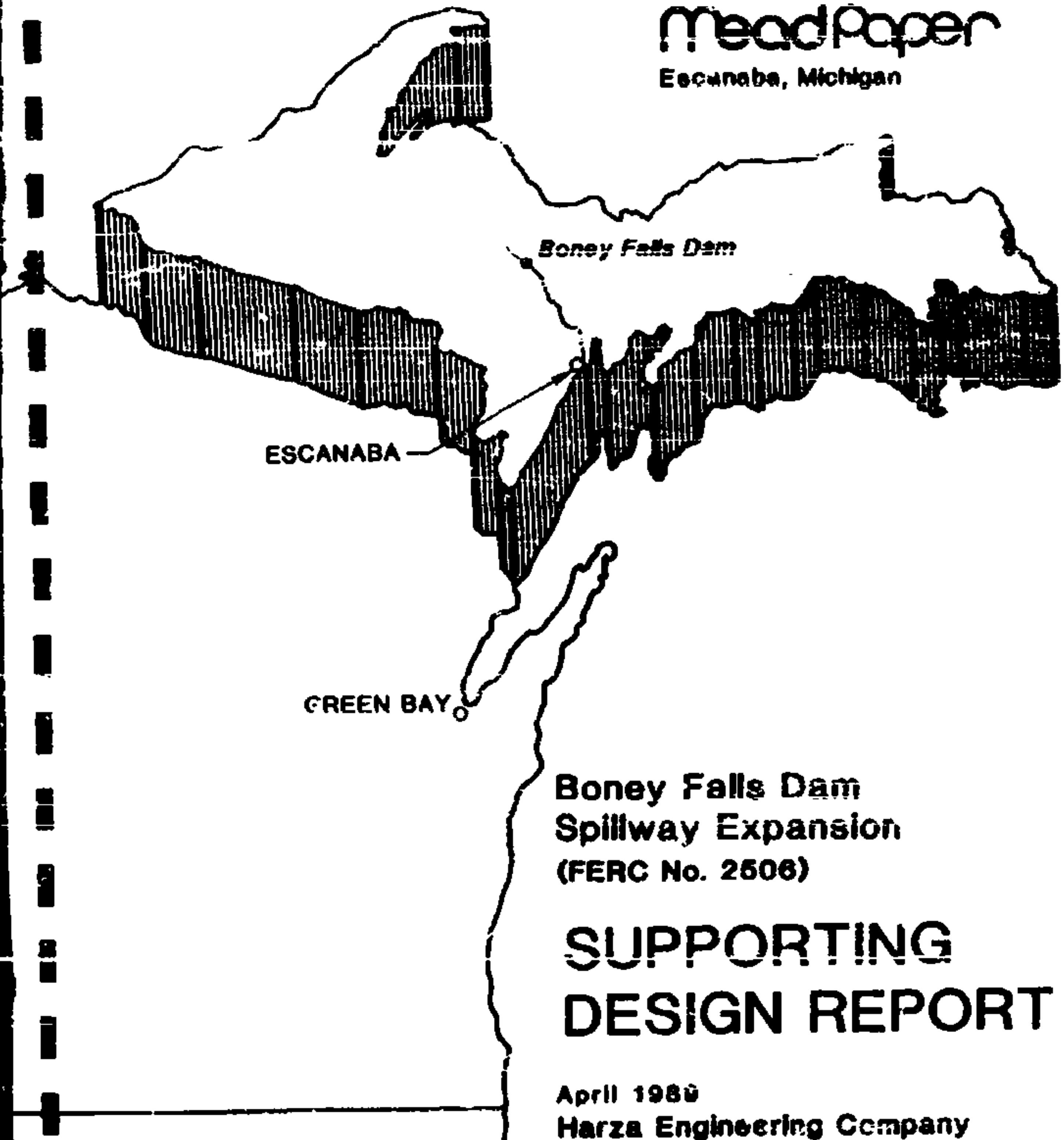

MeadPaper

Escanaba, Michigan

Boney Falls Dam
Spillway Expansion
(FERC No. 2506)

SUPPORTING DESIGN REPORT

April 1989

Harza Engineering Company

P2044

SUPPORTING DESIGN REPORT

BONEY FALLS DAM SPILLWAY EXPANSION
Escanaba River
Escanaba, Michigan

for

MEAD CORPORATION
PUBLISHING PAPER DIVISION
Escanaba, Michigan

April 1989

Prepared by

HARZA ENGINEERING COMPANY
150 South Wacker Drive
Chicago, IL 60606

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INTRODUCTION

General

This General Design Report summarizes the procedures, criteria, and results of analyses used for the design of the Boney Falls Dam Rehabilitation Project. The project has been extensively studied by Harza Engineering Company since 1987. The spillway was cited for apparent spillway capacity inadequacy in the 1986, Second FERC Part 12 Independent Consultant's Safety Report (by others). Since that time Harza has been retained to develop alternatives for spillway capacity expansion [1], evaluate underseepage [2], evaluate the stability of the left embankment [3], evaluate reservoir performance after a scheduled maintenance drawdown [4], develop rationale for new project inflow design flood [5,6], evaluate the stability of the right earth embankment and the integrity of the limestone foundation [7], and prepare the 1989 Third FERC Part 12 Inspection [8,9]. An extensive rehabilitation program has been developed as a result of these studies. Planned activities include: (1) raising the west embankment dam, (2) raising the concrete core wall in the west dam to above maximum IDF pool level, (3) construction of a new roller compacted concrete overflow spillway through a portion of the left earth dam, and (4) construction of a new walkway across the crest of the existing uncontrolled overflow spillway. This report contains a general description of the project, and detailed discussions of the design of the proposed new works.

