ϕ = the riprap material's angle of repose.

The average flow depth and velocity used in equation 6 are main channel values. The main channel is defined as the area between the channel banks (see Figure 17).

Equation 6 is based on a rock riprap specific gravity of 2.65, and a stability factor of 1.2. Equations 8 and 9 present correction factors for other specific gravities and stability factors.

$$C_{sg} = 2.12 / (S_s - 1)^{1.5} \qquad (8)$$

where

S_s = the specific gravity of the rock riprap.

$$C_{sf} = (SF / 1.2)^{1.5} \qquad (9)$$

where

SF = the stability factor to be applied.

The correction factors computed using equations 8 and 9 are multiplied together to form a single correction factor C. This correction factor, C, is then multiplied by the riprap size computed from equation 6 to arrive at a stable riprap size. Chart 2 in appendix C provides a solution to equations 8 and 9 using correction factor C.

The stability factor, SF, used in equations 6 and 9 requires additional explanation. The stability factor is defined as the ratio of the average tractive force exerted by the flow field and the riprap materials critical shear stress. As long as the stability factor is greater than 1, the critical shear stress of the material is greater than the flow induced tractive stress, the riprap is considered to be stable. As mentioned above, a stability factor of 1.2 was used in the development of equation 6.

The stability factor is used to reflect the level of uncertainty in the hydraulic conditions at a particular site. Equation 6 is based on the assumption of uniform or gradually varying flow. In many instances, this assumption is violated or other uncertainties come to bear. For example, debris and/or ice impacts, or the cumulative effect of high shear stresses and forces from wind and/or boat generated waves. The stability factor is used to increase the design rock size when these conditions must be considered. Table 1 presents guidelines for the selection of an appropriate value for the stability factor.

Table 1. Guidelines for the selection of stability factors

Condition	Stability Factor Range
Uniform flow; Straight or mildly curving reach (curve radius/channel width > 30); Impact from wave action and floating debris is minimal; Little or no uncertainty in design parameters.	1.0 - 1.2
Gradually varying flow; Moderate bend curvature (30 > curve radius/channel width > 10); Impact from waves or floating debris moderate.	1.3 - 1.6
Approaching rapidly varying flow; Sharp bend curvature (10 > curve radius/channel width); Significant impact potential from floating debris and/or ice; Significant wind and/or boat generated waves (1 - 2 ft (.30 - .61 m)); High flow turbulence; Turbulently mixing flow at bridge abutments; Significant uncertainty in design parameters.	1.6 - 2.0

4.1.1.2 Application

Application of the relationship in equation 6 is limited to uniform or gradually varying flow conditions. That is in straight or mildly curving channel reaches of relatively uniform cross section. However, design needs dictate that the relationship also be applicable in nonuniform, rapidly varying flow conditions often exhibited in natural channels with sharp bends and steep slopes, and in the vicinity of bridge piers and abutments.

Research efforts to define stable riprap size relationships for nonuniform, rapidly varying flow conditions have been limited. Recently work by Wang and Shen (35) and Maynord (36)has shed some light on the variability of the Shields parameter for large particle sizes in high Reynold's Number flows. However, no definitive relationship has been presented.

To fill the need for a design relationship that can be applied at sharp bends and on steep slopes in natural channels, and at bridge abutments, it is recommended that equation 6 be used with appropriate adjustments in velocity and/or stability factor as outlined in the following sections.

Channel Bends: At channel bends modifications to the stability factor are recommended based on the ratio or curve radius to channel width (R/W) as indicated in the following:

R/W	Stability Factor
> 30	1.2
30 > R/W > 10	1.3 - 1.6
< 10	1.7

Steep Slopes: Flow conditions in steep sloped channels are rarely uniform, and are characterized by high flow velocities and significant flow turbulence. In applying equation 6 to steep slope channels, care must be exercised in the determination of an appropriate velocity. When determining the flow velocity in steep sloped channels, it is recommended that equation 4 be used to determine the channel roughness coefficient. It is also important to thoughtfully consider the guidelines for selection of stability factors as presented in Table 1.

Bridge piers: The FHWA is currently evaluating various equations for selection of riprap at bridge piers. Present research indicates that velocities in the vicinity of the base of a pier can be related to the velocity in the channel upstream of the pier. For this reason, the interim procedure presented below is recommended for designing riprap at piers:

o Determine the D_{50} size of the riprap using the rearranged Ishbash equation to solve for stone diameter (in feet), for fresh water:

$$D_{50} = \frac{1}{2} \frac{1.384\ V^2}{(s-1)\ 2g} \qquad (10)$$

where:

D_{50} = average stone diameter (ft (m))
V = velocity against stone (ft/s (m/s))
s = specific gravity of riprap material
g = 32.2 ft/s^2 (9.81 m/s^2)

To calculate V, first determine the velocity of flow just upstream of the pier. This may be approximated by the velocity in the contracted section. Then multiply this value by a factor of 1.5 to 2.0 to approximate the velocity of flow at the base of the pier. Please note that preliminary research by FHWA indicates that a factor of about 1.5 may be a reasonable design value.

- Provide a mat width that extends horizontally at least two times the pier width measured from the pier face.
- Place the mat below the streambed a depth equivalent to the expected scour. The thickness should be three stone diameters or more.

Abutments: When applying equation 6 for riprap design at abutments a velocity in the vicinity of the abutment should be used instead of the average section velocity. The velocity in the vicinity of bridge abutments is a function of both the abutment type (vertical, wingwalled, or spillthrough), and the amount of constriction caused by the bridge. However, information documenting velocities in the vicinity of bridge abutments is currently unavailable. Until such information becomes available, it is recommended that equation 6 be used with a stability factor of 1.6 to 2.0 for turbulently mixing flow at bridge abutments.

Please take note that the average velocity and depth used in equation 6 for riprap design at bridge constrictions for abutment protection is the average velocity and depth in the constricted cross section at the bridge. Flow profiles at bridge sections are nonuniform as indicated in Figure 17. The recommended procedure for computing the average depth and velocity at bridge constrictions is:

1. Model the reach in the vicinity of the crossing using WSPRO (38), HEC-2 (39), or some other model with bridge loss routines.
2. Compute the average depth and velocity in the constriction as the average of the depth and velocity for modeled cross sections at the entrance to, and exit from the bridge constriction (in the vicinity of cross sections 2 and 3 as illustrated in Figure 18).

In instances where resources are not available to model flow conditions at the constriction as indicated above, normal depth and its associated flow velocity for the constricted section can be used.

As outlined above, the average section flow depth and velocity used in equation 6 are main channel values. The main channel is typically defined as the area between the channel banks (see Figure 17). However, when the bridge abutments are located on the floodplain a sufficient distance from the natural channel banks so as not to be influenced by main channel flows, the average depth and velocity on the floodplain within the constricted section should be used in the riprap design relationship. Most standard computerized bridge backwater routines provide the necessary depths and velocities as a part of their standard output. If hand normal depth computations are being used, the computations must consider conveyance weighted effects of both floodplain, and main channel flows. See reference 5 or standard open channel hydraulics texts for appropriate procedures.

When there is no overbank flow and the bridge spillthrough abutment on the channel bank matches the slope of the main channel banks upstream and downstream, use the design procedure without modification.

4.1.2 Wave Erosion

Waves generated by wind or boat traffic have also been observed to cause bank erosion on inland waterways. The most widely used measure of riprap's resistance to wave is that developed by Hudson (24). The so-called Hudson relationship is given by the following equation:

$$W_{50} = (\gamma_s H^3) / (2.20 [S_s - 1]^3 \cot\theta) \qquad (11)$$

where

H = the wave height; and the other parameters are as defined previously.

Assuming $S_s = 2.65$ and $\gamma_s = 165$ lb/ft^3 (kg/m^3), equation 11 can be reduced to:

$$W_{50} = 16.7\ H^3/\cot\theta \qquad (12)$$

In terms of an equivalent diameter equation 12 can be reduced to:

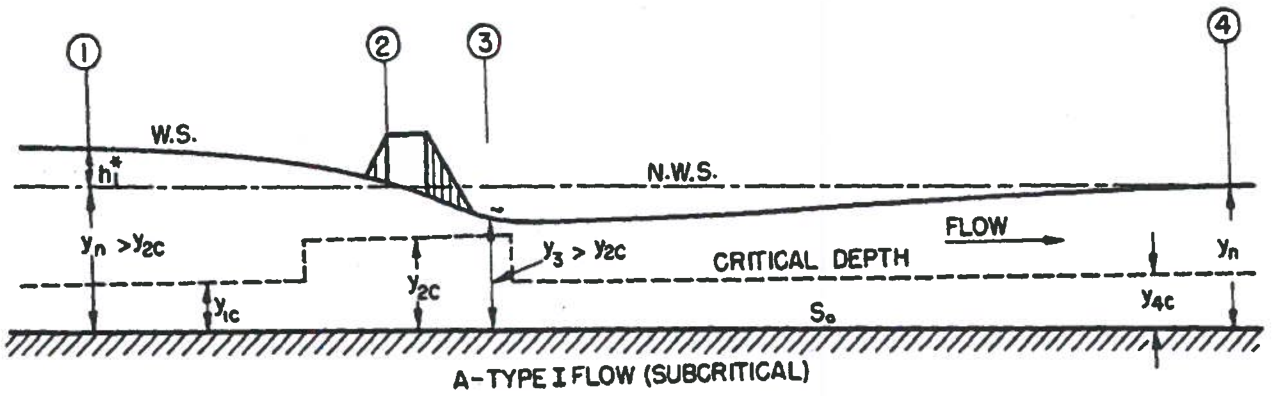
$$D_{50} = 0.75H/\cot^{1/3}\theta \qquad (13)$$

Methods for estimating a design wave height are presented in section 3.6.2. Equation 13 is presented in nomograph form in chart 7 of appendix C. Equations 12 and 13 can be used for preliminary or final design when H is less than 5 ft (1.52 m), and there is no major overtopping of the embankment.

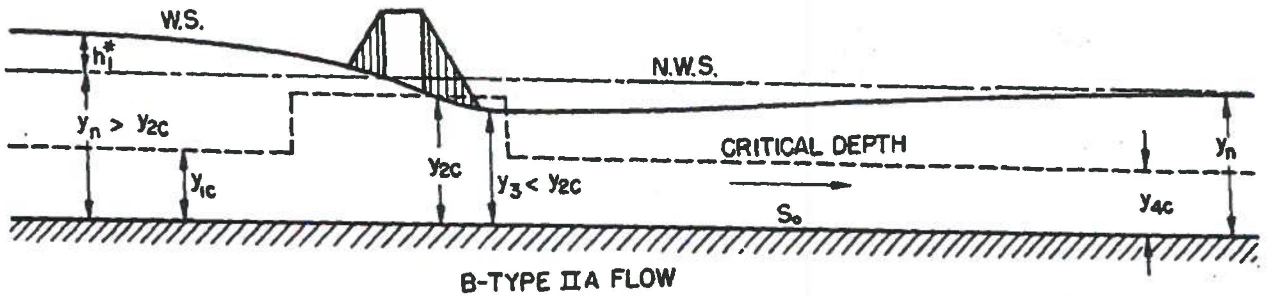
4.1.3 Ice Damage

Ice can affect riprap linings in a number of ways. Moving surface ice can cause crushing and bending forces as well as large impact loadings. The tangential flow of ice along a riprap lined channel bank can also cause excessive shearing forces. Quantitative criteria for evaluating the impact ice has on channel protection schemes are unavailable. However, historic observations of ice flows in New England rivers indicate that riprap sized to resist design flow events will also resist ice forces.

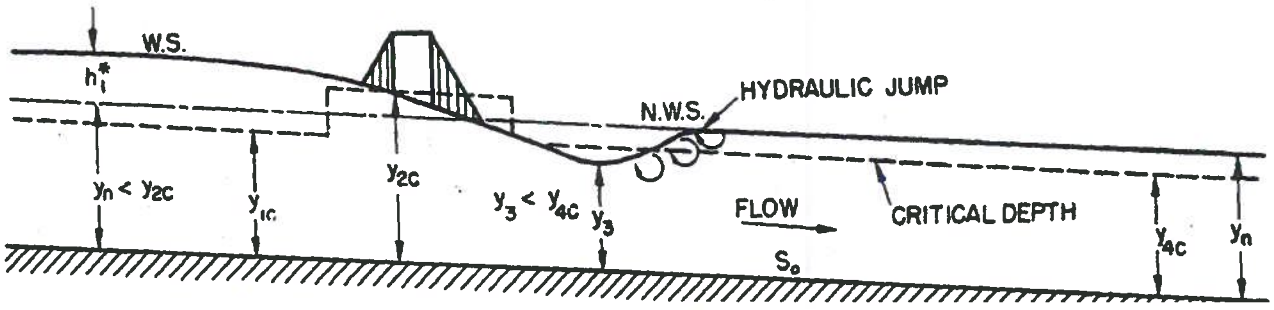
For design, consideration of ice forces should be evaluated on a case by case bases. In most instances, ice flows are not of sufficient magnitude to warrant detailed analysis. Where ice flows have historically caused problems, a stability factor of 1.2 to 1.5 should be used to increase the design rock size. Please note that the selection of an appropriate stability factor to account for ice generated erosive problems should be based on the designers experience.



A-TYPE I FLOW (SUBCRITICAL)



B-TYPE IIA FLOW
(PASSES THROUGH CRITICAL)



4.2 ROCK GRADATION

The gradation of stones in riprap revetment affects the riprap's resistance to erosion. The stone should be reasonably well graded throughout the riprap layer thickness. Specifications should provide for two limiting gradation curves, and the stone gradation (as determined from a field test sample) should lay within these limits. The gradation limits should not be so restrictive that production costs would be excessive. Table 2 presents suggested guidelines for establishing gradation limits. Table 3 presents six (6) suggested gradation classes based on AASHTO specifications. Form 3 (appendix C) can be used as an aid in selecting appropriate gradation limits.

It is recognized that the use of a four (4) point gradation as specified in table 2 might in some cases be too harsh a specification for some smaller quarries. If this is the case, the 85 percent specification can be dropped as is done in table 3. In most instances, a uniform gradation between D_{50} and D_{100} will result in an appropriate D_{85}.

Each load of riprap should be reasonably well graded from the smallest to the maximum size specified. Stones smaller than the specified 5 or 10 percent size should not be permitted in an amount exceeding 20 percent by weight of each load.

Table 2. Rock riprap gradation limits.

Stone Size Range* (ft.)	Stone Weight Range (lb)	Percent of Gradation Smaller Than
1.5 D_{50} to 1.7 D_{50}	3.0 W_{50} to 5.0 W_{50}	100
1.2 D_{50} to 1.4 D_{50}	2.0 W_{50} to 2.75 W_{50}	85
1.0 D_{50} to 1.15 D_{50}	1.0 W_{50} to 1.5 W_{50}	50
0.4 D_{50} to 0.6 D_{50}	0.1 W_{50} to 0.2 W_{50}	15

Table 3. Riprap gradation classes.

Riprap Class	Rock Size[1] (ft.)	Rock Size[2] (lbs)	Percent of Riprap Smaller Than
Facing	1.30	200	100
	0.95	75	50
	0.40	5	10
Light	1.80	500	100
	1.30	200	50
	0.40	5	10
1/4 ton	2.25	1000	100
	1.80	500	50
	0.95	75	10
1/2 ton	2.85	2000	100
	2.25	1000	50
	1.80	500	5
1 ton	3.60	4000	100
	2.85	2000	50
	2.25	1000	5
2 ton	4.50	8000	100
	3.60	4000	50
	2.85	2000	5

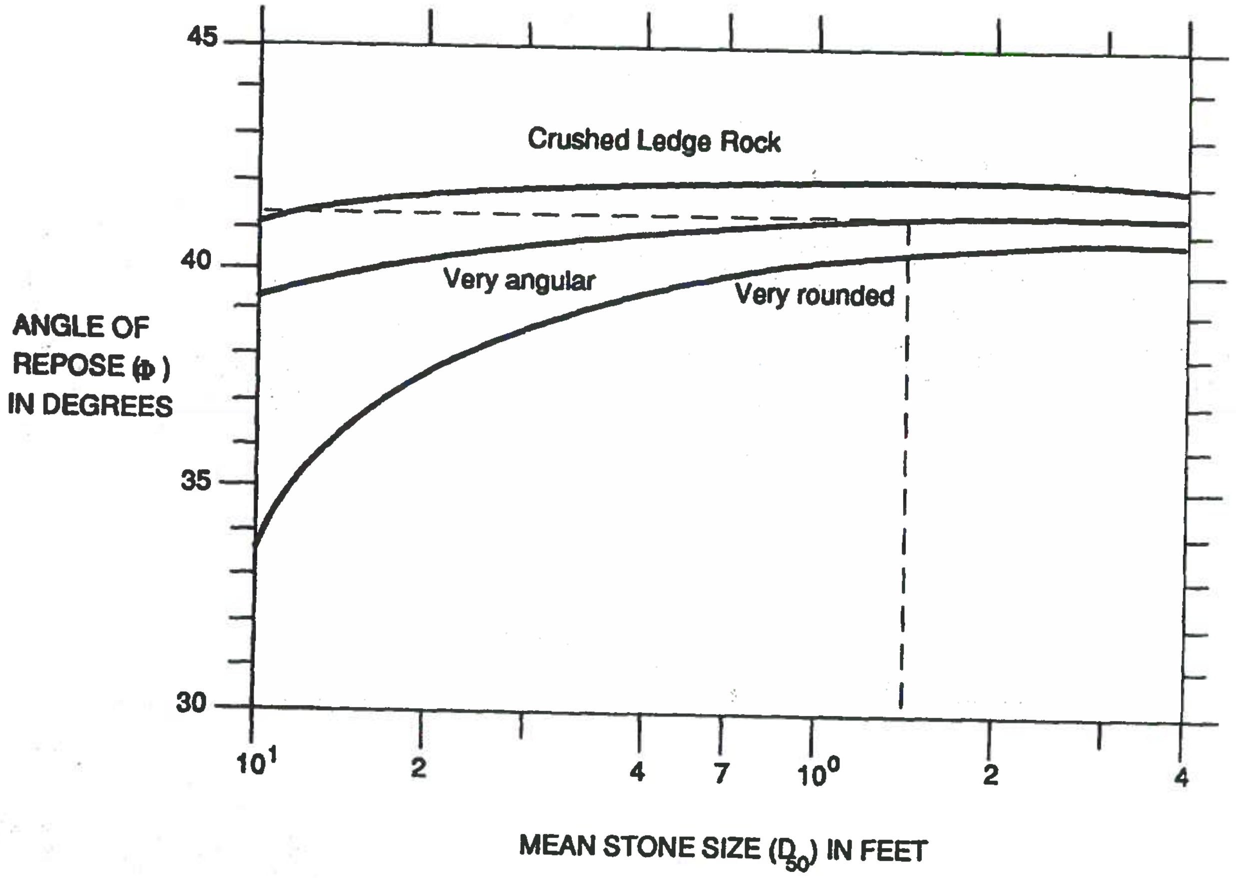


Figure 33. Angle of repose in terms of mean size and shape of stone (chart 4); example 2.

ATTACHMENT D

Calculations and Guidance for Convergence and Freeboard Requirements, USBR, 1987

Attachment D

Convergence Calculation

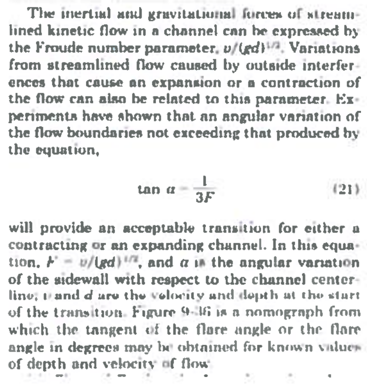
	Type I- Tier Trapazoid			Interim Trapazoidal Spillway			Rectangular Spillway		
	Velocity (ft/s)	depth (ft)	Fr	Velocity (ft/s)	depth (ft)	Fr	Velocity (ft/s)	depth (ft)	FR
100 YR	8.77	6.19	0.8	19.77	2.13	2.69	38.78	0.94	7.04
PMF	14.39	11.43	1	31.01	4.81	3	27.14	1.71	0.13

*Hydraulic parameters taken from spillway channel design spreadsheets

	Type I to Spillway		Interim Spillway to Rectangular Spillway	
θ	4.86	DEG	3.92	DEG
	1:12	V:H	1:15	V:H

516	Total length of transition from Type one channel to rectangular spillway	
200	Transition from non-tiered trapazoidal to rectangular (Spillway transition)	@1.5%
316	Transition from Type I to non-tiered trapazoidal (up on FAR)	@1.0%

Equation 21 from USBR manual



The inertial and gravitational forces of streamlined kinetic flow in a channel can be expressed by the Froude number parameter, $v/(gd)^{1/2}$. Variations from streamlined flow caused by outside interferences that cause an expansion or a contraction of the flow can also be related to this parameter. Experiments have shown that an angular variation of the flow boundaries not exceeding that produced by the equation,

$$\tan \alpha = \frac{1}{3F} \qquad (21)$$

will provide an acceptable transition for either a contracting or an expanding channel. In this equation, $F = v/(gd)^{1/2}$, and α is the angular variation of the sidewall with respect to the channel centerline; v and d are the velocity and depth at the start of the transition. Figure 9-36 is a nomograph from which the tangent of the flare angle or the flare angle in degrees may be obtained for known values of depth and velocity of flow.

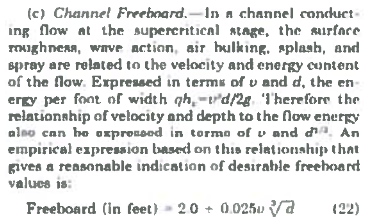
Freeboard

	100 YR		PMF	
	Interim Spillway$^{(1)}$	Spillway$^{(2)}$	Interim Spillway$^{(1)}$	Spillway$^{(2)}$
Freeboard	2.4	2.3	3.2	2.4
Max WSEL	664.94	661.5	669.75	664.5
Minimum Freeboard Elevation	667.3	663.8	673.0	666.9

(1) - WSEL at face of spillway control; cross section 1170 in HEC-RAS model

(2)- WSEL at top of spillway chute; cross section 970 in HEC-RAS model

Equation 22 of the USBR manual



(c) *Channel Freeboard.*—In a channel conducting flow at the supercritical stage, the surface roughness, wave action, air bulking, splash, and spray are related to the velocity and energy content of the flow. Expressed in terms of v and d, the energy per foot of width $qh_v = v^3d/2g$. Therefore the relationship of velocity and depth to the flow energy also can be expressed in terms of v and $d^{1/3}$. An empirical expression based on this relationship that gives a reasonable indication of desirable freeboard values is:

$$\text{Freeboard (in feet)} = 2.0 + 0.025v\sqrt[3]{d} \qquad (22)$$

APPENDIX E2

SPILLWAY DESIGN FOR PROBABLE MAXIMUM FLOOD

Geosyntec consultants

Written by: JNJ **Date:** 10/9/2015
Reviewed by: RM **Date:** 11/09/2016
Client: AEP **Project:** FAR Closure **Project No.:** CHE8273 **Task No.:** 02/05.22

APPENDIX E2:

SPILLWAY DESIGN FOR PROBABLE MAXIMUM FLOOD

PROJECT OVERVIEW

The Stingy Run Fly Ash Reservoir (FAR) is proposed to be closed by draining the reservoir, lowering the existing top of dam, and constructing a cover system. Previously, a Final Cover Stormwater Management Plan (Cover Plan) was developed. The Cover Plan includes the design of primary stormwater collection channels to convey the estimated 100-year peak runoff flows, as descried in Appendix D. A spillway was also sized and designed to convey the estimated 100-year peak runoff to the base of the modified dam. An energy dissipator and reinforced channel were designed at the toe of this spillway to convey the 100-year design events. The design of these system components are described in Appendix E1, Spillway/Energy Dissipator Design for the 100-year Event.

Ohio's Dam Safety Rules require dams to pass flows through their spillways without endangering the integrity of the dam. The magnitude of the design flood is directly related to the classification of the dam, which in turn is determined by evaluating the dam's downstream hazard and/or the dam's height. Based on conversations with Ohio Department of Natural Resources (ODNR), the design flow required to be conveyed through Stingy Run Fly Ash Dam should be 100% of the Probable Maximum Flood (PMF) in order to meet ODNR dam permit requirements. The PMF is the theoretically largest flood that could conceivably occur in a given area based on the estimated Probable Maximum Precipitation (PMP) for the drainage basin size and location. The PMF is an extreme event when compared to the 100-year recurrence interval design event, with a much higher calculated peak runoff rate. Therefore, to accommodate the PMF, the spillway through the crest of the dam and along the downstream dam face must be designed to safely convey higher flows.

This appendix section of the Design Report documents the hydrologic analysis performed to calculate the Probable Maximum Flood (PMF) peak discharge rate, and the hydraulic analysis performed to revise the spillway sizing and configuration. The hydrologic analysis was performed by using ODNR guidance material to determine the appropriate PMP depths and distributions for simulation, and then using hydrologic modeling computer software to estimate hydrographs and peak runoff rates (the PMF) generated from the drainage basins. The hydraulic analysis was performed by using the hydraulic modeling software to simulate possible spillway design configurations to accommodate the PMF, and collaborating with the design team to develop a spillway design that is hydraulically adequate and meets other design criteria such as physical site constraints and structural limitations.

Note that the only elements of the drainage system designed for the PMF are the spillway over the crest of the dam and the downstream face of the dam. Other drainage elements, such as the stormwater conveyance system on the FAR, the energy dissipator at the toe of the dam, and the reinforced channel downstream of the energy dissipator, were only designed to manage 100-year design flow or lesser design events.

This calculations package focuses solely on the PMF for the final cover stormwater management system, for purposes of the dam spillway design.

HYDROLOGIC METHOD OF ANALYSIS

The computer program U.S. Army Corps of Engineers (USACE) Hydrologic Engineering Center- Hydrologic Modeling System V4.0 (HEC-HMS) was used to perform the primary hydrologic analysis for the PMF. A HEC-HMS model was previously developed for the drainage basins upstream of the dam, to support the design of the 100-year spillway and the final cover system. The same HEC-HMS model was used in conjunction with the PMF precipitation input to determine the PMF flow rates as the site characteristics that influence hydrologic conditions were assumed to be the same as those identified during the 100 year peak flow evaluation.

The hydrologic analysis was performed using procedures descried in the HEC-HMS support document and the "Urban Hydrology for Small Watersheds, Technical Release 55," (USDA-SCS, 1986). In addition to the HEC-HMS, various spreadsheet and hand calculations were used to support the analysis. Details of the HEC-HMS model development are provided in Appendix D of the Dam Modification Report (final Cover Stormwater Management Design Calculations).

The HEC-HMS model was set up to follow the same design parameters as established for the final cover. The All Season PMP values are based on the size of the tributary area. The total tributary area is 1.3 square miles. As a conservative approach the rainfall data was based on the 1 square mile instead of interpolating the values between 1 and 10 square miles. A summary of the model structure is presented in the following section.

HYDROLOGIC ANALYTICAL PARAMETERS

The following describes the selection of the various hydrologic parameters used for the PMF analysis.

- Rainfall Distribution and Depth: Rainfall depths and distributions for the PMP are from the Probable Maximum Precipitation Study for the State of Ohio (Applied Weather Associates 2013). Three durations of design storms were simulated in the analysis: 24-hour, 12-hour and 6-hour rainfall durations.
    - The All Season PMP, 24-hour, SCS Type II modified, storm event is 27.3 inches.
    - The All Season PMP, 12-hour storm event is 25.3 inches.
    - The All Season PMP, 6-hour storm event is 19.1 inches.
- Subbasin Drainage Area: The total drainage area tributary to the proposed spillway is 852 acres. For additional details for the subbasin drainage areas refer to the final stormwater calculations in Appendix D. The drainage area inputs for HEC-HMS are shown in Attachment B.
- Pervious and Impervious Area: The tributary watershed was assumed to be pervious, with no impervious cover. For additional detail regarding the curve number analysis refer to the final cover stormwater calculations.
    - The following assumed conditions were used in determining the curve numbers (CN):
        - Cover Soils (CN-78)
            - Cover Type – Meadow
            - Hydrologic Condition- Fair
            - Hydrologic Soil Group- D
        - Existing (offsite) Soils (CN- 73)
            - Cover Type – Woods
            - Hydrologic Condition – Fair
            - Hydrologic Soil Group – C
    - The time of concentrations (Tc) were established with the final cover stormwater calculations. Supporting calculations are presented in Appendix D.

HYDROLOGIC ANALYTICAL PARAMETERS

The peak PMF flows estimated through the hydrologic analysis were used as the basis of design for the proposed spillway. The 6-hour duration event resulted in higher peak flows than the 12-hour and 24-hour events, and therefore the All Season PMP – 6 hour event was used as the design storm for the spillway. Below is a summary of the peak discharge from the cover system at the proposed spillway.

PMF Peak discharge from final cover at proposed spillway

PMF – 6 hour	***PMF – 12 hour***	***PMF – 24 hour***
9,038 cfs	8,540 cfs	5,460 cfs

The HEC-HMS model generated a PMF flow of 9,035 cfs. To provide some design robustness and conservatism a PMF flow of 9,500 cfs was used in the hydraulic design.

HYDRAULIC METHOD OF ANALYSIS

The PMF peak flow of 9,500 cfs as determined from the hydrologic analysis was used as the basis of the updated hydraulic design for the proposed spillway.

Hydraulic modeling was performed using HEC-RAS computer software version 4.1. The model analysis expanded on the HEC-RAS model which was developed for the project design that includes the main stormwater collection conveyance system upstream of the FAD on the FAR, the dam crest and spillway, the energy dissipator downstream of the spillway, a reach of new channel downstream of the energy dissipator leading to a new culvert, and a reach representing the existing Sting Run channel downstream of the culvert location which was designed to convey the 100-year storm event. Documentation of this HEC-RAS model is included in Appendix E-1, Spillway/Energy Dissipator Design for 100-year Event.

The HEC-RAS model was used as a starting point for the hydraulic analysis of the PMF spillway.

HYDRAULIC METHOD OF ANALYSIS

The PMF spillway extends from the upstream (western) edge of the crest of the dam, to the downstream (east) toe of the dam, where the energy dissipator begins. The length of the reach is approximately 750 feet, including the dam crest width (transitional interim spillway) of approximately 200 feet and a spillway chute length along the downstream face of approximately 550 feet. Spillway invert elevations were set at 661 feet at the upper crest of the dam to 586.5 feet at the downstream end of the spillway/beginning of the energy dissipator. The longitudinal bottom slope of the spillway is approximately 1.5% for the upper 200 feet of the spillway (through the crest of the dam and the top area of the face). At that point, the spillway bottom slope steepens to 13% slope, until the toe of the spillway is reached.

The PMF channel includes the 100-year spillway channel. The 100-year spillway was designed using an iterative analysis to determine the optimal geometry based on site and hydraulic conditions (see Appendix E1, Geosyntec October 2015). The 100-year spillway chute downstream of the dam crest includes a rectangular concrete channel consisting of a 55 feet wide channel base with vertical sides to a height of 5 feet. The rectangular shape reduces the potential for wave production during the high flows and the concrete lining will protect the channel against the very high velocities that are anticipated to be experienced in the channel center. In order to provide additional flood conveyance for the PMF event, the overbank area on either side of the 100 year, concrete channel will be lowered and graded, accordingly. A berm will be constructed at the edge of these overbank conveyance areas, with the berm top elevation set to provide appropriate freeboard.

Varying cross section configurations for the overbank/outer spillway areas were iteratively simulated in HEC-RAS in order to select a design configuration that provides adequate hydraulic capacity while maintaining spillway width and expected construction costs. This analysis resulted in a PMF overbank which consists of a flat bench that extends that extends 75 feet in each direction from the top of the 100-year spillway channel. At the edge of the 75-foot wide conveyance overbank a vertical berm will be constructed to contain the PMF flow before tying back into the proposed dam grades.

Freeboard requirements were determined based on USBR guidance (equation 22, USBR 1987).

$$Freeboard(ft) = 2.0 + 0.025v\sqrt[3]{d}$$

The freeboard for the PMF spillway was added to the channel geometry to account for wave action, air bulking, surface roughness and splashing that may be encountered under supercritical flow conditions which are anticipated to occur during this extreme event. At the dam crest where the dam geometry transitions from trapezoidal to the rectangular geometry, a minimum of 3.0 feet of freeboard was used to account for uncertainty in hydraulic conditions. Once the channel geometry transitioned from the trapezoid to the rectangular channel, the top elevation of the berm used to was set to provide a minimum of approximately 2.5 feet of freeboard above the modeled peak water surface elevation observed during the PMF. The berm is stepped so that the most upstream portion of the PMF spillway at the crest has a height of 5 feet and at the bottom of the PMF spillway the berm wall will be 2 feet tall. The spillway bench will be lined with a filter point fabric formed concrete mat that will allow for easier construction. The impounding berm will consist of a concrete wall that varies in thickness from 1 ft to 2 ft at the top. At the crest of the dam (first 200 feet) where the spillway transitions from a trapezoidal to rectangular channel will be constructed entirely of concrete. At the toe of the spillway the PMF channel will be carried downstream at an approximate 1.5% slope until it daylights and ties into the existing grade in order to allow the PMF to safely pass through the spillway.

The design drawings contained in Appendix A of this design report show typical spillway cross sections.

HYDRAULIC ANALYSIS RESULTS

Table 1 presents the results of the HEC-RAS modeling of the PMF. The HEC-RAS simulation confirms that the design of the expanded spillway, as described in the previous section, provides adequate hydraulic capacity to convey the PMF flow of 9,500 cfs. At least 2.5 vertical feet of freeboard will be provided along the spillway between the predicted maximum water elevation during the PMF and the adjacent top of berm bounding the spillway.

Utilizing the hydraulic analysis, and collaborating with the design team regarding other design parameters and criteria, the following cross section was developed to convey the PMF down the eastern (downstream) face of the reconstructed dam.

- A center concrete spillway with a rectangular cross section, a 55 bottom width and vertical side slopes that extends a distance of 5 feet up from the channel bottom.
- From that point, a fabric formed concrete lined PMF flat bench extends 75 feet from the edge of the concrete channel on both sides.
- From the edge of the 75-foot wide fabric formed concrete bench, a vertical berm will be constructed of concrete. The shape of the berm will have a stepped profile that will provide a minimum elevation of at least 2.5 feet above the predicted water surface elevation of the PMF.
- On the other side of the berm, an earthern side slope will extend downward at a 4:1 slope.

Figure 2 shows the profile graphic of the water surface elevation during the PMF from HEC-RAS.

Attachment C shows graphics of representative cross section and computer water surface elevations from HEC-RAS.

SUMMARY AND CONCLUSIONS

Based on a regulatory review directive from the ODNR, the design of the spillway for the Stingy Run fly ash reservoir closure was modified to convey the Probable Maximum Flood (PMF). A previously-constructed HEC-HMS hydrologic runoff was updated to also simulate peak flow from the PMF, using PMP values developed by the Ohio DNR. The PMF flow value of 9,500 cfs for the dam location was used.

Following estimation of the PMF flow, the spillway design was modified to provide additional hydraulic capacity. Hydraulic modeling software, HEC-RAS, was used to assess the hydraulic capacity of alternative design configuration to determine the best geometry to provide safe conveyance of the PMF over the dam crest and down the face of the dam. A channel configuration was selected that combines the center of the rectangular, concrete spillway, as designed to convey the 100-year flood flow, with outer overbank benches and a concrete berm to convey the PMF flow.

REFERENCES

Applied Weather Associates LLC. Probable Maximum Precipitation Study for the State of Ohio. Prepared for the Ohio Department of Natural Resources. February 2013.

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United States Department of Agriculture, Soil Conservation Service (USDA-SCS), "Urban Hydrology for Small Watersheds, Technical Release 55", 2nd ed., Washington, D.C., 1986.

United States Army Corps of Engineers, "Hydrologic Engineering Center- Hydrologic Modeling System". Version 4.0. December 2013. Davis, CA.

Unite States Army Corps of Engineers, "Hydrologic Engineering Center-River Analysis System", Version 4.1. January 2010. Davis CA.

FIGURES AND TABLES

Figure

Table

Attachments

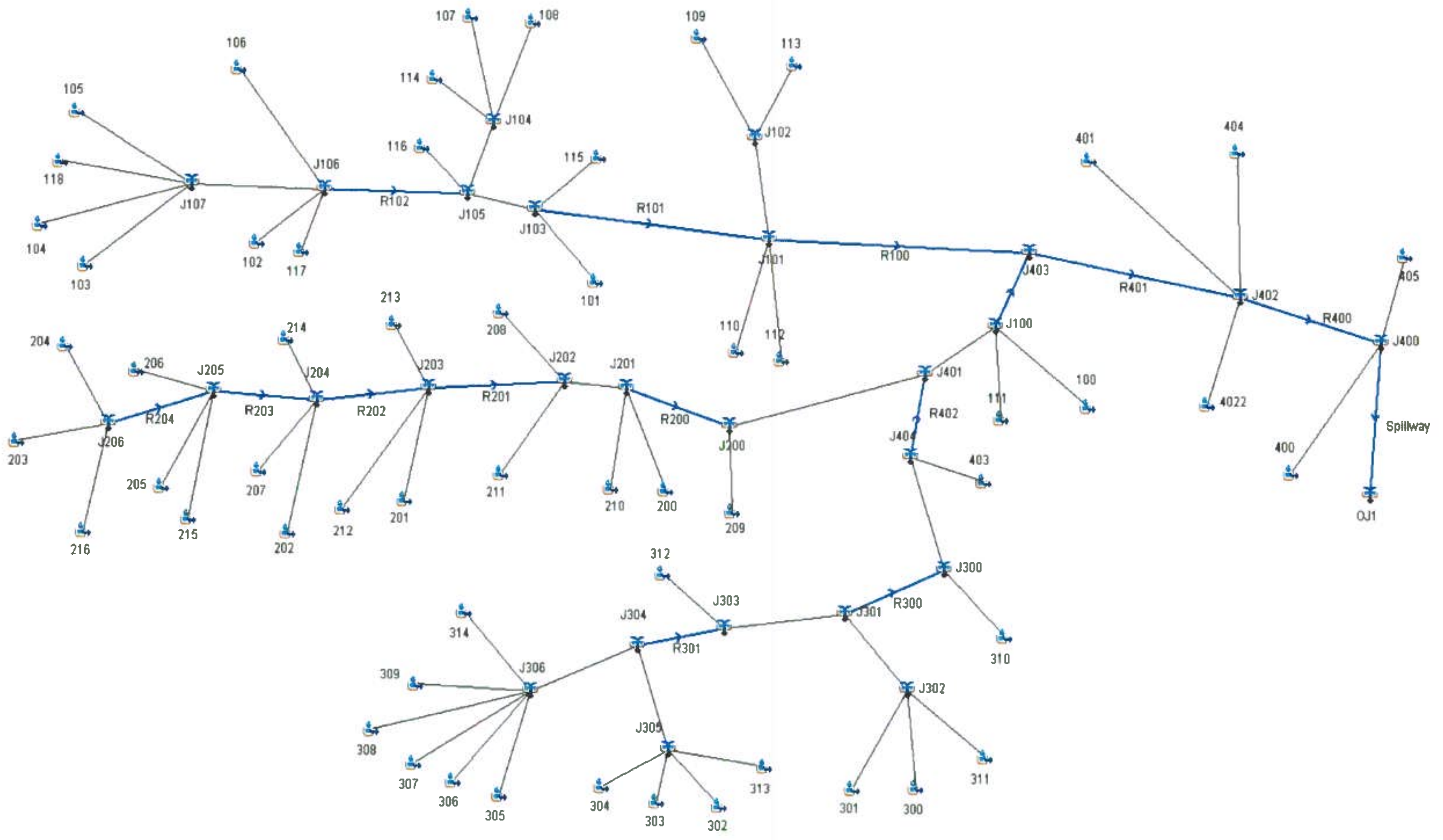

Figure 1. HEC-HMS Model – PMF (Final Cover)

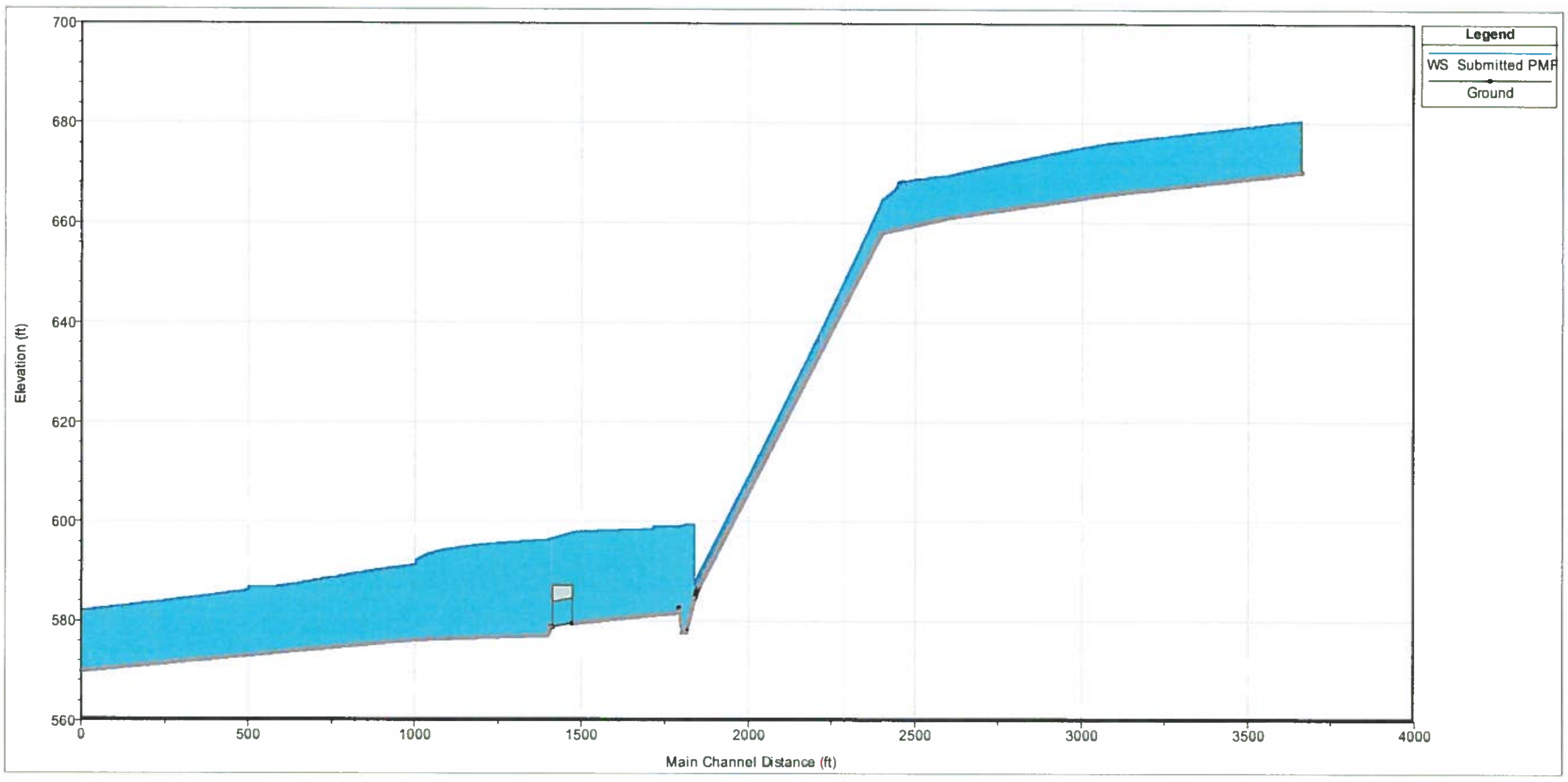

Figure 2. HEC-RAS Water Surface Profile: PMF event

TABLES

Geosyntec
consultants

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Oak Brook, IL
TELEPHONE (630) 203 3340
FAX (630) 203 3341

Table 1

JOB CHE8273-FAR Closure
SHEET NO ____ OF ____
CALCULATED BY Rishab Mahajan DATE 10/1/2015
CHECKED BY Jennifer Jenkins DATE 10/1/2015
SCALE ____
DESCRIPTION HEC-RAS Outputs, PMF Flow