Project Location

Boney Falls Dam is the largest of four dams owned by Mead Paper on the Escanaba River near Escanaba, Michigan as shown on Exhibit 1. The project is located in the upper peninsula of the State of Michigan in Delta County, Section 2, T41N, R25W. The dam was constructed in 1920-1921 over rapids known locally as Boney Falls about 22 miles from the mouth of the Escanaba River at Lake Michigan. The other three Mead Paper dams on the river are downstream of the Boney Falls Dam.

Description of Existing Facilities

The general layout of the project is shown on Exhibit 2. Aerial photographs of the project site are attached (Photographs Nos. 1 and 2). Beginning on the west (right looking downstream) bank of the river, the components of the project include the west earth embankment dam, non-overflow concrete dam, powerhouse, log sluice and fishway, six tainter gate spillway bays, uncontrolled overflow spillway, and east earth embankment dam. All structures are founded on limestone.

<u>Reservoir.</u> The reservoir normal pool level is El. 906.58 (El. 98 local datum). At normal pool the dam impounds 1,700 acre-feet of water. Normal pool reservoir surface area is about 170 acres with an average depth of 10 feet.

<u>West Embankment Dam.</u> The west embankment dam is approximately 1,500 feet long. The earth dam is a low structure which extends parallel to the river on the margin of the reservoir as a freeboard dike for most of its length as shown on Exhibit 3. The highest portion of the structure abuts the non-overflow gravity dam section and extends perpendicular to the river about 880 feet. This section has a maximum height of about 30 feet. The dam contains a concrete corewall to control seepage. The corewall steps down with distance from the non-overflow gravity dam. The design elevation of the corewall at the non-overflow gravity dam is El. 908.08. The wall steps down 5 feet for every 50 feet laterally to a minimum elevation of El. 893.58 as shown on Exhibit 4.

The westerly half of the west embankment dam failed in 1930 and was subsequently reconstructed. The details of this failure are presented in the IDF Addendum Report [6]. The reconstructed portion has a buried pipe drain at the toe for collection of seepage flows. This pipe discharges through a Parshall Flume and into Barney's Creek downstream of the dam. The easterly half of the west embankment dam has a stone masonry lined ditch at the toe for collecting seepage. This ditch drains directly into the river downstream of the powerhouse.

The west embankment crest width is approximately 15 feet. The upstream slope of the embankment dam is approximately 2 horizontal to 1 vertical; the downstream slope is approximately 3 horizontal to 1 vertical. The high portion of the dam has approximately 6 feet of freeboard above normal pool. The low dike portion has approximately 3 feet of freeboard above normal pool. The upstream slopes are protected from erosion by riprap and vegetation. The downstream slope is covered with field grass.

<u>Non-Overflow Gravity Dam.</u> The non-overflow mass concrete gravity dam is approximately 93 feet long and 40 feet high with the crest at El. 912.58. The structure transitions towards the west embankment dam from a mass concrete non-overflow structure with no downstream fill to a slender concrete corewall with fill on both the upstream and downstream sides as shown on Exhibit 5. There are no foundation drains beneath this section.

Powerhouse. The powerhouse is of integral intake design as shown on Exhibit 6. Dual intakes for each unit are provided as shown on Exhibit 7. Individual gates are provided for each unit. Trashracks are provided. The brick and concrete powerhouse superstructure is approximately 70 feet long and 70 feet wide. It contains an operator's office, controls, and three turbine generator units with a total capacity of 4,400 kW (2 at 1,700 kW and 1 at 1,000 kW). The powerhouse substructure is mass concrete with a formed scroll case and draft tube. The tailrace is excavated in bedrock. There are no exposed steel penstocks. Foundation drains are provided.

Log Sluice and Fishway. A log sluice and fishway are integral with the wall extending downstream on the left side of the powerhouse as shown on Exhibit 8. This structure allows logs to be passed through the dam, though the fishladder is non-functional. In 1988 the fishladder and log sluice retaining walls were removed and the remaining structure was capped with concrete. The wall that remains is approximately 7 feet wide by 75 feet long and 11 feet high at the low downstream end.

Mud Gates. This structure also contains two low-level mud gates, each 5 feet by 5 feet in area. These gates are operational. The gates are steel plates with the sill at El. 870.08 on the upstream side. One gate is fitted with a motor for gate operation; the other gate is fitted so that it may be operated with an electric hand drill. Both gates have backup hand cranks available.

Gated Spillway. The gated spillway section has six bays with steel tainter gates, concrete piers, concrete framed operator's bridge with steel grating deck and mass concrete spillway as shown on Exhibit 9. Each bay has a gate 20 feet wide and 12.5 feet high. The operator's deck is at El. 914.58. The mass concrete spillway has a base width of approximately 52 feet. Height from bedrock to the gate sill at El. 893.58 is approximately 33.5 feet. The overall height from the foundation to the operator's deck is about 54.5 feet. The downstream face of the spillway has an ogee shape. Discharge capacity of the gated spillway is 28,200 cfs with water at the crest of the east embankment dam (El. 909.08). There is a compressor on the operating deck that supplies the bubbler system which extends along the upstream side of all concrete structures. A single traveling electric chain hoist on rails is provided for tainter gate operation. The gates can also be raised manually if needed.