Cross-Description	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	Hydr Depth C	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl	Shear Chan
			(cfs)	(ft)	(ft)	(ft)	(ft)	ft	(ft/ft)	(ft/sec)	(sq ft)	(ft)		(lb/sq ft)
Type 1 channel	2230	PMF	9500	670.11	680.5353	680.43	8.62	682.01	0.00595	11.91	1160.35	350.24	0.71	3.15
Begin Type 1 to non-tier trapezoid	1230	PMF	9500	665.65	675.9139		8.46	677.53	0.006636	12.42	1106.97	339.78	0.75	3.44
Transition spillway to non-tier trapezoid	1170	PMF	9500	661	669.5609	669.51	6.98	672.32	0.010172	13.61	739.37	137.73	0.91	4.4
Transition non-tier trapezoid to rectangular spillway	1120	PMF	9500	660.25	669.2082	668.76	7.35	671.61	0.008285	12.74	795.03	142.5	0.83	3.78
Begin Spillway Chute	970	PMF	9500	658	664.7146	666.16	6.71	669.58	0.002917	19.66	626.49	205	1.34	1.03
100' Down Spillway	870 *	PMF	9500	645	650.1323	653.35	5.13	667.43	0.012101	33.47	302.11	205	2.6	3.28
200' Down Spillway	770 *	PMF	9500	632	636.0418	640.46	4.04	664.4	0.026068	42.74	222.3	55	3.75	5.73
300' Down Spillway	670 *	PMF	9500	619	622.5207	627.6	3.52	659.9	0.040391	49.06	193.64	55	4.61	7.87
400' Down Spillway	570 *	PMF	9500	606	609.2	614.72	3.2	654.44	0.054771	53.98	176	55	5.32	9.8
500' Down Spillway	470 *	PMF	9500	593	595.9826	601.82	2.98	648.06	0.06859	57.91	164.04	55	5.91	11.52
Begin transitioning walls for basin	420	PMF	9500	586.5	589.4	595.35	2.9	644.49	0.075052	59.56	159.5	55	6.16	12.29
Start Entrance Curve	419.9	PMF	9500	586.36	589.2582	595.21	2.9	644.41	0.075194	59.6	159.4	55	6.17	12.31
1' Down Dissipator Entrance Curve	419.8	PMF	9500	586.21	589.1063	595.06	2.9	644.34	0.075362	59.64	159.29	55	6.18	12.33
2' Down Dissipator Entrance Curve	419.7	PMF	9500	586.05	588.9437	594.9	2.89	644.27	0.075571	59.69	159.16	55	6.18	12.35
3' Down Dissipator Entrance Curve	419.6	PMF	9500	585.87	588.761	594.72	2.89	644.19	0.075796	59.75	159.01	55	6.19	12.38
4' Down Dissipator Entrance Curve	419.5	PMF	9500	585.67	588.5579	594.52	2.89	644.11	0.076064	59.81	158.83	55	6.2	12.41
5' Down Dissipator Entrance Curve	419.4	PMF	9500	585.47	588.3546	594.32	2.88	644.03	0.076343	59.88	158.65	55	6.21	12.44
6' Down Dissipator Entrance Curve	419.3	PMF	9500	585.25	588.1308	594.1	2.88	643.95	0.076665	59.96	158.44	55	6.23	12.48
7' Down Dissipator Entrance Curve	419.2	PMF	9500	585.01	587.8868	593.86	2.88	643.87	0.07701	60.04	158.22	55	6.24	12.52
End Curve transition to 4H:1V	419.1	PMF	9500	584.76	587.6324	593.61	2.87	643.78	0.077388	60.13	157.98	55	6.25	12.57
Front of chute block	414.69	PMF	9500	578.54	599.4466		20.9	599.91	0.000086	6.23	2504.73	257.7	0.24	0.08
Back of chute block (bottom elev)	413.46	PMF	9500	577.6	599.5247		21.46	599.88	0.000135	5.56	2541.6	257.39	0.21	0.07
Just after chute block	413.45	PMF	9500	577.6	599.42		21.82	599.87	0.00008	6.07	2540.35	256.48	0.23	0.07
Just before baffles	400.97	PMF	9500	577.6	599.4196		21.82	599.87	0.00008	6.08	2540.22	256.47	0.23	0.07
Top edge of baffles	400.96	PMF	9500	577.6	599.4339		20.75	599.81	0.00014	5.75	2484.47	256.6	0.22	0.08
Back top of baffles	400.46	PMF	9500	577.6	599.4338		20.75	599.81	0.00014	5.75	2484.46	256.6	0.22	0.08
Back of baffles	387.754	PMF	9500	577.6	599.3024		21.7	599.76	0.000082	6.13	2510.24	255.46	0.23	0.07
Front edge of sill	380.55	PMF	9500	577.6	599.3022		21.7	599.76	0.000082	6.13	2510.2	255.45	0.23	0.07
Top of sill	370.55	PMF	9500	582.6	599.0613		16.46	599.69	0.000147	7.44	2173.94	253.36	0.32	0.11
Drop after sill and flare out to begin transition	370.54	PMF	9500	581.6	599.068		17.47	599.65	0.000131	7.17	2230.73	253.42	0.3	0.1
End concrete transition	320.54	PMF	9500	581.3	599.1057		14.95	599.48	0.000073	5.79	2797.09	289.75	0.26	0.07
1' Down Exit Channel	316.53	PMF	9500	581.3	599.1392		15.71	599.34	0.000354	4.25	2843.28	289.6	0.19	0.33
2' Down Exit Channel	316.52	PMF	9500	581.3	599.1331		15	599.34	0.000378	4.29	2794.32	289.55	0.2	0.34
4' Down Exit Channel	314.52*	PMF	9500	581.29	599.1323		15	599.34	0.000378	4.29	2794.59	289.55	0.2	0.34
6' Down Exit Channel	312.52*	PMF	9500	581.28	599.1315		15.01	599.34	0.000378	4.29	2794.86	289.54	0.2	0.34
8' Down Exit Channel	310.52*	PMF	9500	581.27	599.1307		15.02	599.34	0.000378	4.29	2795.13	289.53	0.19	0.34
10' Down Exit Channel	308.52*	PMF	9500	581.26	599.1299		15.02	599.33	0.000378	4.29	2795.41	289.53	0.19	0.34
15' Down Exit Channel	303.52*	PMF	9500	581.23	599.1279		15.04	599.33	0.000377	4.29	2796.34	289.51	0.19	0.34
20' Down Exit Channel	298.52*	PMF	9500	581.2	599.126		15.06	599.33	0.000376	4.29	2797.28	289.5	0.19	0.34
25' Down Exit Channel	291.52	PMF	9500	581.16	599.1232		15.08	599.33	0.000376	4.29	2798.49	289.48	0.19	0.34
50' Down Exit Channel	266.949*	PMF	9500	580.99	598.5192		14.9	599.14	0.00095	6.79	1580.26	123.62	0.31	0.86
100' Down Exit Channel	216.805*	PMF	9500	580.65	598.4084		15.1	599.09	0.000997	7.01	1514.85	115.7	0.32	0.91
200' Down Exit Channel	114.471*	PMF	9500	579.94	598.162		15.49	598.97	0.001089	7.45	1386.12	99.54	0.33	1.02
250' Down Exit Channel	63.3039*	PMF	9500	579.59	598.0306		15.67	598.9	0.001133	7.65	1323.85	91.45	0.34	1.07
Cross section just upstream of culvert	50	PMF	9500	579.5	597.9959	590.39	15.72	598.88	0.001144	7.7	1307.75	89.35	0.34	1.08
Cross section downstream culvert	3	PMF	9500	578.8	596.6923	590.27	17.89	597.7	0.00056	9.62	1253.81	89.35	0.4	0.63
Begin Natural Channel	2.94974*	PMF	9500	577.07	596.337	590.51	16.17	597.66	0.000841	10.46	1094.99	79.01	0.46	0.78
Cross Section 2 in Natural Channel	2	PMF	9500	576.17	591.3695	591.35	13.28	596.93	0.004654	21.73	544.31	50.01	1.05	3.6
Cross Section 1 in Natural Channel	1	PMF	9500	573.02	586.8864	586.23	10.23	591.04	0.003884	16.85	597.93	60	0.93	2.35
Cross section just upstream of final culvert	0	PMF	9500	569.87	582.0536	582.05	11.29	587.18	0.004691	19.87	538.54	52.81	1.04	3.16

ATTACHMENT A

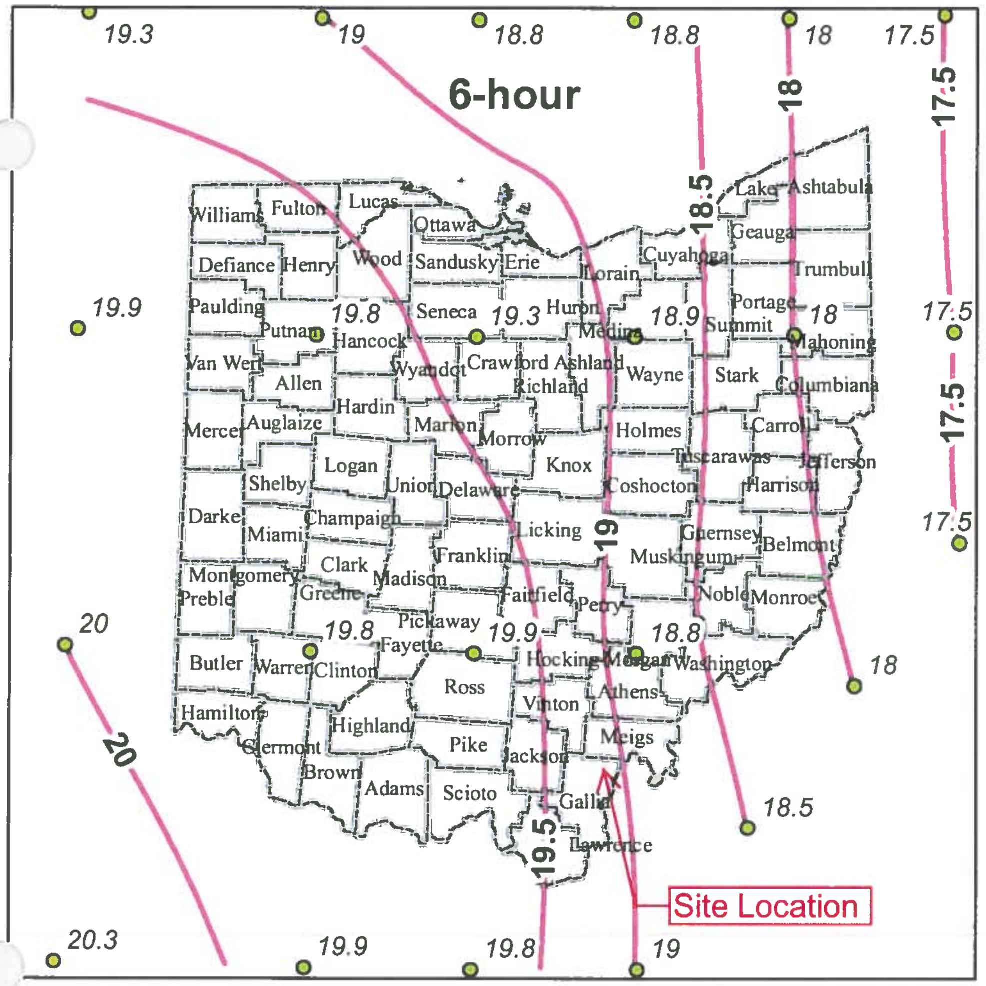
Probable Maximum Precipitation (PMP) Reference Material

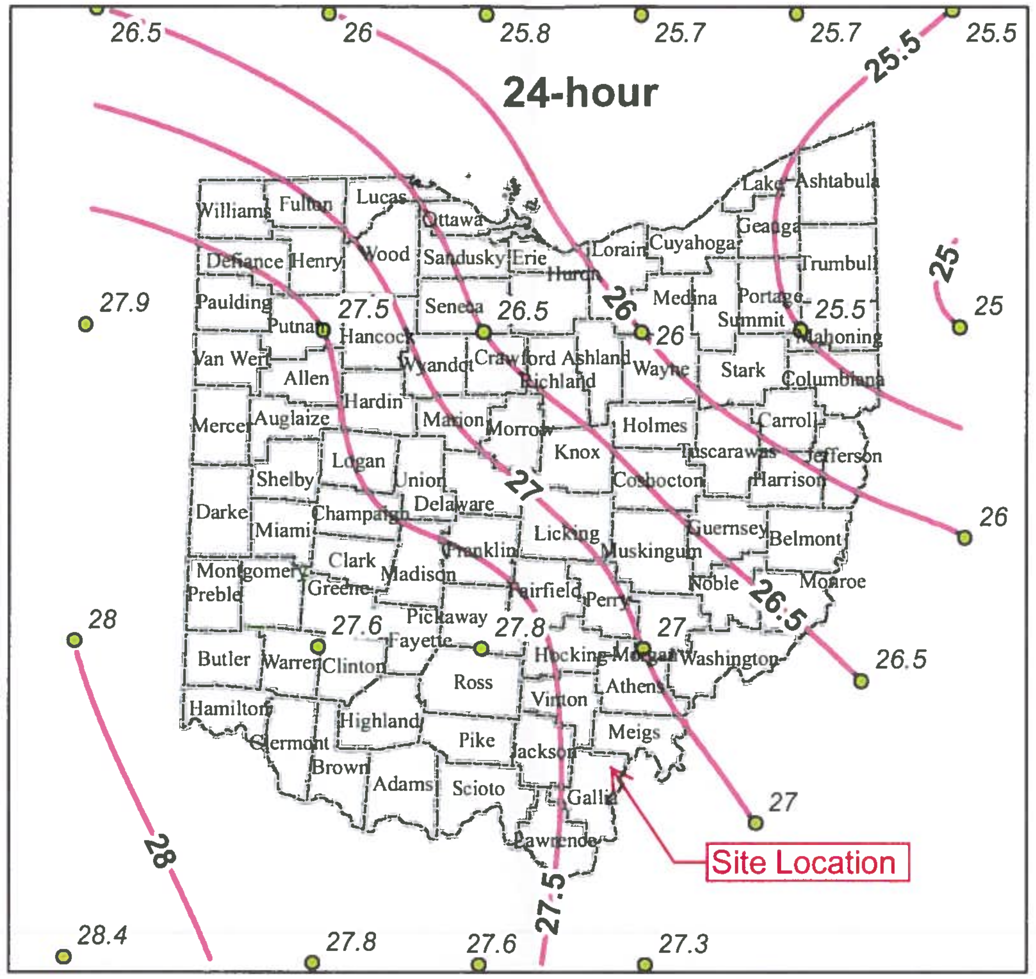
STATE OF OHIO
Department of Natural Resources

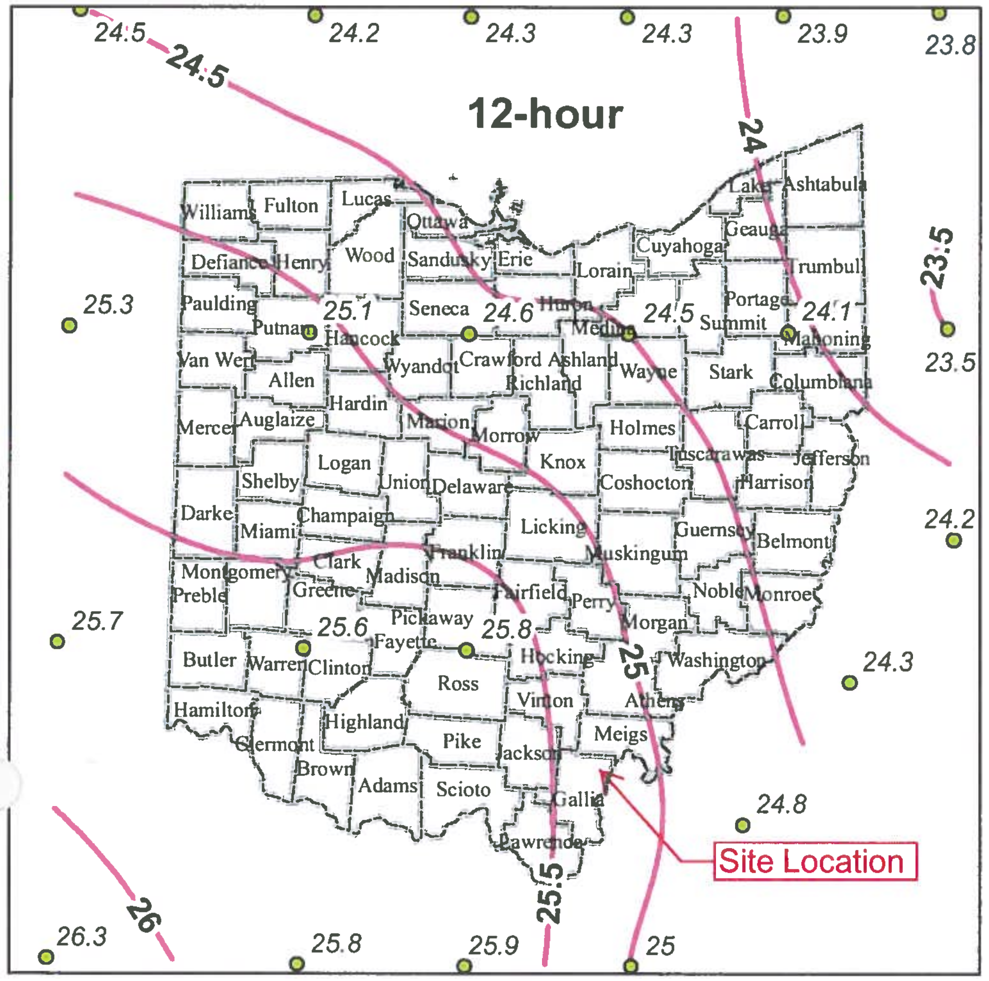
1 square mile

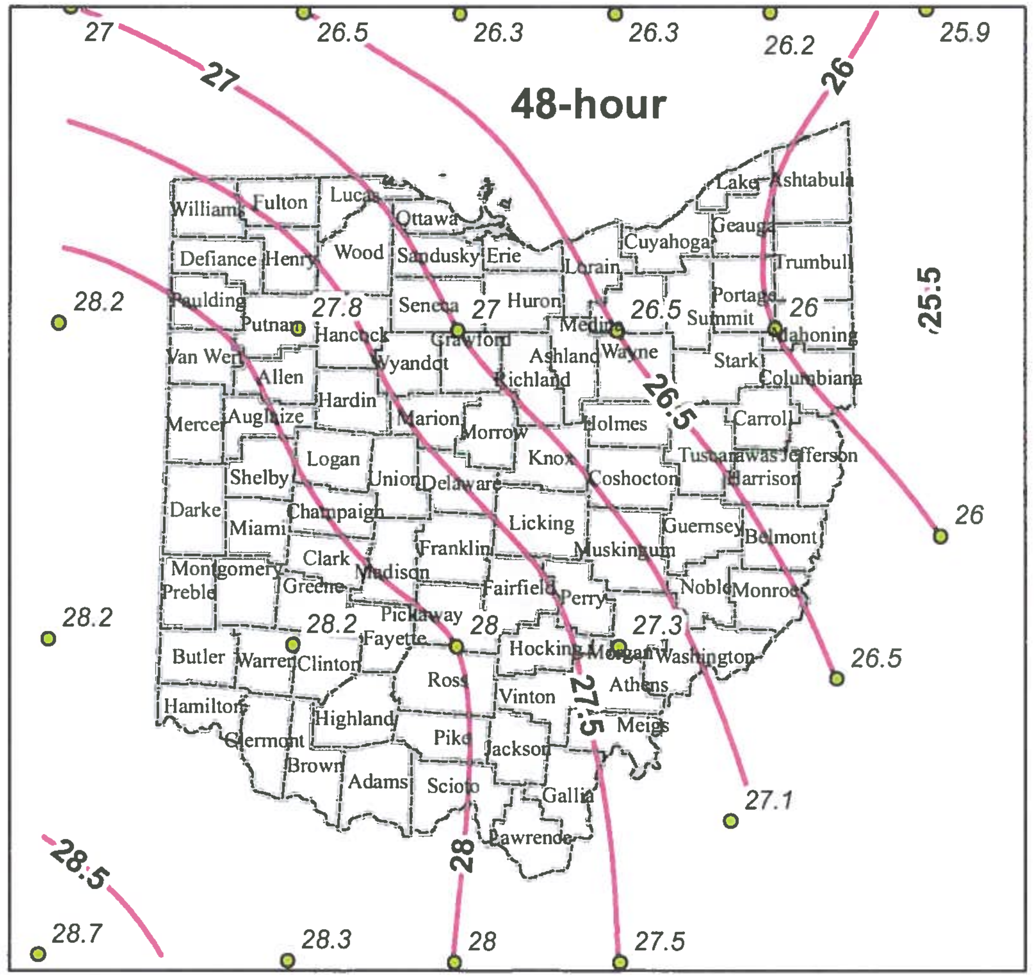
All-Season PMP values (inches)

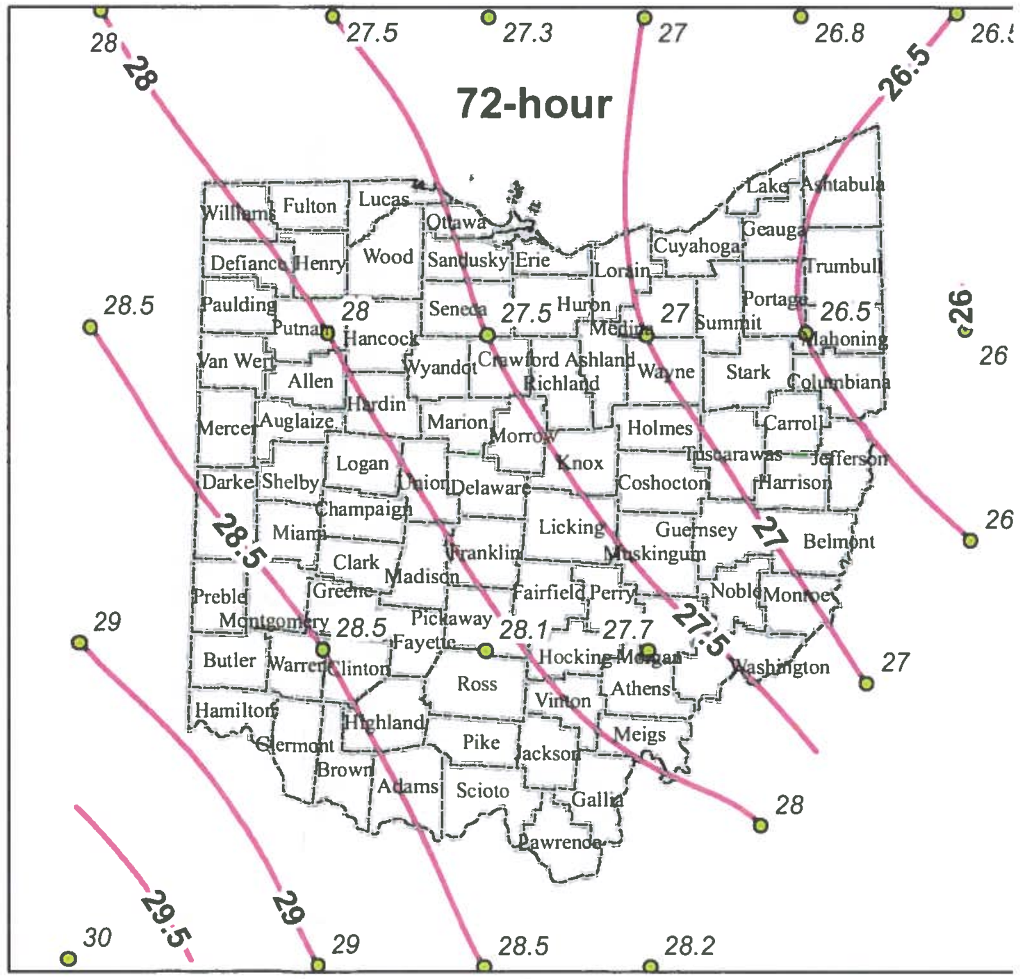
Data obtained from:
Probable Maximum Precipitation Stu
for the State of Ohio
Prepared by:
Applied Weather Associates
2013











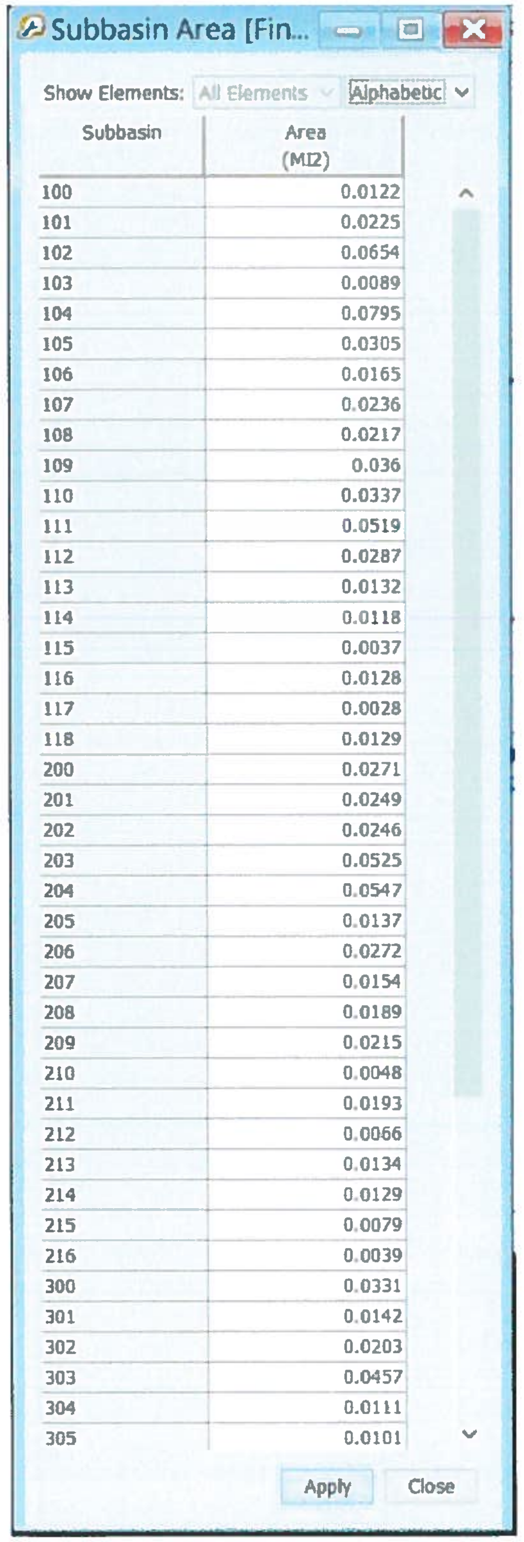
Geosyntec consultants

Written by: JNJ **Date:** 10/9/2015
Reviewed by: RM **Date:** 11/09/2016
Client: AEP **Project:** FAR Closure **Project No.:** CHE8273 **Task No.:** 02/05.22

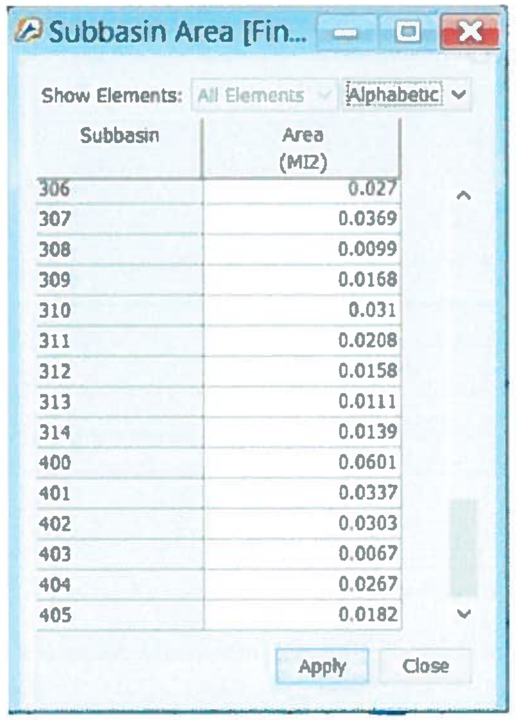
ATTACHMENT B

HEC-HMS input/output screenshots

Summary of Area Inputs for HEC-HMS

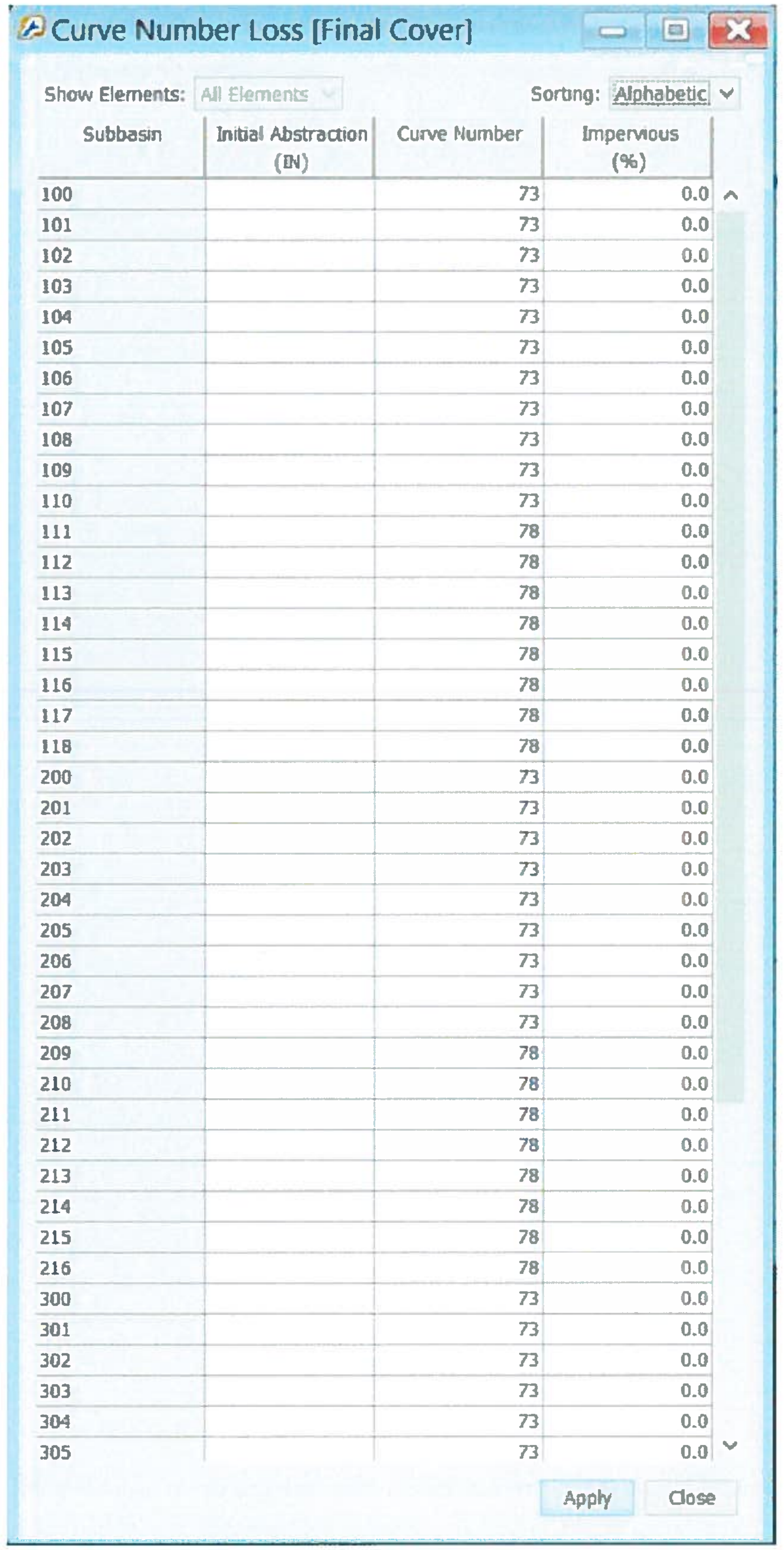


Subbasin	Area (MI2)
100	0.0122
101	0.0225
102	0.0654
103	0.0089
104	0.0795
105	0.0305
106	0.0165
107	0.0236
108	0.0217
109	0.036
110	0.0337
111	0.0519
112	0.0287
113	0.0132
114	0.0118
115	0.0037
116	0.0128
117	0.0028
118	0.0129
200	0.0271
201	0.0249
202	0.0246
203	0.0525
204	0.0547
205	0.0137
206	0.0272
207	0.0154
208	0.0189
209	0.0215
210	0.0048
211	0.0193
212	0.0066
213	0.0134
214	0.0129
215	0.0079
216	0.0039
300	0.0331
301	0.0142
302	0.0203
303	0.0457
304	0.0111
305	0.0101

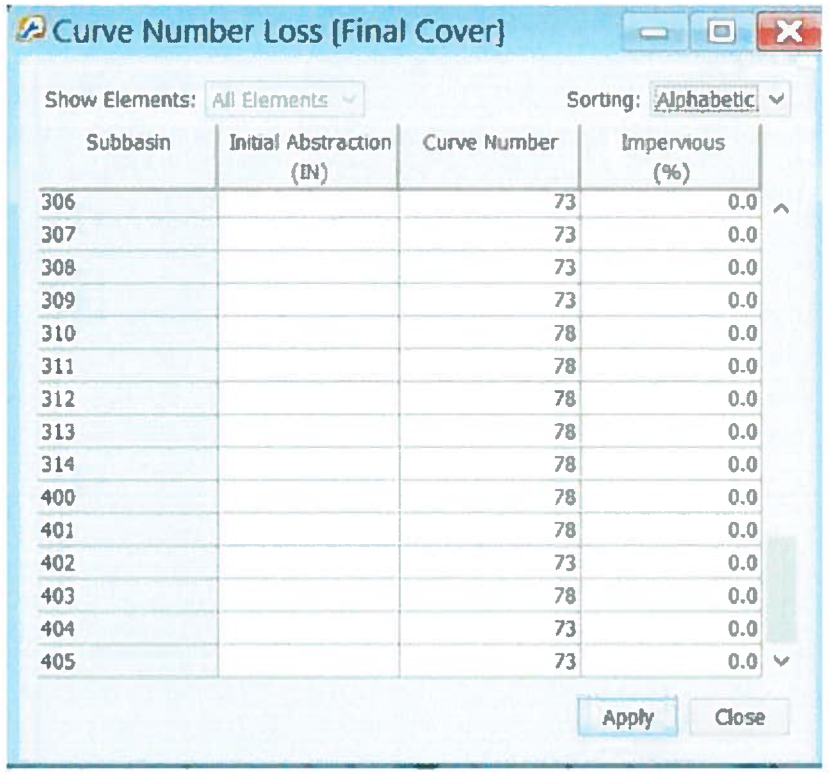


Subbasin	Area (MI2)
306	0.027
307	0.0369
308	0.0099
309	0.0168
310	0.031
311	0.0208
312	0.0158
313	0.0111
314	0.0139
400	0.0601
401	0.0337
402	0.0303
403	0.0067
404	0.0267
405	0.0182

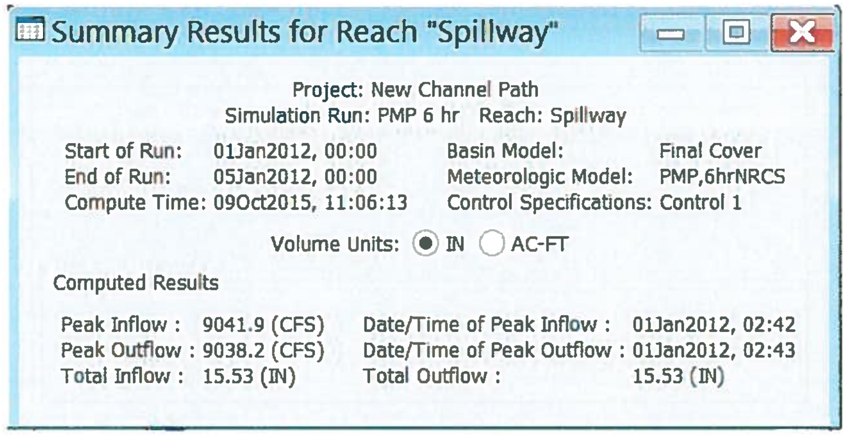
Summary of CN Inputs for HEC-HMS



Subbasin	Initial Abstraction (IN)	Curve Number	Impervious (%)
100		73	0.0
101		73	0.0
102		73	0.0
103		73	0.0
104		73	0.0
105		73	0.0
106		73	0.0
107		73	0.0
108		73	0.0
109		73	0.0
110		73	0.0
111		78	0.0
112		78	0.0
113		78	0.0
114		78	0.0
115		78	0.0
116		78	0.0
117		78	0.0
118		78	0.0
200		73	0.0
201		73	0.0
202		73	0.0
203		73	0.0
204		73	0.0
205		73	0.0
206		73	0.0
207		73	0.0
208		73	0.0
209		78	0.0
210		78	0.0
211		78	0.0
212		78	0.0
213		78	0.0
214		78	0.0
215		78	0.0
216		78	0.0
300		73	0.0
301		73	0.0
302		73	0.0
303		73	0.0
304		73	0.0
305		73	0.0

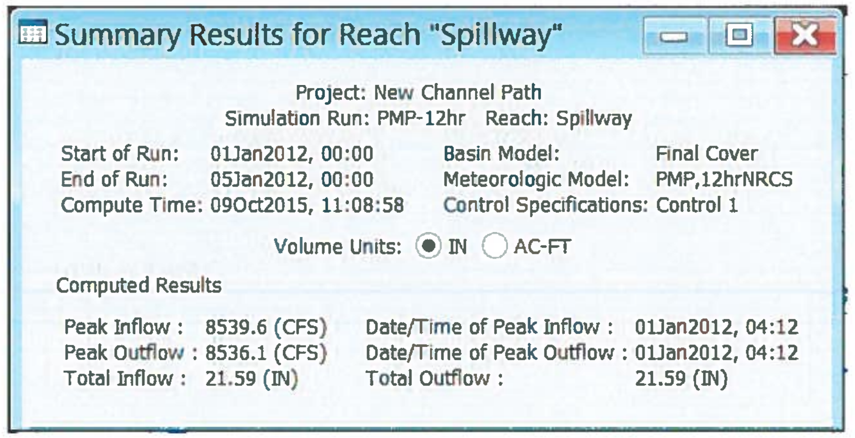


Subbasin	Initial Abstraction (IN)	Curve Number	Impervious (%)
306		73	0.0
307		73	0.0
308		73	0.0
309		73	0.0
310		78	0.0
311		78	0.0
312		78	0.0
313		78	0.0
314		78	0.0
400		78	0.0
401		78	0.0
402		73	0.0
403		78	0.0
404		73	0.0
405		73	0.0



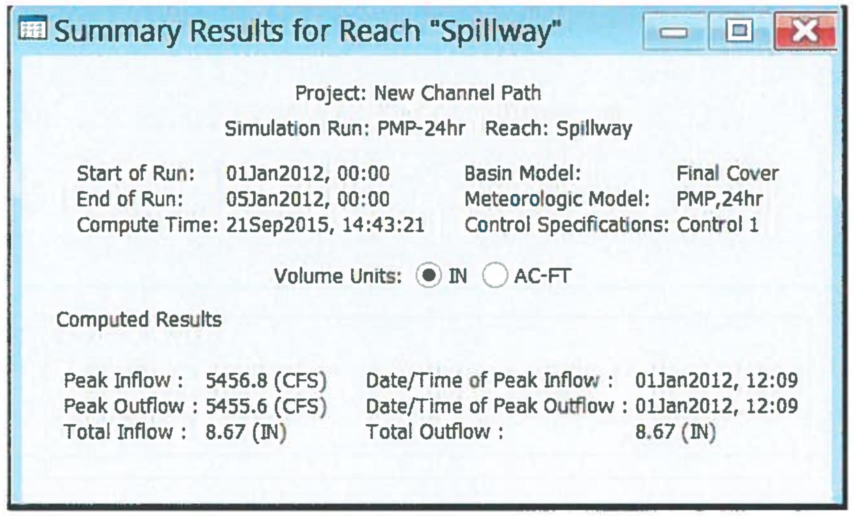
```
Summary Results for Reach "Spillway"

                 Project: New Channel Path
        Simulation Run: PMP 6 hr   Reach: Spillway

Start of Run:    01Jan2012, 00:00       Basin Model:             Final Cover
End of Run:      05Jan2012, 00:00       Meteorologic Model:      PMP,6hrNRCS
Compute Time: 09Oct2015, 11:06:13       Control Specifications: Control 1

                 Volume Units: ● IN  ○ AC-FT

Computed Results

Peak Inflow :   9041.9 (CFS)    Date/Time of Peak Inflow :   01Jan2012, 02:42
Peak Outflow :  9038.2 (CFS)    Date/Time of Peak Outflow :  01Jan2012, 02:43
Total Inflow :  15.53 (IN)      Total Outflow :              15.53 (IN)
```

HEC-HMS summary output: 6-hour PMP event



```
Summary Results for Reach "Spillway"

                 Project: New Channel Path
        Simulation Run: PMP-12hr   Reach: Spillway

Start of Run:    01Jan2012, 00:00       Basin Model:             Final Cover
End of Run:      05Jan2012, 00:00       Meteorologic Model:      PMP,12hrNRCS
Compute Time: 09Oct2015, 11:08:58       Control Specifications: Control 1

                 Volume Units: ● IN  ○ AC-FT

Computed Results

Peak Inflow :   8539.6 (CFS)    Date/Time of Peak Inflow :   01Jan2012, 04:12
Peak Outflow :  8536.1 (CFS)    Date/Time of Peak Outflow :  01Jan2012, 04:12
Total Inflow :  21.59 (IN)      Total Outflow :              21.59 (IN)
```

HEC-HMS summary output: 12-hour PMP event



```
Summary Results for Reach "Spillway"

                 Project: New Channel Path
        Simulation Run: PMP-24hr   Reach: Spillway

Start of Run:    01Jan2012, 00:00       Basin Model:             Final Cover
End of Run:      05Jan2012, 00:00       Meteorologic Model:      PMP,24hr
Compute Time: 21Sep2015, 14:43:21       Control Specifications: Control 1

                 Volume Units: ● IN  ○ AC-FT

Computed Results

Peak Inflow :   5456.8 (CFS)    Date/Time of Peak Inflow :   01Jan2012, 12:09
Peak Outflow :  5455.6 (CFS)    Date/Time of Peak Outflow :  01Jan2012, 12:09
Total Inflow :  8.67 (IN)       Total Outflow :              8.67 (IN)
```

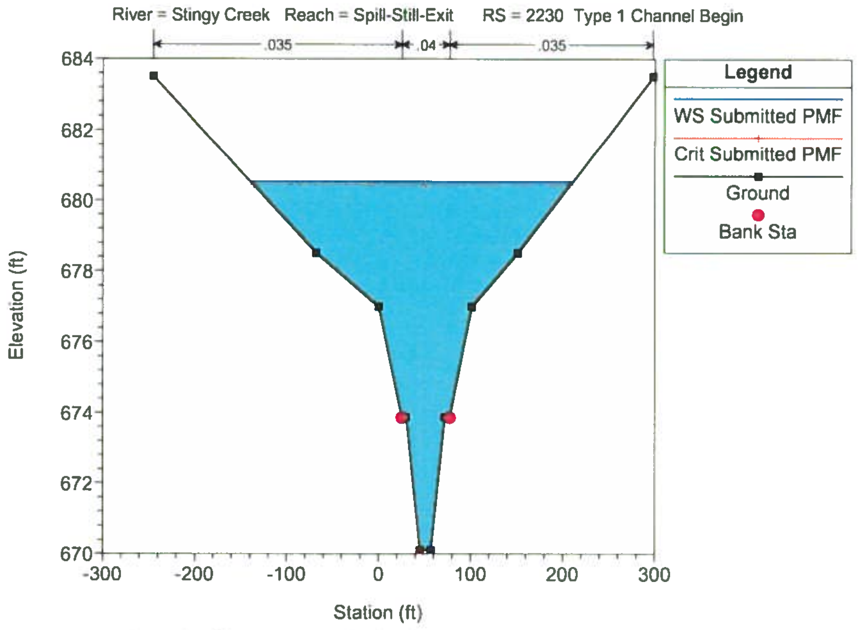
HEC-HMS summary output: 24-hour PMP event

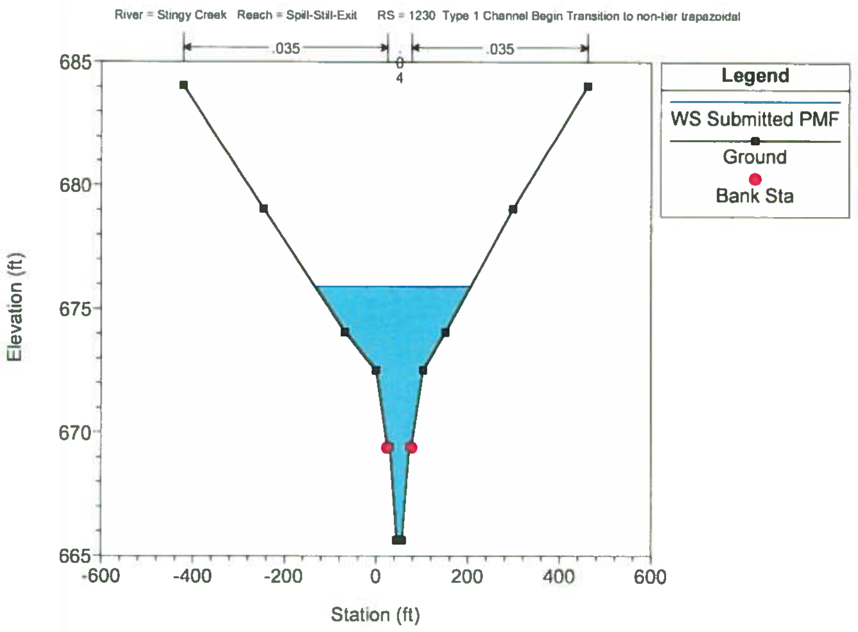
Geosyntec consultants

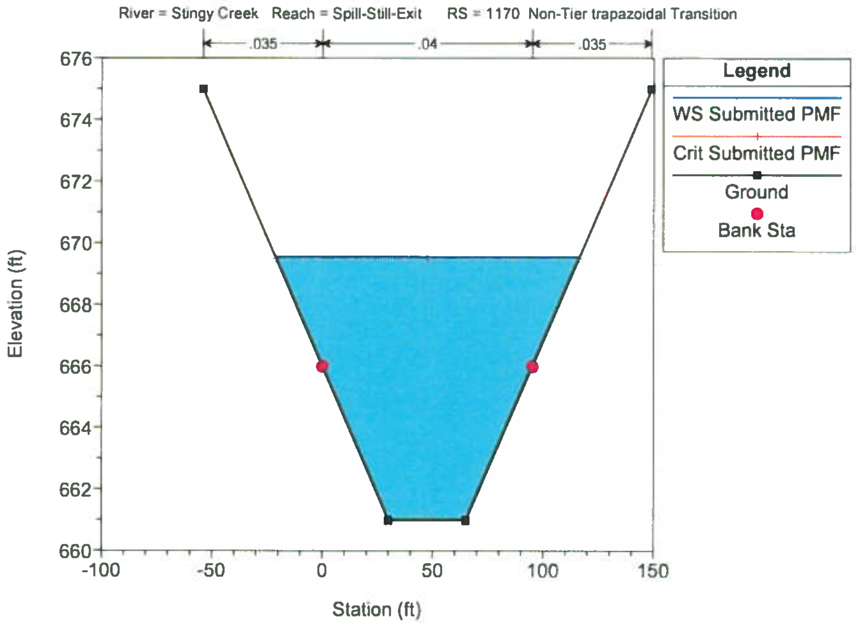
Written by: JNJ **Date:** 10/9/2015
Reviewed by: RM **Date:** 11/09/2016
Client: AEP **Project:** FAR Closure **Project No.:** CHE8273 **Task No.:** 02/05.22

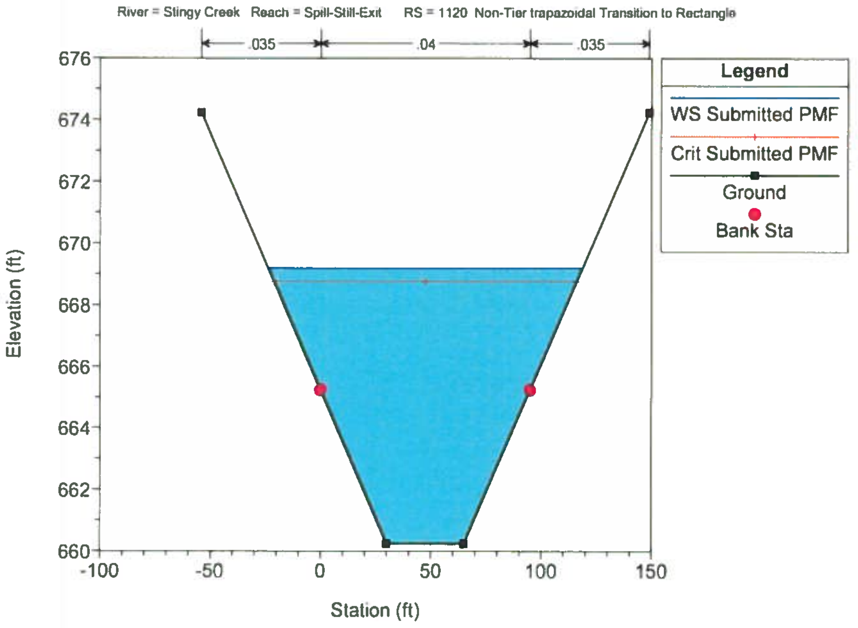
ATTACHMENT C

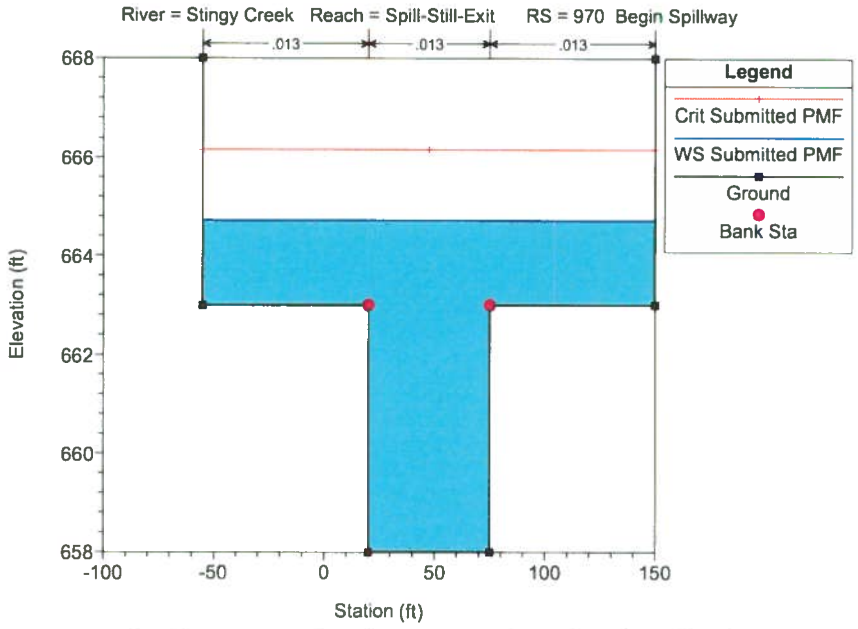
HEC-RAS Cross Section Printouts

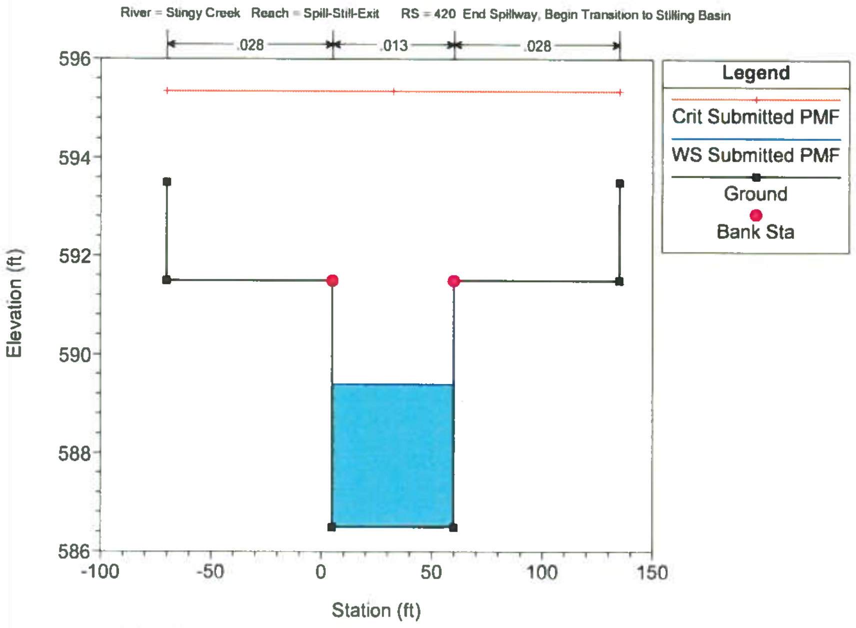

River = Stingy Creek Reach = Spill-Still-Exit RS = 2230 Type 1 Channel Begin

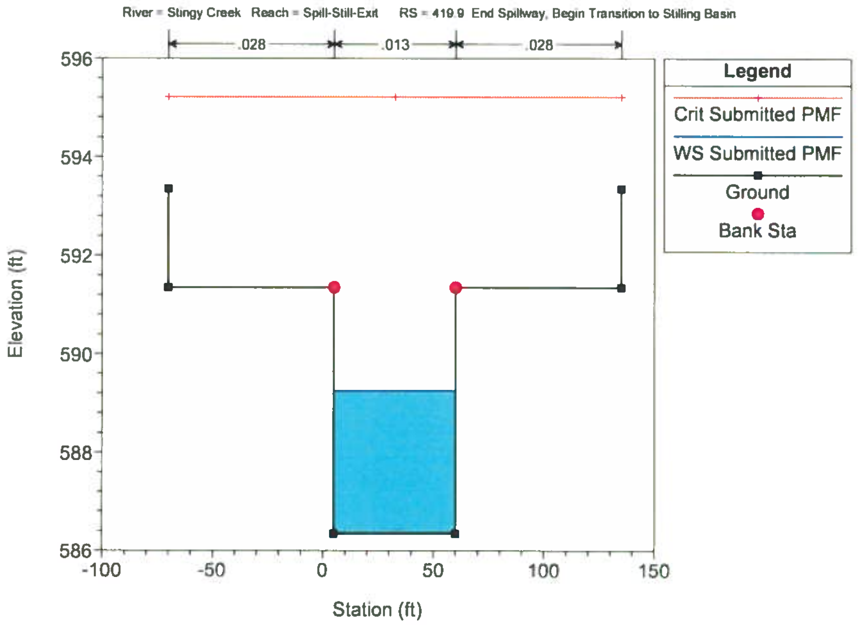

River = Stingy Creek Reach = Spill-Still-Exit RS = 1230 Type 1 Channel Begin Transition to non-tier trapazoidal

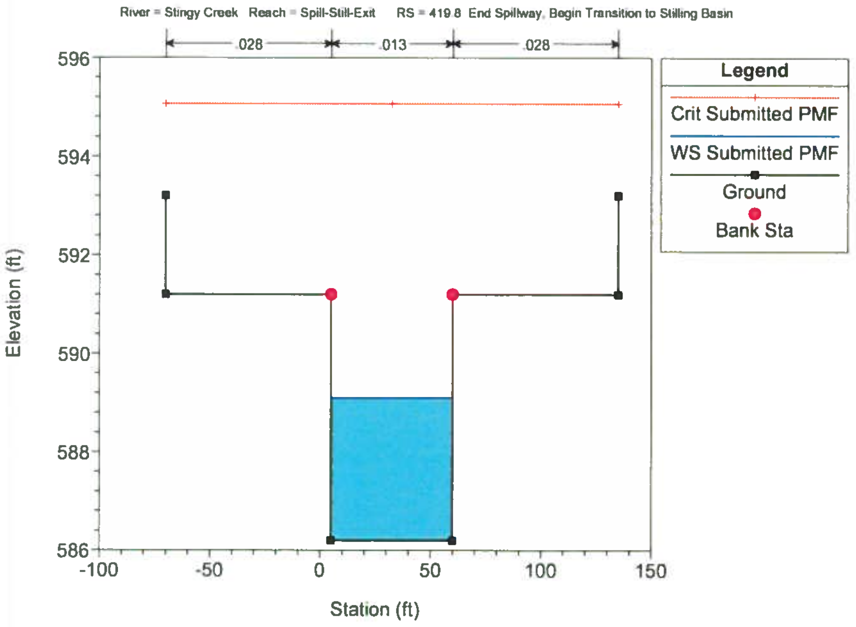

River = Stingy Creek Reach = Spill-Still-Exit RS = 1170 Non-Tier trapazoidal Transition

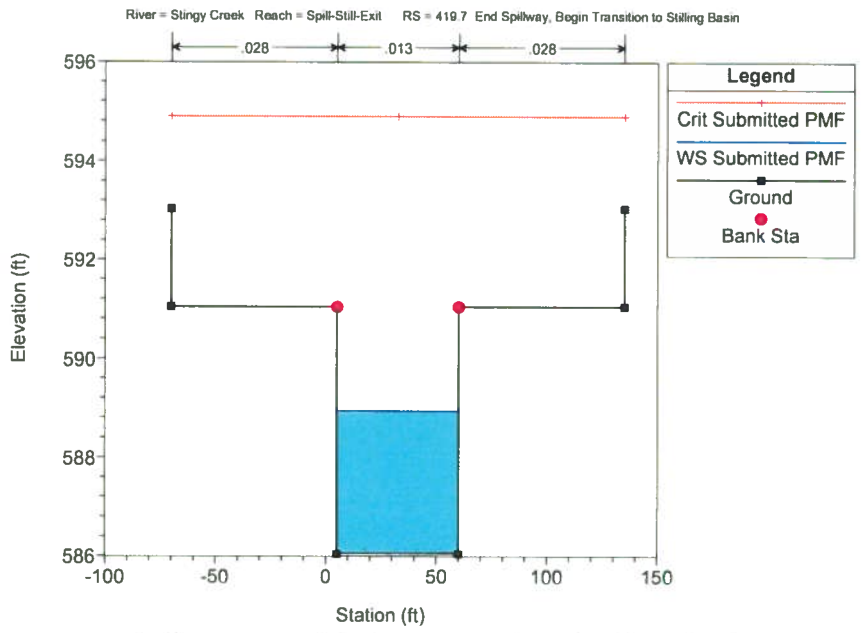

River = Stingy Creek Reach = Spill-Still-Exit RS = 1120 Non-Tier trapazoidal Transition to Rectangle


River = Stingy Creek Reach = Spill-Still-Exit RS = 970 Begin Spillway

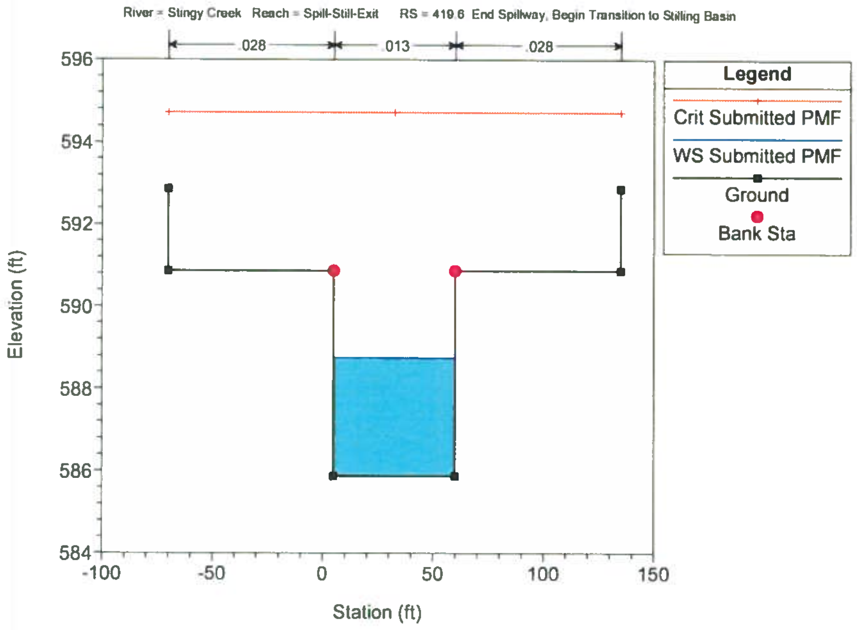

River = Stingy Creek Reach = Spill-Still-Exit RS = 420 End Spillway, Begin Transition to Stilling Basin


River = Stingy Creek Reach = Spill-Still-Exit RS = 419.9 End Spillway, Begin Transition to Stilling Basin

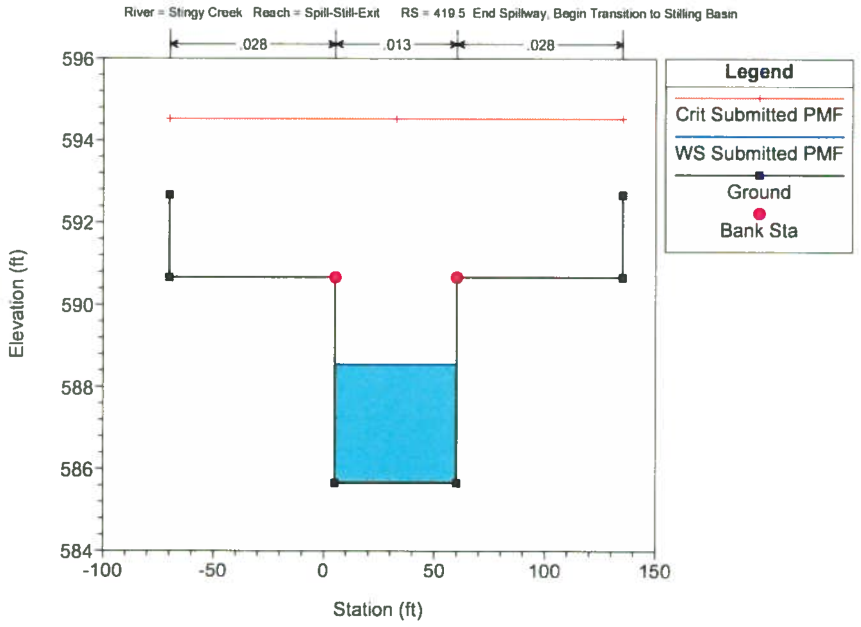

River = Stingy Creek Reach = Spill-Still-Exit RS = 419.8 End Spillway, Begin Transition to Stilling Basin



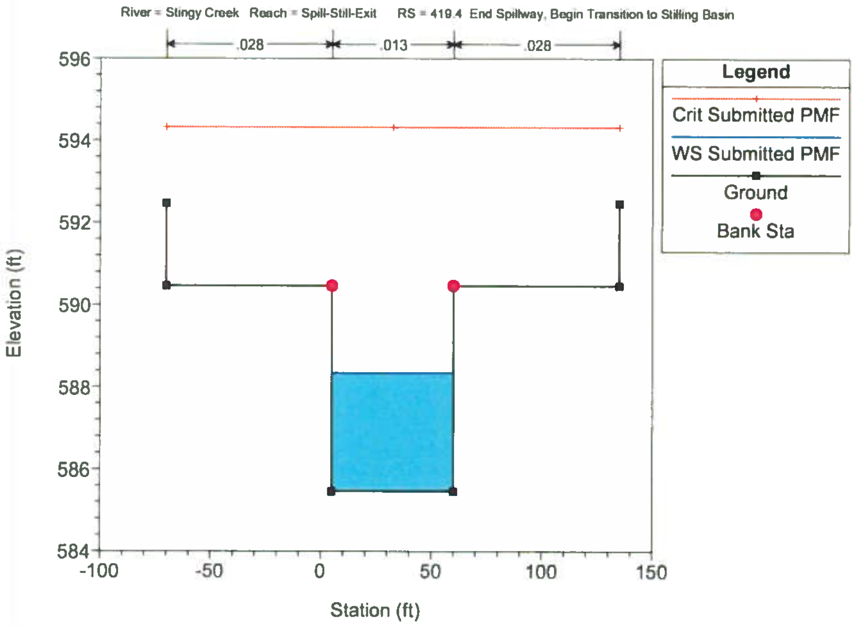
River = Stingy Creek Reach = Spill-Still-Exit RS = 419.7 End Spillway, Begin Transition to Stilling Basin



River = Stingy Creek Reach = Spill-Still-Exit RS = 419.6 End Spillway, Begin Transition to Stilling Basin

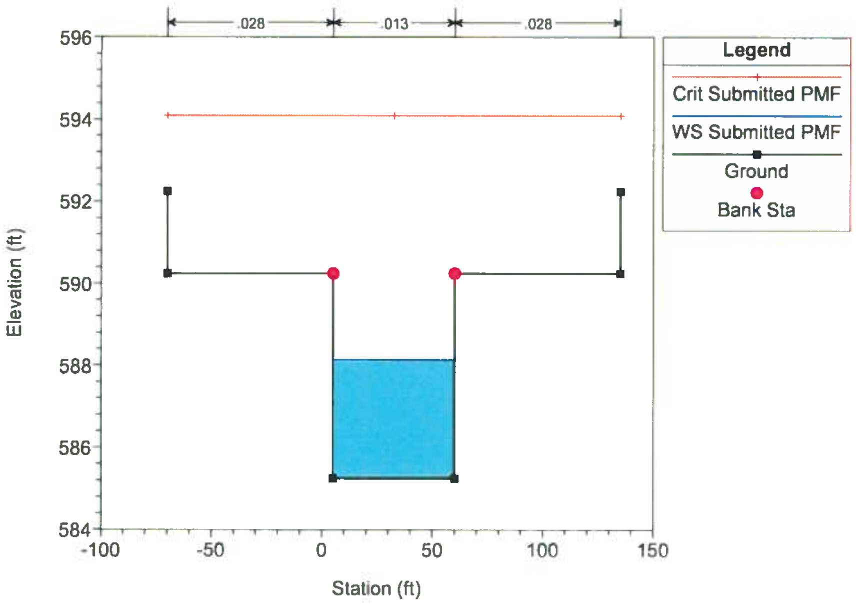


River = Stingy Creek Reach = Spill-Still-Exit RS = 419.5 End Spillway, Begin Transition to Stilling Basin

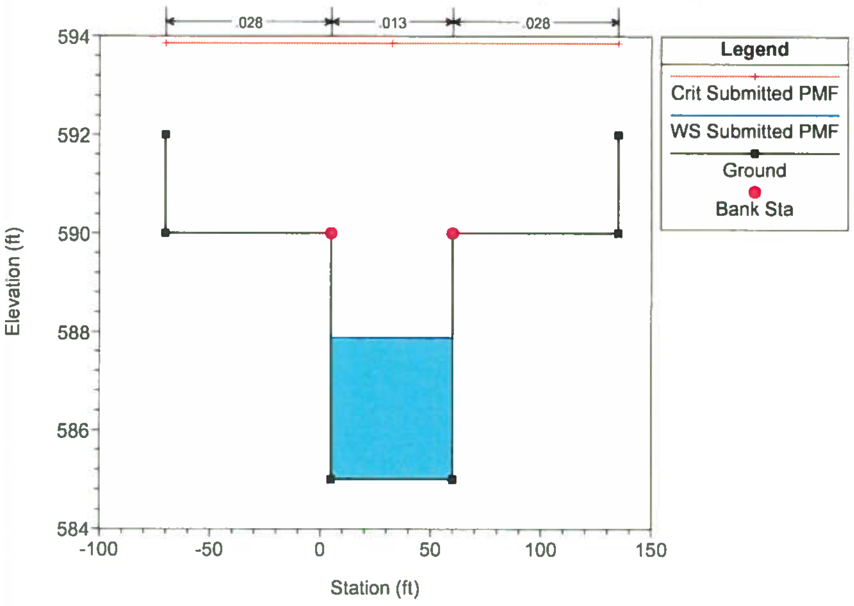


River = Stingy Creek Reach = Spill-Still-Exit RS = 419.4 End Spillway, Begin Transition to Stilling Basin

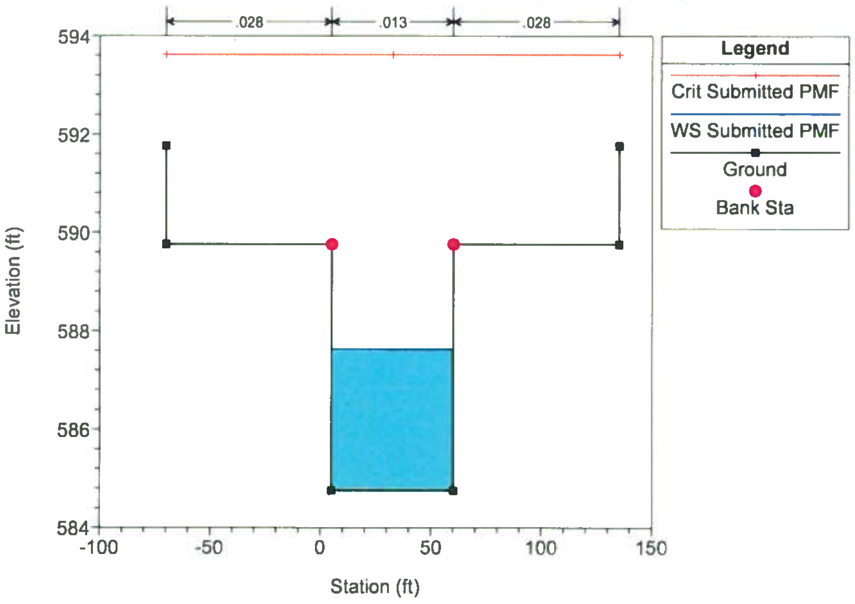
River = Stingy Creek   Reach = Spill-Still-Exit   RS = 419.3 End Spillway, Begin Transition to Stilling Basin



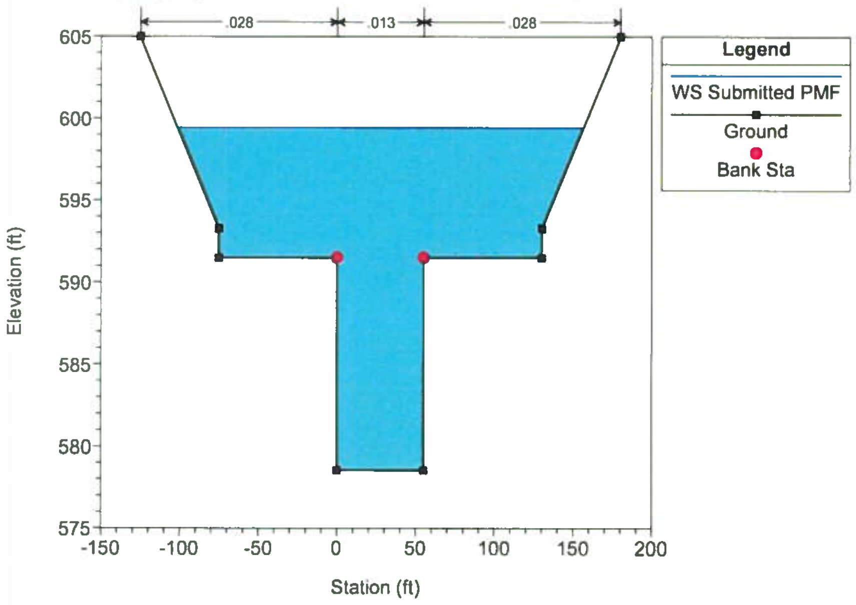
River = Stingy Creek   Reach = Spill-Still-Exit   RS = 419.2 End Spillway, Begin Transition to Stilling Basin



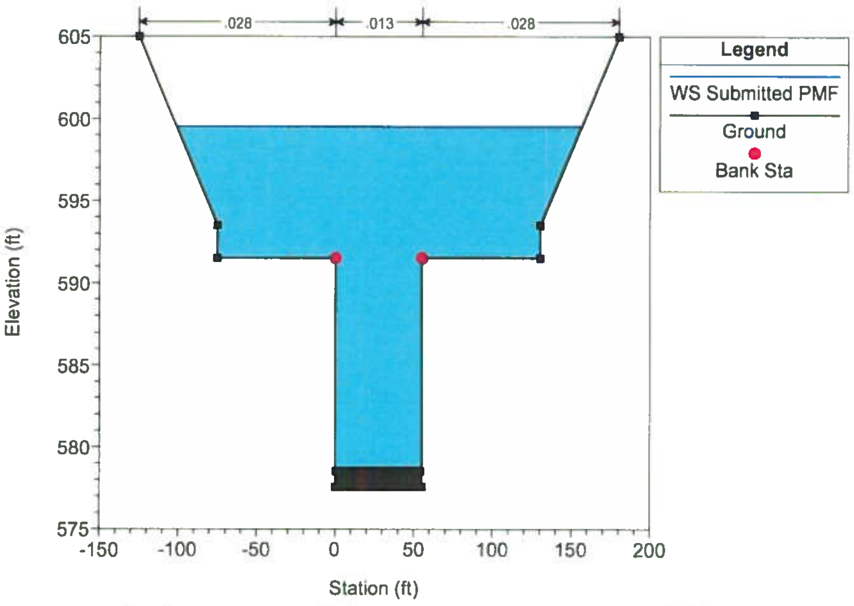
River = Stingy Creek   Reach = Spill-Still-Exit   RS = 419.1 End Spillway, Begin Transition to Stilling Basin



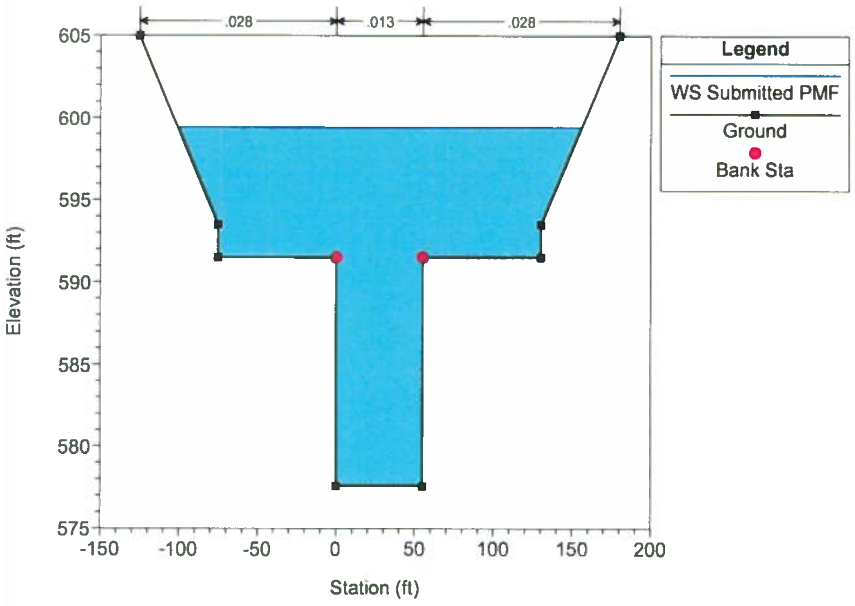
River = Stingy Creek   Reach = Spill-Still-Exit   RS = 414.69 Front of chute blocks Xsec extended 5/1/2014 ACV



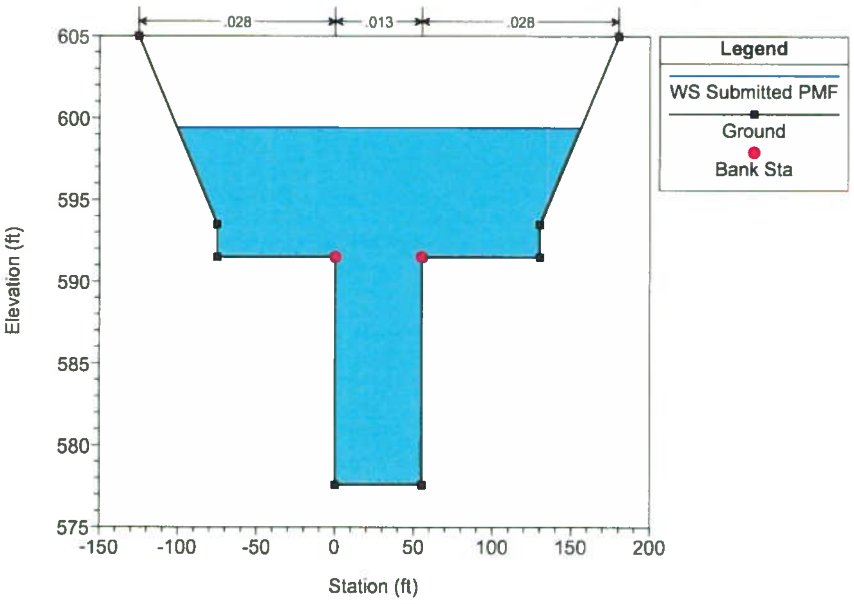
River = Stingy Creek Reach = Spill-Still-Exit RS = 413.46 Top back of chute blocks Xsec extended 5/1/2014 ACV



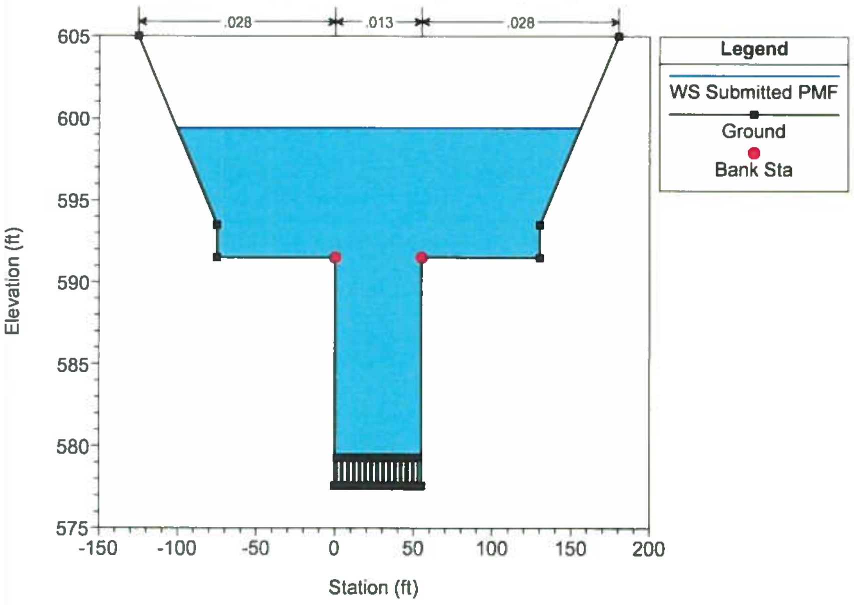
River = Stingy Creek Reach = Spill-Still-Exit RS = 413.45 Bottom Back of chute block Xsec extended 5/1/2014 ACV

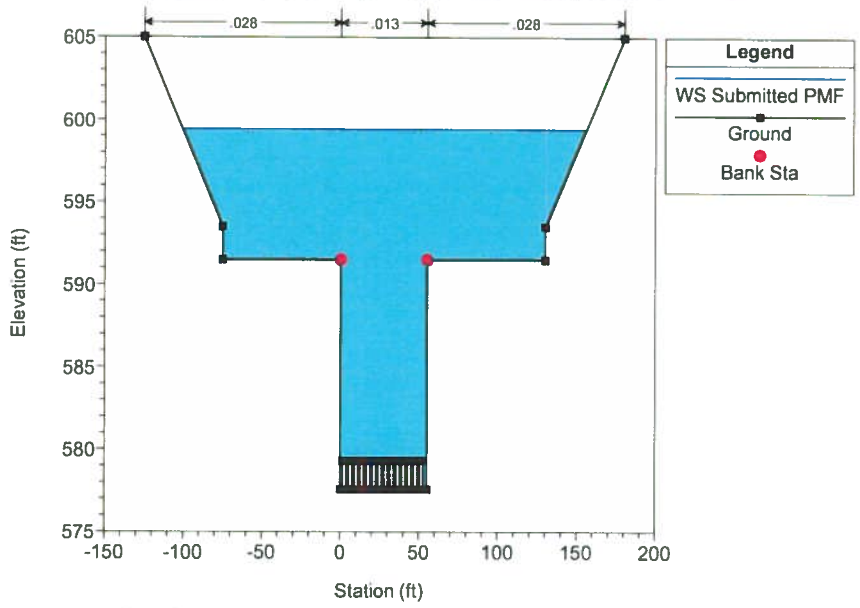


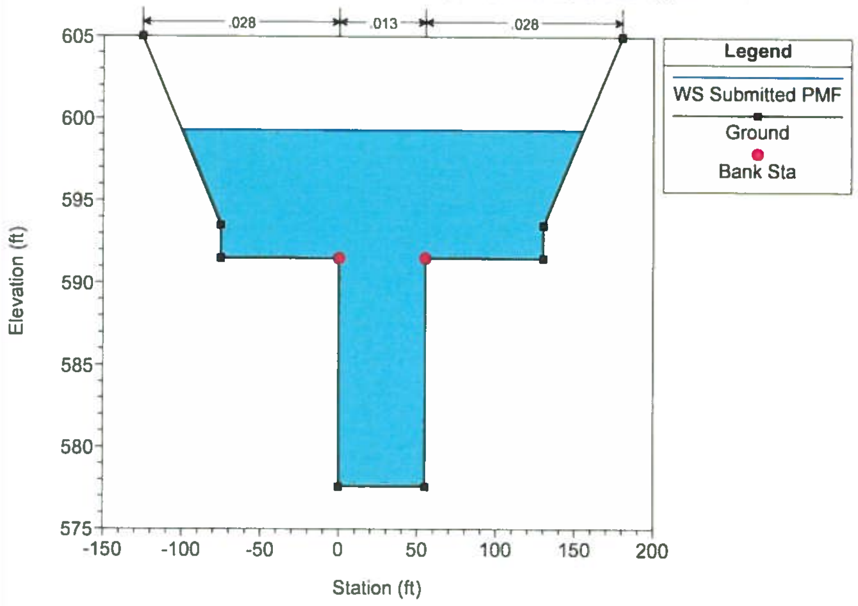
River = Stingy Creek Reach = Spill-Still-Exit RS = 400.97 Bottom front of baffles xsec extended ACV 5/1/2014

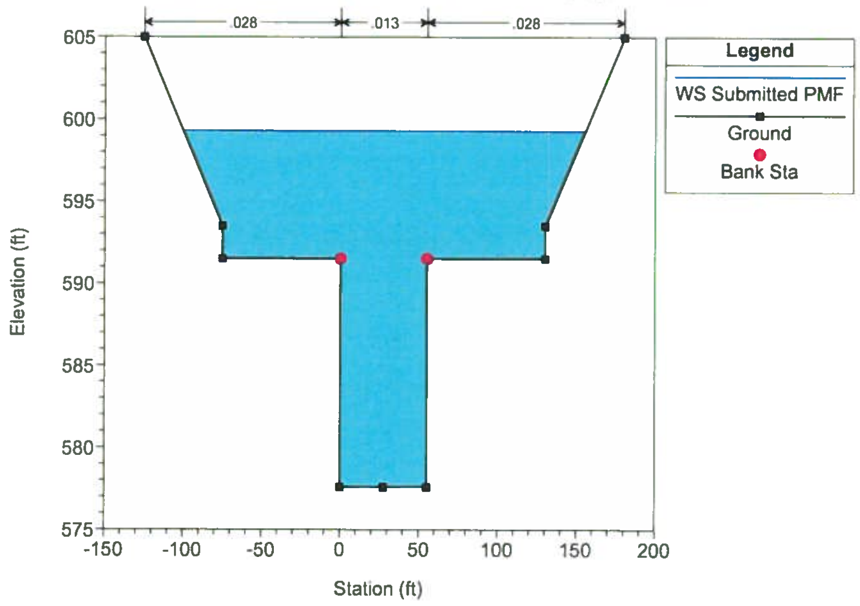


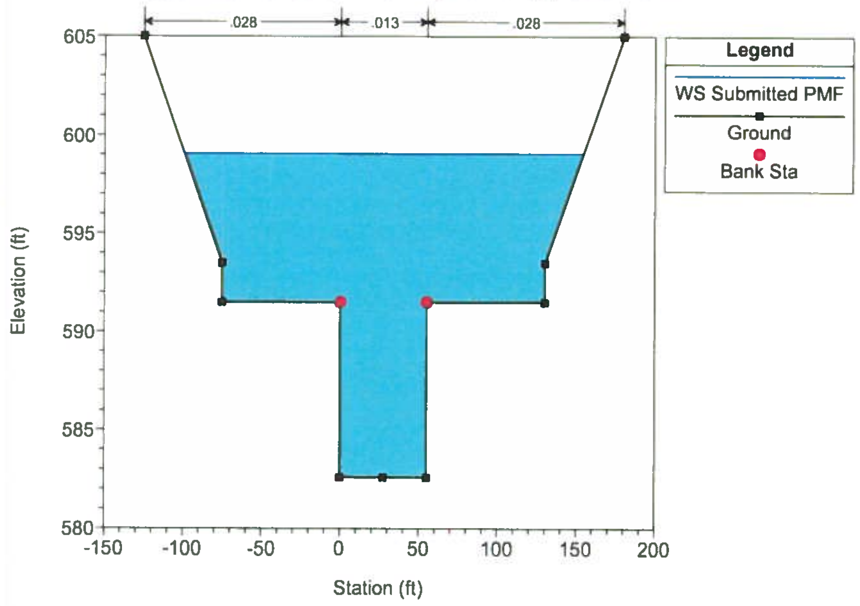
River = Stingy Creek Reach = Spill-Still-Exit RS = 400.98 Top front of baffles Xsec extended 5/1/014 ACV

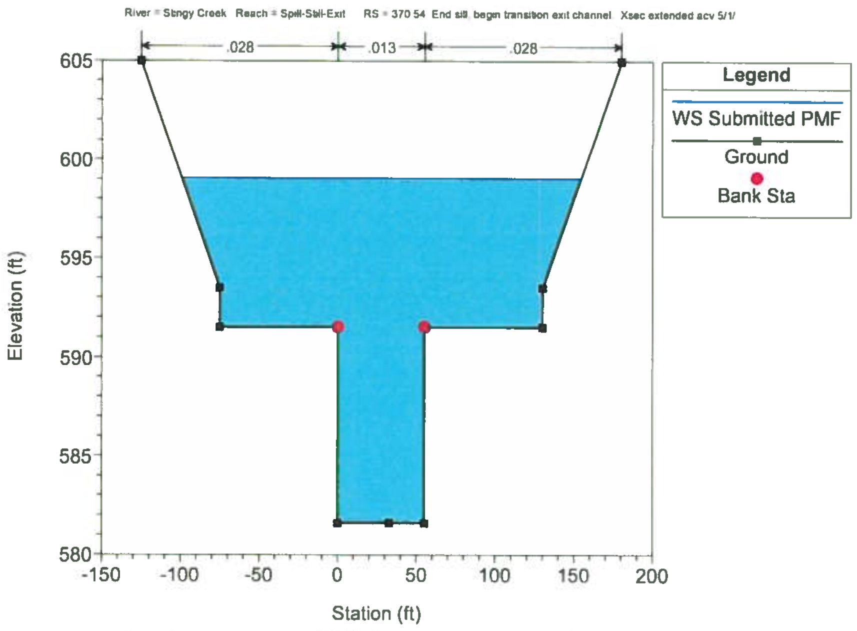


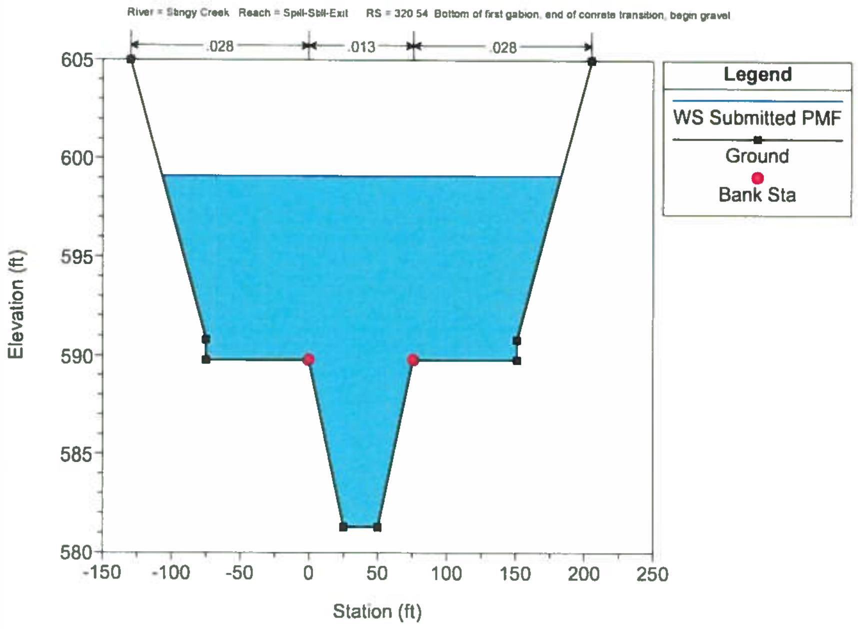

River = Stingy Creek   Reach = Spill-Still-Exit   RS = 400.46 top back of baffles. xsec extended 5/1/2014 ACV

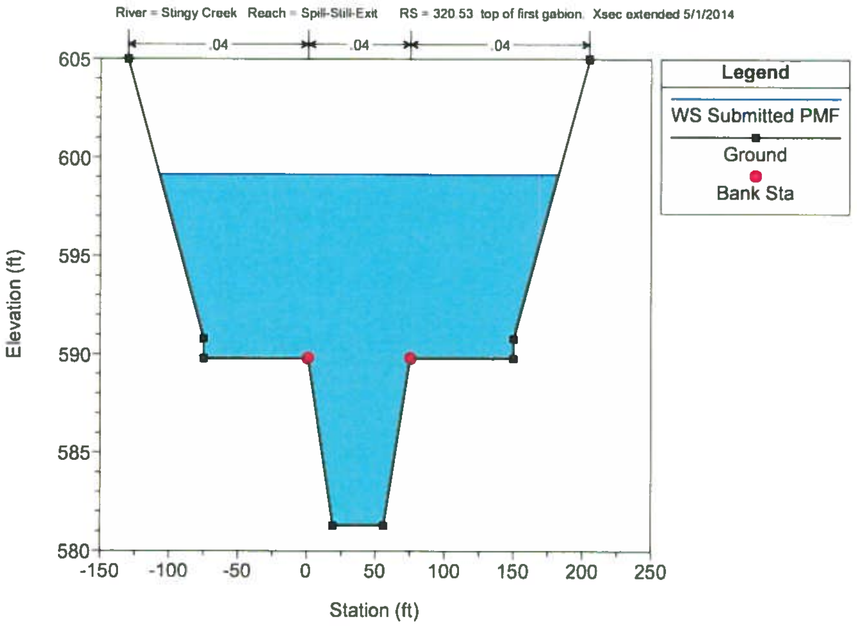

River = Stingy Creek   Reach = Spill-Still-Exit   RS = 387.754 back of baffles. xsec extended ACV 5/1/2014

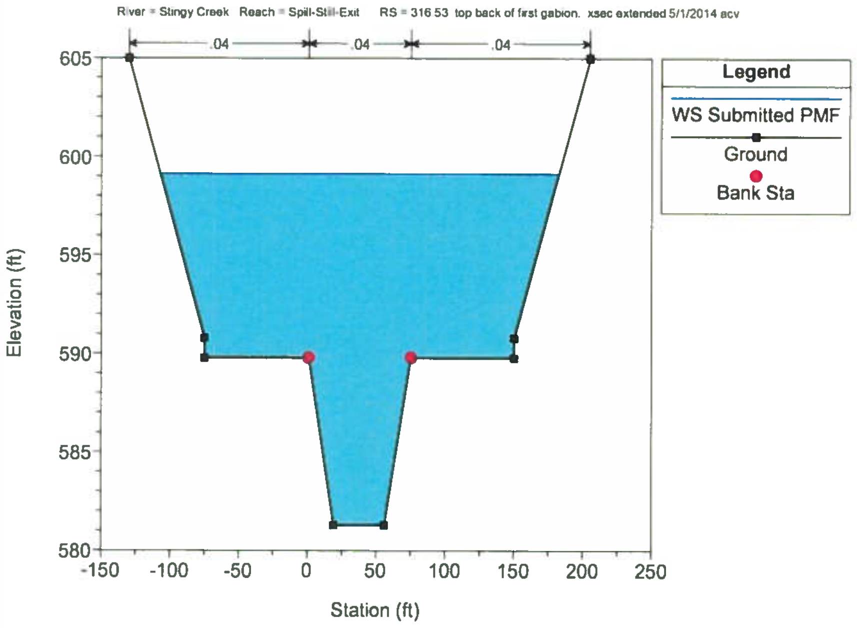

River = Stingy Creek   Reach = Spill-Still-Exit   RS = 380.55 Front edge of sill. xsec extended 5/1/2014 ACV