Uncontrolled Overflow Spillway. The uncontrolled overflow spillway is a concrete gravity structure with flashboards as shown in Exhibit 10. The crest of the spillway is at El. 905.58 with 1 foot high flashboards. The spillway is approximately 200 feet long and has a maximum height of about 40 feet. The east end of the spillway terminates at a concrete training wall abutment adjacent to the east embankment dam as shown on Exhibit 11. The west end of the spillway terminates at the tainter gated spillway structure. Discharge capacity of the uncontrolled spillway is 5,200 cfs with water at the crest of the east embankment dam (El. 909.08). No pedestrian access bridge over the spillway is provided.

East Embankment Dam. The east embankment dam is approximately 2,100 feet long. The earth dam is mostly a low structure extending parallel to the river upstream as a freeboard dike for most of its length as shown on Exhibit 12. The highest portion of the structure abuts the uncontrolled overflow spillway section and extends perpendicular to the river about 440 feet. This section has a maximum height of about 25 feet. The dam contains a concrete corewall to control seepage. The corewall steps down with distance away from the overflow spillway training wall. The design elevation of the corewall at the training wall is El. 908.08. The wall steps down 5 feet for every 50 feet laterally to a minimum elevation of El. 893.58 as shown on Exhibit 12. A filtered pipe drain is provided for seepage collection and measurement from about station 0+00L to 2+17L.

The east embankment crest width is approximately 10 feet. The upstream slope of the embankment dam is approximately 2 horizontal to 1 vertical; the downstream slope is approximately 3 horizontal to 1 vertical. The crest is at about the same elevation for the entire length of the embankment, El. 909.08, which gives about 3 feet of freeboard above normal pool. The upstream slopes are protected from erosion by riprap and vegetation. The downstream slope is covered with field grass.

Historic Failure

On 25 June 1930, a portion of the west embankment dam failed washing away approximately 400 feet of the dam at its extreme western end from about station 4+00R to 8+00R. The failure occurred at about 2:30 a.m. after several days of heavy rains. Historical information collected on the failure was presented in the IDF Addendum No. 1 Report [6] which used the historical failure to calibrate the DAMBRK model. It was concluded by the investigating engineers that the failure was caused primarily because the corewall stopped abruptly at this point instead of being extended until the rock rises above El. 893.58.

The top of the corewall in the area of failure was about 15 feet below normal pool. When the embankment was rebuilt, the corewall was raised to about El. 909 in the section that failed.

Based on field investigations, it seems that the corewall in that section of the dike which did not washout was not raised. This area of the embankment with a low corewall is also the area where a wet spot has been noticed on the downstream face of the dam.

Local Geology

Overburden at the site is a glacial till which generally consists of sandy silt (ML) to silty-sand (SM) material. These materials are slightly plastic and contain a trace amount of gravel. Overburden thickness ranges from about two feet adjacent to the river to a maximum of about 15 feet thick on the upland areas beneath portions of the low earth embankment dams.

The present Escanaba River channel is incised about [illegible] feet into the flat lying bedrock surface found in the area as shown in Photographs 14 and 15. Bedding at the site is sub-horizontal, dipping very gently to the southeast. Three near-vertical joint sets have been identified with orientations of 340 deg., 085 deg. and, to a lesser extent, 015 deg.

The dam was constructed at the waterfalls formed by the headward erosion that caused the formation of the downstream incised channel. Bedrock at the dam consists primarily of a light to medium gray limestone with very thin dark gray shale interbeds and/or partings. Locally bedrock is slightly to moderately weathered to a depth of up to 15 feet in the project area. Karstic features have been observed. Thin vuggy layers and occasional vugs and voids up to 2 inches in diameter were encountered in recently completed borings.

Solution enlarged joints are visible on the eastern side of the reservoir bottom during periods of low reservoir pool. These features were also observed in an inspection trench constructed in 1987 at the toe of the east embankment dam. Numerous joints exposed in the reservoir bottom have been enlarged up to 10 inches in width. Solution enlarged fractures up to 4 inches in width have been observed downstream of the east embankment dam. Some of these joints are clay filled.

Foundation Condition

The limestone foundation of Boney Falls Dam is susceptible to solutioning and development of karstic features as described above. Several sinkholes have been located in the reservoir near the far East and West sides of the reservoir. Geophysical and geochemical evaluations have concluded that water from the reservoir travels from the sinkholes in the reservoir through slightly enlarged solution cavities in the dam foundation to areas downstream of the dam. There are about 9 major springs located approximately 2000 feet downstream of the east embankment dam. Seepage from these springs is monitored on a regular basis by Mead Paper. Flows vary with reservoir head. There is no flow when reservoir pool levels fall below about El. 897. Total seepage at normal pool reservoir level is about 5,000 gpm. Flows are clear.

The springs downstream of the east embankment dam were first reported to the FERC in 1985. There is no mention of the springs in prior Part 12 Reports, but there is evidence from neighboring property owners that the springs have been flowing for many years. Reservoir level has been held at least 3 feet below historic normal pool since 1987.

FIELD INVESTIGATIONS

General

A series of field investigations have been undertaken by Harza since 1987 that forms the basis for a large portion of the planned rehabilitation work. These studies are summarized below.

Geophysical Investigation

A geophysical foundation investigation was conducted in 1987. Results were presented in a report by Weston Geophysical Inc., submitted to the FERC in October 1987 [2]. The purpose of the investigation was to determine the integrity of the embankments and foundation in view of major underseepage discovered in 1987. A summary of results and conclusions from the Geophysical Investigations Report are attached as Appendix A.