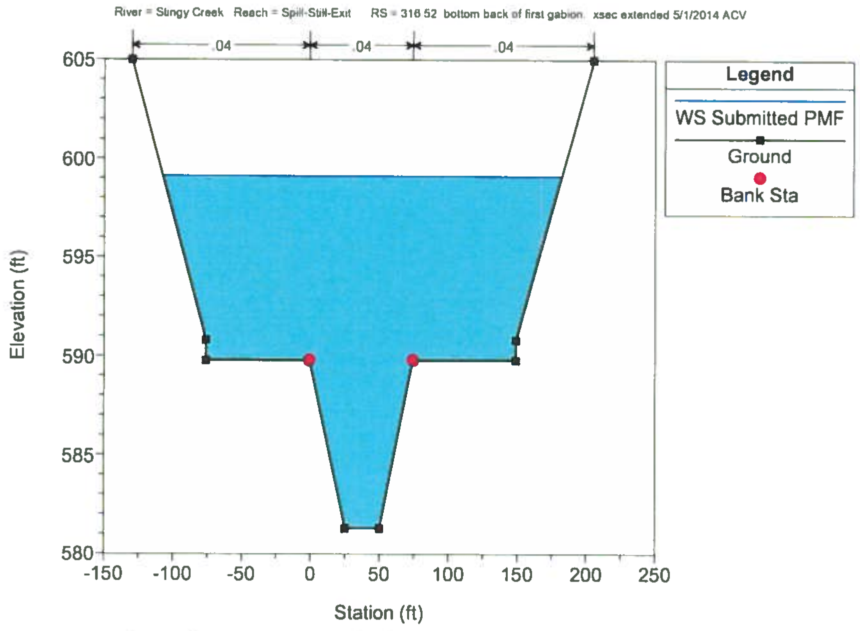

River = Stingy Creek   Reach = Spill-Still-Exit   RS = 370.55 top of sill. Xsec extended 5/1/2014 ACV

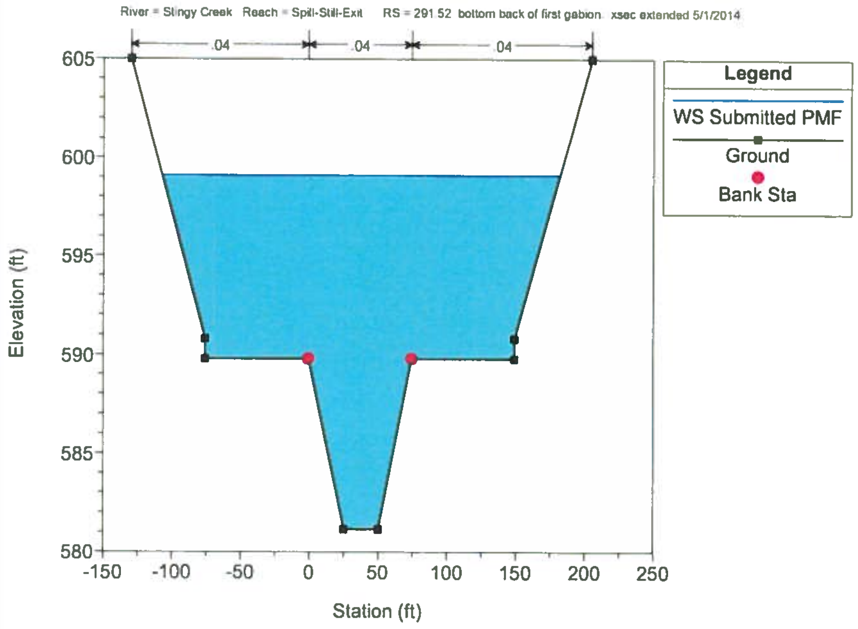

River = Stingy Creek Reach = Spill-Still-Exit RS = 370 54 End sill, begin transition exit channel Xsec extended acv 5/1/
.028
.013
.028
Legend
WS Submitted PMF
Ground
Bank Sta
Elevation (ft)
605
600
595
590
585
580
-150
-100
-50
0
50
100
150
200
Station (ft)

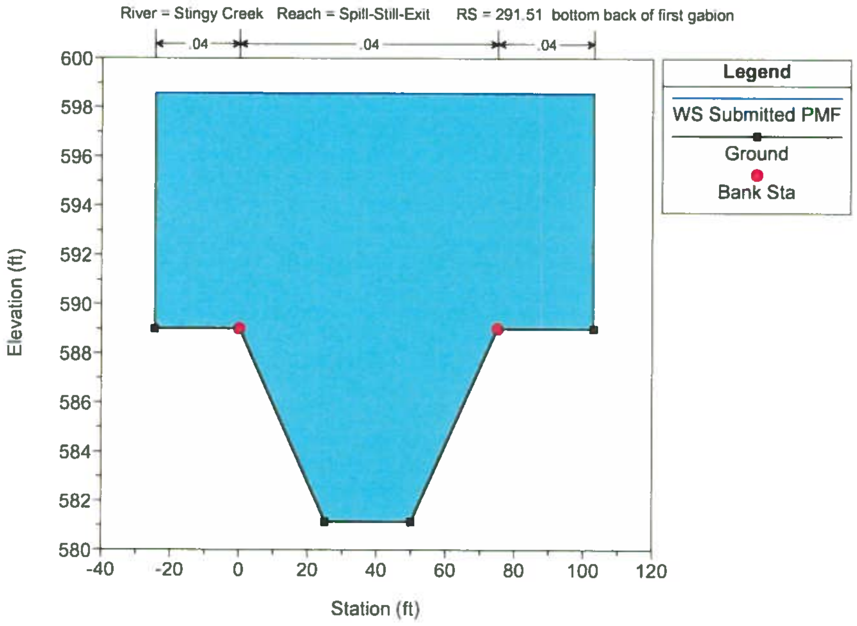

River = Stingy Creek Reach = Spill-Still-Exit RS = 320 54 Bottom of first gabion, end of conrete transition, begin gravel
.028
.013
.028
Legend
WS Submitted PMF
Ground
Bank Sta
Elevation (ft)
605
600
595
590
585
580
-150
-100
-50
0
50
100
150
200
250
Station (ft)

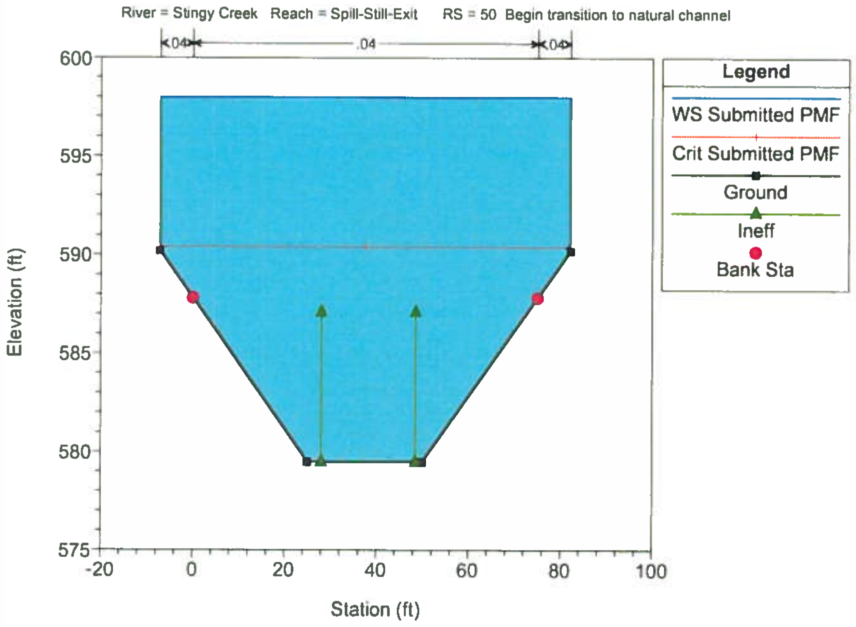

River = Stingy Creek Reach = Spill-Still-Exit RS = 320 53 top of first gabion. Xsec extended 5/1/2014
.04
.04
.04
Legend
WS Submitted PMF
Ground
Bank Sta
Elevation (ft)
605
600
595
590
585
580
-150
-100
-50
0
50
100
150
200
250
Station (ft)

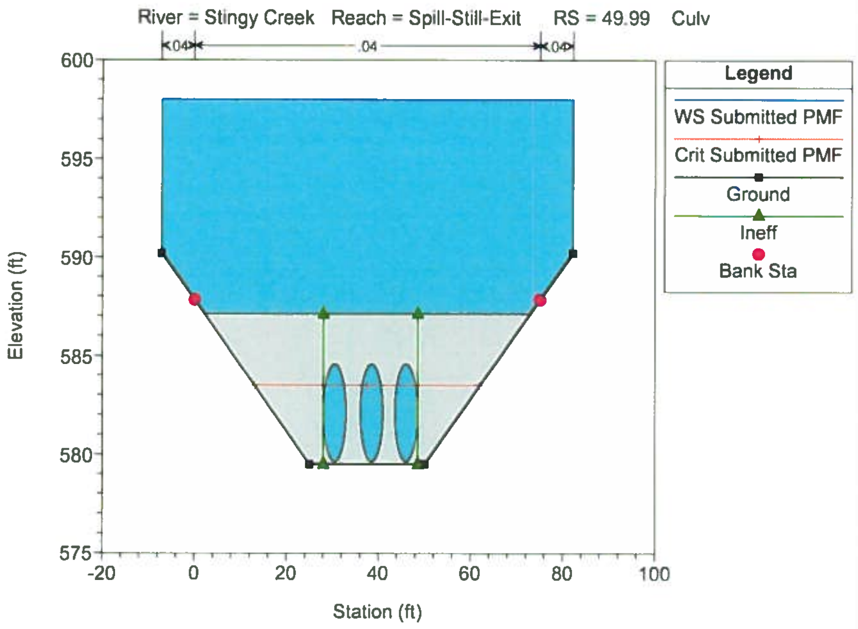

River = Stingy Creek Reach = Spill-Still-Exit RS = 316 53 top back of first gabion. xsec extended 5/1/2014 acv
.04
.04
.04
Legend
WS Submitted PMF
Ground
Bank Sta
Elevation (ft)
605
600
595
590
585
580
-150
-100
-50
0
50
100
150
200
250
Station (ft)

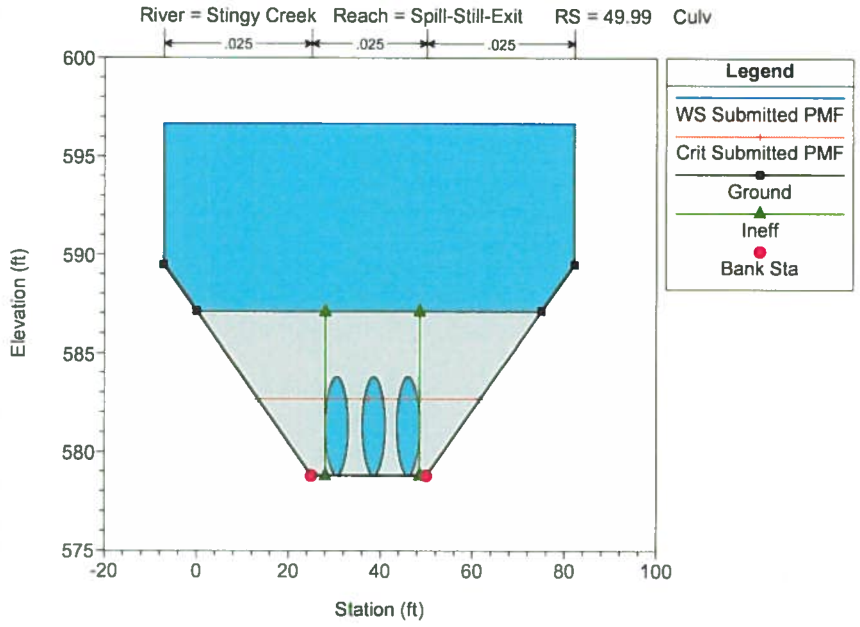

River = Stingy Creek Reach = Spill-Still-Exit RS = 316 52 bottom back of first gabion. xsec extended 5/1/2014 ACV
.04
.04
.04
Legend
WS Submitted PMF
Ground
Bank Sta
Elevation (ft)
605
600
595
590
585
580
-150
-100
-50
0
50
100
150
200
250
Station (ft)

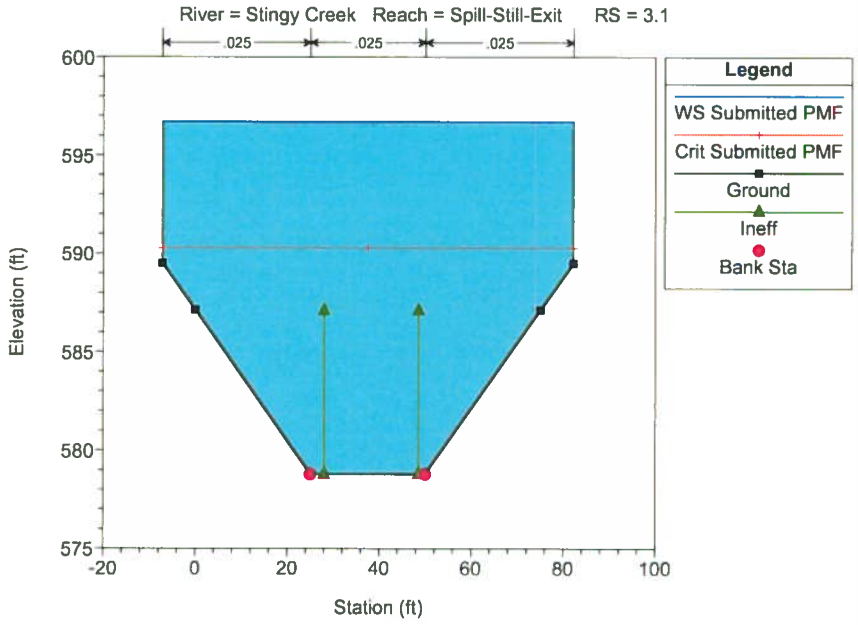

River = Stingy Creek Reach = Spill-Still-Exit RS = 291 52 bottom back of first gabion. xsec extended 5/1/2014
.04
.04
.04
Legend
WS Submitted PMF
Ground
Bank Sta
Elevation (ft)
605
600
595
590
585
580
-150
-100
-50
0
50
100
150
200
250
Station (ft)

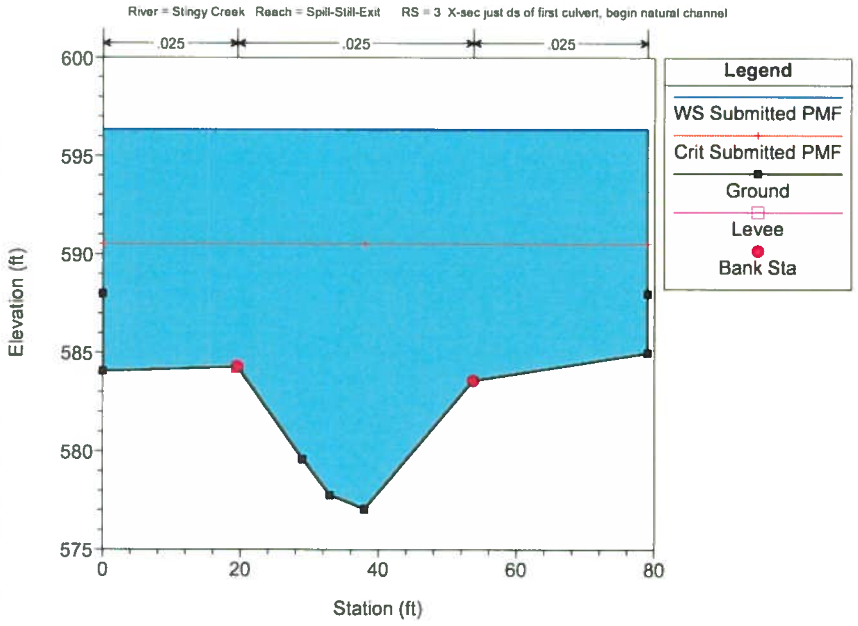

River = Stingy Creek Reach = Spill-Still-Exit RS = 291.51 bottom back of first gabion
.04
.04
.04
Legend
WS Submitted PMF
Ground
Bank Sta
Elevation (ft)
600
598
596
594
592
590
588
586
584
582
580
-40
-20
0
20
40
60
80
100
120
Station (ft)

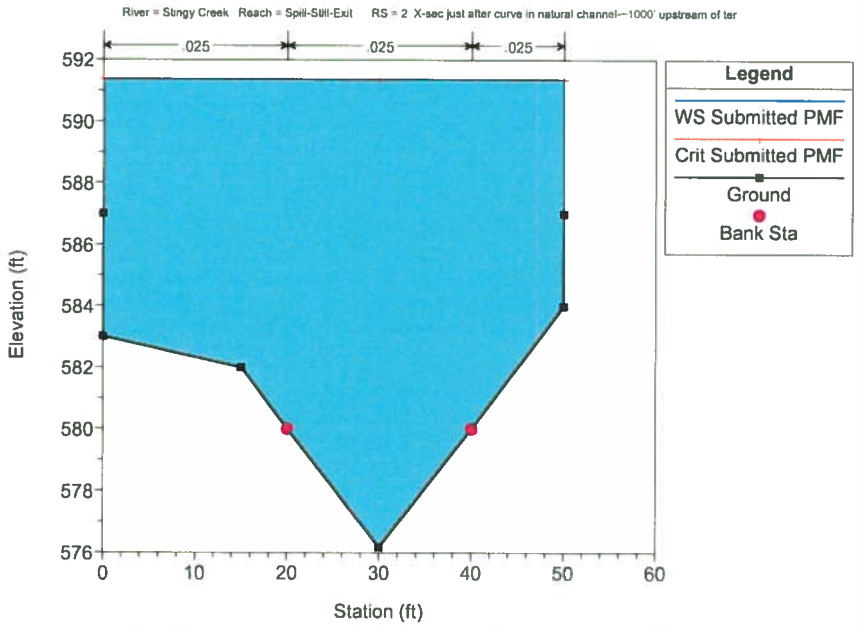

River = Stingy Creek Reach = Spill-Still-Exit RS = 50 Begin transition to natural channel
.04
.04
.04
Legend
WS Submitted PMF
Crit Submitted PMF
Ground
Ineff
Bank Sta
Elevation (ft)
600
595
590
585
580
575
-20
0
20
40
60
80
100
Station (ft)

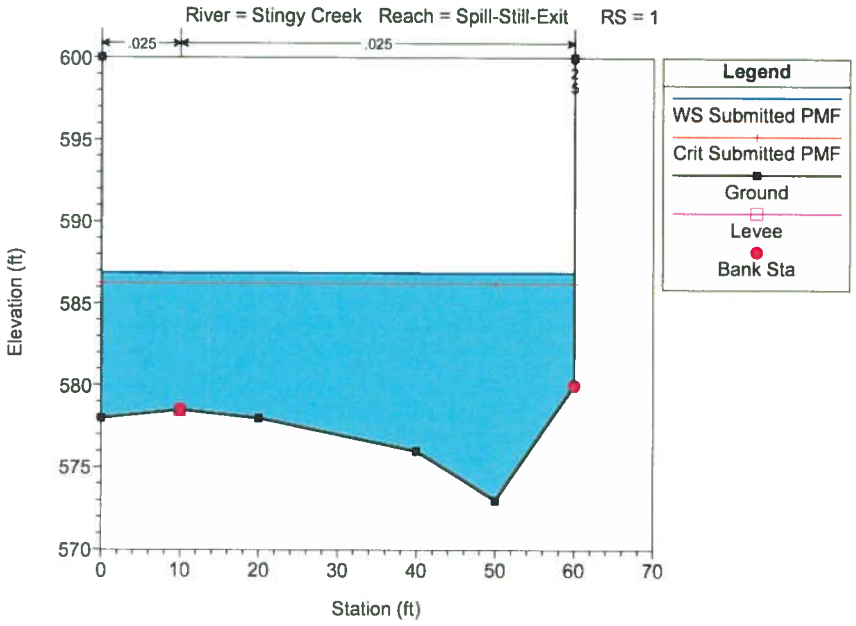

River = Stingy Creek Reach = Spill-Still-Exit RS = 49.99 Culv

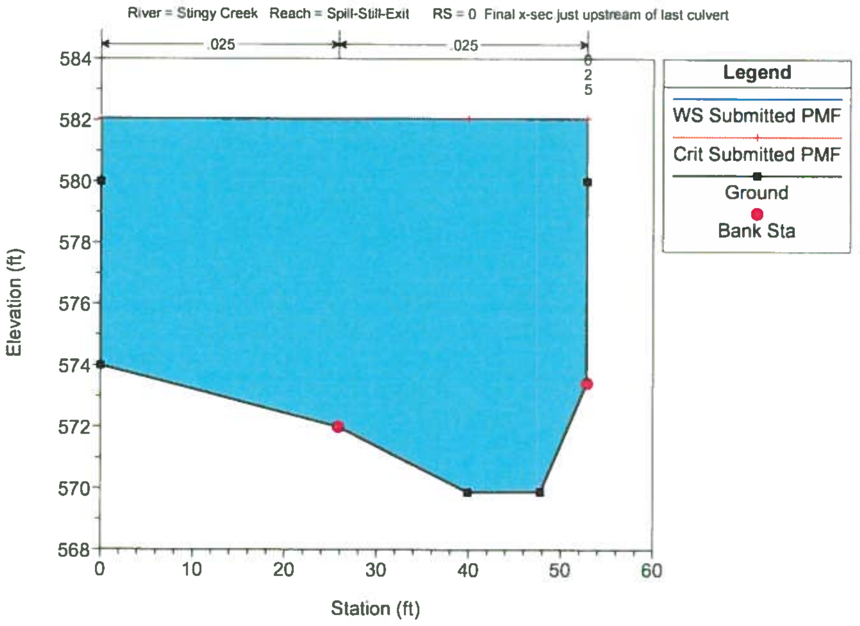

River = Stingy Creek Reach = Spill-Still-Exit RS = 49.99 Culv


River = Stingy Creek Reach = Spill-Still-Exit RS = 3.1


River = Stingy Creek Reach = Spill-Still-Exit RS = 3 X-sec just ds of first culvert, begin natural channel


River = Stingy Creek Reach = Spill-Still-Exit RS = 2 X-sec just after curve in natural channel--~1000' upstream of ter
.025
.025
.025
Legend
WS Submitted PMF
Crit Submitted PMF
Ground
Bank Sta
Elevation (ft)
592
590
588
586
584
582
580
578
576
0
10
20
30
40
50
60
Station (ft)


River = Stingy Creek Reach = Spill-Still-Exit RS = 1
.025
.025
.025
Legend
WS Submitted PMF
Crit Submitted PMF
Ground
Levee
Bank Sta
Elevation (ft)
600
595
590
585
580
575
570
0
10
20
30
40
50
60
70
Station (ft)


River = Stingy Creek Reach = Spill-Still-Exit RS = 0 Final x-sec just upstream of last culvert
.025
.025
.025
Legend
WS Submitted PMF
Crit Submitted PMF
Ground
Bank Sta
Elevation (ft)
584
582
580
578
576
574
572
570
568
0
10
20
30
40
50
60
Station (ft)

APPENDIX F

LOCAL DRAINAGE AND STORMWATER MANAGEMENT FINAL AND INTERIM CONDITIONS DESIGN CALCULATIONS

(DOWNSTREAM OF DAM CREST)

TABLE OF CONTENTS

Geosyntec consultants

Written by: JNJ **Date:** 10/15/2015
Reviewed by: RM **Date:** 11/10/2015
Client: AEP **Project:** FAR Closure **Project No.:** CHE8273 **Task No.:** 02/05.22

ATTACHMENTS

1. Reference Documentation
    - NOAA Rainfall Point Precipitation Frequency Estimates
    - TR-55 Curve Number Chart
    - TR-55 Sheet Flow Velocity Chart
2. Drainage Exhibits
3. Summary tables
    - Table 3-1: Swale Design Summary
    - Table 3-2: Culvert Design Summary
    - Table 3-3: Riprap Design Summary
4. HEC-HMS and HY-8 inputs
5. Conveyance feature design calculations (swales & culverts)
6. Flow vs Area rating curves and tables
7. Erosion Control Materials Design Software Input
8. Sediment Basin Stage vs. Storage

LOCAL DRAINAGE AND STORMWATER MANAGEMENT FINAL AND INTERIM CONDITIONS

(DOWNSTREAM OF DAM CREST)

INTRODUCTION

The purpose of the local drainage and interim conditions package is to present the hydrologic analysis for the sizing of stormwater management and pollution prevention features during construction and post-development. The local drainage features include two drainage swales, three ditches, and four culverts that convey local runoff from the face of the proposed dam (i.e., local drainage features do not include runoff from the reservoir). The interim conditions design features include sizing sediment basins and riprap pads. The specific goals of the analysis include calculating peak discharge of stormwater from the site via drainage features for stormwater management during construction and post-development. The local drainage features consist of permanent features that will convey stormwater. This calculation package addresses the peak flow, peak velocity, and peak depth of water on drainage features that are necessary for the design of the local drainage and interim conditions features, to demonstrate that these features are adequately sized.

BASIS OF DESIGN

Local Drainage Calculations

- 25-year, 24-hour design storm (Safely convey 100-year 24-hour design storm)

Stormwater Conveyance Features – Drainage Terraces and Benches, Surface Water Drainage Channels, and Culverts

- Convey 25-year 24-hour design storm
- Resist erosion from 25-year 24-hour design storm

Interim Conditions Calculations

- Interim SWP3 Calculations
    - *Rainwater and Land Development; Ohio's Standards for Stormwater Management Land Development and Urban Stream Protection*, Third Edition 2006, Updated March 3, 2014 (herein referred to as "ODNR Manual").
    - OEPA's *General Permit Authorization for Storm Water Discharges Associated with Construction Activity under the National Pollutant Discharge Elimination System* (Permit No. OHC000004; herein referred to as "General Permit")
    - Convey 2-year, 24-hour design storm (Riprap Pads)

METHOD OF ANALYSIS

Local Drainage Features

The watershed analysis, including subbasin delineation and time of concentration and curve number calculations, was performed using procedures described in the documents, "Urban Hydrology for Small Watersheds, Technical Release 55", (USDA-SCS, 1986). The computer program HEC-HMS was used to perform the hydrologic analysis to calculate the peak flows for drainage areas of varying sizes. The computer program HY-8 was used to perform the hydraulic analysis of the culverts. Microsoft Excel ® spreadsheets were developed to perform several of the other supporting calculations, such as time of concentration, and swale conveyance capacity.

Interim Construction Stages

Interim construction control features were designed based on guidance from the Ohio DNR Stormwater Management Manual and the General Permit. Sizing of riprap for interim features was performed using procedures described in the document, "Urban Hydrology for Small Watersheds, Technical Release 55", (USDA-SCS, 1986). The computer program HEC-HMS was used to perform the hydrologic analysis to calculate the peak flows for drainage areas of varying sizes. The methodology presented in the Illinois Urban Manual Practice Standard Code 872 was used to size the riprap.

ANALYZED CONDITIONS

HEC-HMS was used to determine the stormwater volume and peak flows that must be conveyed for the selected design storm for the local drainage post-development condition and during construction. A Drainage exhibit is presented in Attachment 2. Summary tables of the hydrologic parameters and results are presented in Attachment 3.

Interim Conditions:

There will be temporary swales, diversion berms, and sediment basins to convey stormwater during construction. The construction grading and areas modeled in HEC-HMS are shown in Attachment 2.

Local Drainage:

There will be permanent swales and culverts to convey the flow from local drainage. The local drainage grading and areas modeled in HEC-HMS are shown in Attachment 2.

PARAMETERS USED IN ANALYSIS

The following describes the selection of the various hydrologic parameters used for watershed analysis.

- **Rainfall Distribution and Depth**: Based on precipitation data from the *"Precipitation-Frequency Atlas of the United States"*, (NOAA National Weather Service, 2011, http://hdsc.nws.noaa.gov/hdsc/pdfs/); the 24-hour precipitation magnitudes for 2-year, 10-year, 25-year, and 100-year return periods are used in the hydrologic analyses (see Attachment 1):

- **Hydrologic Soil Group**:

Construction Stage: For the various construction stages, the newly graded areas are anticipated to exhibit similar characteristics to soils of Hydrologic Soils Group C

Final Cover Case: Soil used to construct the final grading for the local drainage will consist of low permeability and topsoil material borrowed from nearby areas, which will also exhibit characteristics of Hydrologic Soils Group C.

- **Curve Number (CN):**

Existing Condition and Construction Stages: A curve number (CN) of 91 is selected to represent newly graded areas.

Final Cover Case: For the final cover system, a CN of 75 is used, the value recommended by SCS for 80% hydrologic soil group C for "open spaces in good condition (grass cover > 75%)" and 20% hydrologic soil group C for "open spaces in fair condition (grass cover 50% - 75%). A summary of runoff CN values provided by SCS are provided in Appendix 1.

- **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, 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 peak length of 150 ft. TR-55 allows up to 300 feet, however a variety of publications suggests using a length shorter than 300 ft. A length of 150 ft was selected;
    - **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:

$$T_t = \frac{L}{3{,}600\ V}$$

- **Open Channel Flow:**

$$T_t = \frac{nL}{3{,}600\ (1.49)\ r^{2/3}\ s^{1/2}}$$

where: T_t = travel time (hours);
n = Manning's roughness coefficient (dimensionless);
  - n = 0.4 for woods, light underbrush
  - n = 0.15 for short grass
  - n = 0.05 for fallow
  - n = 0.03 for open channel, earth, grass and winding
  - n = 0.025 for open channel, earth, clean and straight
  - n = 0.022 for slope drain, provided by manufacturer

L = length of flow (ft.);
P_2 = rainfall from a 2-year, 24-hour storm (in.);
s = Bed or surface slope in the flow direction (ft/ft);
V = velocity (ft/sec); and
r = hydraulic radius (ft.).

Hydraulic radius, r, is defined as A/P, where A is flow area and P is the wetted perimeter of the cross section.

TR-55 provides a graphical solution for T_t for shallow concentrated flow over "paved" and "unpaved areas". The "*National Engineering Handbook, Section 4 Hydrology (NEH-4)*", (SCS, 1985) provides the following equation for V:

$$V = K_v \sqrt{s}$$

where K_v is a velocity factor based on various surface conditions (i.e., paved, unpaved, grassed waterway, short grass pasture, etc.) and s is the slope of the land surface. TR-55 provides a graph of velocity factors (K_v), which are used to calculate V in the above equation. The value of K_V used for these calculations were obtain from Figure 3-1 in the TR-55 manual.

INTERIM CONDITIONS DESIGN

The ODNR Manual and General Permit were used to size the majority of stormwater management controls during construction. Hydraulic analysis was only performed to size the riprap pads and channel protection. The riprap pads are designed to effectively armor channels and slopes from the 2-year, 24-hour storm; however, selected riprap will protect up to the 25-year, 24-hour storm. The riprap and channel protection are designed to effectively convey the runoff from the 2 year, 24 hour design storm. The riprap design summary is presented in Table 3-3 in Attachment 3. The interim conditions drainage figure is presented in Attachment 2.

The sediment basins were designed to meet the requirements of the General Permit that requires 1000 cubic feet of dead storage per acre of disturbed drainage area and 1800 cubic feet of live storage per acre of disturbed drainage area. Two sediment basins were designed with skimmers. A summary table and stage vs. storage table and skimmer sizing calculations are presented in Attachment 8.

SWALE AND CULVERT DESIGN – HYDRAULIC ANALYSIS

The post-development local drainage figure in Attachment 2 of this Appendix shows a plan view of the swales with culvert locations and designations.

The permanent swales and culverts are designed to effectively convey the runoff from the 25 year, 24 hour design storm. Attachment 5 presents the hydraulic design calculations of the proposed swales. Tables 3-1 and 3-2 in Attachment 3 summarize the peak flows of each swale and culvert. Microsoft Excel ® spreadsheets were developed to analyze the conveyance capacity of the select drainage swales. Attachment 5 presents the hydraulic design calculations of the proposed swales. The computer program HY-8 was used to perform the hydraulic analysis of the culverts.The HEC-HMS rating curves and tables are presented in Attachment 6.

CHANNEL AND SLOPE PROTECTION DESIGN

Slope and channel protection materials were selected from the Erosion Control Materials Design Software developed by North American Green (ECMDS Version 5.0). Hydraulic inputs were calculated from the conveyance feature design tables presented in Attachment 5. Model inputs are presented in Attachment 7 and selected erosion control materials are presented in the summary table in Attachment 3.

HYDRAULIC ANALYSIS RESULTS

The maximum flow capacity of all swales and culverts (as presented in Attachment 5 respectively) is greater than the estimated peak discharge from the 25 year, 24 hour design storm. In addition, swales are designed to safely convey the 100 year, 24 hour design storm.

SUMMARY AND CONCLUSIONS

The swales and culverts for the local drainage have been designed to effectively convey the runoff from the 25 year, 24 hour design storm and safely convey the 100 year, 24 hour design storm. The interim conditions features have been designed to effectively convey the 2 year, 24 hour storm.

REFERENCES

NOAA National Weather Service, *"Precipitation-Frequency Atlas of the United States"* Atlas 14, Volume 2, Version 3, Silver Spring Maryland, 2004.

United States Department of Agriculture, Soil Conservation Service (USDA-SCS), "*Computer Program for Project Formulation Hydrology, Technical Release 20*", Washington D.C., 1982.

United States Department of Agriculture, Soil Conservation Service (USDA-SCS), *"National Engineering Handbook, Section 4 Hydrology (NEH-4)"*, National Technical Information Service, 1985.

United States Department of Agriculture, Soil Conservation Service (USDA-SCS), "*Urban Hydrology for Small Watersheds, Technical Release 55*", 2nd ed., Washington, D.C., 1986.

US Army Corps of Engineers, Hydrologic Engineering Center, "Hydrologic Modeling System HEC-HMS User's Manual," Version 3, Davis California, August 2010.

ATTACHMENTS

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Written by: JNJ **Date:** 10/15/2015
Reviewed by: RM **Date:** 11/10/2015
Client: AEP **Project:** FAR Closure **Project No.:** CHE8273 **Task No.:** 02/05.22

ATTACHMENT 1

Reference Documentation

a) NOAA Rainfall Point Precipitation Frequency Estimates
b) TR-55 Curve Number Chart
c) TR-55 Sheet Flow Velocity Chart
d) Manning's n values

NOAA Atlas 14, Volume 2, Version 3
Location name: Cheshire, Ohio, US*
Coordinates: 38.9400, -82.1200
Elevation: 566ft*
* 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

http://hdsc.nws.noaa.gov/hdsc/pfds/

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	0.340 (0.306–0.374)	0.404 (0.367–0.446)	0.486 (0.441–0.534)	0.548 (0.496–0.602)	0.625 (0.564–0.686)	0.684 (0.616–0.749)	0.740 (0.664–0.808)	0.795 (0.711–0.867)	0.866 (0.771–0.944)	0.918 (0.813–0.995)
10-min	0.528 (0.475–0.581)	0.631 (0.573–0.696)	0.755 (0.685–0.830)	0.846 (0.766–0.930)	0.956 (0.863–1.05)	1.04 (0.934–1.14)	1.11 (0.992–1.22)	1.19 (1.06–1.29)	1.27 (1.13–1.39)	1.34 (1.18–1.45)
15-min	0.647 (0.586–0.712)	0.772 (0.701–0.851)	0.927 (0.841–1.02)	1.04 (0.943–1.14)	1.18 (1.07–1.30)	1.28 (1.16–1.41)	1.38 (1.24–1.51)	1.48 (1.32–1.61)	1.59 (1.41–1.73)	1.67 (1.48–1.82)
30-min	0.857 (0.776–0.942)	1.03 (0.936–1.14)	1.27 (1.15–1.40)	1.45 (1.31–1.59)	1.67 (1.51–1.83)	1.84 (1.65–2.01)	2.00 (1.79–2.18)	2.15 (1.93–2.35)	2.36 (2.10–2.57)	2.50 (2.22–2.72)
60-min	1.05 (0.947–1.15)	1.27 (1.15–1.40)	1.59 (1.45–1.75)	1.84 (1.67–2.02)	2.16 (1.95–2.37)	2.42 (2.18–2.65)	2.67 (2.40–2.92)	2.92 (2.61–3.19)	3.26 (2.90–3.56)	3.52 (3.12–3.83)
2-hr	1.23 (1.11–1.35)	1.48 (1.35–1.63)	1.86 (1.69–2.04)	2.16 (1.96–2.36)	2.56 (2.30–2.80)	2.88 (2.58–3.14)	3.20 (2.85–3.49)	3.54 (3.14–3.85)	3.99 (3.52–4.33)	4.35 (3.81–4.71)
3-hr	1.30 (1.18–1.44)	1.57 (1.42–1.74)	1.96 (1.78–2.17)	2.28 (2.06–2.51)	2.71 (2.44–2.98)	3.06 (2.74–3.35)	3.42 (3.05–3.74)	3.78 (3.36–4.13)	4.29 (3.78–4.68)	4.70 (4.11–5.11)
6-hr	1.55 (1.42–1.70)	1.86 (1.70–2.05)	2.31 (2.11–2.54)	2.67 (2.43–2.93)	3.18 (2.88–3.48)	3.60 (3.24–3.93)	4.03 (3.61–4.39)	4.48 (3.99–4.88)	5.12 (4.51–5.56)	5.63 (4.92–6.09)
12-hr	1.82 (1.65–1.99)	2.18 (2.01–2.37)	2.67 (2.46–2.92)	3.08 (2.83–3.35)	3.67 (3.35–3.97)	4.15 (3.77–4.46)	4.66 (4.21–5.02)	5.19 (4.66–5.59)	5.95 (5.29–6.38)	6.57 (5.79–7.03)
24-hr	2.20 (2.07–2.35)	2.62 (2.46–2.81)	3.19 (2.99–3.40)	3.64 (3.41–3.89)	4.28 (4.00–4.56)	4.79 (4.46–5.10)	5.33 (4.94–5.66)	5.89 (5.44–6.25)	6.66 (6.11–7.07)	7.28 (6.64–7.71)
2-day	2.59 (2.44–2.76)	3.08 (2.89–3.29)	3.70 (3.48–3.94)	4.20 (3.94–4.47)	4.88 (4.56–5.19)	5.42 (5.06–5.76)	5.98 (5.56–6.35)	6.55 (6.07–6.97)	7.34 (6.75–7.80)	7.94 (7.27–8.45)
3-day	2.78 (2.62–2.96)	3.30 (3.11–3.52)	3.95 (3.72–4.20)	4.46 (4.19–4.75)	5.16 (4.84–5.48)	5.71 (5.34–6.06)	6.27 (5.85–6.65)	6.84 (6.36–7.26)	7.61 (7.03–8.07)	8.19 (7.54–8.70)
4-day	2.98 (2.81–3.16)	3.53 (3.32–3.75)	4.20 (3.96–4.47)	4.73 (4.45–5.02)	5.45 (5.11–5.77)	6.00 (5.62–6.36)	6.56 (6.13–6.95)	7.13 (6.64–7.55)	7.88 (7.31–8.34)	8.45 (7.80–8.95)
7-day	3.58 (3.38–3.79)	4.23 (4.00–4.49)	4.99 (4.71–5.29)	5.57 (5.25–5.90)	6.34 (5.97–6.71)	6.93 (6.51–7.33)	7.51 (7.04–7.94)	8.08 (7.56–8.55)	8.84 (8.22–9.34)	9.40 (8.72–9.95)
10-day	4.09 (3.87–4.32)	4.82 (4.56–5.10)	5.62 (5.32–5.94)	6.24 (5.90–6.59)	7.03 (6.64–7.42)	7.63 (7.19–8.06)	8.22 (7.72–8.67)	8.78 (8.24–9.28)	9.51 (8.89–10.0)	10.0 (9.36–10.6)
20-day	5.69 (5.40–5.98)	6.67 (6.34–7.01)	7.67 (7.28–8.06)	8.42 (7.99–8.84)	9.37 (8.88–9.84)	10.1 (9.54–10.6)	10.7 (10.2–11.3)	11.4 (10.7–11.9)	12.1 (11.4–12.8)	12.7 (11.9–13.4)
30-day	7.10 (6.79–7.43)	8.31 (7.94–8.69)	9.44 (9.02–9.88)	10.3 (9.84–10.6)	11.4 (10.8–11.9)	12.2 (11.6–12.7)	12.9 (12.3–13.5)	13.6 (12.9–14.2)	14.4 (13.7–15.1)	15.0 (14.2–15.7)
45-day	9.08 (8.70–9.48)	10.6 (10.1–11.0)	11.9 (11.4–12.4)	12.9 (12.3–13.4)	14.1 (13.5–14.7)	14.9 (14.3–15.6)	15.7 (15.0–16.4)	16.4 (15.7–17.2)	17.3 (16.5–18.1)	17.9 (17.0–18.8)
60-day	10.9 (10.5–11.4)	12.7 (12.2–13.2)	14.1 (13.5–14.7)	15.2 (14.6–15.8)	16.5 (15.8–17.2)	17.4 (16.7–18.2)	18.3 (17.5–19.0)	19.0 (18.2–19.8)	19.9 (19.0–20.8)	20.5 (19.5–21.4)

[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.

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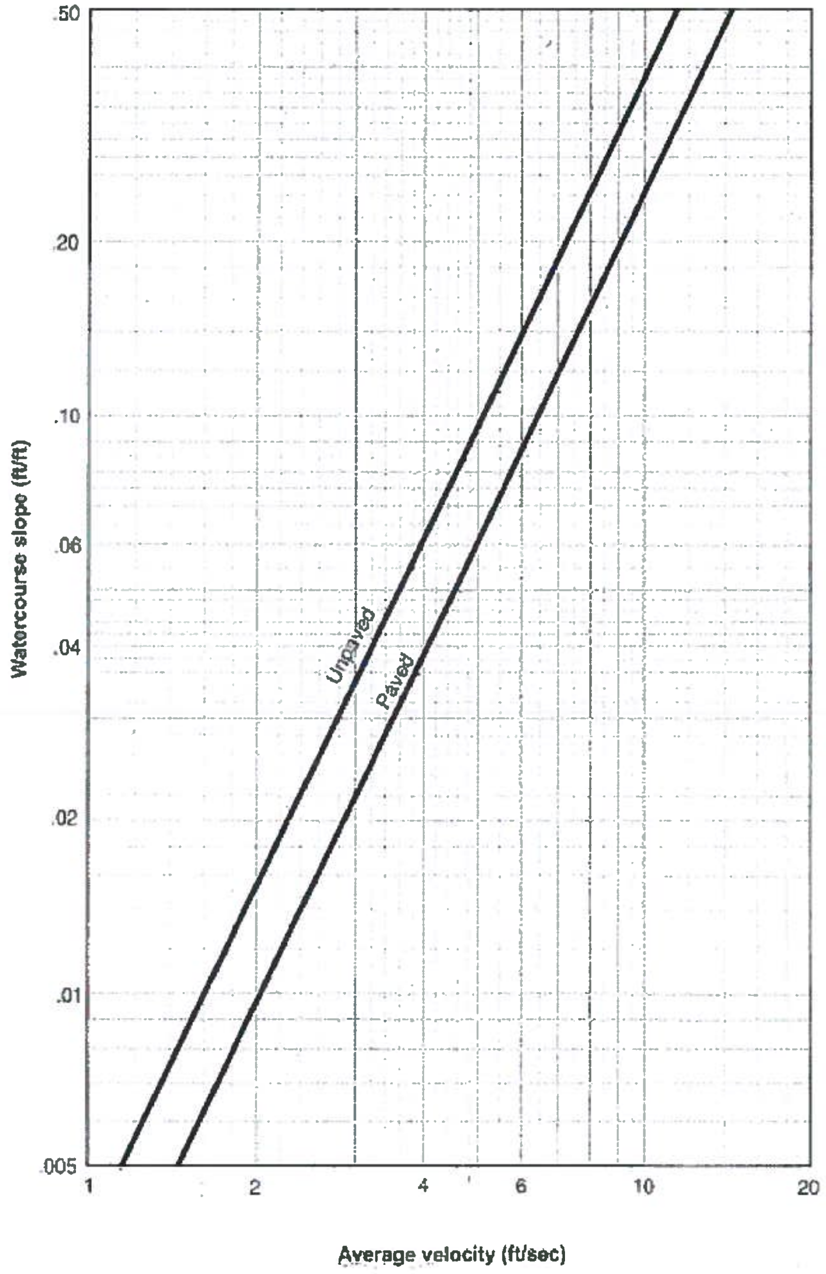
Table 2-2a Runoff curve numbers for urban areas [1]

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

Figure 3-1 Average velocities for estimating travel time for shallow concentrated flow



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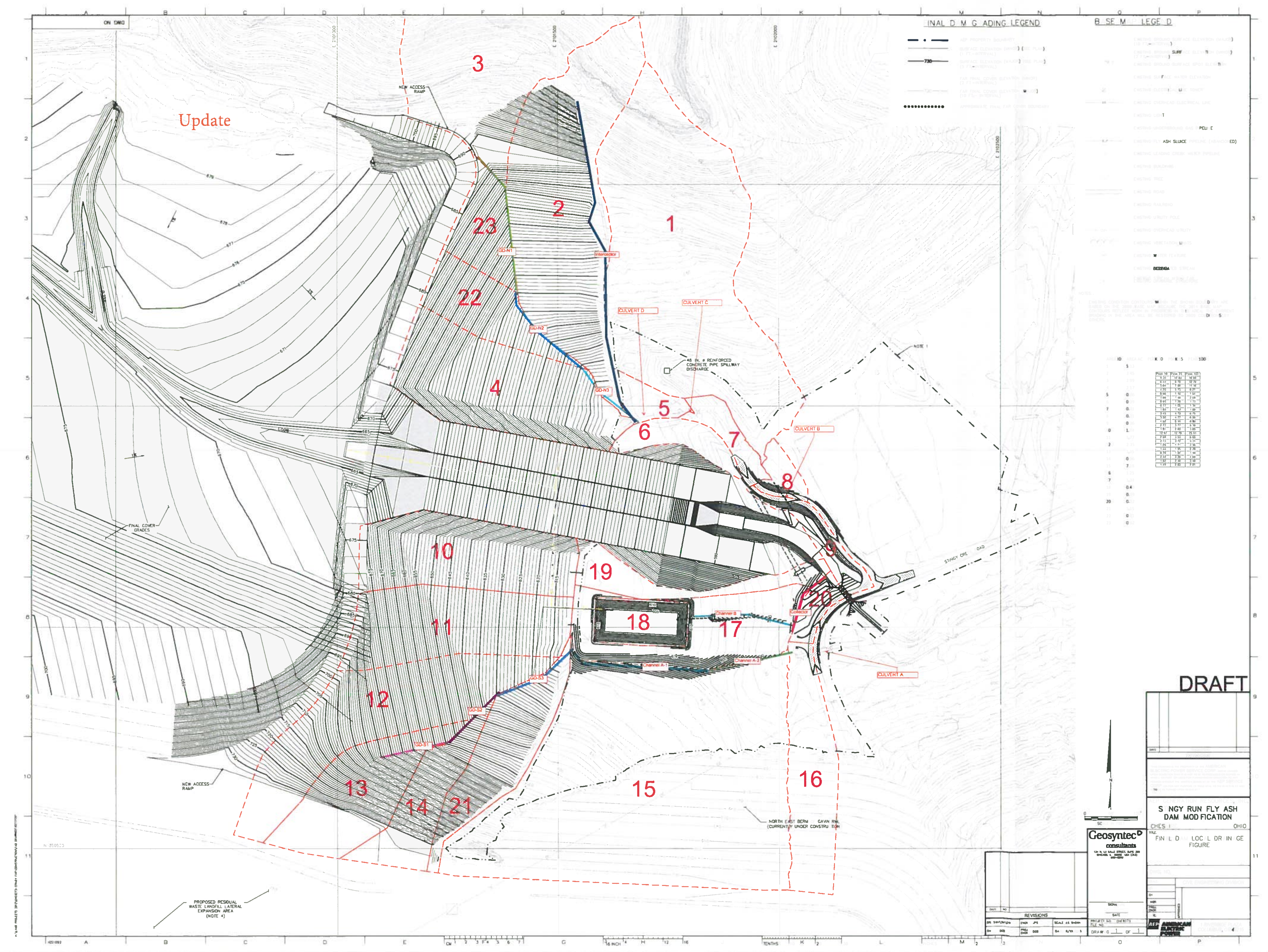
		Written by:	JNJ		**Date:**	10/15/2015	
		Reviewed by:	RM		**Date:**	11/10/2015	
Client:	AEP	**Project:**	FAR Closure	**Project No.:**	CHE8273	**Task No.:**	02/05.22

ATTACHMENT 2

Drainage Exhibits


Update
DRAFT
FINAL DAM GRADING LEGEND
BASE MAP LEGEND
NEW ACCESS RAMP
FINAL COVER GRADES
PROPOSED RESIDUAL WASTE LANDFILL LATERAL EXPANSION AREA (NOTE 4)
48 IN. Ø REINFORCED CONCRETE PIPE SPILLWAY DISCHARGE
NORTH EAST BERM GAVIN RWL (CURRENTLY UNDER CONSTRUCTION)
STINGY CREEK ROAD
NOTE 1
CULVERT A
CULVERT B
CULVERT C
CULVERT D
GD-N1
GD-N2
GD-N3
GD-S1
GD-S2
GD-S3
Interceptor
Channel A-1
Channel A-2
Channel B
Collector
1
2
3
4
5
6
7
8
9
10
11
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21
22
23
REVISIONS
SCALE AS SHOWN
Geosyntec consultants
DRAWING 1 OF 1
STINGY RUN FLY ASH DAM MODIFICATION
CHESHIRE OHIO
TITLE
FINAL DAM LOCAL DRAINAGE FIGURE
AEP AMERICAN ELECTRIC POWER

ATTACHMENT 3

Summary Tables

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JOB CHE 8273 Stingy Run FAR Closure
SHEET NO OF
CALCULATED BY CAC (updated JNJ 10/12/15) DATE 5/27/2014
CHECKED BY DATE
SCALE
DESCRIPTION Swale Summary

TABLE 3-1
Swale Design Summary
(Reference Swale Calculation Spreadsheets)

Groin and Interface Ditches

Groin and Interface Ditches	Contributing Areas	Drainage Area (ac)	Peak Flow (10yr) (cfs)	Peak Flow (25yr) (cfs)	Peak Flow (100yr) (cfs)	Slope (%)	**Flow Depth (25yr (ft)**	**Flow Depth (100yr (ft)**	Capacity (cfs) (at 1.5 ft flow depth)	Meets Design Capacity	Primary Renforcment
GD-N1	3, 23	3.61	7.12	9.71	13.96	0.15	**0.58**	**0.66**	126	Y	SC 250
GD-N2	2, 3, 22, 23	7.87	15.39	20.98	30.22	0.13	**0.80**	**0.90**	117	Y	SC 250
GD-N3	2, 3, 4, 22, 23	10.04	19.59	26.71	38.49	0.08	**0.94**	**1.08**	92	Y	SC 250
GD-S1	13	1.35	2.77	3.77	5.38	0.19	**0.38**	**0.44**	141	Y	SC 250
GD-S2	13, 14	2.21	4.68	6.37	9.03	0.19	**0.48**	**0.54**	141	Y	SC 250
GD-S3	12, 13, 14, 21	5.71	11.61	15.82	22.61	0.10	**0.74**	**0.86**	103	Y	SC 250
Interface Ditch N-1	1	5.13	9.38	12.80	18.69	0.19	**0.62**	**0.70**	141	Y	SC 250

Swales

Swales	Contributing Areas	Drainage Area (ac)	Peak Flow (10yr) (cfs)	Peak Flow (25yr) (cfs)	Peak Flow (100yr) (cfs)	Slope (%)	**Flow Depth (25yr (ft)**	**Flow Depth (100yr (ft)**	Capacity (cfs) (1.5ft flow depth)	Meets Design Capacity	Primary Reinforcment
A-1	12, 13, 14, 15, 21	12.89	24.57	33.52	48.52	0.03	**0.96**	**1.14**	149	Y	SC 250
A-2	12, 13, 14, 15, 21	12.89	24.57	33.52	48.52	0.08	**0.76**	**0.90**	70	Y	S150
B	11, 18	2.26	4.76	6.49	9.21	0.11	**0.50**	**0.60**	174	Y	SC150BN
Collector	10, 11, 12, 13, 14, 15, 16, 17, 19, 20	18.64	36.66	49.98	71.90	6.66	**0.71**	**0.89**	208	Y	Type C Rip Rap

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JOB CHE 8273 Stingy Run FAR Closure
SHEET NO ______ OF ______
CALCULATED BY CAC (updated RM 12/23/15) DATE 5/27/2014
CHECKED BY ______ DATE ______
SCALE ______
DESCRIPTION Culvert Summary

TABLE 3-2
Culvert Design

Culvert	Contributing Areas	Drainage Area (ac)	10yr Peak (cfs)	25yr Peak (cfs)	100yr Peak (cfs)	Approximate Length (ft)	US invert (ft)	DS invert (ft)	Slope (%)	Description
A	12, 13, 14, 15, 16, 21	12.89	25	34	49	90	584	582	0.022	36" Concrete
B	1, 2,3, 4, 5, 6, 7, 22, 23	6.49	5	6	9	75	584	582	0.027	36" Concrete
C	2, 3, 4, 5, 22, 23	11.32	22	30	43	51	602.5	600.6	0.037	36" Concrete
D	2, 3, 4, , 22, 23	7.87	15	21	30	60	604.9	603.7	0.020	36" Concrete

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SHEET NO OF
CALCULATED BY CAC (updated JNJ 10/12/15) DATE 5/27/2014
CHECKED BY DATE
SCALE
DESCRIPTION Riprap Design

TABLE 3-3
Riprap Design for Interim Conditions

Storm(yr)	Bed Gradient, S (ft/ft)	Total Discharge (ft^3/s)	Bottom Width, W (ft)	Flow Depth, Z (ft)	Unit Discharge, Q (ft^3/s/ft)	Particle Size 50% Finer D50, (in.)	Manning's, N	Riprap
2	0.33	22.5	6	0.42	3.75	7.94	0.05	ODOT TYPE C
10	0.33	35	6	0.56	5.83	10.03	0.06	ODOT TYPE C
25	0.33	42.5	6	0.64	7.08	11.11	0.06	ODOT TYPE C

Geosyntec consultants

Written by: JNJ **Date:** 10/15/2015

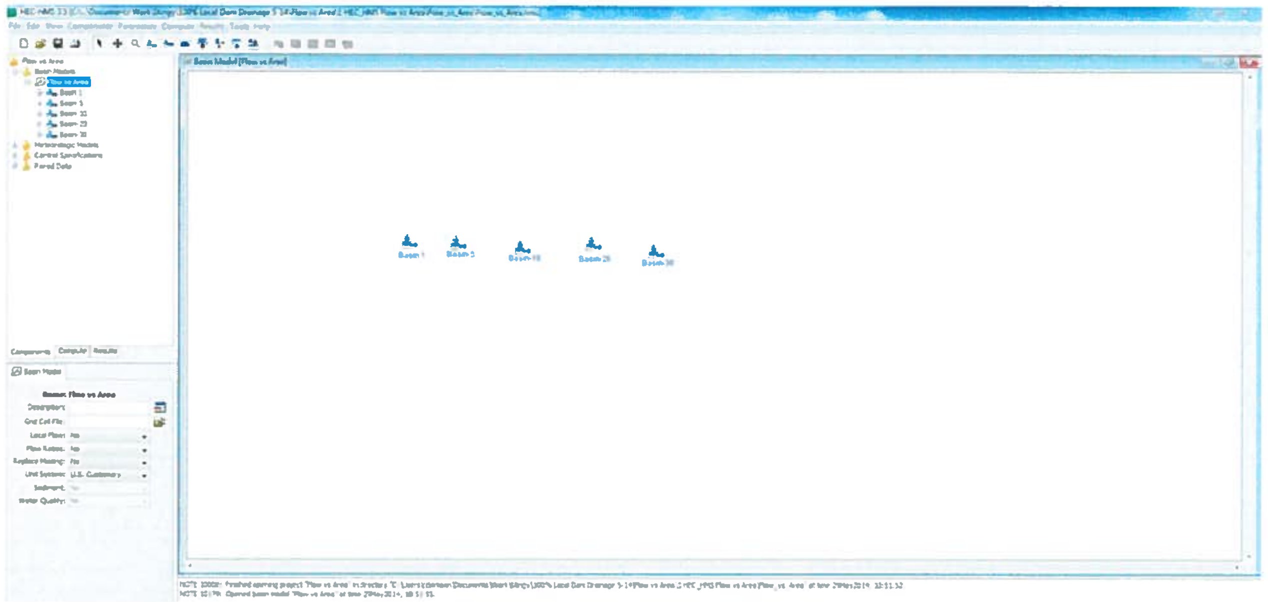
Reviewed by: RM **Date:** 11/10/2015

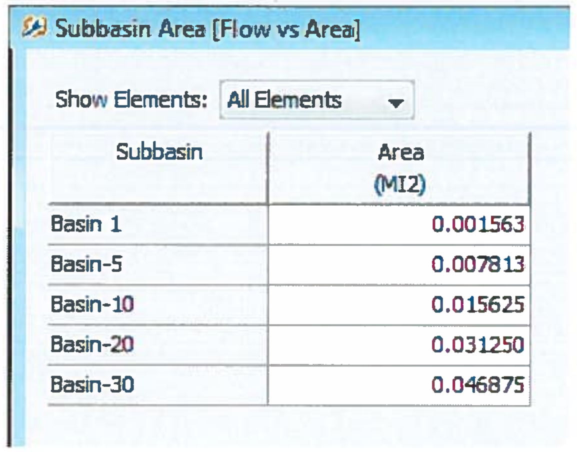
Client: AEP **Project:** FAR Closure **Project No.:** CHE8273 **Task No.:** 02/05.22

ATTACHMENT 4

HEC-HMS and HY-8 Input

HEC-HMS Input



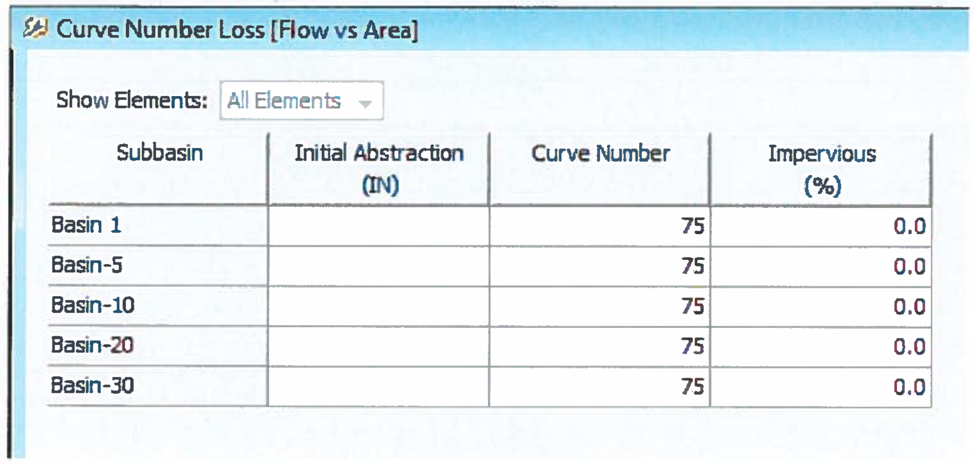


Subbasin Area [Flow vs Area]

Show Elements: All Elements

Subbasin	Area (MI2)
Basin 1	0.001563
Basin-5	0.007813
Basin-10	0.015625
Basin-20	0.031250
Basin-30	0.046875

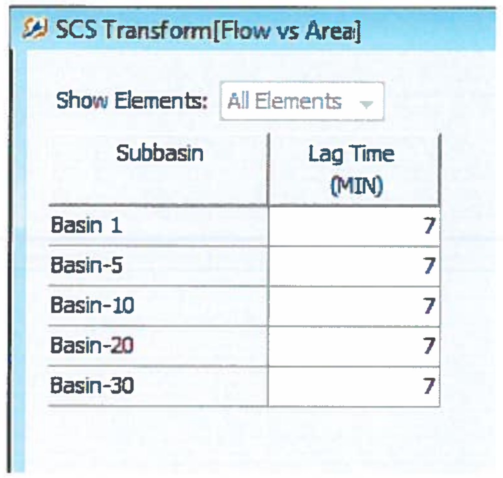
HEC-HMS Input



Curve Number Loss [Flow vs Area]

Show Elements: All Elements

Subbasin	Initial Abstraction (IN)	Curve Number	Impervious (%)
Basin 1		75	0.0
Basin-5		75	0.0
Basin-10		75	0.0
Basin-20		75	0.0
Basin-30		75	0.0



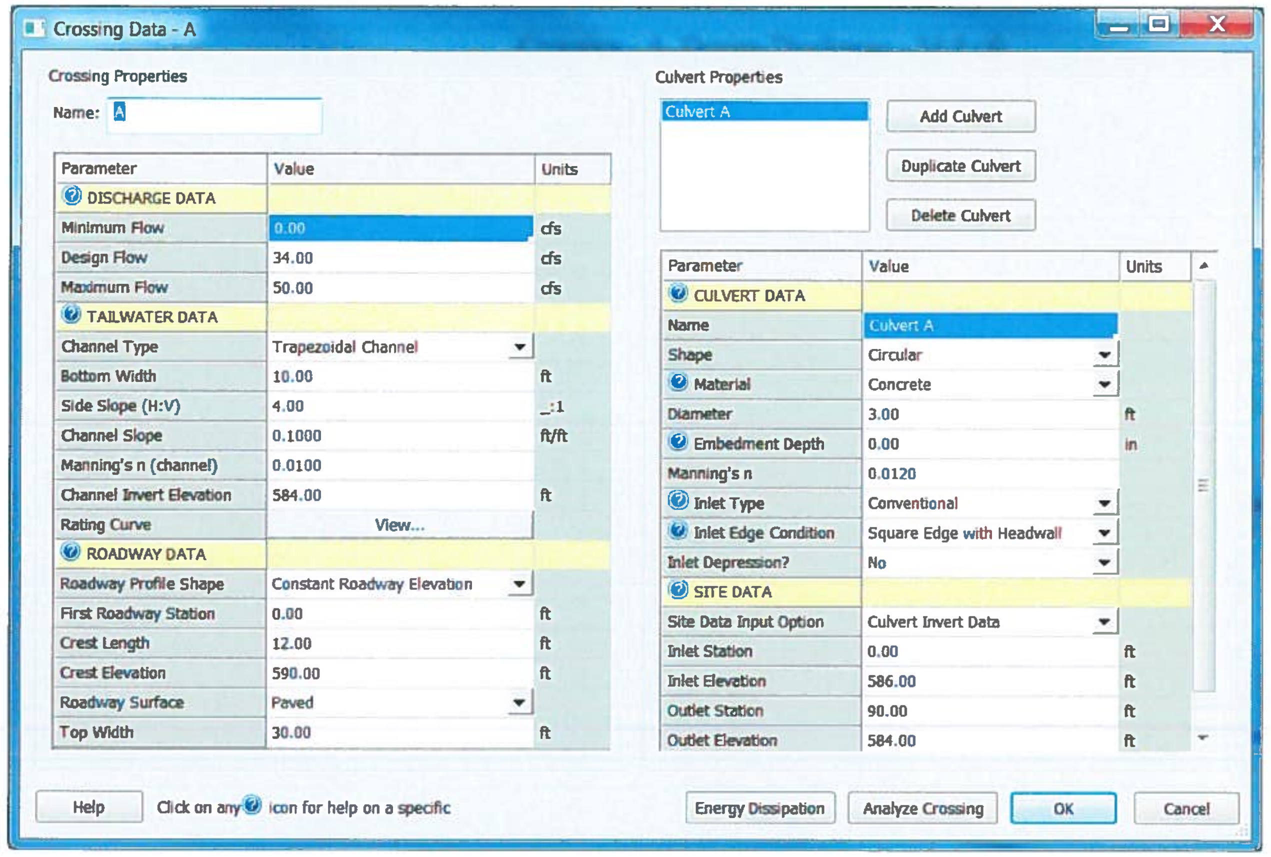
SCS Transform[Flow vs Area]

Show Elements: All Elements

Subbasin	Lag Time (MIN)
Basin 1	7
Basin-5	7
Basin-10	7
Basin-20	7
Basin-30	7

HY-8 Inputs

Culvert A



Crossing Data - A

Crossing Properties

Name: A

Parameter	Value	Units
DISCHARGE DATA		
Minimum Flow	0.00	cfs
Design Flow	34.00	cfs
Maximum Flow	50.00	cfs
TAILWATER DATA		
Channel Type	Trapezoidal Channel	
Bottom Width	10.00	ft
Side Slope (H:V)	4.00	_:1
Channel Slope	0.1000	ft/ft
Manning's n (channel)	0.0100	
Channel Invert Elevation	584.00	ft
Rating Curve	View...	
ROADWAY DATA		
Roadway Profile Shape	Constant Roadway Elevation	
First Roadway Station	0.00	ft
Crest Length	12.00	ft
Crest Elevation	590.00	ft
Roadway Surface	Paved	
Top Width	30.00	ft

Culvert Properties

Culvert A

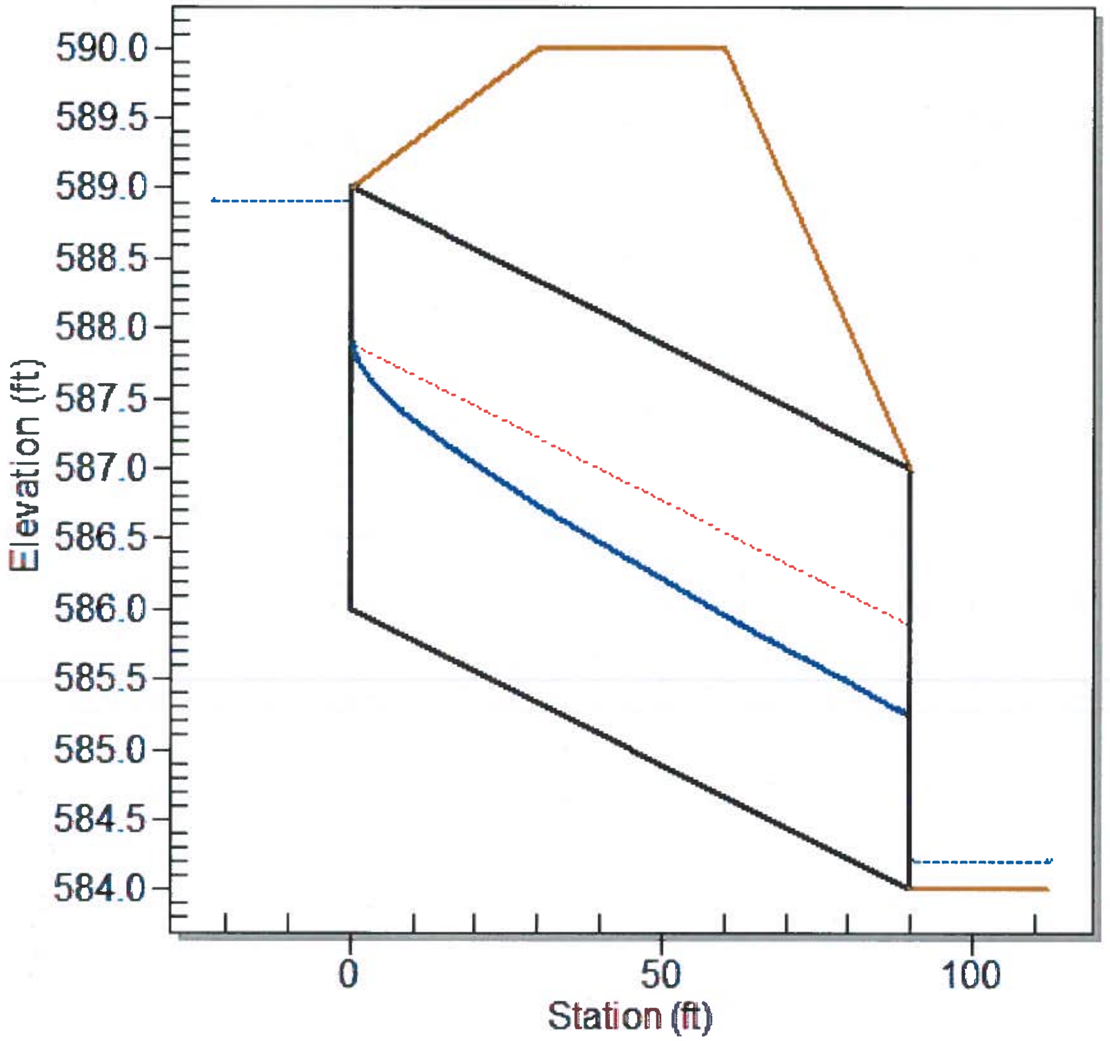
Parameter	Value	Units
CULVERT DATA		
Name	Culvert A	
Shape	Circular	
Material	Concrete	
Diameter	3.00	ft
Embedment Depth	0.00	in
Manning's n	0.0120	
Inlet Type	Conventional	
Inlet Edge Condition	Square Edge with Headwall	
Inlet Depression?	No	
SITE DATA		
Site Data Input Option	Culvert Invert Data	
Inlet Station	0.00	ft
Inlet Elevation	586.00	ft
Outlet Station	90.00	ft
Outlet Elevation	584.00	ft

Click on any icon for help on a specific

HY-8 Inputs

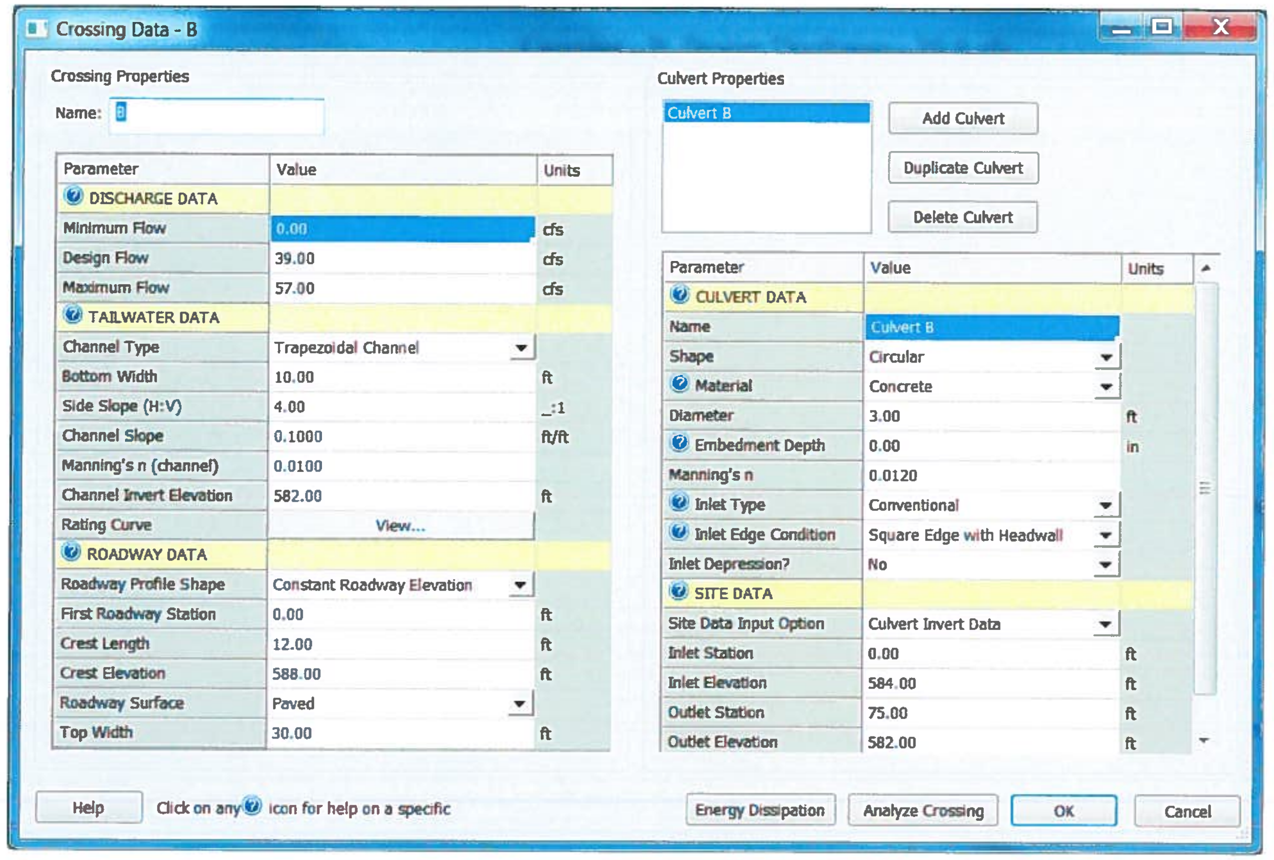
Crossing - A, Design Discharge - 34.0 cfs

Culvert - Culvert A, Culvert Discharge - 34.0 cfs



Culvert B

HY-8 Inputs



Crossing Properties

Name: B

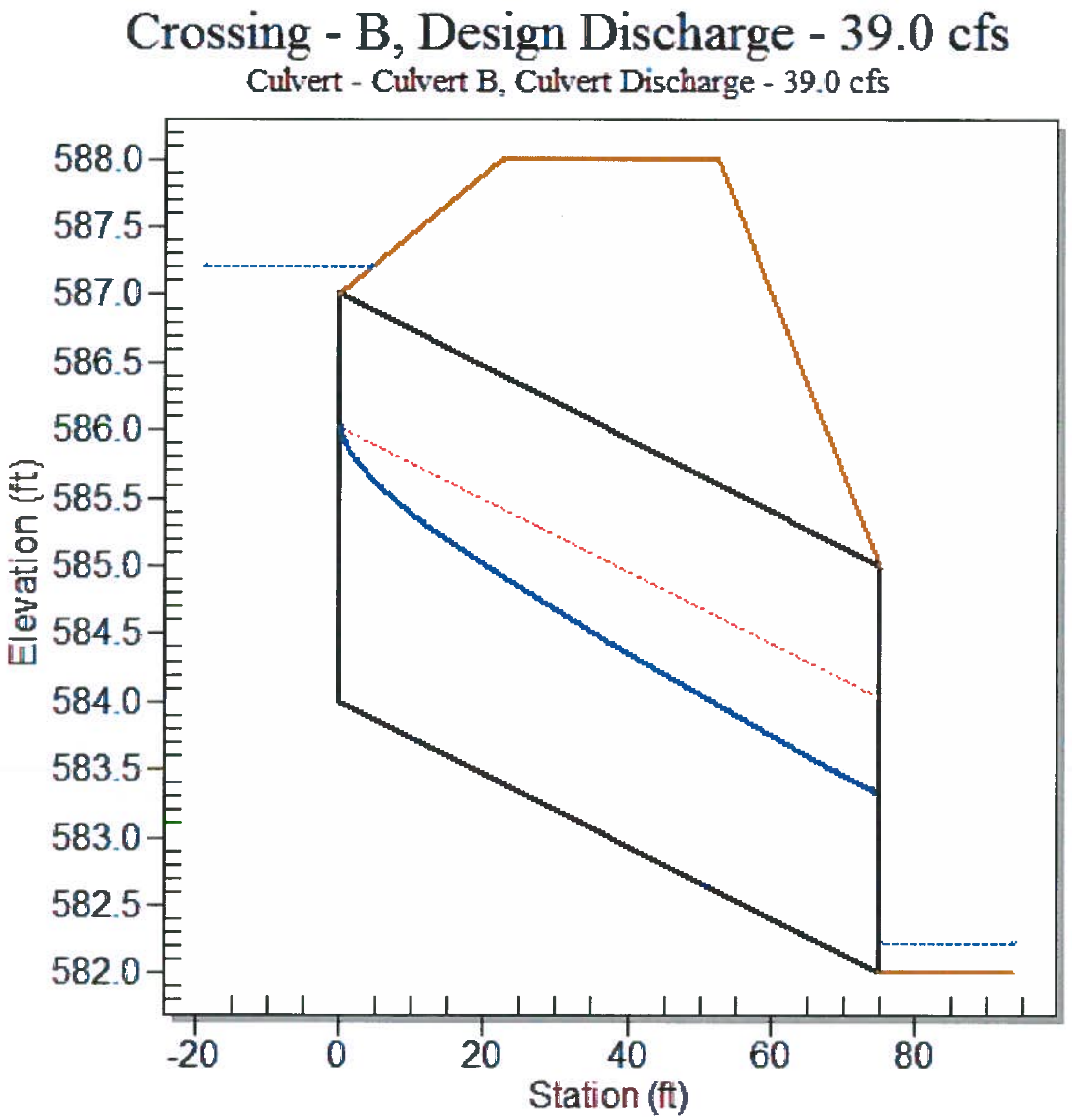
Parameter	Value	Units
DISCHARGE DATA		
Minimum Flow	0.00	cfs
Design Flow	39.00	cfs
Maximum Flow	57.00	cfs
TAILWATER DATA		
Channel Type	Trapezoidal Channel	
Bottom Width	10.00	ft
Side Slope (H:V)	4.00	_:1
Channel Slope	0.1000	ft/ft
Manning's n (channel)	0.0100	
Channel Invert Elevation	582.00	ft
Rating Curve	View...	
ROADWAY DATA		
Roadway Profile Shape	Constant Roadway Elevation	
First Roadway Station	0.00	ft
Crest Length	12.00	ft
Crest Elevation	588.00	ft
Roadway Surface	Paved	
Top Width	30.00	ft

Culvert Properties

Culvert B

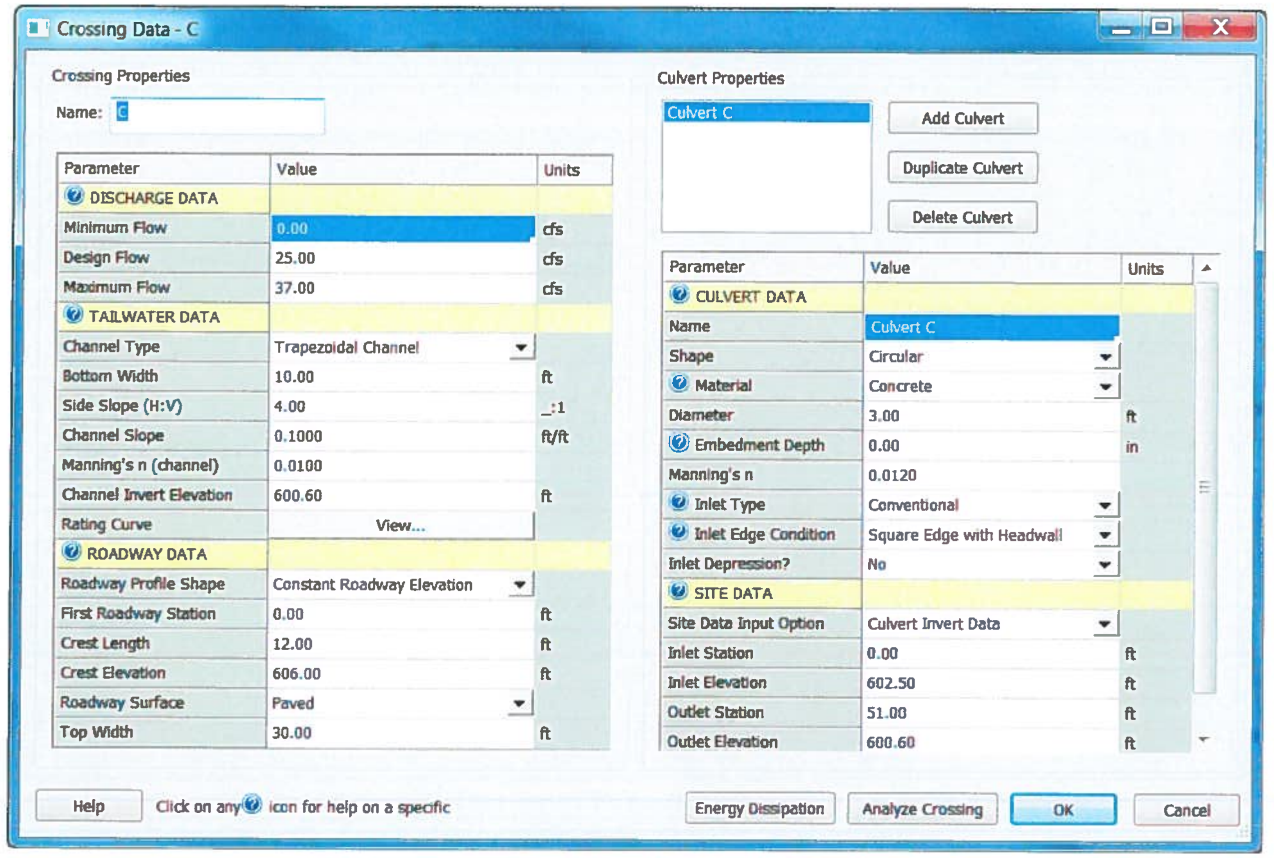
Parameter	Value	Units
CULVERT DATA		
Name	Culvert B	
Shape	Circular	
Material	Concrete	
Diameter	3.00	ft
Embedment Depth	0.00	in
Manning's n	0.0120	
Inlet Type	Conventional	
Inlet Edge Condition	Square Edge with Headwall	
Inlet Depression?	No	
SITE DATA		
Site Data Input Option	Culvert Invert Data	
Inlet Station	0.00	ft
Inlet Elevation	584.00	ft
Outlet Station	75.00	ft
Outlet Elevation	582.00	ft

Click on any icon for help on a specific


Crossing - B, Design Discharge - 39.0 cfs
Culvert - Culvert B, Culvert Discharge - 39.0 cfs
Elevation (ft)
588.0
587.5
587.0
586.5
586.0
585.5
585.0
584.5
584.0
583.5
583.0
582.5
582.0
-20
0
20
40
60
80
Station (ft)

Culvert C

HY-8 Inputs



Crossing Properties

Name: C

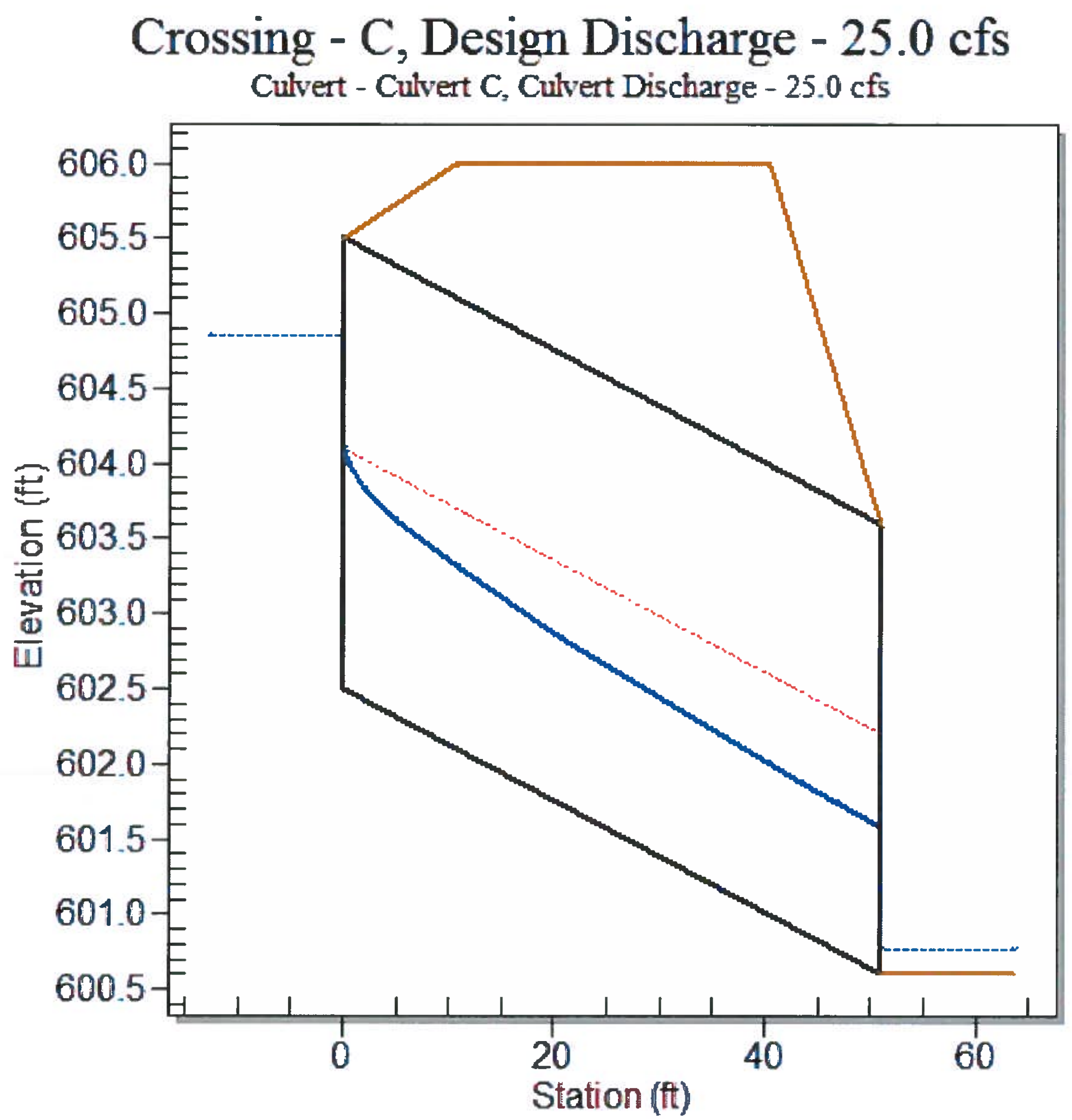
Parameter	Value	Units
DISCHARGE DATA		
Minimum Flow	0.00	cfs
Design Flow	25.00	cfs
Maximum Flow	37.00	cfs
TAILWATER DATA		
Channel Type	Trapezoidal Channel	
Bottom Width	10.00	ft
Side Slope (H:V)	4.00	_:1
Channel Slope	0.1000	ft/ft
Manning's n (channel)	0.0100	
Channel Invert Elevation	600.60	ft
Rating Curve	View...	
ROADWAY DATA		
Roadway Profile Shape	Constant Roadway Elevation	
First Roadway Station	0.00	ft
Crest Length	12.00	ft
Crest Elevation	606.00	ft
Roadway Surface	Paved	
Top Width	30.00	ft

Culvert Properties

Culvert C

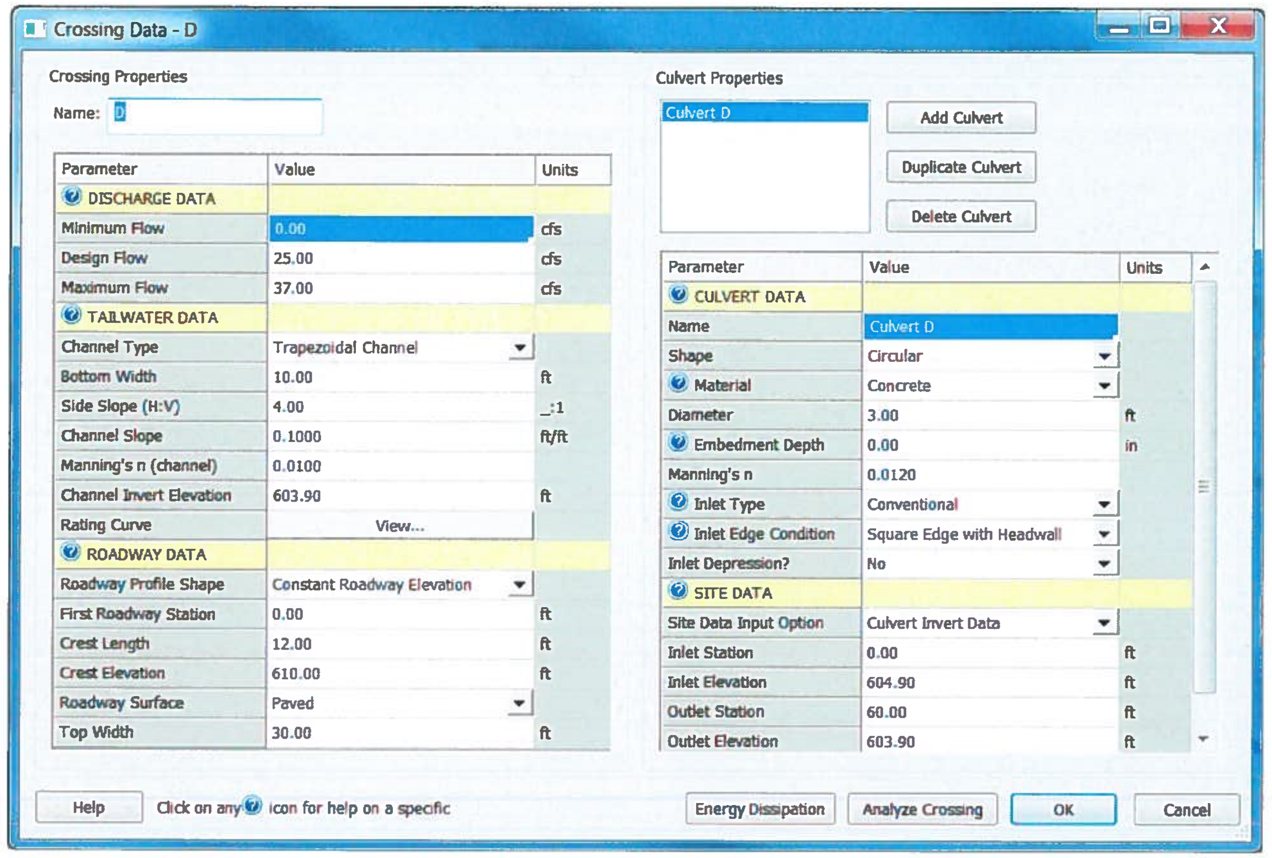
Parameter	Value	Units
CULVERT DATA		
Name	Culvert C	
Shape	Circular	
Material	Concrete	
Diameter	3.00	ft
Embedment Depth	0.00	in
Manning's n	0.0120	
Inlet Type	Conventional	
Inlet Edge Condition	Square Edge with Headwall	
Inlet Depression?	No	
SITE DATA		
Site Data Input Option	Culvert Invert Data	
Inlet Station	0.00	ft
Inlet Elevation	602.50	ft
Outlet Station	51.00	ft
Outlet Elevation	600.60	ft