Geotechnical Investigation

A geotechnical exploratory drilling program was conducted in 1988. The purpose of the program was to sample embankment and foundation materials, confirm the presence of features noted in the 1987 geophysical program, determine in-situ uplift pressures under project structures, determine core wall presence and location within the embankment dams, evaluate phreatic conditions in the embankment dams, and develop shear strength parameters for use in stability analyses.

The description of the Geotechnical Investigations Program and results are presented in Harza's Geotechnical Investigations Report [7]. A summary of results and conclusions from the Geotechnical Report are attached as Appendix B.

FERC Part 12 Safety Inspection.

The Third FERC Part 12 5-Year Inspection and Safety Report was prepared by Harza Engineering Company (Mr. David R. Baier, Independent Consultant). The Part 12 Report [8] and Stability Analysis Supplement [9] were submitted to the FERC in January, 1989. A summary of Mr. Baier's inspection and recommendations are attached as Appendix C.

HYDRAULIC DESIGN

General

This section describes the hydraulic design criteria and computations used to design the proposed new spillway. All supporting hydraulic design rating curves for existing and proposed conditions are attached in Appendix D.

Floods of Record

The drainage area at Boney Falls Dam is 770 sq. miles. The peak discharge of record is 10,700 cfs on 22 April 1985. According to the 1986 Part 12 Report, the 100-year flood peak discharge is about 13,000 cfs.

Existing Spillway Rating Curve

The rating curves for the project gated and ungated spillways are attached in Appendix D of this report. Prior to rehabilitation, the maximum discharge capacity with the reservoir pool at the crest of the earth embankment dam (El. 909.08) is about 33,000 cfs.

Probable Maximum Flood

An estimate of the Probable Maximum Flood (PMF) was presented in the 1986 Part 12 Report. The conventional hydrometeorological approach, using probable maximum precipitation (PMP) data from the National Weather Service Hydrometeorlogical Report (HMR) No. 51, was used. Peak PMF discharge was estimated at 147,000 cfs.

Inflow Design Flood

Flood surcharge storage in the reservoir is negligible compared to the PMF volume. There is little reservoir peak attenuation during major floods. The PMF peak is several times greater than the capacity of the existing spillway. If the PMF were to occur, the project's earth embankments would be overtopped and would likely fail and release water from storage in the reservoir. If the embankments were to survive overtopping, maximum reservoir stage would overtop the existing east embankment dam by about 6 feet.

A detailed IDF study was undertaken by Harza in 1988. The work was based on earlier studies in support of the 1986 Part 12 Report. New Harza studies were prepared using the 1984 version of the DAMBRK computer program. The model was calibrated using observations from the 1930 failure of the west embankment dam. River cross sections and first floor elevations of several downstream structures were surveyed specifically for use in the IDF study.

On the basis of the studies presented in Harza's 1988 studies, the appropriate IDF for the Boney Falls Project is 100,000 cfs. This IDF has been accepted by the FERC in a letter dated March 15, 1989.

Spillway Expansion Plan

A variety of spillway capacity expansion alternatives have been studied by Harza for the Boney Falls Project [1]. The design scheme adopted calls for the construction of a Roller Compacted Concrete (RCC) emergency overflow spillway in place of the existing east embankment dam. The following project modifications are planned so that the project can safely accommodate the IDF. IDF maximum pool will be at El. 912.58. Several lengths of existing embankment will be raised to prevent overtopping during th IDF.

East Embankment Dam. An uncontrolled, broad-crest RCC spillway will be constructed from station 0+00L to 5+00L. The crest of the spillway will be 10 feet wide at El. 905.58. The upstream face of the RCC will be vertical using the existing embankment core wall as a form. The downstream slope will be 0.5H:1V and will be formed in a stair-step appearance to provide for energy dissipation of overflow.

The RCC will be placed using the existing concrete core wall as the upstream form. The core wall is also expected to perform as the impervious barrier to protect against seepage through horizontal RCC lift joints. The existing core wall will be lowered and raised where needed to a constant elevation of 905.58. The core wall will also be extended from about station 3+50L to 5+00L.

A small earthfill fuse plug will be placed on the crest of the new RCC structure to El.909.58 and on the downstream slope of the RCC spillway so that the spillway will only operate during severe floods. The earthfill will also protect the RCC material from weathering; including freeze-thaw damage. Five "pilot channels" will be provided in the fuse plug to initiate erosion when required. The channel will be 6-inches lower than the earthfill crest. Each channel will be 10 feet wide, spaced every 100-feet starting at station 0+50L. Pea gravel near the downstream face will increase the time for erosion during overtopping.

A gravel chimney drain will be placed against the downstream slope of the RCC. This drain is needed for collection of any seepage along the crest since the RCC will be about one foot lower than normal pool level. This drain will also collect seepage, if any, from the horizontal RCC lift joints.

The portion of the existing east embankment from station 5+00L to 16+65L will be raised to a uniform elevation of 911.08. This is needed to assure that the RCC spillway portion of the east embankment is overtopped first.

The portion of the existing east embankment beyond station 16+65 will not be modified since this section of the east embankment is less than 4-feet high and is heavily forested. The section will be overtopped by about 3.5 feet during the IDF.

An area downstream of the east embankment from station 0+00L to 16+65L will be cleared and grubbed of large trees and shrubs to channel overtopping flow back to the Escanaba River.