Click on any icon for help on a specific


Crossing - C, Design Discharge - 25.0 cfs
Culvert - Culvert C, Culvert Discharge - 25.0 cfs
Elevation (ft)
606.0
605.5
605.0
604.5
604.0
603.5
603.0
602.5
602.0
601.5
601.0
600.5
0
20
40
60
Station (ft)

Culvert D

HY-8 Inputs



Crossing Properties

Name: D

Parameter	Value	Units
DISCHARGE DATA		
Minimum Flow	0.00	cfs
Design Flow	25.00	cfs
Maximum Flow	37.00	cfs
TAILWATER DATA		
Channel Type	Trapezoidal Channel	
Bottom Width	10.00	ft
Side Slope (H:V)	4.00	_:1
Channel Slope	0.1000	ft/ft
Manning's n (channel)	0.0100	
Channel Invert Elevation	603.90	ft
Rating Curve	View...	
ROADWAY DATA		
Roadway Profile Shape	Constant Roadway Elevation	
First Roadway Station	0.00	ft
Crest Length	12.00	ft
Crest Elevation	610.00	ft
Roadway Surface	Paved	
Top Width	30.00	ft

Culvert Properties

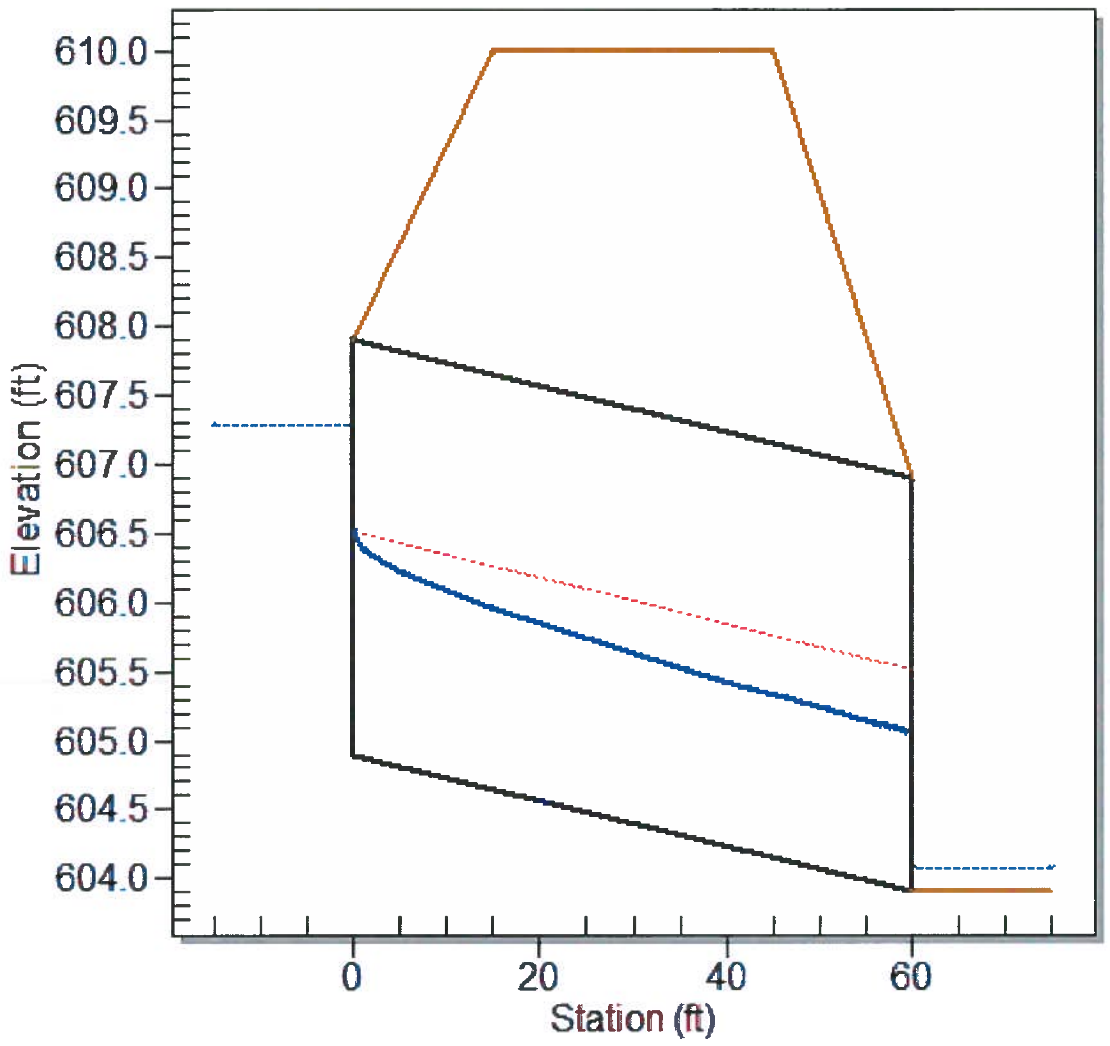
Culvert D

Parameter	Value	Units
CULVERT DATA		
Name	Culvert D	
Shape	Circular	
Material	Concrete	
Diameter	3.00	ft
Embedment Depth	0.00	in
Manning's n	0.0120	
Inlet Type	Conventional	
Inlet Edge Condition	Square Edge with Headwall	
Inlet Depression?	No	
SITE DATA		
Site Data Input Option	Culvert Invert Data	
Inlet Station	0.00	ft
Inlet Elevation	604.90	ft
Outlet Station	60.00	ft
Outlet Elevation	603.90	ft

Click on any icon for help on a specific

Crossing - D, Design Discharge - 25.0 cfs

Culvert - Culvert D, Culvert Discharge - 25.0 cfs



Geosyntec consultants

Written by: JNJ **Date:** 10/15/2015

Reviewed by: RM **Date:** 11/10/2015

Client: AEP **Project:** FAR Closure **Project No.:** CHE8273 **Task No.:** 02/05.22

ATTACHMENT 5

Conveyance Feature Design Calculations

(Culverts)

Design/Check: Trapezoidal/Triangular Channel

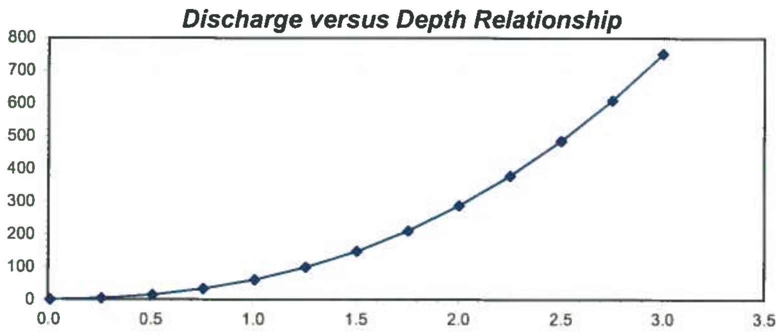
Methodology: Manning's Equation

Project: Stingy Run FAR Closure

Ditch ID: **A-1**

Peak Discharge, Q_{max} = 33.52 cfs
Bottom Width, B = 2.00 ft
Left Side Slope, Z_1 = 3.00 horizontal :1 vertical
Right Side Slope, Z_2 = 3.00 horizontal :1 vertical
Manning's Roughness Coeff., n = 0.025
Longitudinal Channel Slope, S_o = 0.0800 ft/ft

Depth of Flow Y ft	Area of Flow A ft^2	Wetted Perimeter P ft	Hydraulic Radius R=A/P ft	Average Velocity V ft/s	Discharge (Flow Rate) Q=AV ft^3/s	Avg. Tractive Stress τ_o lb/ft^2	Comments
0.01	0.02	2.06	0.01	0.77	0.02	0.05	
0.26	0.72	3.64	0.20	5.72	4.12	0.99	
0.51	1.79	5.21	0.34	8.27	14.81	1.72	
0.76	3.24	6.79	0.48	10.28	33.28	2.38	
1.01	5.05	8.37	0.60	12.04	60.86	3.02	
1.26	7.24	9.94	0.73	13.65	98.84	3.64	
1.51	9.81	11.52	0.85	15.14	148.45	4.25	
1.75	12.74	13.09	0.97	16.55	210.86	4.86	
2.00	16.05	14.67	1.09	17.90	287.18	5.46	
2.25	19.73	16.25	1.21	19.19	378.50	6.06	
2.50	23.78	17.82	1.33	20.43	485.85	6.66	
2.75	28.20	19.40	1.45	21.64	610.24	7.26	
3.00	33.00	20.97	1.57	22.81	752.66	7.85	
0.77	3.32	6.87	0.48	10.38	34.43	2.41	DESIGN Q


Discharge versus Depth Relationship

Design/Check: Trapezoidal/Triangular Channel

Methodology: Manning's Equation

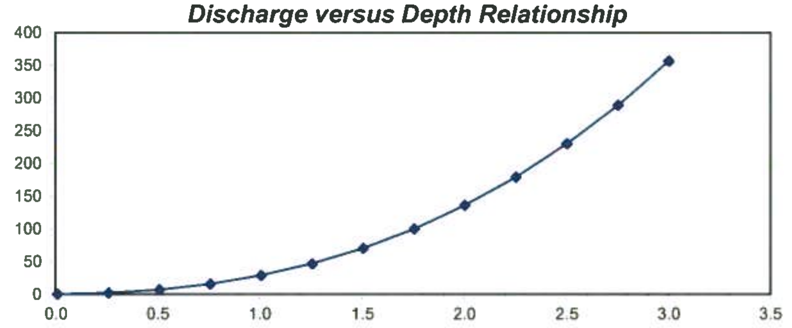
Project: Stingy Run FAR Closure

Ditch ID: **A-1**

Peak Discharge, Q_{max} = 31.27 cfs
Bottom Width, B = 2.00 ft
Left Side Slope, Z_1 = 3.00 horizontal :1 vertical
Right Side Slope, Z_2 = 3.00 horizontal :1 vertical
Manning's Roughness Coeff., n = 0.025
Longitudinal Channel Slope, S_o = 0.0180 ft/ft

Depth of Flow Y ft	Area of Flow A ft^2	Wetted Perimeter P ft	Hydraulic Radius R=A/P ft	Average Velocity V ft/s	Discharge (Flow Rate) Q=AV ft^3/s	Avg. Tractive Stress τ_o lb/ft^2	Comments
0.01	0.02	2.06	0.01	0.37	0.01	0.01	
0.26	0.72	3.64	0.20	2.71	1.95	0.22	
0.51	1.79	5.21	0.34	3.92	7.03	0.39	
0.76	3.24	6.79	0.48	4.88	15.79	0.54	
1.01	5.05	8.37	0.60	5.71	28.87	0.68	
1.26	7.24	9.94	0.73	6.47	46.89	0.82	
1.51	9.81	11.52	0.85	7.18	70.42	0.96	
1.75	12.74	13.09	0.97	7.85	100.02	1.09	
2.00	16.05	14.67	1.09	8.49	136.22	1.23	
2.25	19.73	16.25	1.21	9.10	179.54	1.36	
2.50	23.78	17.82	1.33	9.69	230.46	1.50	
2.75	28.20	19.40	1.45	10.26	289.46	1.63	
3.00	33.00	20.97	1.57	10.82	357.02	1.77	
1.05	5.41	8.64	0.63	5.85	31.63	0.70	DESIGN Q

Discharge versus Depth Relationship



Design/Check: Trapezoidal/Triangular Channel

Methodology: Manning's Equation

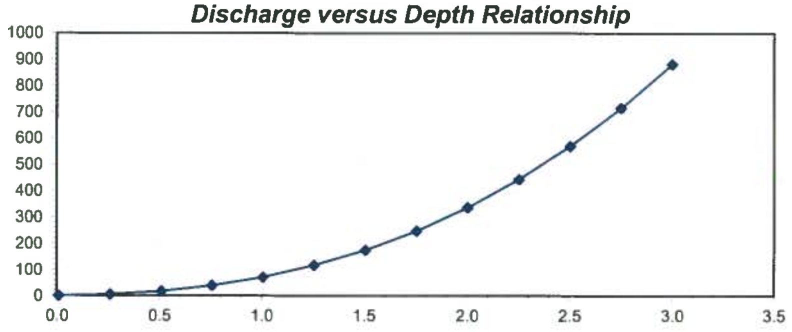
Project: Stingy Run FAR Closure

Ditch ID: B

Peak Discharge, Q_{max} =	23.97	cfs
Bottom Width, B =	2.00	ft
Left Side Slope, Z_1 =	3.00	horizontal :1 vertical
Right Side Slope, Z_2 =	3.00	horizontal :1 vertical
Manning's Roughness Coeff., n =	0.025	
Longitudinal Channel Slope, S_o =	0.1100	ft/ft

Depth of Flow Y ft	Area of Flow A ft^2	Wetted Perimeter P ft	Hydraulic Radius R=A/P ft	Average Velocity V ft/s	Discharge (Flow Rate) Q=AV ft^3/s	Avg. Tractive Stress τ_o lb/ft^2	Comments
0.01	0.02	2.06	0.01	0.91	0.02	0.07	
0.26	0.72	3.64	0.20	6.71	4.83	1.36	
0.51	1.79	5.21	0.34	9.69	17.37	2.36	
0.76	3.24	6.79	0.48	12.06	39.02	3.27	
1.01	5.05	8.37	0.60	14.12	71.36	4.15	
1.26	7.24	9.94	0.73	16.00	115.90	5.00	
1.51	9.81	11.52	0.85	17.75	174.08	5.84	
1.75	12.74	13.09	0.97	19.41	247.25	6.68	
2.00	16.05	14.67	1.09	20.99	336.75	7.51	
2.25	19.73	16.25	1.21	22.50	443.83	8.33	
2.50	23.78	17.82	1.33	23.96	569.71	9.16	
2.75	28.20	19.40	1.45	25.37	715.57	9.98	
3.00	33.00	20.97	1.57	26.74	882.57	10.80	
0.59	2.22	5.73	0.39	10.51	23.39	2.66	DESIGN Q

Discharge versus Depth Relationship



Design/Check: Trapezoidal/Triangular Channel

Methodology: Manning's Equation

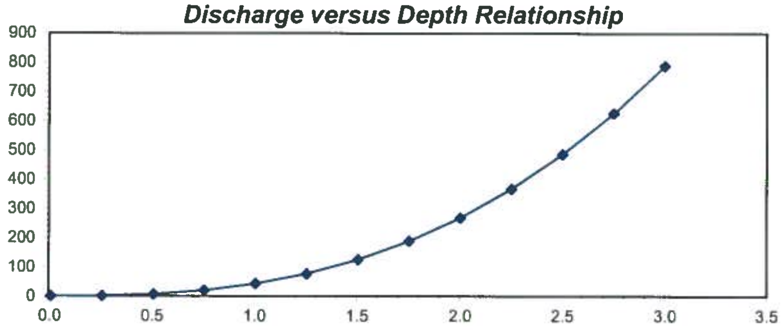
Project: Stingy Run FAR Closure

Ditch ID: **GD N-1**

Peak Discharge, Q_{max} = 13.96 cfs
Bottom Width, B = 0.00 ft
Left Side Slope, Z_1 = 3.00 horizontal : 1 vertical
Right Side Slope, Z_2 = 3.00 horizontal : 1 vertical
Manning's Roughness Coeff., n = 0.025
Longitudinal Channel Slope, S_o = 0.1500 ft/ft

Depth of Flow Y ft	Area of Flow A ft^2	Wetted Perimeter P ft	Hydraulic Radius R=A/P ft	Average Velocity V ft/s	Discharge (Flow Rate) Q=AV ft^3/s	Avg. Tractive Stress τ_o lb/ft^2	Comments
0.01	0.00	0.06	0.00	0.65	0.00	0.04	
0.26	0.20	1.64	0.12	5.70	1.15	1.15	
0.51	0.78	3.21	0.24	8.94	6.93	2.26	
0.76	1.72	4.79	0.36	11.66	20.08	3.36	
1.01	3.04	6.37	0.48	14.10	42.86	4.47	
1.26	4.73	7.94	0.60	16.34	77.31	5.58	
1.51	6.80	9.52	0.71	18.44	125.27	6.68	
1.75	9.23	11.09	0.83	20.42	188.50	7.79	
2.00	12.04	12.67	0.95	22.31	268.62	8.89	
2.25	15.22	14.25	1.07	24.13	367.22	10.00	
2.50	18.78	15.82	1.19	25.87	485.78	11.11	
2.75	22.70	17.40	1.30	27.57	625.77	12.21	
3.00	27.00	18.97	1.42	29.21	788.58	13.32	
0.67	1.35	4.24	0.32	10.75	14.47	2.97	DESIGN Q

Discharge versus Depth Relationship



Design/Check: Trapezoidal/Triangular Channel

Methodology: Manning's Equation

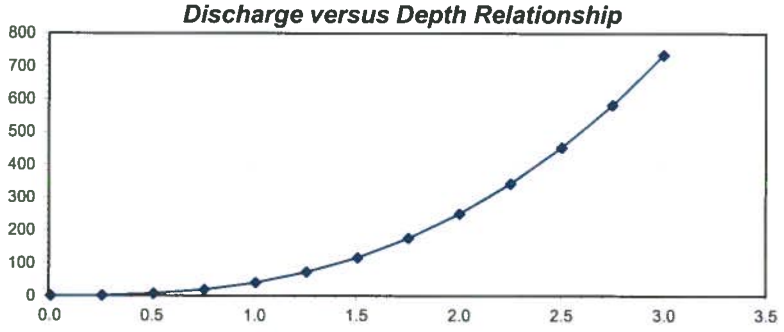
Project: Stingy Run FAR Closure

Ditch ID: **GD N-2**

Peak Discharge, Q_{max} = 38.49 cfs
Bottom Width, B = 0.00 ft
Left Side Slope, Z_1 = 3.00 horizontal :1 vertical
Right Side Slope, Z_2 = 3.00 horizontal :1 vertical
Manning's Roughness Coeff., n = 0.025
Longitudinal Channel Slope, S_o = 0.1300 ft/ft

Depth of Flow Y ft	Area of Flow A ft^2	Wetted Perimeter P ft	Hydraulic Radius R=A/P ft	Average Velocity V ft/s	Discharge (Flow Rate) Q=AV ft^3/s	Avg. Tractive Stress τ_o lb/ft^2	Comments
0.01	0.00	0.06	0.00	0.61	0.00	0.04	
0.26	0.20	1.64	0.12	5.31	1.07	1.00	
0.51	0.78	3.21	0.24	8.32	6.45	1.96	
0.76	1.72	4.79	0.36	10.86	18.69	2.91	
1.01	3.04	6.37	0.48	13.12	39.90	3.87	
1.26	4.73	7.94	0.60	15.21	71.97	4.83	
1.51	6.80	9.52	0.71	17.16	116.62	5.79	
1.75	9.23	11.09	0.83	19.01	175.48	6.75	
2.00	12.04	12.67	0.95	20.77	250.07	7.71	
2.25	15.22	14.25	1.07	22.46	341.86	8.67	
2.50	18.78	15.82	1.19	24.09	452.24	9.63	
2.75	22.70	17.40	1.30	25.66	582.56	10.58	
3.00	27.00	18.97	1.42	27.19	734.13	11.54	
0.99	2.94	6.26	0.47	12.98	38.16	3.81	DESIGN Q

Discharge versus Depth Relationship



Design/Check: Trapezoidal/Triangular Channel

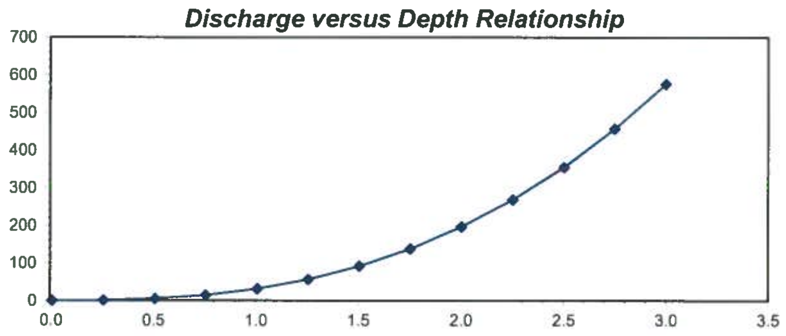
Methodology: Manning's Equation

Project: Stingy Run FAR Closure

Ditch ID: **GD N-2**

Peak Discharge, Q_{max} = 26.71 cfs
Bottom Width, B = 0.00 ft
Left Side Slope, Z_1 = 3.00 horizontal :1 vertical
Right Side Slope, Z_2 = 3.00 horizontal :1 vertical
Manning's Roughness Coeff., n = 0.025
Longitudinal Channel Slope, S_o = 0.0800 ft/ft

Depth of Flow Y ft	Area of Flow A ft^2	Wetted Perimeter P ft	Hydraulic Radius R=A/P ft	Average Velocity V ft/s	Discharge (Flow Rate) Q=AV ft^3/s	Avg. Tractive Stress τ_o lb/ft^2	Comments
0.01	0.00	0.06	0.00	0.48	0.00	0.02	
0.26	0.20	1.64	0.12	4.16	0.84	0.61	
0.51	0.78	3.21	0.24	6.53	5.06	1.20	
0.76	1.72	4.79	0.36	8.52	14.66	1.79	
1.01	3.04	6.37	0.48	10.30	31.30	2.38	
1.26	4.73	7.94	0.60	11.93	56.46	2.97	
1.51	6.80	9.52	0.71	13.46	91.49	3.56	
1.75	9.23	11.09	0.83	14.91	137.66	4.15	
2.00	12.04	12.67	0.95	16.29	196.17	4.74	
2.25	15.22	14.25	1.07	17.62	268.18	5.33	
2.50	18.78	15.82	1.19	18.90	354.77	5.92	
2.75	22.70	17.40	1.30	20.13	457.00	6.51	
3.00	27.00	18.97	1.42	21.33	575.90	7.10	
0.95	2.71	6.01	0.45	9.91	26.82	2.25	DESIGN Q


Discharge versus Depth Relationship

Design/Check: Trapezoidal/Triangular Channel

Methodology: Manning's Equation

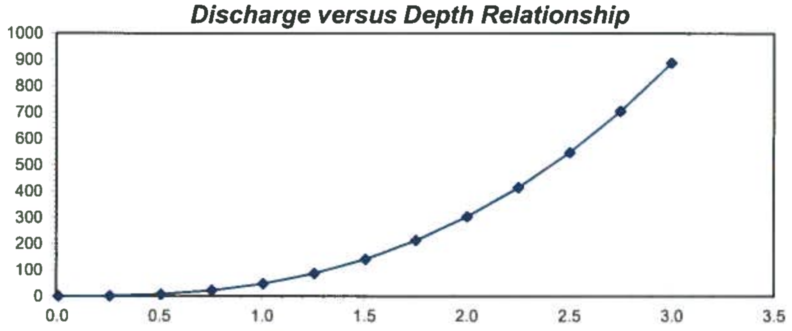
Project: Stingy Run FAR Closure

Ditch ID: **GD S-1**

Peak Discharge, Q_{max} =	5.38	cfs
Bottom Width, B =	0.00	ft
Left Side Slope, Z_1 =	3.00	horizontal :1 vertical
Right Side Slope, Z_2 =	3.00	horizontal :1 vertical
Manning's Roughness Coeff., n =	0.025	
Longitudinal Channel Slope, S_o =	0.1900	ft/ft

Depth of Flow Y ft	Area of Flow A ft^2	Wetted Perimeter P ft	Hydraulic Radius R=A/P ft	Average Velocity V ft/s	Discharge (Flow Rate) Q=AV ft^3/s	Avg. Tractive Stress τ_o lb/ft^2	Comments
0.01	0.00	0.06	0.00	0.73	0.00	0.06	
0.26	0.20	1.64	0.12	6.42	1.29	1.46	
0.51	0.78	3.21	0.24	10.06	7.80	2.86	
0.76	1.72	4.79	0.36	13.13	22.59	4.26	
1.01	3.04	6.37	0.48	15.87	48.24	5.66	
1.26	4.73	7.94	0.60	18.39	87.01	7.06	
1.51	6.80	9.52	0.71	20.75	140.99	8.46	
1.75	9.23	11.09	0.83	22.98	212.15	9.87	
2.00	12.04	12.67	0.95	25.11	302.32	11.27	
2.25	15.22	14.25	1.07	27.15	413.29	12.67	
2.50	18.78	15.82	1.19	29.12	546.73	14.07	
2.75	22.70	17.40	1.30	31.02	704.28	15.47	
3.00	27.00	18.97	1.42	32.87	887.52	16.87	
0.45	0.61	2.85	0.21	9.27	5.63	2.53	DESIGN Q

Discharge versus Depth Relationship



Design/Check: Trapezoidal/Triangular Channel

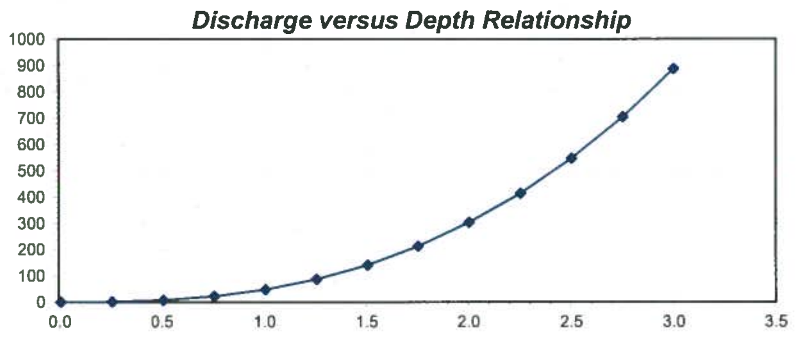
Methodology: Manning's Equation

Project: Stingy Run FAR Closure

Ditch ID: **GD S-2**

Peak Discharge, Q_{max} =	9.03	cfs
Bottom Width, B =	0.00	ft
Left Side Slope, Z_1 =	3.00	horizontal :1 vertical
Right Side Slope, Z_2 =	3.00	horizontal :1 vertical
Manning's Roughness Coeff., n =	0.025	
Longitudinal Channel Slope, S_o =	0.1900	ft/ft

Depth of Flow Y ft	Area of Flow A ft^2	Wetted Perimeter P ft	Hydraulic Radius R=A/P ft	Average Velocity V ft/s	Discharge (Flow Rate) Q=AV ft^3/s	Avg. Tractive Stress τ_o lb/ft^2	Comments
0.01	0.00	0.06	0.00	0.73	0.00	0.06	
0.26	0.20	1.64	0.12	6.42	1.29	1.46	
0.51	0.78	3.21	0.24	10.06	7.80	2.86	
0.76	1.72	4.79	0.36	13.13	22.59	4.26	
1.01	3.04	6.37	0.48	15.87	48.24	5.66	
1.26	4.73	7.94	0.60	18.39	87.01	7.06	
1.51	6.80	9.52	0.71	20.75	140.99	8.46	
1.75	9.23	11.09	0.83	22.98	212.15	9.87	
2.00	12.04	12.67	0.95	25.11	302.32	11.27	
2.25	15.22	14.25	1.07	27.15	413.29	12.67	
2.50	18.78	15.82	1.19	29.12	546.73	14.07	
2.75	22.70	17.40	1.30	31.02	704.28	15.47	
3.00	27.00	18.97	1.42	32.87	887.52	16.87	
0.53	0.84	3.35	0.25	10.34	8.72	2.98	DESIGN Q


Discharge versus Depth Relationship

Design/Check: Trapezoidal/Triangular Channel

Methodology: Manning's Equation

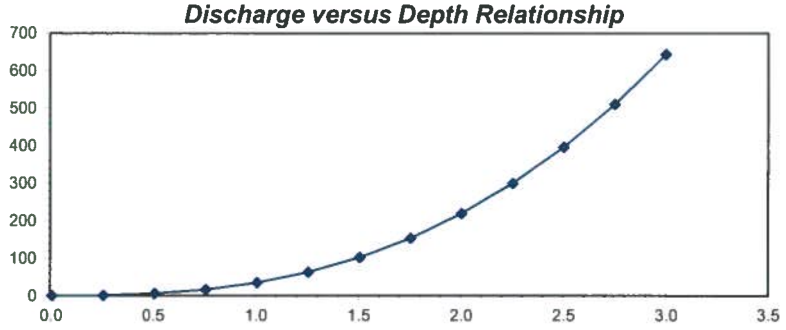
Project: Stingy Run FAR Closure

Ditch ID: **GD S-3**

Peak Discharge, Q_{max} =	22.61	cfs
Bottom Width, B =	0.00	ft
Left Side Slope, Z_1 =	3.00	horizontal :1 vertical
Right Side Slope, Z_2 =	3.00	horizontal :1 vertical
Manning's Roughness Coeff., n =	0.025	
Longitudinal Channel Slope, S_o =	0.1000	ft/ft

Depth of Flow Y ft	Area of Flow A ft^2	Wetted Perimeter P ft	Hydraulic Radius R=A/P ft	Average Velocity V ft/s	Discharge (Flow Rate) Q=AV ft^3/s	Avg. Tractive Stress τ_o lb/ft^2	Comments
0.01	0.00	0.06	0.00	0.53	0.00	0.03	
0.26	0.20	1.64	0.12	4.66	0.94	0.77	
0.51	0.78	3.21	0.24	7.30	5.66	1.50	
0.76	1.72	4.79	0.36	9.52	16.39	2.24	
1.01	3.04	6.37	0.48	11.51	35.00	2.98	
1.26	4.73	7.94	0.60	13.34	63.12	3.72	
1.51	6.80	9.52	0.71	15.05	102.28	4.45	
1.75	9.23	11.09	0.83	16.67	153.91	5.19	
2.00	12.04	12.67	0.95	18.22	219.33	5.93	
2.25	15.22	14.25	1.07	19.70	299.83	6.67	
2.50	18.78	15.82	1.19	21.13	396.64	7.40	
2.75	22.70	17.40	1.30	22.51	510.94	8.14	
3.00	27.00	18.97	1.42	23.85	643.88	8.88	
0.85	2.17	5.38	0.40	10.28	22.29	2.52	DESIGN Q

Discharge versus Depth Relationship



Design/Check: Trapezoidal/Triangular Channel

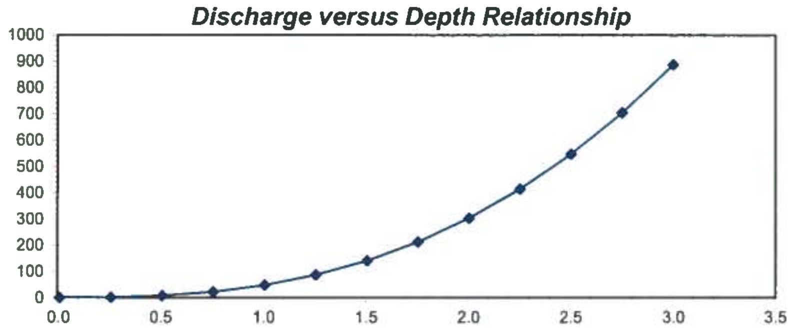
Methodology: Manning's Equation

Project: Stingy Run FAR Closure

Ditch ID: **Interface**

Peak Discharge, Q_{max} = 18.69 cfs
Bottom Width, B = 0.00 ft
Left Side Slope, Z_1 = 3.00 horizontal :1 vertical
Right Side Slope, Z_2 = 3.00 horizontal :1 vertical
Manning's Roughness Coeff., n = 0.025
Longitudinal Channel Slope, S_o = 0.1900 ft/ft

Depth of Flow Y ft	Area of Flow A ft^2	Wetted Perimeter P ft	Hydraulic Radius R=A/P ft	Average Velocity V ft/s	Discharge (Flow Rate) Q=AV ft^3/s	Avg. Tractive Stress τ_o lb/ft^2	Comments
0.01	0.00	0.06	0.00	0.73	0.00	0.06	
0.26	0.20	1.64	0.12	6.42	1.29	1.46	
0.51	0.78	3.21	0.24	10.06	7.80	2.86	
0.76	1.72	4.79	0.36	13.13	22.59	4.26	
1.01	3.04	6.37	0.48	15.87	48.24	5.66	
1.26	4.73	7.94	0.60	18.39	87.01	7.06	
1.51	6.80	9.52	0.71	20.75	140.99	8.46	
1.75	9.23	11.09	0.83	22.98	212.15	9.87	
2.00	12.04	12.67	0.95	25.11	302.32	11.27	
2.25	15.22	14.25	1.07	27.15	413.29	12.67	
2.50	18.78	15.82	1.19	29.12	546.73	14.07	
2.75	22.70	17.40	1.30	31.02	704.28	15.47	
3.00	27.00	18.97	1.42	32.87	887.52	16.87	
0.71	1.51	4.49	0.34	12.57	19.01	3.99	DESIGN Q



Design/Check: Trapezoidal/Triangular Channel

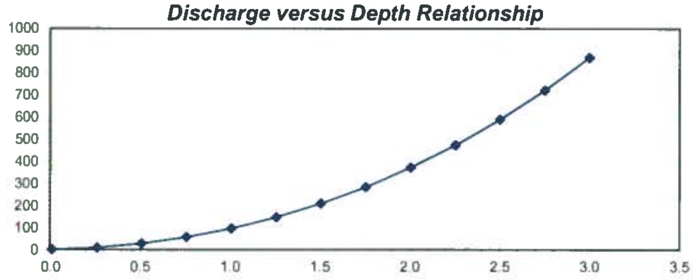
Methodology: Manning's Equation

Project: Stingy Run FAR Closure

Ditch ID: **Collector Swale**

Peak Discharge, Q_{max} = 50.00 cfs
Bottom Width, B = 10.00 ft
Left Side Slope, Z_1 = 5.00 horizontal :1 vertical
Right Side Slope, Z_2 = 5.00 horizontal :1 vertical
Manning's Roughness Coeff., n = 0.050
Longitudinal Channel Slope, S_o = 0.0667 ft/ft

Depth of Flow Y ft	Area of Flow A ft^2	Wetted Perimeter P ft	Hydraulic Radius R=A/P ft	Average Velocity V ft/s	Discharge (Flow Rate) Q=AV ft^3/s	Avg. Tractive Stress τ_o lb/ft^2	Comments
0.01	0.10	10.10	0.01	0.36	0.04	0.04	
0.26	2.93	12.64	0.23	2.90	8.49	0.96	
0.51	6.38	15.18	0.42	4.31	27.50	1.75	
0.76	10.44	17.73	0.59	5.41	56.47	2.45	
1.01	15.13	20.27	0.75	6.33	95.84	3.11	
1.26	20.44	22.81	0.90	7.15	146.24	3.73	
1.51	26.38	25.35	1.04	7.90	208.39	4.33	
1.75	32.93	27.89	1.18	8.60	283.04	4.91	
2.00	40.10	30.43	1.32	9.25	370.90	5.48	
2.25	47.89	32.97	1.45	9.87	472.73	6.04	
2.50	56.31	35.51	1.59	10.46	589.23	6.60	
2.75	65.34	38.05	1.72	11.04	721.12	7.14	
3.00	75.00	40.59	1.85	11.59	869.09	7.69	
0.71	9.62	17.24	0.56	5.21	50.16	2.32	DESIGN Q


Discharge versus Depth Relationship

Riprap Collector Swale Design

Max velocity in channel	5.962	ft/sec	See Above	
Max hydraulic depth in swale	0.89	ft	See Above	
Max shear stress	2.8	lb/sq ft	See Above	
Angle of repose	42	degrees	From Chart 4, for d50 of 12" or 24"	
angle with 5:1 side slopes	11.3099	degrees	tan^-1 (1/3)	
Angle of repose	0.73303829	radians	From Chart 4, for d50 of 12" or 24"	
angle with 5:1 side slopes	0.19739499	radians	tan^-1 (1/3)*180/pi	
K1 constant for equation 6 for d50 in HEC-11	0.95608481		From equation 7 in HEC-11	
Shield's equation constant, tc*	0.054		From Fischenrich, 2001	
Max allowable shear stress based on Shield's equation f	5.5404	lb/sq ft	Shield Equation	
Specific gravity of riprap	2.64	g/cm3	Assumed	GOOD
Calculated d50 according to HEC-11	0.21	ft	d50: 0.001 *V^3 / (d^0.5 * K1^0.5)	
Ohio Type C Rip Rap	1	ft	Assumed	

* Note: Swale velocity is acceptable for D50 of 0.5' but shear stress is not acceptable; use D50 of 1.0'

4	ft/2	Max velocity for 2.5" stone based on Isbach Curve	NO
6.25	ft/s	Max velocity for 6" stone based on Isbach Curve	GOOD
10	ft/s	Max velocity for 12" stone based on Isbach Curve	GOOD

ATTACHMENT 6

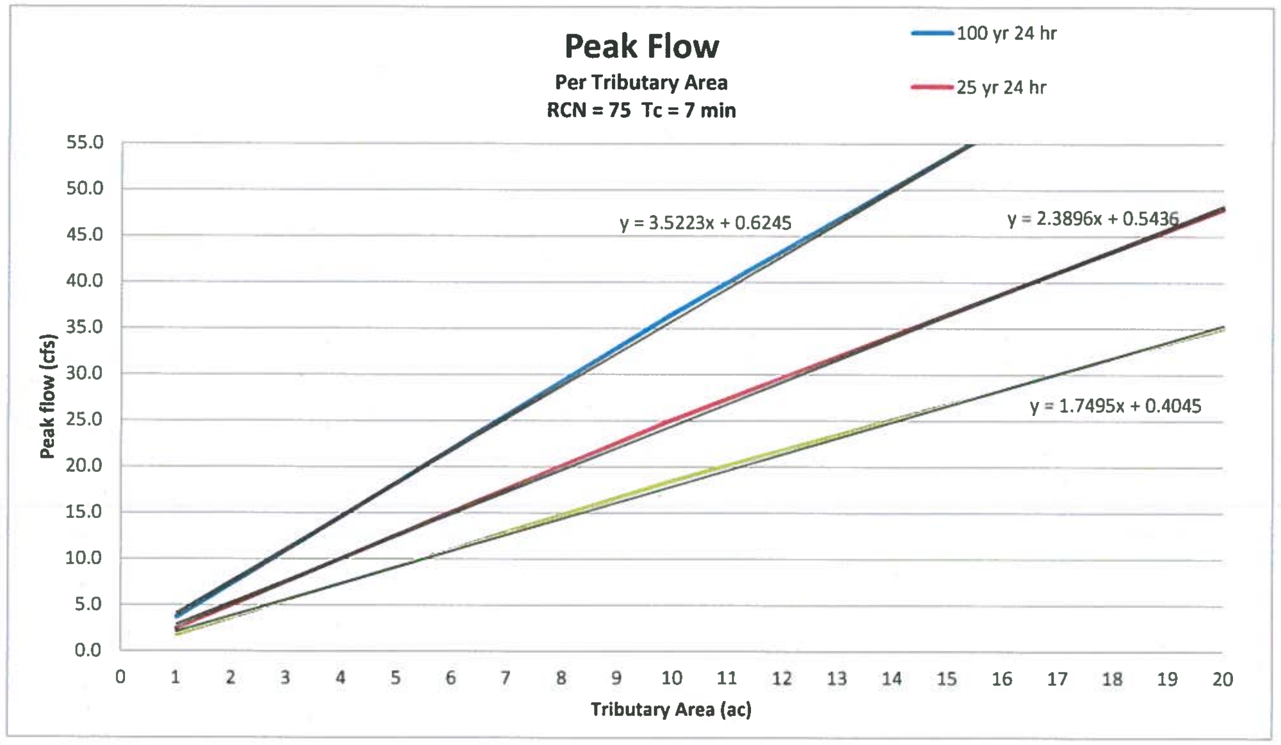
Flow vs. Area Rate Curves and Tables

ATTACHMENT 6: Flow VS Area Rating Curves

Post Construction

	area (sq mi)	area (ac)	10 yr 24 hr	25 yr 24 hr	100 yr 24 hr
Basin-1	0.001563	1	1.8	2.5	3.7
Basin-5	0.007813	5	9.2	12.6	18.3
Basin-10	0.015625	10	18.5	25.1	36.6
Basin-20	0.031250	20	35.1	48.0	70.7

* Used HEC-HMS model

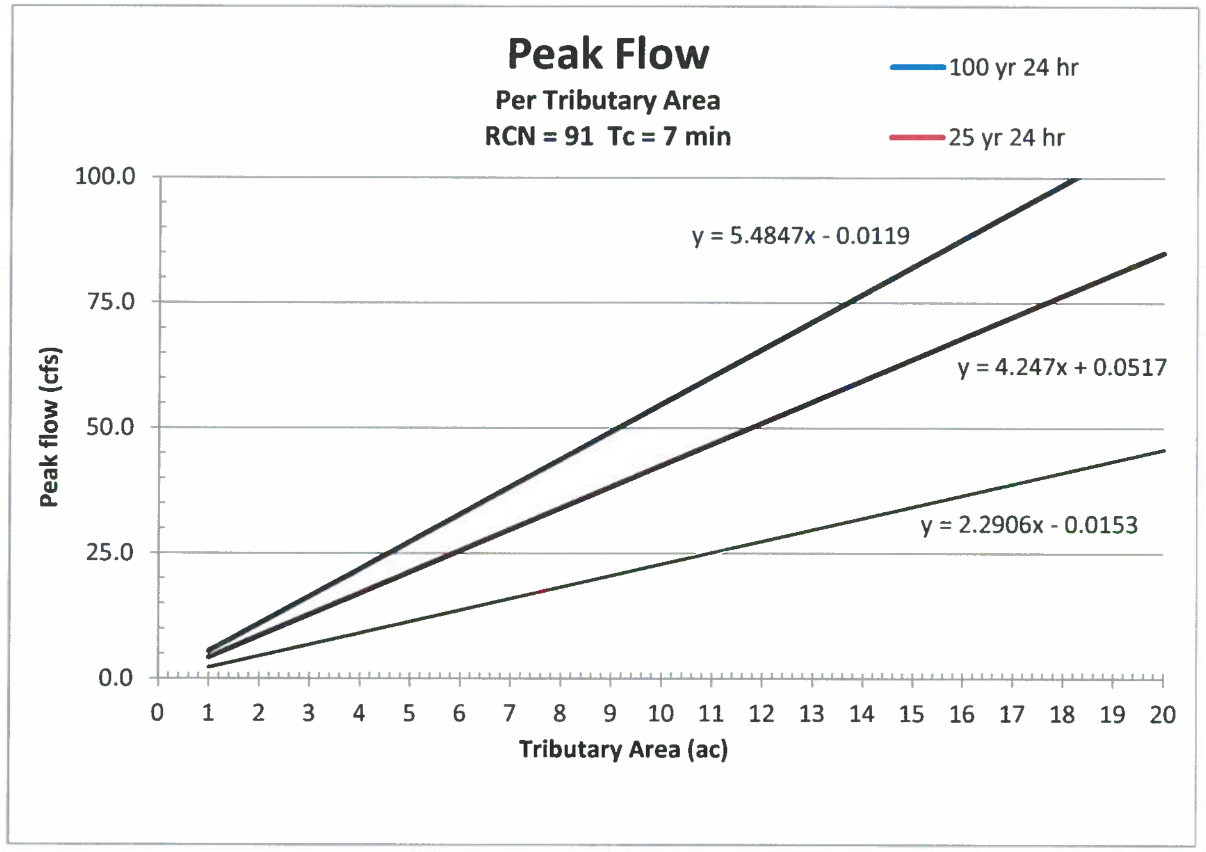


ATTACHMENT 6: Flow VS Area Rating Curves

Constructoin

	area (sq mi)	area (ac)	2yr 24 hour	25 yr 24 hr	100 yr 24 hr
Basin-1	0.001563	1	2	4.3	5.5
Basin-5	0.007813	5	11	21.3	27.4
Basin-10	0.015625	10	23	42.5	54.8
Basin-20	0.031250	20	46	85.0	109.7

* Used HEC-HMS model



Flow Vs. Area Table RNC = 75; Tc = 7 min

	area (sq mi)	area (ac)	bench width (ft)	bench lenth (ft)	10 yr 24 hr	25 yr 24 hr	100 yr 24 hr
Basin-1	0.001563	1	120	363	1.8	2.5	3.7
Basin-5	0.007813	5	120	1815	9.2	12.6	18.3
Basin-10	0.015625	10	120	3630	18.5	25.1	36.6
Basin-20	0.031250	20	120	7260	35.1	48.0	70.7
Basin-30	0.046875	30	120	10890	52.6	72.1	106.0
Basin-40	0.062500	40	120	14520	70.1	96.1	141.4
Basin-50	0.078125	50	120	18150	87.6	120.1	176.7
Basin-60	0.093750	60	120	21780	105.2	144.1	212.0
Basin-70	0.109375	70	120	25410	122.7	168.2	247.4
Basin-80	0.125000	80	120	29040	140.2	192.2	282.7

* Used HEC-HMS model "Flow vs Area"

Flow Vs. Area Table RNC = 91; Tc = 7 min

	area (sq mi)	area (ac)	bench width (ft)	bench lenth (ft)	25 yr 24 hr	100 yr 24 hr
Basin-1	0.001563	1	120	363	4.3	5.5
Basin-5	0.007813	5	120	1815	21.3	27.4
Basin-10	0.015625	10	120	3630	42.5	54.8
Basin-20	0.031250	20	120	7260	85.0	109.7
Basin-30	0.046875	30	120	10890	127.5	164.5
Basin-40	0.062500	40	120	14520	170.1	219.4
Basin-50	0.078125	50	120	18150	212.6	274.2
Basin-60	0.093750	60	120	21780	255.1	329.1
Basin-70	0.109375	70	120	25410	297.6	383.9
Basin-80	0.125000	80	120	29040	340.1	438.8

* Used HEC-HMS model "Flow vs Area"

ATTACHMENT 7

Erosion Control Materials Design Software Input and Results

Tensar

NORTH AMERICAN GREEN®

Tensar International Corporation
5401 St. Wendel-Cynthiana Road
Poseyville, Indiana 47633
Tel. 800.772.2040
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www.nagreen.com

Erosion Control Materials Design Software
Version 5.0

Project Name: FAD
Project Number: 78135
Project Location: Oak Brook, Illinois
Channel Name: GD-N1

Discharge	9.71
Peak Flow Period	24
Channel Slope	0.15
Channel Bottom Width	0
Left Side Slope	3
Right Side Slope	3
Low Flow Liner	
Retardance Class	D
Vegtation Type	Bunch Type
Vegetation Density	Good 75-95%
Soil Type	Clay

DS75

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
DS75 Unvegetated	Straight	9.71 cfs	9.71 ft/s	0.58 ft	0.025	1.55 lbs/ft2	5.4 lbs/ft2	0.29	UNSTABLE	D

S75BN

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
S75BN Unvegetated	Straight	9.71 cfs	9.71 ft/s	0.58 ft	0.025	1.6 lbs/ft2	5.4 lbs/ft2	0.3	UNSTABLE	D

P300 - Class D - Bunch Type - Good 75-95%

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
P300 Unvegetated	Straight	9.71 cfs	9.71 ft/s	0.58 ft	0.025	2 lbs/ft2	5.4 lbs/ft2	0.37	UNSTABLE	E
P300 Reinforced Vegetation	Straight	9.71 cfs	9.71 ft/s	0.58 ft	0.025	8 lbs/ft2	5.4 lbs/ft2	1.48	STABLE	E
Underlying Substrate	Straight	9.71 cfs	9.71 ft/s	0.58 ft	--	2 lbs/ft2	4.788 lbs/ft2	0.42	UNSTABLE	--

ShoreMax - Class D - Bunch Type - Good 75-95%

Phase	Reach	Discharge	Velocity	Normal	Mannings	Permissible	Calculated	Safety	Remarks	Staple

				Depth	N	Shear Stress	Shear Stress	Factor		Pattern
ShoreMax w/ SC250 Unvegetated	Straight	9.71 cfs	9.71 ft/s	0.58 ft	0.025	7.5 lbs/ft2	5.4 lbs/ft2	1.39	STABLE	F
ShoreMax w/ SC250 Reinforced Vegetation	Straight	9.71 cfs	9.71 ft/s	0.58 ft	0.025	8 lbs/ft2	5.4 lbs/ft2	1.48	STABLE	F
Underlying Substrate	Straight	9.71 cfs	9.71 ft/s	0.58 ft	--	3.25 lbs/ft2	6.765 lbs/ft2	0.48	UNSTABLE	--

S75

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
S75 Unvegetated	Straight	9.71 cfs	9.71 ft/s	0.58 ft	0.025	1.55 lbs/ft2	5.4 lbs/ft2	0.29	UNSTABLE	D

S150BN

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
S150BN Unvegetated	Straight	9.71 cfs	9.71 ft/s	0.58 ft	0.025	1.85 lbs/ft2	5.4 lbs/ft2	0.34	UNSTABLE	D

SC250 - Class D - Bunch Type - Good 75-95%

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
SC250 Unvegetated	Straight	9.71 cfs	9.71 ft/s	0.58 ft	0.025	2.5 lbs/ft2	5.4 lbs/ft2	0.46	UNSTABLE	E
SC250 Reinforced Vegetation	Straight	9.71 cfs	9.71 ft/s	0.58 ft	0.025	8 lbs/ft2	5.4 lbs/ft2	1.48	STABLE	E
Underlying Substrate	Straight	9.71 cfs	9.71 ft/s	0.58 ft	--	0.8 lbs/ft2	6.41 lbs/ft2	0.12	UNSTABLE	--

Rock Riprap - Class D - Bunch Type - Good 75-95%

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
Rock Riprap Unvegetated	Straight	9.71 cfs	9.71 ft/s	0.58 ft	0.025	2 lbs/ft2	5.4 lbs/ft2	0.37	UNSTABLE	--

DS150

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
DS150 Unvegetated	Straight	9.71 cfs	9.71 ft/s	0.58 ft	0.025	1.75 lbs/ft2	5.4 lbs/ft2	0.32	UNSTABLE	D

SC150BN

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
SC150BN Unvegetated	Straight	9.71 cfs	9.71 ft/s	0.58 ft	0.025	2.1 lbs/ft2	5.4 lbs/ft2	0.39	UNSTABLE	D

C350 - Class D - Bunch Type - Good 75-95%

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
C350 Unvegetated	Straight	9.71 cfs	9.71 ft/s	0.58 ft	0.025	3 lbs/ft2	5.4 lbs/ft2	0.56	UNSTABLE	E

C350 Reinforced Vegetation	Straight	9.71 cfs	9.71 ft/s	0.58 ft	0.025	10 lbs/ft2	5.4 lbs/ft2	1.85	STABLE	E
Underlying Substrate	Straight	9.71 cfs	9.71 ft/s	0.58 ft	--	1.2 lbs/ft2	6.747 lbs/ft2	0.18	UNSTABLE	--

S150

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
S150 Unvegetated	Straight	9.71 cfs	9.71 ft/s	0.58 ft	0.025	1.75 lbs/ft2	5.4 lbs/ft2	0.32	UNSTABLE	D

C125BN

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
C125BN Unvegetated	Straight	9.71 cfs	9.71 ft/s	0.58 ft	0.025	2.35 lbs/ft2	5.4 lbs/ft2	0.43	UNSTABLE	D

P550 - Class D - Bunch Type - Good 75-95%

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
P550 Unvegetated	Straight	9.71 cfs	9.71 ft/s	0.58 ft	0.025	3.25 lbs/ft2	5.4 lbs/ft2	0.6	UNSTABLE	E
P550 Reinforced Vegetation	Straight	9.71 cfs	9.71 ft/s	0.58 ft	0.025	12 lbs/ft2	5.4 lbs/ft2	2.22	STABLE	E
Underlying Substrate	Straight	9.71 cfs	9.71 ft/s	0.58 ft	--	3.25 lbs/ft2	6.765 lbs/ft2	0.48	UNSTABLE	--

SC150

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
SC150 Unvegetated	Straight	9.71 cfs	9.71 ft/s	0.58 ft	0.025	2 lbs/ft2	5.4 lbs/ft2	0.37	UNSTABLE	D

C125

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
C125 Unvegetated	Straight	9.71 cfs	9.71 ft/s	0.58 ft	0.025	2.25 lbs/ft2	5.4 lbs/ft2	0.42	UNSTABLE	D

Unreinforced Vegetation - Class D - Bunch Type - Good 75-95%

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
Unreinforced Vegetation	Straight	9.71 cfs	9.71 ft/s	0.58 ft	0.025	3.33 lbs/ft2	5.4 lbs/ft2	0.62	UNSTABLE	--
Underlying Substrate	Straight	9.71 cfs	9.71 ft/s	0.58 ft	--	0.07 lbs/ft2	1.052 lbs/ft2	0.07	UNSTABLE	--

Tensar
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www.nagreen.com

Erosion Control Materials Design Software
Version 5.0

Project Name: FAD
Project Number: 78135
Project Location: Oak Brook, Illinois
Channel Name: GD-N2

Discharge	20.98
Peak Flow Period	24
Channel Slope	0.13
Channel Bottom Width	0
Left Side Slope	3
Right Side Slope	3
Low Flow Liner	
Retardance Class	D
Vegtation Type	Bunch Type
Vegetation Density	Good 75-95%
Soil Type	Clay

DS75

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
DS75 Unvegetated	Straight	20.98 cfs	11.16 ft/s	0.79 ft	0.025	1.55 lbs/ft2	6.42 lbs/ft2	0.24	UNSTABLE	D

S75BN

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
S75BN Unvegetated	Straight	20.98 cfs	11.16 ft/s	0.79 ft	0.025	1.6 lbs/ft2	6.42 lbs/ft2	0.25	UNSTABLE	D

P300 - Class D - Bunch Type - Good 75-95%

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
P300 Unvegetated	Straight	20.98 cfs	11.16 ft/s	0.79 ft	0.025	2 lbs/ft2	6.42 lbs/ft2	0.31	UNSTABLE	E
P300 Reinforced Vegetation	Straight	20.98 cfs	11.16 ft/s	0.79 ft	0.025	8 lbs/ft2	6.42 lbs/ft2	1.25	STABLE	E
Underlying Substrate	Straight	20.98 cfs	11.16 ft/s	0.79 ft	--	2 lbs/ft2	5.027 lbs/ft2	0.4	UNSTABLE	--

ShoreMax - Class D - Bunch Type - Good 75-95%

Phase	Reach	Discharge	Velocity	Normal	Mannings	Permissible	Calculated	Safety	Remarks	Staple

				Depth	N	Shear Stress	Shear Stress	Factor		Pattern
ShoreMax w/ SC250 Unvegetated	Straight	20.98 cfs	11.16 ft/s	0.79 ft	0.025	7.5 lbs/ft2	6.42 lbs/ft2	1.17	STABLE	G
ShoreMax w/ SC250 Reinforced Vegetation	Straight	20.98 cfs	11.16 ft/s	0.79 ft	0.025	8 lbs/ft2	6.42 lbs/ft2	1.25	STABLE	G
Underlying Substrate	Straight	20.98 cfs	11.16 ft/s	0.79 ft	--	3.25 lbs/ft2	6.495 lbs/ft2	0.5	UNSTABLE	--

S75

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
S75 Unvegetated	Straight	20.98 cfs	11.16 ft/s	0.79 ft	0.025	1.55 lbs/ft2	6.42 lbs/ft2	0.24	UNSTABLE	D

S150BN

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
S150BN Unvegetated	Straight	20.98 cfs	11.16 ft/s	0.79 ft	0.025	1.85 lbs/ft2	6.42 lbs/ft2	0.29	UNSTABLE	D

SC250 - Class D - Bunch Type - Good 75-95%

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
SC250 Unvegetated	Straight	20.98 cfs	11.16 ft/s	0.79 ft	0.025	2.5 lbs/ft2	6.42 lbs/ft2	0.39	UNSTABLE	E
SC250 Reinforced Vegetation	Straight	20.98 cfs	11.16 ft/s	0.79 ft	0.025	8 lbs/ft2	6.42 lbs/ft2	1.25	STABLE	E
Underlying Substrate	Straight	20.98 cfs	11.16 ft/s	0.79 ft	--	0.8 lbs/ft2	6.066 lbs/ft2	0.13	UNSTABLE	--

Rock Riprap - Class D - Bunch Type - Good 75-95%

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
Rock Riprap Unvegetated	Straight	20.98 cfs	11.16 ft/s	0.79 ft	0.025	2 lbs/ft2	6.42 lbs/ft2	0.31	UNSTABLE	--

DS150

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
DS150 Unvegetated	Straight	20.98 cfs	11.16 ft/s	0.79 ft	0.025	1.75 lbs/ft2	6.42 lbs/ft2	0.27	UNSTABLE	D

SC150BN

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
SC150BN Unvegetated	Straight	20.98 cfs	11.16 ft/s	0.79 ft	0.025	2.1 lbs/ft2	6.42 lbs/ft2	0.33	UNSTABLE	D

C350 - Class D - Bunch Type - Good 75-95%

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
C350 Unvegetated	Straight	20.98 cfs	11.16 ft/s	0.79 ft	0.025	3 lbs/ft2	6.42 lbs/ft2	0.47	UNSTABLE	E

C350 Reinforced Vegetation	Straight	20.98 cfs	11.16 ft/s	0.79 ft	0.025	10 lbs/ft2	6.42 lbs/ft2	1.56	STABLE	E
Underlying Substrate	Straight	20.98 cfs	11.16 ft/s	0.79 ft	--	1.2 lbs/ft2	6.424 lbs/ft2	0.19	UNSTABLE	--

S150

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
S150 Unvegetated	Straight	20.98 cfs	11.16 ft/s	0.79 ft	0.025	1.75 lbs/ft2	6.42 lbs/ft2	0.27	UNSTABLE	D

C125BN

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
C125BN Unvegetated	Straight	20.98 cfs	11.16 ft/s	0.79 ft	0.025	2.35 lbs/ft2	6.42 lbs/ft2	0.37	UNSTABLE	D

P550 - Class D - Bunch Type - Good 75-95%

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
P550 Unvegetated	Straight	20.98 cfs	11.16 ft/s	0.79 ft	0.025	3.25 lbs/ft2	6.42 lbs/ft2	0.51	UNSTABLE	E
P550 Reinforced Vegetation	Straight	20.98 cfs	11.16 ft/s	0.79 ft	0.025	12 lbs/ft2	6.42 lbs/ft2	1.87	STABLE	E
Underlying Substrate	Straight	20.98 cfs	11.16 ft/s	0.79 ft	--	3.25 lbs/ft2	6.495 lbs/ft2	0.5	UNSTABLE	--

SC150

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
SC150 Unvegetated	Straight	20.98 cfs	11.16 ft/s	0.79 ft	0.025	2 lbs/ft2	6.42 lbs/ft2	0.31	UNSTABLE	D

C125

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
C125 Unvegetated	Straight	20.98 cfs	11.16 ft/s	0.79 ft	0.025	2.25 lbs/ft2	6.42 lbs/ft2	0.35	UNSTABLE	D

Unreinforced Vegetation - Class D - Bunch Type - Good 75-95%

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
Unreinforced Vegetation	Straight	20.98 cfs	11.16 ft/s	0.79 ft	0.025	3.33 lbs/ft2	6.42 lbs/ft2	0.52	UNSTABLE	--
Underlying Substrate	Straight	20.98 cfs	11.16 ft/s	0.79 ft	--	0.07 lbs/ft2	1.25 lbs/ft2	0.06	UNSTABLE	--

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www.nagreen.com

Erosion Control Materials Design Software
Version 5.0

Project Name: FAD
Project Number: 78135
Project Location: Oak Brook, Illinois
Channel Name: GD-N3

Discharge	26.71
Peak Flow Period	24
Channel Slope	.08
Channel Bottom Width	0
Left Side Slope	3
Right Side Slope	3
Low Flow Liner	
Retardance Class	D
Vegtation Type	Bunch Type
Vegetation Density	Good 75-95%
Soil Type	Clay

DS75

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
DS75 Unvegetated	Straight	26.71 cfs	9.88 ft/s	0.95 ft	0.025	1.55 lbs/ft2	4.74 lbs/ft2	0.33	UNSTABLE	D

S75BN

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
S75BN Unvegetated	Straight	26.71 cfs	9.88 ft/s	0.95 ft	0.025	1.6 lbs/ft2	4.74 lbs/ft2	0.34	UNSTABLE	D

P300 - Class D - Bunch Type - Good 75-95%

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
P300 Unvegetated	Straight	26.71 cfs	9.88 ft/s	0.95 ft	0.025	2 lbs/ft2	4.74 lbs/ft2	0.42	UNSTABLE	E
P300 Reinforced Vegetation	Straight	26.71 cfs	9.88 ft/s	0.95 ft	0.025	8 lbs/ft2	4.74 lbs/ft2	1.69	STABLE	E
Underlying Substrate	Straight	26.71 cfs	9.88 ft/s	0.95 ft	--	2 lbs/ft2	3.368 lbs/ft2	0.59	UNSTABLE	--

ShoreMax - Class D - Bunch Type - Good 75-95%

Phase	Reach	Discharge	Velocity	Normal	Mannings	Permissible	Calculated	Safety	Remarks	Staple

				Depth	N	Shear Stress	Shear Stress	Factor		Pattern
ShoreMax w/ SC250 Unvegetated	Straight	26.71 cfs	9.88 ft/s	0.95 ft	0.025	7.5 lbs/ft2	4.74 lbs/ft2	1.58	STABLE	F
ShoreMax w/ SC250 Reinforced Vegetation	Straight	26.71 cfs	9.88 ft/s	0.95 ft	0.025	8 lbs/ft2	4.74 lbs/ft2	1.69	STABLE	F
Underlying Substrate	Straight	26.71 cfs	9.88 ft/s	0.95 ft	--	3.25 lbs/ft2	4.032 lbs/ft2	0.81	UNSTABLE	--

S75

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
S75 Unvegetated	Straight	26.71 cfs	9.88 ft/s	0.95 ft	0.025	1.55 lbs/ft2	4.74 lbs/ft2	0.33	UNSTABLE	D

S150BN

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
S150BN Unvegetated	Straight	26.71 cfs	9.88 ft/s	0.95 ft	0.025	1.85 lbs/ft2	4.74 lbs/ft2	0.39	UNSTABLE	D

SC250 - Class D - Bunch Type - Good 75-95%

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
SC250 Unvegetated	Straight	26.71 cfs	9.88 ft/s	0.95 ft	0.025	2.5 lbs/ft2	4.74 lbs/ft2	0.53	UNSTABLE	E
SC250 Reinforced Vegetation	Straight	26.71 cfs	9.88 ft/s	0.95 ft	0.025	8 lbs/ft2	4.74 lbs/ft2	1.69	STABLE	E
Underlying Substrate	Straight	26.71 cfs	9.88 ft/s	0.95 ft	--	0.8 lbs/ft2	3.717 lbs/ft2	0.22	UNSTABLE	--

Rock Riprap - Class D - Bunch Type - Good 75-95%

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
Rock Riprap Unvegetated	Straight	26.71 cfs	9.88 ft/s	0.95 ft	0.025	2 lbs/ft2	4.74 lbs/ft2	0.42	UNSTABLE	--

DS150

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
DS150 Unvegetated	Straight	26.71 cfs	9.88 ft/s	0.95 ft	0.025	1.75 lbs/ft2	4.74 lbs/ft2	0.37	UNSTABLE	D

SC150BN

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
SC150BN Unvegetated	Straight	26.71 cfs	9.88 ft/s	0.95 ft	0.025	2.1 lbs/ft2	4.74 lbs/ft2	0.44	UNSTABLE	D

C350 - Class D - Bunch Type - Good 75-95%

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
C350 Unvegetated	Straight	26.71 cfs	9.88 ft/s	0.95 ft	0.025	3 lbs/ft2	4.74 lbs/ft2	0.63	UNSTABLE	E

C350 Reinforced Vegetation	Straight	26.71 cfs	9.88 ft/s	0.95 ft	0.025	10 lbs/ft2	4.74 lbs/ft2	2.11	STABLE	E
Underlying Substrate	Straight	26.71 cfs	9.88 ft/s	0.95 ft	--	1.2 lbs/ft2	3.958 lbs/ft2	0.3	UNSTABLE	--

S150

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
S150 Unvegetated	Straight	26.71 cfs	9.88 ft/s	0.95 ft	0.025	1.75 lbs/ft2	4.74 lbs/ft2	0.37	UNSTABLE	D

C125BN

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
C125BN Unvegetated	Straight	26.71 cfs	9.88 ft/s	0.95 ft	0.025	2.35 lbs/ft2	4.74 lbs/ft2	0.5	UNSTABLE	D

P550 - Class D - Bunch Type - Good 75-95%

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
P550 Unvegetated	Straight	26.71 cfs	9.88 ft/s	0.95 ft	0.025	3.25 lbs/ft2	4.74 lbs/ft2	0.69	UNSTABLE	E
P550 Reinforced Vegetation	Straight	26.71 cfs	9.88 ft/s	0.95 ft	0.025	12 lbs/ft2	4.74 lbs/ft2	2.53	STABLE	E
Underlying Substrate	Straight	26.71 cfs	9.88 ft/s	0.95 ft	--	3.25 lbs/ft2	4.032 lbs/ft2	0.81	UNSTABLE	--

SC150

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
SC150 Unvegetated	Straight	26.71 cfs	9.88 ft/s	0.95 ft	0.025	2 lbs/ft2	4.74 lbs/ft2	0.42	UNSTABLE	D

C125

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
C125 Unvegetated	Straight	26.71 cfs	9.88 ft/s	0.95 ft	0.025	2.25 lbs/ft2	4.74 lbs/ft2	0.47	UNSTABLE	D

Unreinforced Vegetation - Class D - Bunch Type - Good 75-95%

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
Unreinforced Vegetation	Straight	26.71 cfs	9.88 ft/s	0.95 ft	0.025	3.33 lbs/ft2	4.74 lbs/ft2	0.7	UNSTABLE	--
Underlying Substrate	Straight	26.71 cfs	9.88 ft/s	0.95 ft	--	0.07 lbs/ft2	0.923 lbs/ft2	0.08	UNSTABLE	--

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Erosion Control Materials Design Software
Version 5.0

Project Name: FAD
Project Number: 78135
Project Location: Oak Brook, Illinois
Channel Name: GD-S1

Discharge	3.77
Peak Flow Period	24
Channel Slope	0.19
Channel Bottom Width	0
Left Side Slope	3
Right Side Slope	3
Low Flow Liner	
Retardance Class	D
Vegtation Type	Bunch Type
Vegetation Density	Good 75-95%
Soil Type	Clay

DS75

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
DS75 Unvegetated	Straight	3.77 cfs	8.37 ft/s	0.39 ft	0.025	1.55 lbs/ft2	4.59 lbs/ft2	0.34	UNSTABLE	D

S75BN

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
S75BN Unvegetated	Straight	3.77 cfs	8.37 ft/s	0.39 ft	0.025	1.6 lbs/ft2	4.59 lbs/ft2	0.35	UNSTABLE	D

P300 - Class D - Bunch Type - Good 75-95%

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
P300 Unvegetated	Straight	3.77 cfs	8.37 ft/s	0.39 ft	0.025	2 lbs/ft2	4.59 lbs/ft2	0.44	UNSTABLE	E
P300 Reinforced Vegetation	Straight	3.77 cfs	8.37 ft/s	0.39 ft	0.025	8 lbs/ft2	4.59 lbs/ft2	1.74	STABLE	E
Underlying Substrate	Straight	3.77 cfs	8.37 ft/s	0.39 ft	--	2 lbs/ft2	4.248 lbs/ft2	0.47	UNSTABLE	--

ShoreMax - Class D - Bunch Type - Good 75-95%

Phase	Reach	Discharge	Velocity	Normal	Mannings	Permissible	Calculated	Safety	Remarks	Staple

				Depth	N	Shear Stress	Shear Stress	Factor		Pattern
ShoreMax w/ SC250 Unvegetated	Straight	3.77 cfs	8.37 ft/s	0.39 ft	0.025	7.5 lbs/ft2	4.59 lbs/ft2	1.63	STABLE	F
ShoreMax w/ SC250 Reinforced Vegetation	Straight	3.77 cfs	8.37 ft/s	0.39 ft	0.025	8 lbs/ft2	4.59 lbs/ft2	1.74	STABLE	F
Underlying Substrate	Straight	3.77 cfs	8.37 ft/s	0.39 ft	--	3.25 lbs/ft2	6.177 lbs/ft2	0.53	UNSTABLE	--

S75

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
S75 Unvegetated	Straight	3.77 cfs	8.37 ft/s	0.39 ft	0.025	1.55 lbs/ft2	4.59 lbs/ft2	0.34	UNSTABLE	D

S150BN

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
S150BN Unvegetated	Straight	3.77 cfs	8.37 ft/s	0.39 ft	0.025	1.85 lbs/ft2	4.59 lbs/ft2	0.4	UNSTABLE	D

SC250 - Class D - Bunch Type - Good 75-95%

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
SC250 Unvegetated	Straight	3.77 cfs	8.37 ft/s	0.39 ft	0.025	2.5 lbs/ft2	4.59 lbs/ft2	0.54	UNSTABLE	E
SC250 Reinforced Vegetation	Straight	3.77 cfs	8.37 ft/s	0.39 ft	0.025	8 lbs/ft2	4.59 lbs/ft2	1.74	STABLE	E
Underlying Substrate	Straight	3.77 cfs	8.37 ft/s	0.39 ft	--	0.8 lbs/ft2	5.879 lbs/ft2	0.14	UNSTABLE	--

Rock Riprap - Class D - Bunch Type - Good 75-95%

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
Rock Riprap Unvegetated	Straight	3.77 cfs	8.37 ft/s	0.39 ft	0.025	2 lbs/ft2	4.59 lbs/ft2	0.44	UNSTABLE	--

DS150

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
DS150 Unvegetated	Straight	3.77 cfs	8.37 ft/s	0.39 ft	0.025	1.75 lbs/ft2	4.59 lbs/ft2	0.38	UNSTABLE	D

SC150BN

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
SC150BN Unvegetated	Straight	3.77 cfs	8.37 ft/s	0.39 ft	0.025	2.1 lbs/ft2	4.59 lbs/ft2	0.46	UNSTABLE	D

C350 - Class D - Bunch Type - Good 75-95%

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
C350 Unvegetated	Straight	3.77 cfs	8.37 ft/s	0.39 ft	0.025	3 lbs/ft2	4.59 lbs/ft2	0.65	UNSTABLE	E

C350 Reinforced Vegetation	Straight	3.77 cfs	8.37 ft/s	0.39 ft	0.025	10 lbs/ft2	4.59 lbs/ft2	2.18	STABLE	E
Underlying Substrate	Straight	3.77 cfs	8.37 ft/s	0.39 ft	--	1.2 lbs/ft2	6.177 lbs/ft2	0.19	UNSTABLE	--

S150

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
S150 Unvegetated	Straight	3.77 cfs	8.37 ft/s	0.39 ft	0.025	1.75 lbs/ft2	4.59 lbs/ft2	0.38	UNSTABLE	D

C125BN

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
C125BN Unvegetated	Straight	3.77 cfs	8.37 ft/s	0.39 ft	0.025	2.35 lbs/ft2	4.59 lbs/ft2	0.51	UNSTABLE	D

P550 - Class D - Bunch Type - Good 75-95%

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
P550 Unvegetated	Straight	3.77 cfs	8.37 ft/s	0.39 ft	0.025	3.25 lbs/ft2	4.59 lbs/ft2	0.71	UNSTABLE	E
P550 Reinforced Vegetation	Straight	3.77 cfs	8.37 ft/s	0.39 ft	0.025	12 lbs/ft2	4.59 lbs/ft2	2.61	STABLE	E
Underlying Substrate	Straight	3.77 cfs	8.37 ft/s	0.39 ft	--	3.25 lbs/ft2	6.177 lbs/ft2	0.53	UNSTABLE	--

SC150

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
SC150 Unvegetated	Straight	3.77 cfs	8.37 ft/s	0.39 ft	0.025	2 lbs/ft2	4.59 lbs/ft2	0.44	UNSTABLE	D

C125

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
C125 Unvegetated	Straight	3.77 cfs	8.37 ft/s	0.39 ft	0.025	2.25 lbs/ft2	4.59 lbs/ft2	0.49	UNSTABLE	D

Unreinforced Vegetation - Class D - Bunch Type - Good 75-95%

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
Unreinforced Vegetation	Straight	3.77 cfs	8.37 ft/s	0.39 ft	0.025	3.33 lbs/ft2	4.59 lbs/ft2	0.73	UNSTABLE	--
Underlying Substrate	Straight	3.77 cfs	8.37 ft/s	0.39 ft	--	0.07 lbs/ft2	0.894 lbs/ft2	0.08	UNSTABLE	--

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Erosion Control Materials Design Software
Version 5.0

Project Name: FAD
Project Number: 78135
Project Location: Oak Brook, Illinois
Channel Name: GD-S2

Discharge	6.37
Peak Flow Period	24
Channel Slope	0.19
Channel Bottom Width	0
Left Side Slope	3
Right Side Slope	3
Low Flow Liner	
Retardance Class	D
Vegtation Type	Bunch Type
Vegetation Density	Good 75-95%
Soil Type	Clay

DS75

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
DS75 Unvegetated	Straight	6.37 cfs	9.55 ft/s	0.47 ft	0.025	1.55 lbs/ft2	5.59 lbs/ft2	0.28	UNSTABLE	D

S75BN

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
S75BN Unvegetated	Straight	6.37 cfs	9.55 ft/s	0.47 ft	0.025	1.6 lbs/ft2	5.59 lbs/ft2	0.29	UNSTABLE	D

P300 - Class D - Bunch Type - Good 75-95%

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
P300 Unvegetated	Straight	6.37 cfs	9.55 ft/s	0.47 ft	0.025	2 lbs/ft2	5.59 lbs/ft2	0.36	UNSTABLE	E
P300 Reinforced Vegetation	Straight	6.37 cfs	9.55 ft/s	0.47 ft	0.025	8 lbs/ft2	5.59 lbs/ft2	1.43	STABLE	E
Underlying Substrate	Straight	6.37 cfs	9.55 ft/s	0.47 ft	--	2 lbs/ft2	5.171 lbs/ft2	0.39	UNSTABLE	--

ShoreMax - Class D - Bunch Type - Good 75-95%

Phase	Reach	Discharge	Velocity	Normal	Mannings	Permissible	Calculated	Safety	Remarks	Staple

				Depth	N	Shear Stress	Shear Stress	Factor		Pattern
ShoreMax w/ SC250 Unvegetated	Straight	6.37 cfs	9.55 ft/s	0.47 ft	0.025	7.5 lbs/ft2	5.59 lbs/ft2	1.34	STABLE	F
ShoreMax w/ SC250 Reinforced Vegetation	Straight	6.37 cfs	9.55 ft/s	0.47 ft	0.025	8 lbs/ft2	5.59 lbs/ft2	1.43	STABLE	F
Underlying Substrate	Straight	6.37 cfs	9.55 ft/s	0.47 ft	--	3.25 lbs/ft2	7.519 lbs/ft2	0.43	UNSTABLE	--

S75

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
S75 Unvegetated	Straight	6.37 cfs	9.55 ft/s	0.47 ft	0.025	1.55 lbs/ft2	5.59 lbs/ft2	0.28	UNSTABLE	D

S150BN

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
S150BN Unvegetated	Straight	6.37 cfs	9.55 ft/s	0.47 ft	0.025	1.85 lbs/ft2	5.59 lbs/ft2	0.33	UNSTABLE	D

SC250 - Class D - Bunch Type - Good 75-95%

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
SC250 Unvegetated	Straight	6.37 cfs	9.55 ft/s	0.47 ft	0.025	2.5 lbs/ft2	5.59 lbs/ft2	0.45	UNSTABLE	E
SC250 Reinforced Vegetation	Straight	6.37 cfs	9.55 ft/s	0.47 ft	0.025	8 lbs/ft2	5.59 lbs/ft2	1.43	STABLE	E
Underlying Substrate	Straight	6.37 cfs	9.55 ft/s	0.47 ft	--	0.8 lbs/ft2	7.157 lbs/ft2	0.11	UNSTABLE	--

Rock Riprap - Class D - Bunch Type - Good 75-95%

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
Rock Riprap Unvegetated	Straight	6.37 cfs	9.55 ft/s	0.47 ft	0.025	2 lbs/ft2	5.59 lbs/ft2	0.36	UNSTABLE	--

DS150

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
DS150 Unvegetated	Straight	6.37 cfs	9.55 ft/s	0.47 ft	0.025	1.75 lbs/ft2	5.59 lbs/ft2	0.31	UNSTABLE	D

SC150BN

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
SC150BN Unvegetated	Straight	6.37 cfs	9.55 ft/s	0.47 ft	0.025	2.1 lbs/ft2	5.59 lbs/ft2	0.38	UNSTABLE	D

C350 - Class D - Bunch Type - Good 75-95%

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
C350 Unvegetated	Straight	6.37 cfs	9.55 ft/s	0.47 ft	0.025	3 lbs/ft2	5.59 lbs/ft2	0.54	UNSTABLE	E

C350 Reinforced Vegetation	Straight	6.37 cfs	9.55 ft/s	0.47 ft	0.025	10 lbs/ft2	5.59 lbs/ft2	1.79	STABLE	E
Underlying Substrate	Straight	6.37 cfs	9.55 ft/s	0.47 ft	--	1.2 lbs/ft2	7.519 lbs/ft2	0.16	UNSTABLE	--

S150

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
S150 Unvegetated	Straight	6.37 cfs	9.55 ft/s	0.47 ft	0.025	1.75 lbs/ft2	5.59 lbs/ft2	0.31	UNSTABLE	D

C125BN

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
C125BN Unvegetated	Straight	6.37 cfs	9.55 ft/s	0.47 ft	0.025	2.35 lbs/ft2	5.59 lbs/ft2	0.42	UNSTABLE	D

P550 - Class D - Bunch Type - Good 75-95%

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
P550 Unvegetated	Straight	6.37 cfs	9.55 ft/s	0.47 ft	0.025	3.25 lbs/ft2	5.59 lbs/ft2	0.58	UNSTABLE	E
P550 Reinforced Vegetation	Straight	6.37 cfs	9.55 ft/s	0.47 ft	0.025	12 lbs/ft2	5.59 lbs/ft2	2.15	STABLE	E
Underlying Substrate	Straight	6.37 cfs	9.55 ft/s	0.47 ft	--	3.25 lbs/ft2	7.519 lbs/ft2	0.43	UNSTABLE	--

SC150

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
SC150 Unvegetated	Straight	6.37 cfs	9.55 ft/s	0.47 ft	0.025	2 lbs/ft2	5.59 lbs/ft2	0.36	UNSTABLE	D

C125

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
C125 Unvegetated	Straight	6.37 cfs	9.55 ft/s	0.47 ft	0.025	2.25 lbs/ft2	5.59 lbs/ft2	0.4	UNSTABLE	D

Unreinforced Vegetation - Class D - Bunch Type - Good 75-95%

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
Unreinforced Vegetation	Straight	6.37 cfs	9.55 ft/s	0.47 ft	0.025	3.33 lbs/ft2	5.59 lbs/ft2	0.6	UNSTABLE	--
Underlying Substrate	Straight	6.37 cfs	9.55 ft/s	0.47 ft	--	0.07 lbs/ft2	1.089 lbs/ft2	0.06	UNSTABLE	--

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Erosion Control Materials Design Software
Version 5.0

Project Name: FAD
Project Number: 78135
Project Location: Oak Brook, Illinois
Channel Name: GD-S3

Discharge	15.82
Peak Flow Period	24
Channel Slope	.10
Channel Bottom Width	0
Left Side Slope	3
Right Side Slope	3
Low Flow Liner	
Retardance Class	D
Vegtation Type	Bunch Type
Vegetation Density	Good 75-95%
Soil Type	Clay

DS75

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
DS75 Unvegetated	Straight	15.82 cfs	9.42 ft/s	0.75 ft	0.025	1.55 lbs/ft2	4.67 lbs/ft2	0.33	UNSTABLE	D

S75BN

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
S75BN Unvegetated	Straight	15.82 cfs	9.42 ft/s	0.75 ft	0.025	1.6 lbs/ft2	4.67 lbs/ft2	0.34	UNSTABLE	D

P300 - Class D - Bunch Type - Good 75-95%

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
P300 Unvegetated	Straight	15.82 cfs	9.42 ft/s	0.75 ft	0.025	2 lbs/ft2	4.67 lbs/ft2	0.43	UNSTABLE	E
P300 Reinforced Vegetation	Straight	15.82 cfs	9.42 ft/s	0.75 ft	0.025	8 lbs/ft2	4.67 lbs/ft2	1.71	STABLE	E
Underlying Substrate	Straight	15.82 cfs	9.42 ft/s	0.75 ft	--	2 lbs/ft2	3.749 lbs/ft2	0.53	UNSTABLE	--

ShoreMax - Class D - Bunch Type - Good 75-95%

Phase	Reach	Discharge	Velocity	Normal	Mannings	Permissible	Calculated	Safety	Remarks	Staple

				Depth	N	Shear Stress	Shear Stress	Factor		Pattern
ShoreMax w/ SC250 Unvegetated	Straight	15.82 cfs	9.42 ft/s	0.75 ft	0.025	7.5 lbs/ft2	4.67 lbs/ft2	1.61	STABLE	F
ShoreMax w/ SC250 Reinforced Vegetation	Straight	15.82 cfs	9.42 ft/s	0.75 ft	0.025	8 lbs/ft2	4.67 lbs/ft2	1.71	STABLE	F
Underlying Substrate	Straight	15.82 cfs	9.42 ft/s	0.75 ft	--	3.25 lbs/ft2	4.94 lbs/ft2	0.66	UNSTABLE	--

S75

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
S75 Unvegetated	Straight	15.82 cfs	9.42 ft/s	0.75 ft	0.025	1.55 lbs/ft2	4.67 lbs/ft2	0.33	UNSTABLE	D

S150BN

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
S150BN Unvegetated	Straight	15.82 cfs	9.42 ft/s	0.75 ft	0.025	1.85 lbs/ft2	4.67 lbs/ft2	0.4	UNSTABLE	D

SC250 - Class D - Bunch Type - Good 75-95%

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
SC250 Unvegetated	Straight	15.82 cfs	9.42 ft/s	0.75 ft	0.025	2.5 lbs/ft2	4.67 lbs/ft2	0.54	UNSTABLE	E
SC250 Reinforced Vegetation	Straight	15.82 cfs	9.42 ft/s	0.75 ft	0.025	8 lbs/ft2	4.67 lbs/ft2	1.71	STABLE	E
Underlying Substrate	Straight	15.82 cfs	9.42 ft/s	0.75 ft	--	0.8 lbs/ft2	4.628 lbs/ft2	0.17	UNSTABLE	--

Rock Riprap - Class D - Bunch Type - Good 75-95%

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
Rock Riprap Unvegetated	Straight	15.82 cfs	9.42 ft/s	0.75 ft	0.025	2 lbs/ft2	4.67 lbs/ft2	0.43	UNSTABLE	--

DS150

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
DS150 Unvegetated	Straight	15.82 cfs	9.42 ft/s	0.75 ft	0.025	1.75 lbs/ft2	4.67 lbs/ft2	0.37	UNSTABLE	D

SC150BN

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
SC150BN Unvegetated	Straight	15.82 cfs	9.42 ft/s	0.75 ft	0.025	2.1 lbs/ft2	4.67 lbs/ft2	0.45	UNSTABLE	D

C350 - Class D - Bunch Type - Good 75-95%

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
C350 Unvegetated	Straight	15.82 cfs	9.42 ft/s	0.75 ft	0.025	3 lbs/ft2	4.67 lbs/ft2	0.64	UNSTABLE	E

C350 Reinforced Vegetation	Straight	15.82 cfs	9.42 ft/s	0.75 ft	0.025	10 lbs/ft2	4.67 lbs/ft2	2.14	STABLE	E
Underlying Substrate	Straight	15.82 cfs	9.42 ft/s	0.75 ft	--	1.2 lbs/ft2	4.895 lbs/ft2	0.25	UNSTABLE	--

S150

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
S150 Unvegetated	Straight	15.82 cfs	9.42 ft/s	0.75 ft	0.025	1.75 lbs/ft2	4.67 lbs/ft2	0.37	UNSTABLE	D

C125BN

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
C125BN Unvegetated	Straight	15.82 cfs	9.42 ft/s	0.75 ft	0.025	2.35 lbs/ft2	4.67 lbs/ft2	0.5	UNSTABLE	D

P550 - Class D - Bunch Type - Good 75-95%

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
P550 Unvegetated	Straight	15.82 cfs	9.42 ft/s	0.75 ft	0.025	3.25 lbs/ft2	4.67 lbs/ft2	0.7	UNSTABLE	E
P550 Reinforced Vegetation	Straight	15.82 cfs	9.42 ft/s	0.75 ft	0.025	12 lbs/ft2	4.67 lbs/ft2	2.57	STABLE	E
Underlying Substrate	Straight	15.82 cfs	9.42 ft/s	0.75 ft	--	3.25 lbs/ft2	4.94 lbs/ft2	0.66	UNSTABLE	--

SC150

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
SC150 Unvegetated	Straight	15.82 cfs	9.42 ft/s	0.75 ft	0.025	2 lbs/ft2	4.67 lbs/ft2	0.43	UNSTABLE	D

C125

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
C125 Unvegetated	Straight	15.82 cfs	9.42 ft/s	0.75 ft	0.025	2.25 lbs/ft2	4.67 lbs/ft2	0.48	UNSTABLE	D

Unreinforced Vegetation - Class D - Bunch Type - Good 75-95%

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
Unreinforced Vegetation	Straight	15.82 cfs	9.42 ft/s	0.75 ft	0.025	3.33 lbs/ft2	4.67 lbs/ft2	0.71	UNSTABLE	--
Underlying Substrate	Straight	15.82 cfs	9.42 ft/s	0.75 ft	--	0.07 lbs/ft2	0.909 lbs/ft2	0.08	UNSTABLE	--

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Erosion Control Materials Design Software
Version 5.0

Project Name: FAD
Project Number: 78135
Project Location: Oak Brook, Illinois
Channel Name: Interface Ditch

Discharge	12.80
Peak Flow Period	24
Channel Slope	0.19
Channel Bottom Width	0
Left Side Slope	3
Right Side Slope	3
Low Flow Liner	
Retardance Class	D
Vegtation Type	Bunch Type
Vegetation Density	Good 75-95%
Soil Type	Clay

DS75

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
DS75 Unvegetated	Straight	12.8 cfs	11.37 ft/s	0.61 ft	0.025	1.55 lbs/ft2	7.26 lbs/ft2	0.21	UNSTABLE	D

S75BN

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
S75BN Unvegetated	Straight	12.8 cfs	11.37 ft/s	0.61 ft	0.025	1.6 lbs/ft2	7.26 lbs/ft2	0.22	UNSTABLE	D

P300 - Class D - Bunch Type - Good 75-95%

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
P300 Unvegetated	Straight	12.8 cfs	11.37 ft/s	0.61 ft	0.025	2 lbs/ft2	7.26 lbs/ft2	0.28	UNSTABLE	E
P300 Reinforced Vegetation	Straight	12.8 cfs	11.37 ft/s	0.61 ft	0.025	8 lbs/ft2	7.26 lbs/ft2	1.1	STABLE	E
Underlying Substrate	Straight	12.8 cfs	11.37 ft/s	0.61 ft	--	2 lbs/ft2	6.309 lbs/ft2	0.32	UNSTABLE	--

ShoreMax - Class D - Bunch Type - Good 75-95%

Phase	Reach	Discharge	Velocity	Normal	Mannings	Permissible	Calculated	Safety	Remarks	Staple

				Depth	N	Shear Stress	Shear Stress	Factor		Pattern
ShoreMax w/ SC250 Unvegetated	Straight	12.8 cfs	11.37 ft/s	0.61 ft	0.025	7.5 lbs/ft2	7.26 lbs/ft2	1.03	STABLE	G
ShoreMax w/ SC250 Reinforced Vegetation	Straight	12.8 cfs	11.37 ft/s	0.61 ft	0.025	8 lbs/ft2	7.26 lbs/ft2	1.1	STABLE	G
Underlying Substrate	Straight	12.8 cfs	11.37 ft/s	0.61 ft	--	3.25 lbs/ft2	8.792 lbs/ft2	0.37	UNSTABLE	--

S75

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
S75 Unvegetated	Straight	12.8 cfs	11.37 ft/s	0.61 ft	0.025	1.55 lbs/ft2	7.26 lbs/ft2	0.21	UNSTABLE	D

S150BN

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
S150BN Unvegetated	Straight	12.8 cfs	11.37 ft/s	0.61 ft	0.025	1.85 lbs/ft2	7.26 lbs/ft2	0.25	UNSTABLE	D

SC250 - Class D - Bunch Type - Good 75-95%

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
SC250 Unvegetated	Straight	12.8 cfs	11.37 ft/s	0.61 ft	0.025	2.5 lbs/ft2	7.26 lbs/ft2	0.34	UNSTABLE	E
SC250 Reinforced Vegetation	Straight	12.8 cfs	11.37 ft/s	0.61 ft	0.025	8 lbs/ft2	7.26 lbs/ft2	1.1	STABLE	E
Underlying Substrate	Straight	12.8 cfs	11.37 ft/s	0.61 ft	--	0.8 lbs/ft2	8.313 lbs/ft2	0.1	UNSTABLE	--

Rock Riprap - Class D - Bunch Type - Good 75-95%

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
Rock Riprap Unvegetated	Straight	12.8 cfs	11.37 ft/s	0.61 ft	0.025	2 lbs/ft2	7.26 lbs/ft2	0.28	UNSTABLE	--

DS150

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
DS150 Unvegetated	Straight	12.8 cfs	11.37 ft/s	0.61 ft	0.025	1.75 lbs/ft2	7.26 lbs/ft2	0.24	UNSTABLE	D

SC150BN

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
SC150BN Unvegetated	Straight	12.8 cfs	11.37 ft/s	0.61 ft	0.025	2.1 lbs/ft2	7.26 lbs/ft2	0.29	UNSTABLE	D

C350 - Class D - Bunch Type - Good 75-95%

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
C350 Unvegetated	Straight	12.8 cfs	11.37 ft/s	0.61 ft	0.025	3 lbs/ft2	7.26 lbs/ft2	0.41	UNSTABLE	E

C350 Reinforced Vegetation	Straight	12.8 cfs	11.37 ft/s	0.61 ft	0.025	10 lbs/ft2	7.26 lbs/ft2	1.38	STABLE	E
Underlying Substrate	Straight	12.8 cfs	11.37 ft/s	0.61 ft	--	1.2 lbs/ft2	8.758 lbs/ft2	0.14	UNSTABLE	--