West Embankment Dam Core Wall. As previously reported by Harza [6,7,8], there is a portion of the west embankment where the existing core wall is significantly below normal pool level. The west embankment core wall will be raised to the peak IDF reservoir level, El. 912.58, from the gravity dam section, station 2+00R, to about station 8+00R.

West Embankment Crest. The crest of the west embankment dam will be raised to a uniform elevation of 913.58. This raising will prevent overtopping, with one foot of freeboard, during the IDF.

CIVIL DESIGN

General

Civil/structural design criteria for the proposed new works are described in this section.

Access Roads

There is an existing service road that provides access to all proposed construction areas of the west embankment dam. It will be possible for the rehabilitation contractor to get equipment onto the west dam crest with minimal effort.

There is no developed access road on the east bank of the river. There is, however, an unimproved service road to the east embankment dam that passes through the lands of an adjacent farmer. Mead Paper is currently negotiating with the farmer for access rights during construction of the proposed new spillway.

Mead will construct a new access road on it's property along with clearing of trees downstream of the east embankment dam. The area of clearing is shown on the civil design drawings 1979L-C8.

Cofferdams/Diversion of Water

No cofferdams will be needed at any time during construction. Mead has received a permit from the Michigan State DNR to lower the reservoir to the crest of the gated spillway, El. 893.58. This will be sufficient to dewater the necessary construction areas.

East Embankment RCC Spillway

The following hydraulic conditions were used as design criteria for the new RCC spillway.

Crest Elevation:	905.58
Crest Length:	500.00 ft (Sta. 0+00L to 5+00L)
Discharge Coefficient:	2.67 (Broad-Crested Weir)
Crest of Earthfill Cover:	909.58
Invert of Pilot Channel:	909.08
Lowest Base Elevation:	880.00±
Highest Section:	25.58 ft

Normal Headwater Elevation:	906.58
Normal Tailwater Elevation:	857.58
IDF Headwater Elevation:	912.58
IDF Tailwater Elevation:	894.58

Discharge rating curves for the project structures (including the RCC spillway) and a tailwater rating curve are attached as Appendix D to this report.

Stability analyses of the maximum RCC spillway section are attached as Appendix E to this report. The new spillway structure was proportioned in accordance with FERC criteria. The stability of all other project structures was presented to the FERC in the supplement to the 1989 Part 12 Report.

The design computations to determine the length of the new RCC spillway are attached as Appendix F.

East Embankment Crest Raising

The east embankment dam from the end of the RCC spillway section, station 5+00 to station 16+65 is being raised to El. 911.08 to prevent overtopping until the earthfill over the RCC spillway is washed out. It is anticipated that this raising will be made utilizing spoil material removed from the RCC spillway excavation.

The east embankment beyond station 16+65 will not be raised since this portion of the dike is only a few feet high, and has many trees both upstream and downstream.

It is assumed that a failure (washout) of any portion of the east embankment dam not replaced by the RCC spillway section, would rerult in very little increased flooding downstream since these portions of the dike are very low.

West Embankment Core Wall Raising

The west embankment core wall will be raised to a uniform elevation of 912.58 from the non-overflow gravity dam to the end of the existing core wall (about station 8+00). Elevation 912.58 is the peak reservoir level during the IDF flood. The new core wall will be constructed of reinforced concrete with a top width of 2 feet. The new wall will be dowelled into the existing core wall. Further details of this construction are shown on Civil Design Drawing No. 1979L-C3.

West Embankment Crest Raising

The crest of the west embankment dam will be raised to a uniform crest elevation of 913.58. This will provide one-foot of freeboard during the IDF flood assuming no failure of those portions of the east that dam will not be raised. Further details of this construction are shown on Civil Design Drawing No. 1979L-C2.

Ungated Spillway Walkway

A new walkway will be constructed over the ungated spillway to provide for pedestrian traffic to the east embankment dam. The walkway will be 3-feet wide supported by 8-inch diameter metal pipe anchored into the spillway crest concrete. The structure will be fabricated from A36 structural steel; designed for a live load of 50 psf. The top of the grating will be at El. 914.58. Further details of this construction are shown on Civil Design Drawing 1979L-C4.

REFERENCES

[1] "Evaluation of Spillway Capacity Expansion Alternatives at Boney Falls Hydroelectric Project," by Harza Engineering Company, August, 1987.

[2] "Geophysical Investigation, Boney Falls Dam, Escanaba, Michigan", October 1987, prepared for Harza Engineering Company by Weston Geophysical Corporation and Harza's letter to Mead Paper date 7 October 1987, subject, "Report of Geophysical Survey Results, Boney Falls Dam FERC Project No. 2506."

[3] Report and Letter dated 3 December 1987, subject, "Boney Falls Dam, FERC Project 2506, Interim Report on Left Embankment" prepared by Harza Engineering Company.

[4] Report and letter dated 16 February 1988, subject, "Boney Falls Dam, FERC Project No. 2506, Report on Dam Performance During Reservoir Raising and Field Investigation Program Description" prepared by Harza Engineering Company.

[5] "Design Flood Documentation Report for Boney Falls Dam," and letter dated 2 May 1988 prepared by Harza Engineering Company.

[6] "Inflow Design Flood Report - Addendum No. 1," prepared by Harza Engineering Company, October, 1988.