S150

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
S150 Unvegetated	Straight	12.8 cfs	11.37 ft/s	0.61 ft	0.025	1.75 lbs/ft2	7.26 lbs/ft2	0.24	UNSTABLE	D

C125BN

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
C125BN Unvegetated	Straight	12.8 cfs	11.37 ft/s	0.61 ft	0.025	2.35 lbs/ft2	7.26 lbs/ft2	0.32	UNSTABLE	D

P550 - Class D - Bunch Type - Good 75-95%

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
P550 Unvegetated	Straight	12.8 cfs	11.37 ft/s	0.61 ft	0.025	3.25 lbs/ft2	7.26 lbs/ft2	0.45	UNSTABLE	E
P550 Reinforced Vegetation	Straight	12.8 cfs	11.37 ft/s	0.61 ft	0.025	12 lbs/ft2	7.26 lbs/ft2	1.65	STABLE	E
Underlying Substrate	Straight	12.8 cfs	11.37 ft/s	0.61 ft	--	3.25 lbs/ft2	8.792 lbs/ft2	0.37	UNSTABLE	--

SC150

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
SC150 Unvegetated	Straight	12.8 cfs	11.37 ft/s	0.61 ft	0.025	2 lbs/ft2	7.26 lbs/ft2	0.28	UNSTABLE	D

C125

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
C125 Unvegetated	Straight	12.8 cfs	11.37 ft/s	0.61 ft	0.025	2.25 lbs/ft2	7.26 lbs/ft2	0.31	UNSTABLE	D

Unreinforced Vegetation - Class D - Bunch Type - Good 75-95%

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
Unreinforced Vegetation	Straight	12.8 cfs	11.37 ft/s	0.61 ft	0.025	3.33 lbs/ft2	7.26 lbs/ft2	0.46	UNSTABLE	--
Underlying Substrate	Straight	12.8 cfs	11.37 ft/s	0.61 ft	--	0.07 lbs/ft2	1.414 lbs/ft2	0.05	UNSTABLE	--

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Erosion Control Materials Design Software
Version 5.0

Project Name: FAD
Project Number: 78135
Project Location: Oak Brook, Illinois
Channel Name: Swale A-1

Discharge	33.52
Peak Flow Period	24
Channel Slope	.03
Channel Bottom Width	0
Left Side Slope	3
Right Side Slope	3
Low Flow Liner	
Retardance Class	D
Vegtation Type	Bunch Type
Vegetation Density	Good 75-95%
Soil Type	Clay

DS75

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
DS75 Unvegetated	Straight	33.52 cfs	7.24 ft/s	1.24 ft	0.025	1.55 lbs/ft2	2.33 lbs/ft2	0.67	UNSTABLE	D

S75BN

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
S75BN Unvegetated	Straight	33.52 cfs	7.24 ft/s	1.24 ft	0.025	1.6 lbs/ft2	2.33 lbs/ft2	0.69	UNSTABLE	D

P300 - Class D - Bunch Type - Good 75-95%

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
P300 Unvegetated	Straight	33.52 cfs	7.24 ft/s	1.24 ft	0.025	2 lbs/ft2	2.33 lbs/ft2	0.86	UNSTABLE	E
P300 Reinforced Vegetation	Straight	33.52 cfs	7.24 ft/s	1.24 ft	0.025	8 lbs/ft2	2.33 lbs/ft2	3.44	STABLE	E
Underlying Substrate	Straight	33.52 cfs	7.24 ft/s	1.24 ft	--	2 lbs/ft2	1.364 lbs/ft2	1.47	STABLE	--

ShoreMax - Class D - Bunch Type - Good 75-95%

Phase	Reach	Discharge	Velocity	Normal	Mannings	Permissible	Calculated	Safety	Remarks	Staple

				Depth	N	Shear Stress	Shear Stress	Factor		Pattern
ShoreMax w/ SC250 Unvegetated	Straight	33.52 cfs	7.24 ft/s	1.24 ft	0.025	7.5 lbs/ft2	2.33 lbs/ft2	3.22	STABLE	F
ShoreMax w/ SC250 Reinforced Vegetation	Straight	33.52 cfs	7.24 ft/s	1.24 ft	0.025	8 lbs/ft2	2.33 lbs/ft2	3.44	STABLE	F
Underlying Substrate	Straight	33.52 cfs	7.24 ft/s	1.24 ft	--	3.25 lbs/ft2	1.371 lbs/ft2	2.37	STABLE	--

S75

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
S75 Unvegetated	Straight	33.52 cfs	7.24 ft/s	1.24 ft	0.025	1.55 lbs/ft2	2.33 lbs/ft2	0.67	UNSTABLE	D

S150BN

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
S150BN Unvegetated	Straight	33.52 cfs	7.24 ft/s	1.24 ft	0.025	1.85 lbs/ft2	2.33 lbs/ft2	0.8	UNSTABLE	D

SC250 - Class D - Bunch Type - Good 75-95%

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
SC250 Unvegetated	Straight	33.52 cfs	7.24 ft/s	1.24 ft	0.025	2.5 lbs/ft2	2.33 lbs/ft2	1.07	STABLE	E
SC250 Reinforced Vegetation	Straight	33.52 cfs	7.24 ft/s	1.24 ft	0.025	8 lbs/ft2	2.33 lbs/ft2	3.44	STABLE	E
Underlying Substrate	Straight	33.52 cfs	7.24 ft/s	1.24 ft	--	0.8 lbs/ft2	1.224 lbs/ft2	0.65	UNSTABLE	--

Rock Riprap - Class D - Bunch Type - Good 75-95%

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
Rock Riprap Unvegetated	Straight	33.52 cfs	7.24 ft/s	1.24 ft	0.025	2 lbs/ft2	2.33 lbs/ft2	0.86	UNSTABLE	--

DS150

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
DS150 Unvegetated	Straight	33.52 cfs	7.24 ft/s	1.24 ft	0.025	1.75 lbs/ft2	2.33 lbs/ft2	0.75	UNSTABLE	D

SC150BN

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
SC150BN Unvegetated	Straight	33.52 cfs	7.24 ft/s	1.24 ft	0.025	2.1 lbs/ft2	2.33 lbs/ft2	0.9	UNSTABLE	D

C350 - Class D - Bunch Type - Good 75-95%

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
C350 Unvegetated	Straight	33.52 cfs	7.24 ft/s	1.24 ft	0.025	3 lbs/ft2	2.33 lbs/ft2	1.29	STABLE	E

C350 Reinforced Vegetation	Straight	33.52 cfs	7.24 ft/s	1.24 ft	0.025	10 lbs/ft2	2.33 lbs/ft2	4.3	STABLE	E
Underlying Substrate	Straight	33.52 cfs	7.24 ft/s	1.24 ft	--	1.2 lbs/ft2	1.321 lbs/ft2	0.91	UNSTABLE	--

S150

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
S150 Unvegetated	Straight	33.52 cfs	7.24 ft/s	1.24 ft	0.025	1.75 lbs/ft2	2.33 lbs/ft2	0.75	UNSTABLE	D

C125BN

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
C125BN Unvegetated	Straight	33.52 cfs	7.24 ft/s	1.24 ft	0.025	2.35 lbs/ft2	2.33 lbs/ft2	1.01	STABLE	D

P550 - Class D - Bunch Type - Good 75-95%

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
P550 Unvegetated	Straight	33.52 cfs	7.24 ft/s	1.24 ft	0.025	3.25 lbs/ft2	2.33 lbs/ft2	1.4	STABLE	E
P550 Reinforced Vegetation	Straight	33.52 cfs	7.24 ft/s	1.24 ft	0.025	12 lbs/ft2	2.33 lbs/ft2	5.16	STABLE	E
Underlying Substrate	Straight	33.52 cfs	7.24 ft/s	1.24 ft	--	3.25 lbs/ft2	1.371 lbs/ft2	2.37	STABLE	--

SC150

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
SC150 Unvegetated	Straight	33.52 cfs	7.24 ft/s	1.24 ft	0.025	2 lbs/ft2	2.33 lbs/ft2	0.86	UNSTABLE	D

C125

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
C125 Unvegetated	Straight	33.52 cfs	7.24 ft/s	1.24 ft	0.025	2.25 lbs/ft2	2.33 lbs/ft2	0.97	UNSTABLE	D

Unreinforced Vegetation - Class D - Bunch Type - Good 75-95%

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
Unreinforced Vegetation	Straight	33.52 cfs	7.24 ft/s	1.24 ft	0.025	3.33 lbs/ft2	2.33 lbs/ft2	1.43	STABLE	--
Underlying Substrate	Straight	33.52 cfs	7.24 ft/s	1.24 ft	--	0.07 lbs/ft2	0.453 lbs/ft2	0.15	UNSTABLE	--

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Version 5.0

Project Name: FAD
Project Number: 78135
Project Location: Oak Brook, Illinois
Channel Name: Swale A-2

Discharge	33.52
Peak Flow Period	24
Channel Slope	0.08
Channel Bottom Width	0
Left Side Slope	3
Right Side Slope	3
Low Flow Liner	
Retardance Class	D
Vegtation Type	Bunch Type
Vegetation Density	Good 75-95%
Soil Type	Clay

DS75

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
DS75 Unvegetated	Straight	33.52 cfs	10.46 ft/s	1.03 ft	0.025	1.55 lbs/ft2	5.16 lbs/ft2	0.3	UNSTABLE	D

S75BN

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
S75BN Unvegetated	Straight	33.52 cfs	10.46 ft/s	1.03 ft	0.025	1.6 lbs/ft2	5.16 lbs/ft2	0.31	UNSTABLE	D

P300 - Class D - Bunch Type - Good 75-95%

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
P300 Unvegetated	Straight	33.52 cfs	10.46 ft/s	1.03 ft	0.025	2 lbs/ft2	5.16 lbs/ft2	0.39	UNSTABLE	E
P300 Reinforced Vegetation	Straight	33.52 cfs	10.46 ft/s	1.03 ft	0.025	8 lbs/ft2	5.16 lbs/ft2	1.55	STABLE	E
Underlying Substrate	Straight	33.52 cfs	10.46 ft/s	1.03 ft	--	2 lbs/ft2	3.477 lbs/ft2	0.58	UNSTABLE	--

ShoreMax - Class D - Bunch Type - Good 75-95%

Phase	Reach	Discharge	Velocity	Normal	Mannings	Permissible	Calculated	Safety	Remarks	Staple

				Depth	N	Shear Stress	Shear Stress	Factor		Pattern
ShoreMax w/ SC250 Unvegetated	Straight	33.52 cfs	10.46 ft/s	1.03 ft	0.025	7.5 lbs/ft2	5.16 lbs/ft2	1.45	STABLE	F
ShoreMax w/ SC250 Reinforced Vegetation	Straight	33.52 cfs	10.46 ft/s	1.03 ft	0.025	8 lbs/ft2	5.16 lbs/ft2	1.55	STABLE	F
Underlying Substrate	Straight	33.52 cfs	10.46 ft/s	1.03 ft	--	3.25 lbs/ft2	3.977 lbs/ft2	0.82	UNSTABLE	--

S75

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
S75 Unvegetated	Straight	33.52 cfs	10.46 ft/s	1.03 ft	0.025	1.55 lbs/ft2	5.16 lbs/ft2	0.3	UNSTABLE	D

S150BN

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
S150BN Unvegetated	Straight	33.52 cfs	10.46 ft/s	1.03 ft	0.025	1.85 lbs/ft2	5.16 lbs/ft2	0.36	UNSTABLE	D

SC250 - Class D - Bunch Type - Good 75-95%

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
SC250 Unvegetated	Straight	33.52 cfs	10.46 ft/s	1.03 ft	0.025	2.5 lbs/ft2	5.16 lbs/ft2	0.48	UNSTABLE	E
SC250 Reinforced Vegetation	Straight	33.52 cfs	10.46 ft/s	1.03 ft	0.025	8 lbs/ft2	5.16 lbs/ft2	1.55	STABLE	E
Underlying Substrate	Straight	33.52 cfs	10.46 ft/s	1.03 ft	--	0.8 lbs/ft2	3.637 lbs/ft2	0.22	UNSTABLE	--

Rock Riprap - Class D - Bunch Type - Good 75-95%

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
Rock Riprap Unvegetated	Straight	33.52 cfs	10.46 ft/s	1.03 ft	0.025	2 lbs/ft2	5.16 lbs/ft2	0.39	UNSTABLE	--

DS150

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
DS150 Unvegetated	Straight	33.52 cfs	10.46 ft/s	1.03 ft	0.025	1.75 lbs/ft2	5.16 lbs/ft2	0.34	UNSTABLE	D

SC150BN

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
SC150BN Unvegetated	Straight	33.52 cfs	10.46 ft/s	1.03 ft	0.025	2.1 lbs/ft2	5.16 lbs/ft2	0.41	UNSTABLE	D

C350 - Class D - Bunch Type - Good 75-95%

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
C350 Unvegetated	Straight	33.52 cfs	10.46 ft/s	1.03 ft	0.025	3 lbs/ft2	5.16 lbs/ft2	0.58	UNSTABLE	E

C350 Reinforced Vegetation	Straight	33.52 cfs	10.46 ft/s	1.03 ft	0.025	10 lbs/ft2	5.16 lbs/ft2	1.94	STABLE	E
Underlying Substrate	Straight	33.52 cfs	10.46 ft/s	1.03 ft	--	1.2 lbs/ft2	3.886 lbs/ft2	0.31	UNSTABLE	--

S150

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
S150 Unvegetated	Straight	33.52 cfs	10.46 ft/s	1.03 ft	0.025	1.75 lbs/ft2	5.16 lbs/ft2	0.34	UNSTABLE	D

C125BN

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
C125BN Unvegetated	Straight	33.52 cfs	10.46 ft/s	1.03 ft	0.025	2.35 lbs/ft2	5.16 lbs/ft2	0.46	UNSTABLE	D

P550 - Class D - Bunch Type - Good 75-95%

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
P550 Unvegetated	Straight	33.52 cfs	10.46 ft/s	1.03 ft	0.025	3.25 lbs/ft2	5.16 lbs/ft2	0.63	UNSTABLE	E
P550 Reinforced Vegetation	Straight	33.52 cfs	10.46 ft/s	1.03 ft	0.025	12 lbs/ft2	5.16 lbs/ft2	2.33	STABLE	E
Underlying Substrate	Straight	33.52 cfs	10.46 ft/s	1.03 ft	--	3.25 lbs/ft2	3.977 lbs/ft2	0.82	UNSTABLE	--

SC150

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
SC150 Unvegetated	Straight	33.52 cfs	10.46 ft/s	1.03 ft	0.025	2 lbs/ft2	5.16 lbs/ft2	0.39	UNSTABLE	D

C125

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
C125 Unvegetated	Straight	33.52 cfs	10.46 ft/s	1.03 ft	0.025	2.25 lbs/ft2	5.16 lbs/ft2	0.44	UNSTABLE	D

Unreinforced Vegetation - Class D - Bunch Type - Good 75-95%

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
Unreinforced Vegetation	Straight	33.52 cfs	10.46 ft/s	1.03 ft	0.025	3.33 lbs/ft2	5.16 lbs/ft2	0.65	UNSTABLE	--
Underlying Substrate	Straight	33.52 cfs	10.46 ft/s	1.03 ft	--	0.07 lbs/ft2	1.005 lbs/ft2	0.07	UNSTABLE	--

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Version 5.0

Project Name: FAD
Project Number: 78135
Project Location: Oak Brook, Illinois
Channel Name: Swale B-1

Discharge	6.49
Peak Flow Period	24
Channel Slope	.11
Channel Bottom Width	0
Left Side Slope	3
Right Side Slope	3
Low Flow Liner	
Retardance Class	D
Vegtation Type	Bunch Type
Vegetation Density	Good 75-95%
Soil Type	Clay

DS75

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
DS75 Unvegetated	Straight	6.49 cfs	7.82 ft/s	0.53 ft	0.025	1.55 lbs/ft2	3.61 lbs/ft2	0.43	UNSTABLE	D

S75BN

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
S75BN Unvegetated	Straight	6.49 cfs	7.82 ft/s	0.53 ft	0.025	1.6 lbs/ft2	3.61 lbs/ft2	0.44	UNSTABLE	D

P300 - Class D - Bunch Type - Good 75-95%

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
P300 Unvegetated	Straight	6.49 cfs	7.82 ft/s	0.53 ft	0.025	2 lbs/ft2	3.61 lbs/ft2	0.55	UNSTABLE	E
P300 Reinforced Vegetation	Straight	6.49 cfs	7.82 ft/s	0.53 ft	0.025	8 lbs/ft2	3.61 lbs/ft2	2.21	STABLE	E
Underlying Substrate	Straight	6.49 cfs	7.82 ft/s	0.53 ft	--	2 lbs/ft2	3.292 lbs/ft2	0.61	UNSTABLE	--

ShoreMax - Class D - Bunch Type - Good 75-95%

Phase	Reach	Discharge	Velocity	Normal	Mannings	Permissible	Calculated	Safety	Remarks	Staple

				Depth	N	Shear Stress	Shear Stress	Factor		Pattern
ShoreMax w/ SC250 Unvegetated	Straight	6.49 cfs	7.82 ft/s	0.53 ft	0.025	7.5 lbs/ft2	3.61 lbs/ft2	2.08	STABLE	F
ShoreMax w/ SC250 Reinforced Vegetation	Straight	6.49 cfs	7.82 ft/s	0.53 ft	0.025	8 lbs/ft2	3.61 lbs/ft2	2.21	STABLE	F
Underlying Substrate	Straight	6.49 cfs	7.82 ft/s	0.53 ft	--	3.25 lbs/ft2	4.742 lbs/ft2	0.69	UNSTABLE	--

S75

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
S75 Unvegetated	Straight	6.49 cfs	7.82 ft/s	0.53 ft	0.025	1.55 lbs/ft2	3.61 lbs/ft2	0.43	UNSTABLE	D

S150BN

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
S150BN Unvegetated	Straight	6.49 cfs	7.82 ft/s	0.53 ft	0.025	1.85 lbs/ft2	3.61 lbs/ft2	0.51	UNSTABLE	D

SC250 - Class D - Bunch Type - Good 75-95%

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
SC250 Unvegetated	Straight	6.49 cfs	7.82 ft/s	0.53 ft	0.025	2.5 lbs/ft2	3.61 lbs/ft2	0.69	UNSTABLE	E
SC250 Reinforced Vegetation	Straight	6.49 cfs	7.82 ft/s	0.53 ft	0.025	8 lbs/ft2	3.61 lbs/ft2	2.21	STABLE	E
Underlying Substrate	Straight	6.49 cfs	7.82 ft/s	0.53 ft	--	0.8 lbs/ft2	4.507 lbs/ft2	0.18	UNSTABLE	--

Rock Riprap - Class D - Bunch Type - Good 75-95%

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
Rock Riprap Unvegetated	Straight	6.49 cfs	7.82 ft/s	0.53 ft	0.025	2 lbs/ft2	3.61 lbs/ft2	0.55	UNSTABLE	--

DS150

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
DS150 Unvegetated	Straight	6.49 cfs	7.82 ft/s	0.53 ft	0.025	1.75 lbs/ft2	3.61 lbs/ft2	0.48	UNSTABLE	D

SC150BN

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
SC150BN Unvegetated	Straight	6.49 cfs	7.82 ft/s	0.53 ft	0.025	2.1 lbs/ft2	3.61 lbs/ft2	0.58	UNSTABLE	D

C350 - Class D - Bunch Type - Good 75-95%

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
C350 Unvegetated	Straight	6.49 cfs	7.82 ft/s	0.53 ft	0.025	3 lbs/ft2	3.61 lbs/ft2	0.83	UNSTABLE	E

C350 Reinforced Vegetation	Straight	6.49 cfs	7.82 ft/s	0.53 ft	0.025	10 lbs/ft2	3.61 lbs/ft2	2.77	STABLE	E
Underlying Substrate	Straight	6.49 cfs	7.82 ft/s	0.53 ft	--	1.2 lbs/ft2	4.738 lbs/ft2	0.25	UNSTABLE	--

S150

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
S150 Unvegetated	Straight	6.49 cfs	7.82 ft/s	0.53 ft	0.025	1.75 lbs/ft2	3.61 lbs/ft2	0.48	UNSTABLE	D

C125BN

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
C125BN Unvegetated	Straight	6.49 cfs	7.82 ft/s	0.53 ft	0.025	2.35 lbs/ft2	3.61 lbs/ft2	0.65	UNSTABLE	D

P550 - Class D - Bunch Type - Good 75-95%

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
P550 Unvegetated	Straight	6.49 cfs	7.82 ft/s	0.53 ft	0.025	3.25 lbs/ft2	3.61 lbs/ft2	0.9	UNSTABLE	E
P550 Reinforced Vegetation	Straight	6.49 cfs	7.82 ft/s	0.53 ft	0.025	12 lbs/ft2	3.61 lbs/ft2	3.32	STABLE	E
Underlying Substrate	Straight	6.49 cfs	7.82 ft/s	0.53 ft	--	3.25 lbs/ft2	4.742 lbs/ft2	0.69	UNSTABLE	--

SC150

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
SC150 Unvegetated	Straight	6.49 cfs	7.82 ft/s	0.53 ft	0.025	2 lbs/ft2	3.61 lbs/ft2	0.55	UNSTABLE	D

C125

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
C125 Unvegetated	Straight	6.49 cfs	7.82 ft/s	0.53 ft	0.025	2.25 lbs/ft2	3.61 lbs/ft2	0.62	UNSTABLE	D

Unreinforced Vegetation - Class D - Bunch Type - Good 75-95%

Phase	Reach	Discharge	Velocity	Normal Depth	Mannings N	Permissible Shear Stress	Calculated Shear Stress	Safety Factor	Remarks	Staple Pattern
Unreinforced Vegetation	Straight	6.49 cfs	7.82 ft/s	0.53 ft	0.025	3.33 lbs/ft2	3.61 lbs/ft2	0.92	UNSTABLE	--
Underlying Substrate	Straight	6.49 cfs	7.82 ft/s	0.53 ft	--	0.07 lbs/ft2	0.703 lbs/ft2	0.1	UNSTABLE	--

Geosyntec consultants

Written by: JNJ **Date:** 10/15/2015
Reviewed by: RM **Date:** 11/10/2015
Client: AEP **Project:** FAR Closure **Project No.:** CHE8273 **Task No.:** 02/05.22

ATTACHMENT 8

Sediment Basin Stage vs. Storage

Geosyntec
consultants

engineers | scientists | innovators
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Oak Brook, IL
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Attachement 8: Sediment Basin Design

JOB CHE8273-FAR Closure
SHEET NO.
CALCULATED BY Jennifer Jenkins
CHECKED BY Rishab Mahajan
SCALE

Sediment Basin 1 (North)

Total Disturbed Area	**6.29**	Acres
Dead Load	6300	cubic feet
Live Load	11300	cubic feet
Total Required Volume	**17600**	cubic feet

Sediment Basin 2 (South)

Total Disturbed Area	9.22	Acres
Dead Load	9500	cubic feet
Live Load	17100	cubic feet
Total Required Volume	**26600**	cubic feet

	Contour	**Depth**	**Area**	**Depth incremet**	**inc vo**
	FT	**FT**	**Sq. Ft**	**FT**	**Cub**
Bottom of Basin	604	0	7110	0.2	(
	604.2	0.2	7355	0.2	14
	604.4	0.4	7603	0.2	14
	604.6	0.6	7854	0.2	15
	604.8	0.8	8107	0.2	15
Skimmer	605	1	8364	0.2	16
	605.2	1.2	8623	0.2	16
	605.4	1.4	8886	0.2	17
	605.6	1.6	9151	0.2	18
	605.8	1.8	9419	0.2	18
	606.2	2.2	9964	0.2	19
	606.4	2.4	10241	0.2	20
	606.6	2.6	10520	0.2	20
	606.8	2.8	10803	0.2	21
	607	3	11088	0.2	21
	607.2	3.2	11376	0.2	22
Normal Depth	607.4	3.4	11667	0.2	23
	607.6	3.6	11961	0.2	23
	607.8	3.8	12258	0.2	24
	608	4	12558	0.2	24
	608.2	4.2	12861	0.2	25
	608.4	4.4	13166	0.2	26
	608.8	4.8	13786	0.2	27

	609.4	5.4	14737	0.2	29
	609.6	5.6	15060	0.2	29
	609.8	5.8	15385	0.2	30
	610	6	15714	0.2	31
	610.2	6.2	16045	0.2	31
	610.4	6.4	16380	0.2	32
	610.6	6.6	16717	0.2	33
	610.8	6.8	17057	0.2	33
	611	7	17400	0.2	34
	611.2	7.2	17746	0.2	35
Emergency spillway	611.4	7.4	18095	0.2	35
	611.6	7.6	18446	0.2	36
	611.8	7.8	18801	0.2	37
Spillway	612	8	19158	0.2	37

6. DAM MODIFICATION DESIGN ANALYSIS

6.1 Hydrology and Hydraulics

6.1.1 FAR Final Cover Stormwater Management

6.1.1.1 Overview

This section summarizes the hydrologic and hydraulic analysis supporting the design of the FAR final cover stormwater management system. This component of the design focuses primarily on the FAR closure area west of (upstream of) the FAD. A later section discusses the hydrologic/hydraulic design of the dam spillway itself. This summary of the FAR final cover stormwater management is included because the hydrologic analysis conducted to develop design flows for the final cover channels was also used to develop design flows for the dam spillway.

The overall approach to the FAR final cover stormwater management is to convey flow through channels on top of the cover system that generally mirrors the original drainage ways and then over the FAD through a new spillway. The final cover system upstream of the dam was designed to convey the 100-year, 24-hour storm event. The slope of the drainage ways and cover system were designed to be the lowest reasonable slopes to reduce the amount of contouring fill necessary to provide positive drainage from the upstream ends of the cover drainage ways to the downstream FAD spillway. Energy dissipation will be provided at the bottom of the spillway.

The final cover stormwater management analysis consisted of two elements: i) a hydrologic analysis to determine peak flows at specific locations and ii) a hydraulic analysis of the proposed conveyance channels. The complete design analysis and supporting calculations are presented in **Appendix D**.

The existing FAR consists of three main valleys or branches (north, central, south) that converge into a main body adjacent to the dam. The proposed conveyance swales will be approximately aligned along the centerline of each of the original valleys, corresponding to the alignment of the natural valley. Each of the three branches will have a primary conveyance channel along its center. These three channels will converge and the combined flow will be conveyed within a single channel to the modified FAD proposed spillway.

The peak flows calculated through the hydrologic analysis have been used as the basis of design for the cover channels (conveyance and stabilization) and the proposed FAD spillway. The cover conveyance swales consist of a two stage channel design. The first stage is a low flow channel designed to convey the 2–year, 24-hour storm event, which was selected due to the frequency of this potential flow. The second stage was designed to convey the 100-year, 24-hour storm event to protect the adjacent cover system. The intent is to reduce flow across the cover cap by concentrating the flows within stabilized channels that will safely convey the 100-year design storm.

6.1.1.2 Hydrologic and Hydraulic Analysis

The computer program U.S. Army Corps of Engineers (USACE) Hydrologic Engineering Center-Hydrologic Modeling System V.3.5 (HEC-HMS) was used to perform the primary hydrologic analysis. The hydrologic analysis was performed using procedures described in the HEC-HMS support document and the "Urban Hydrology for Small Watersheds, Technical Release 55," (USDA-SCS, 1986). In addition to HEC-HMS, various spreadsheet and hand calculations were used to support the analysis. The HEC-HMS model was set up to include the primary sub-basins and main final cover conveyance swales.

The peak flows calculated through the hydrologic analysis were used as the basis of design for the conveyance channels on the cover system and the proposed spillway. The 2-year, 24-hour and the 100-year, 24-hour events were used as the design storms for the cover system. The Probable Maximum Precipitation (PMP) was used as the design storm for the spillway. A six-hour duration of the PMP was used to simulate the PMF, because the 6-hour duration/intensity generated a higher peak flow than the simulations of the 12- and 24-hour PMPs. Below is a summary of the design rainfall depths and peak discharges from the cover system at the proposed spillway.

Design Rainfall Depths and Peak Discharges

Storm event:	2-year, 24-hour	100-year, 24-hour	Probable Max. Flood
Rainfall depth:	2.62	5.33	19.1[2]
Peak discharge from final cover at proposed spillway	423 cfs	1,914 cfs	9,500 cfs

The results of the hydrologic analysis were used to design the surface conveyance channels across the proposed cover system. Based on the range of flows and the grading approach, three types of channels were used. Details of the final cover channel design, upstream of the dam, are presented in **Appendix D**.

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[2] The 6-hour PMP generates higher peak flow that was used to design the spillway.

6.1.2 Spillway Design and Energy Dissipater

6.1.2.1 Overview

Section 6.1.1 focuses on the stormwater management for the cover system extending to the proposed FAD spillway. This section picks up the stormwater design from this point, detailing the conveyance system downstream of the cover system from the proposed spillway to the confluence with the existing channel. The hydrologic calculations for the proposed condition are presented in **Appendix E.** The peak flows in the spillway during the 100-year, 24-hour storm event are approximately 1,960 cfs. The PMF has a magnitude of approximately 9,500 cfs. Conveying flow down the spillway and to Stingy Run Creek in a safe and efficient manner will require significant channel protection and energy dissipation.

This section focuses on the design of the portion of the conveyance system from the modified FAD to the existing Stingy Run Creek channel. Further details on the analysis are included in **Appendix E**.

6.1.2.2 Design Methodology

ODNR has indicated that the spillway through the crest of the dam and on the downstream face of the dam should be designed for the Probable Maximum Flood (PMF) to meet ODNR dam permit requirements. The PMF is an estimation of the theoretically largest flood that could conceivably occur in a given area, based on the estimated Probable Maximum Precipitation (PMP) for the drainage basin size and location. Therefore, the PMF flow was calculated by using the HEC-HMS watershed model previously developed for the final cover stormwater management plan to simulate the PMP. The HEC-HMS model generated a PMF flow of 9,120 cfs. To provide some design robustness and conservatism, a PMF flow of 9,500 cfs was used in the hydraulic design.

The spillway was first designed for the 100-year, 24-hour flood event. The design was later modified to include the additional capacity to convey the PMF. When considering design alternatives to expand the spillway to convey the PMF flow, it was decided to maintain the center rectangular concrete spillway design. This allows a continuation of previous preliminary design work, and the concrete lining will protect the channel against the very high velocities experienced in the channel center. To provide additional flood conveyance for the PMF, the overbank area on either side of the previously-designed concrete channel will be graded to construct another rectangular channel. The top of the channel was set to provide a minimum of 2.5 ft of freeboard elevation above the modeled peak water surface elevation during the PMF. Minimum regulatory criteria requires 2 ft of freeboard. The additional half-foot of freeboard provides a greater factor of safety and added design flexibility if modifications are required in the future.

The hydraulic design was performed by using the hydraulic modeling software HEC-RAS to simulate possible spillway design configurations to accommodate first the 100-year event and second the PMF, and collaborating with the design team to develop a spillway design that is hydraulically adequate and meets other design criteria. All of the 100-year flow is contained in the center concrete spillway. Calculated average velocities in the center concrete spillway range from 10 feet per second to 38 feet per second for the 100-year flow. The details of hydraulic analysis for the 100 year flow event are presented in **Appendix E1**.

Several options for surface material for the overbank PMF spillway were considered. Constructing the overbank areas from riprap would require a D-50 stone of approximately 2.5 ft, a very large riprap size. This would be very costly, for reasons such as obtaining and transporting a sufficient supply of rock in that size range. An all-concrete PMF spillway was considered but would require additional structural design; AEP directed the design team to consider fabric formed concrete. The use of fabric formed lined PMF flat bench was evaluated and determined to be acceptable. The details of hydraulic analysis for the PMF flow event are presented in **Appendix E2**. The PMF channel is a rectangle with fabric formed slab and perimeter reinforced concrete wall. From the edge of the concrete wall a vertical berm will be constructed of structural compacted fill. The berm will have a 4H:1V slope, be vegetated, and will provide a minimum freeboard of at least 2.5 ft above the maximum predicted water surface elevation of the PMF.

Energy dissipation for flows up to the 100-year flow is provided within and downstream of the spillway. Supercritical flow conditions within the spillway require the need for energy dissipation measures to be used at the base of the spillway to protect the downstream channel and surrounding area from significant erosion.

The energy dissipation design was performed in accordance with design guidance from the U.S. Bureau of Reclamation (USBR, 1987) and the U.S. Army Corps of Engineers (USACE, 1990). Based on the spillway flow having a calculated velocity of 38.6 ft/sec and a Froude number of 7 immediately upstream of the energy dissipater during the 100-year event, a stilling basin was designed in accordance with the Type III Basin characteristics recommended by the USBR (USBR, 1987). Drawing 4 of **Appendix A** shows a plan view of the basin design. This type of energy dissipater is known as a "stilling basin". Dimensions and design parameters from the final design are described in the calculation package presented in **Appendix E**.

The channel downstream of the energy dissipater is designed for the 100-year peak flow. Downstream channel design was first performed based on topographical conditions and necessary hydraulic capacity. A trapezoidal channel design with 3H:1V side slopes and width of 25 ft was assumed. Scour protection was analyzed in accordance with the U.S. Department of Transportation Federal Highway Administration's (FHA) guidance set forth in Hydraulic

Engineering Circular No. 11 (FHA, 1989), as well as a more recent article published by Craig Fischenich at the Ecosystem Management & Restoration Research Program (Fischenich, 2001). Both of these specifications were in agreement on suggesting channel stability measures based on flow velocities and scour potential. Details of this design can be found in Hydraulic Analysis Results below. See **Appendix E1** for excerpts from the Hydraulic Engineering Circular No. 11 paper that was used for the selection of rip-rap armament.

6.1.2.3 Hydraulic Analysis Results

A detailed summary of the results of the hydraulic analysis using HEC-RAS are presented in **Appendix E**. The spillway conveyance system is comprised of several distinct components, summarized below. Each of these components was included in HEC-RAS hydraulic models.

FAD Transition: The FAD transition area will start at the face of the dam and extend approximately 200 ft downstream. The channel will have a 35 ft bottom width and side walls that slope at a 6:1 (horizontal: vertical) grade. The intent of the FAD interim geometry is to transition the FAR Type I conveyance channels to the rectangular spillway chute.

Spillway Chute: The spillway chute will extend for a length of 550 ft at a 13% slope down the face of the dam. The spillway will consist of a rectangular, concrete channel with a bottom width of 55 ft. The sides of the channel will extend to a height of 5 ft to convey the 100 year flow while providing a 2.5 ft minimum freeboard throughout the extent of the spillway.

Spillway Transition: From the end of the rectangular spillway chute, the entry transition to the stilling basin will consist of a concrete channel which follows the recommended USBR curvature until it transitions to a 4H:1V slope. This transition will be approximately 36.6 ft in length, extending from Elevation 586.5 ft to 577.6 ft. The transition will have a base width of 55 ft with vertical side walls which will be constructed at a steady Elevation 591.5 ft to provide adequate freeboard (minimum of 2 ft) for the stilling basin. Chute blocks measuring 1.0 ft high by 1 ft wide will be equally spaced across the bottom of the transition channel, where the transition meets the stilling basin. A total of 22 chute blocks will be constructed along the base of the spillway transition. See **Appendix E1**, Attachment B for schematic showing a general design of the chute blocks.

Stilling Basin: The stilling basin was designed in accordance with the Type III Basin characteristics recommended by USBR (USBR, 1987). See **Appendix E1**, Attachment B for a schematic of the stilling basin design parameters. The concrete basin will be 55 ft wide and will extend downstream from the spillway with a flat bottom. The hydraulic jump required out of the spillway was calculated using the USBR methods at approximately 8.9 ft. The floor of the stilling basin will be installed at an Elevation of 577.6 ft giving the basin a total depth of approximately 13.9 ft including 5 ft of freeboard.

Baffle blocks will be constructed 7.1 ft from the beginning of the stilling basin. The baffle blocks will be 2.0 ft wide and 1.7 ft high with a vertical face. The tops of the blocks will be

0.3 ft wide. The downstream side of each baffle will be sloped 1H:1V. A total of 14 blocks will be constructed in place and will be equally spaced across the stilling basin.

At the downstream end of the basin, a sill will be constructed. The sill will be sloped upward at 2H:1V and the bottom of the sill will begin 11.25 ft from the beginning of the stilling basin, and will slope upwards for a vertical height of 5 ft to an Elevation of 582.6 ft. The sill height was a parameter that was adjusted through trial and error to achieve appropriate tailwater depth as well as allow the system to have an adequate channel slope downstream of the dissipater. The total length of the basin will be 23 ft from the entry transition to the end of the sill. The downstream side of the sill will drop vertically 1 ft to the basin exit transition channel at Elevation 581.6 ft to allow for an adequate downstream channel slope to tie into the natural channel while maintaining the necessary tailwater depth. A summary of design parameters for the stilling basin is provided in Appendix E1, Table 1B.

<u>Basin Exit Transition</u>: From the base of the sill at an Elevation of 581.6 ft the conveyance system will be graded at a steady slope of 0.5% until the channel reaches the new culvert system at Stingy Creek Road (Outfall 001). Exiting the stilling basin, a 50 ft long concrete transition channel will be built to transition the conveyance system from the stilling basin to the reinforced channel. The channel bottom will be 55 ft wide at the base of the stilling basin exit and will taper to a 25 ft wide bottom approximately 50 ft downstream of dissipater to an Elevation of 581.3 ft. The side slopes will also taper over this transition from vertical side walls at the stilling basin to 3H:1V sloped walls 50 ft downstream. The depth of the channel will be determined by the grading in the surrounding area for the PMF spillway exit transition, but at no point should the minimum channel depth within the transition be less than 9 ft. A summary of design parameters for the dissipater basin exit transition is provided in **Appendix E1**, Table 1C.

<u>Reinforced Channel</u>: The basin exit transition will tie into a trapezoidal channel constructed to be consistent with the bed slope to Stingy Creek Road and the new culvert system. The invert culvert elevation (upstream side) at this point will be approximately 579.6 ft based on current topographic data. The channel will have a 25 ft wide base with 3H:1V side slopes. The depth of the channel will be dependent on the surrounding grading, but at no point will the depth be less than 6 ft. The channel will be reinforced with hard armoring for its entire length to prevent scour and erosion during the 100 year event. Rip-rap will be used to armor the entire bottom of the channel as well as the sidewalls to a depth of three ft. Above a depth of 3 ft, armoring of the channel is not necessary. Rip-rap sizing was based on FHA Hydraulic Engineering Circular No. 11 (FHA, 1989) using shear stresses and velocities provided by the HEC-RAS model. Ohio Type C Rip Rap with a D_{50} of 1 ft will be used to armor the downstream channel. Appendix E1, Table 1D presents the design parameters for the reinforced channel.

<u>New Culvert System at Outfall 001</u>: Currently, twin 48" corrugated metal culverts are installed at the Stingy Creek Road crossing (Outfall 001); however, based on the HEC-RAS model results, the existing culverts do not have the capacity to pass the 2 year peak event. Therefore, the existing culverts will be replaced with three 60" diameter culverts. Culverts will be ADS N-12 or approved equivalent, which has a smooth interior with corrugated outer

shell. The new culverts can sufficiently handle the 2 year storm event. The upstream invert of the culverts will be at an elevation 579.6 feet and the downstream invert will be at an elevation 578.8 feet.

6.1.2.4 Management of Probable Maximum Flood (PMF) During Construction/Closure

The permanent, post-closure dam crest and spillway will accommodate the peak discharges from the Probable Maximum Flood (PMF). Per ODNR guidance, the PMF must also be managed during construction until the final spillway is constructed. The management approach that will be used is to contain the full PMF runoff volume upstream of the dam without overtopping the dam prior to construction of the final spillway.

An analysis was performed to compare the volumes of stormwater runoff generated during the PMF for the series of construction phases to the storage volume available upstream of the dam during that construction phase.

The estimation of stormwater runoff volume for the PMF was based on rainfall depths from PMP data. At the time the analysis was performed, a PMP depth of 34.1 inches[3] was used. This depth is the all-season, 24-hour duration PMP for the project site from NOAA's Hydrometeorological Report 51, Probable Maximum Precipitation Estimates, United States East of the 105th Meridian. Using this rainfall depth, the Soil Conservation Service curve number procedure (from Urban Hydrology for Small Watersheds, Technical Release 55, 1986) was used to calculate total runoff depth. The runoff volume was computed by multiplying the runoff depth by the drainage area for the construction phase. A runoff volume of approximately 2,000 acre-feet was calculated.

Based on the hydraulic analysis considering the final FAR cover grades, the 100-year and PMF flow water levels are expected to be at Elevations 664.7 ft and 669.5 ft, respectively, by the time flow enters into the FAD spillway. During lowering of the FAD, the crest of the dam will be lowered to Elevation 670 ft or above until the upstream end of spillway is ready to be constructed. Prior to lowering the FAD crest, the minimum elevation that the dam crest will be lowered to will be analyzed based on the existing grades within FAR to provide adequate freeboard for PMF flow. Drawing 18 of **Appendix A** provided the general construction sequence of the spillway and energy dissipater.

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[3] The 24-hour PMP was used to calculate the maximum runoff to be contained or managed behind the FAD; for the spillway design, the 6-hour PMP was used because it generates a higher peak discharge.

6.1.2.5 Summary and Conclusions

The FAD spillway was designed to handle flows from the Probable Maximum Flood (PMF). A conveyance system downstream of the Stingy Run FAD spillway was designed to handle flows from the 100–yr, 24-hour storm event. To convey these flows from the base of the spillway to Stingy Run Creek in a safe and efficient manner, a stilling basin and rip rap lined channel were designed using HEC-RAS. The stilling basin was designed in accordance with U.S. Bureau of Reclamation standards (USBR, 1987), complete with chute blocks at the entrance, baffle blocks, and a 5 ft exit sill to create a hydraulic jump within the basin. The downstream conveyance channel was designed with hard armoring to protect the channel from scour and erosion. Additionally, a new culvert system was designed at the first creek crossing of Stingy Creek Road to effectively convey the 2–yr, 24-hour storm event under the road.

6.1.3 Local Drainage and Stormwater Management Design – Post Construction

The "local drainage" management system consists of engineered elements that convey and manage runoff from the face of the dam, downstream of the crest, leading to Stingy Run Creek and excluding the main spillway. The local drainage features include two drainage swales and five culverts that convey local runoff from the downstream face of the proposed dam.

The basis of design for the two swales is to effectively convey the 100-yr, 24 hour storm event. The channels have been designed to convey the peak flow and resist scour for the design storm. The basis of design for the culverts is to have sufficient capacity to convey the 25-year, 24-hour storm event. The system has been designed to safely pass the 100-year, 24-hour event. Under these conditions, the roadways that cross the culverts will overtop; however, they will allow for the safe passage of the 100-year design storm. The Local Drainage and Stormwater Management Final and Interim Conditions Design Calculations are presented in **Appendix F.**

The analysis was performed using procedures described in the document "Urban Hydrology for Small Watersheds, Technical Release 55", (USDA-SCS, 1986). The computer program HEC-HMS was used to perform the hydrologic analysis. The computer program HY-8 was used to perform the hydraulic analysis of the culverts. Microsoft Excel ® spreadsheets were developed to perform several of the other supporting calculations, such as time of concentration, and swale conveyance capacity. Local drainage figures and design calculations are presented in **Appendix F.**

ATTACHMENT F

MAINTENANCE PLAN

7-DAY INSPECTION CHECKLIST
GAVIN PLANT
FLY ASH POND

FORM ID: GV-FAP-7DayIns
FILE NAME: GV-FAP-7DayIns-**mmddyy**.pdf
REVISION: 0 ISSUE DATE: 10/14/2015

Date:	
Weather & Temperature:	
Inspector(s):	
Pool Level:	
Rainfall:	Last 7 days: inch, Last 24 hours: inch

Are the following components of this dam in satisfactory condition, Yes or No?

ITEM	YES	NO	REMARKS
EMBANKMENT			
1. Crest (e.g. settlement, cracks, misalignment, rutting, potholes)	☐	☐	
2. Upstream Slope (e.g. vegetation, rodent holes, sloughing, erosion, seepage, riprap)	☐	☐	
3. Downstream Slope (e.g. vegetation, rodent holes, sloughing, erosion, seepage, riprap)	☐	☐	
4. Groins/Abutments	☐	☐	
OVERFLOW STRUCTURE			
5. Visible concrete, skimmer structure, no obstruction to flow	☐	☐	
6. Access and walkway	☐	☐	
APPURTENANCES			
7. Seepage Outlet ditch and weirs	☐	☐	
8. Downstream channel/pipe culverts	☐	☐	
9. pH treatment system	☐	☐	

Were any of the potential deficiencies discovered, Yes or No?

ITEM	YES	NO	MONITORING?	LOCATION
10. Seepage	☐	☐	☐	
11. Muddy Seepage	☐	☐	☐	
12. Embankment Cracks	☐	☐	☐	
13. Settlement	☐	☐	☐	
14. Sloughing / Slides	☐	☐	☐	
15. Ruts	☐	☐	☐	
16. Animal Activity	☐	☐	☐	
17. Excessive Debris at Outlets	☐	☐	☐	
18. Excessive Vegetation	☐	☐	☐	
19. Other__________	☐	☐	☐	

Comments on any deficiency found:

Contact AEP Geotechnical Engineering if any unsatisfactory conditions or deficiencies are discovered.

INSPECTOR'S SIGNATURE: ____

NOTE: The Master Record is controlled in accordance with "MESHCORP 4.4.5 Control of Documents and Records", is available at http:\\ MESH site and supersedes all paper copies.