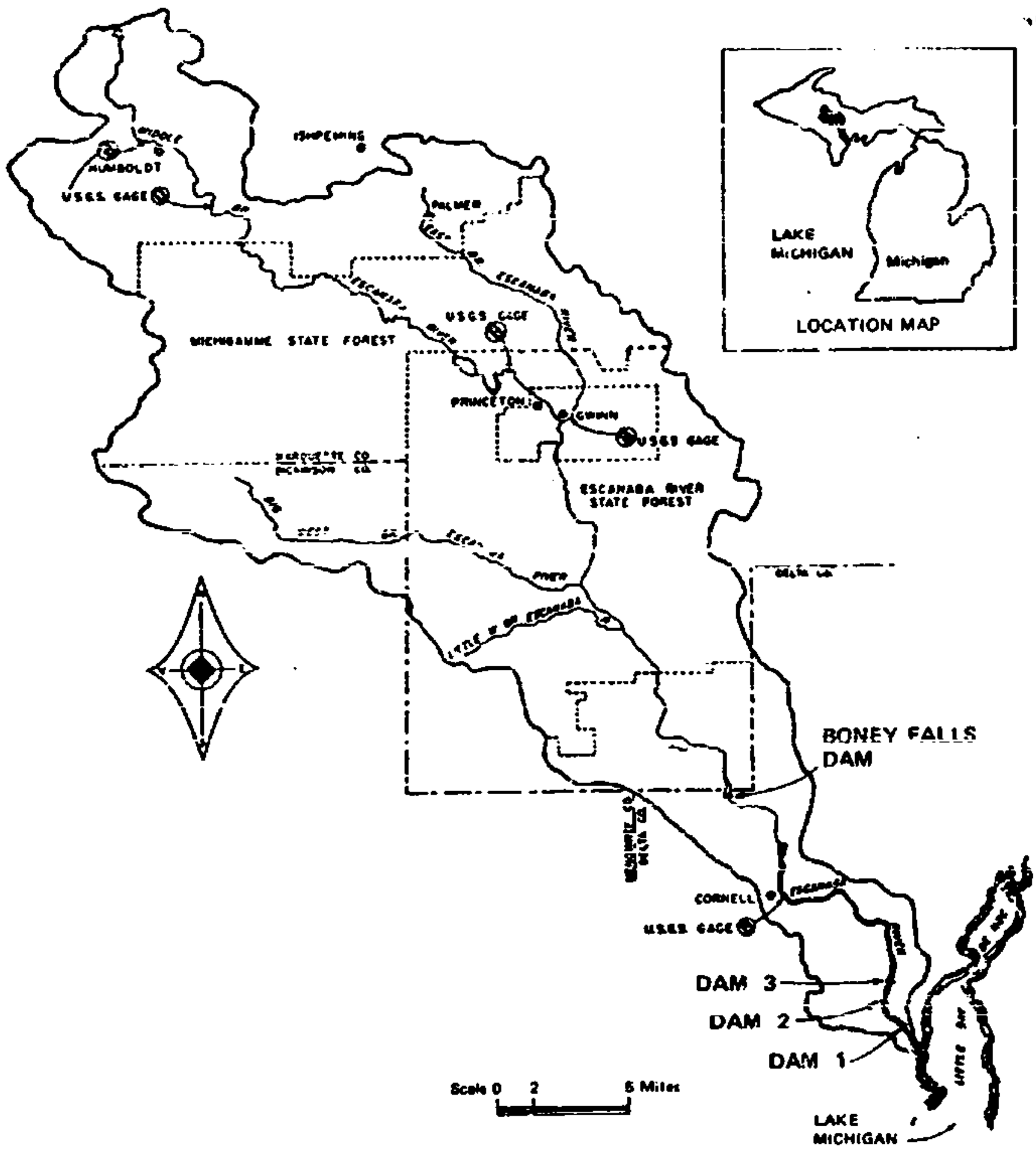
[7] "Boney Falls Dam, Licensed Project No. 2506(4), Geotechnical Investigations Report," prepared by Harza Engineering Company, October 1988.

[8] "Boney Falls Dam, Licensed Project No. 2506(4), Third 5-Year Inspection and Safety Report," prepared by Harza Engineering Company, January 1989.

[9] "Boney Falls Dam, Licensed Project No. 2506(4), Third 5-Year Inspection and Safety Report, Stability Analysis Supplement" prepared by Harza Engineering Company, January 1989.

Exhibits

Exhibit 1



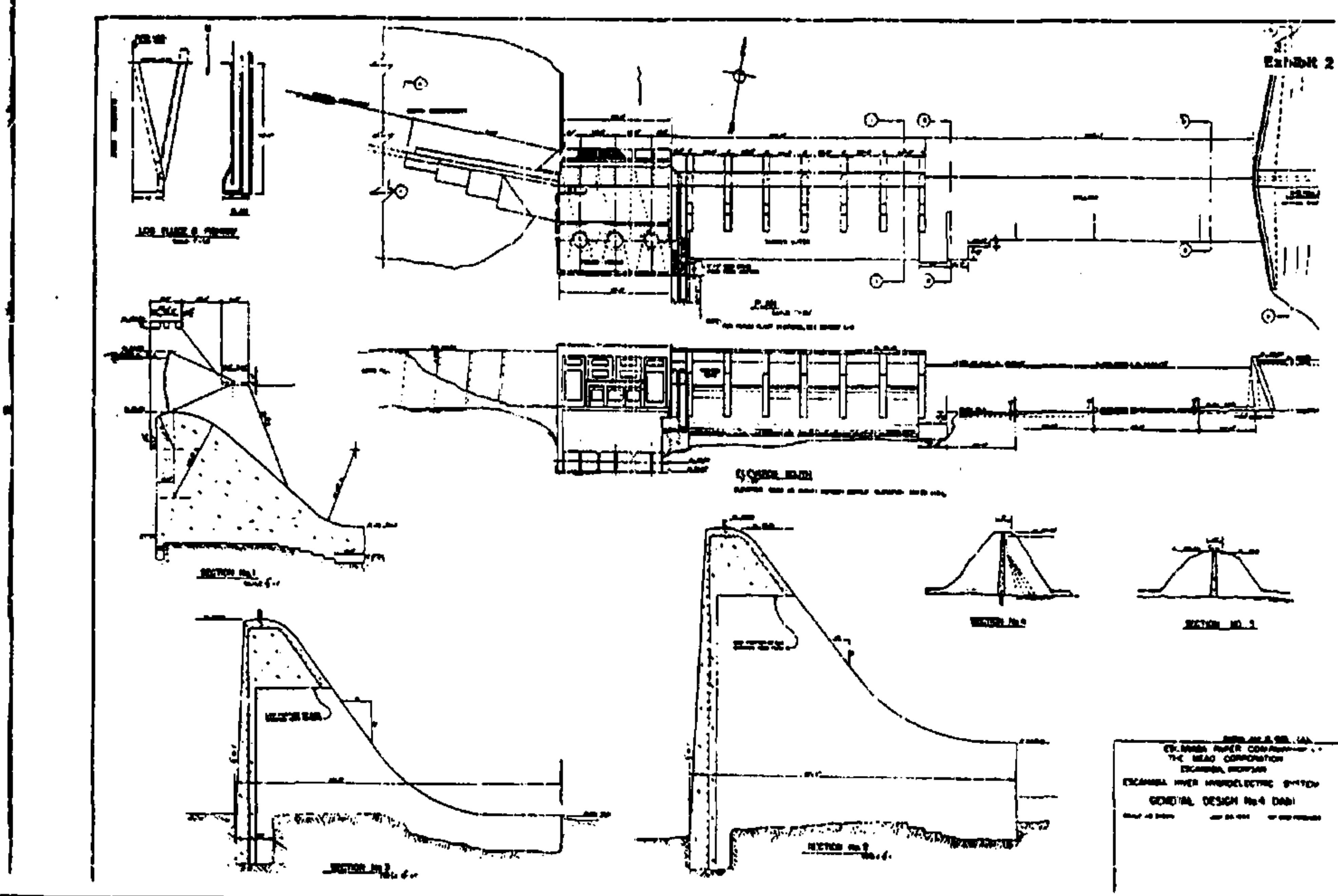
ESCANABA RIVER
HYDROELECTRIC PROJECT

Escanaba River Basin

Escanaba Paper Company
The Mead Corporation
Escanaba, Michigan

November 1988

Exhibit 2



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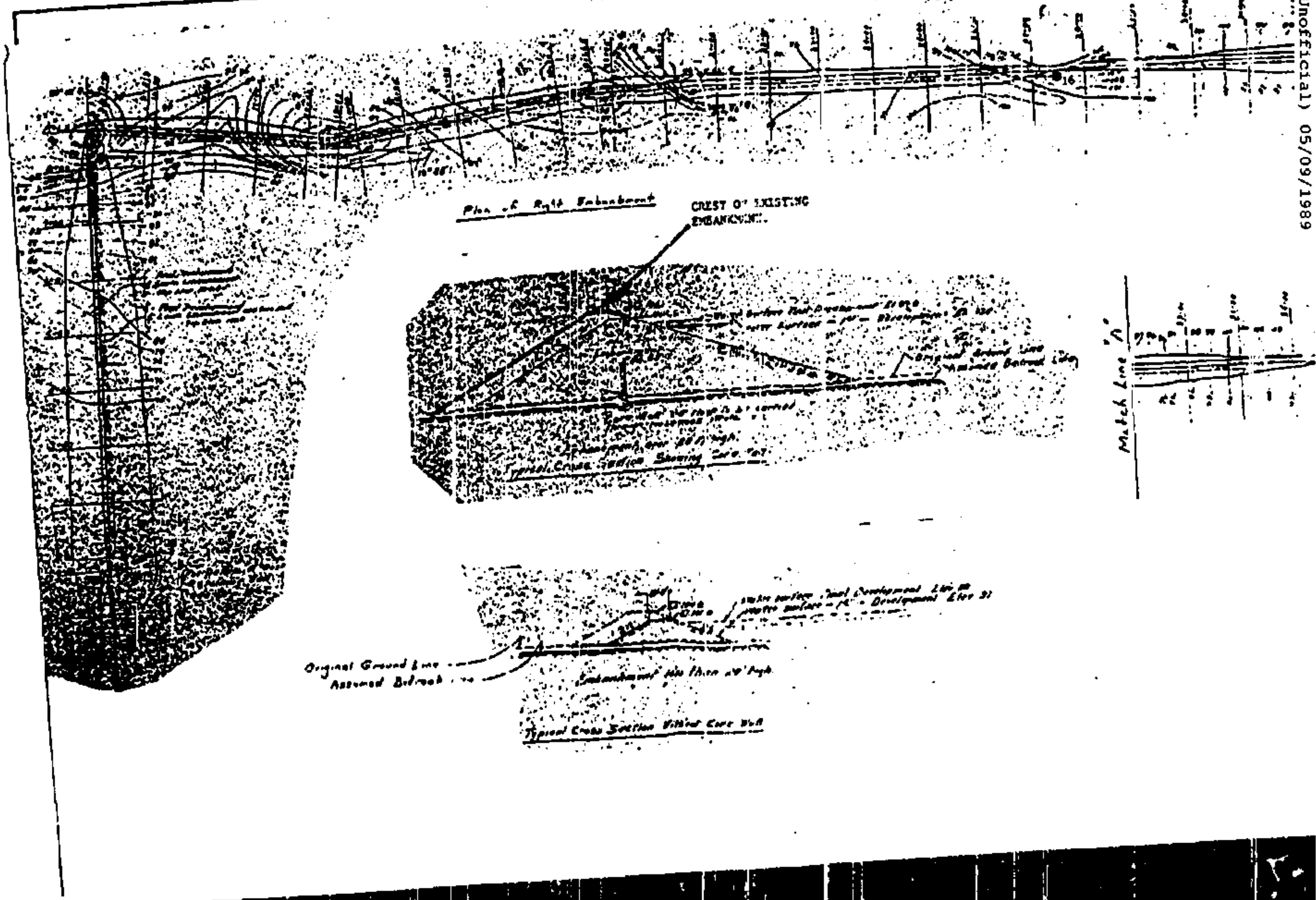

CREST OF EXISTING
EMBANKMENT
Match Line "A"
Original Ground Line
Assumed Bedrock Line
Typical Cross Section Without Core Wall

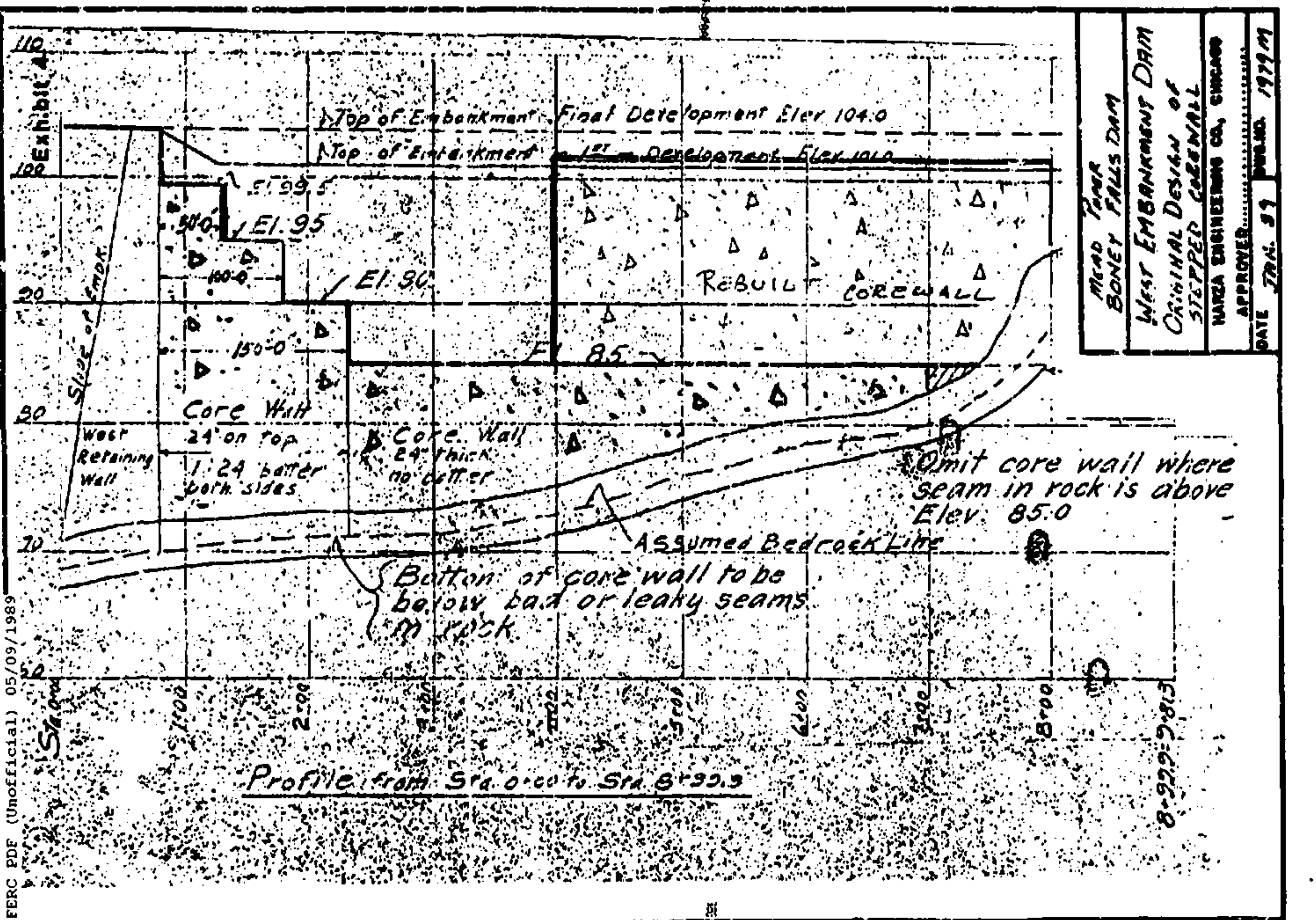
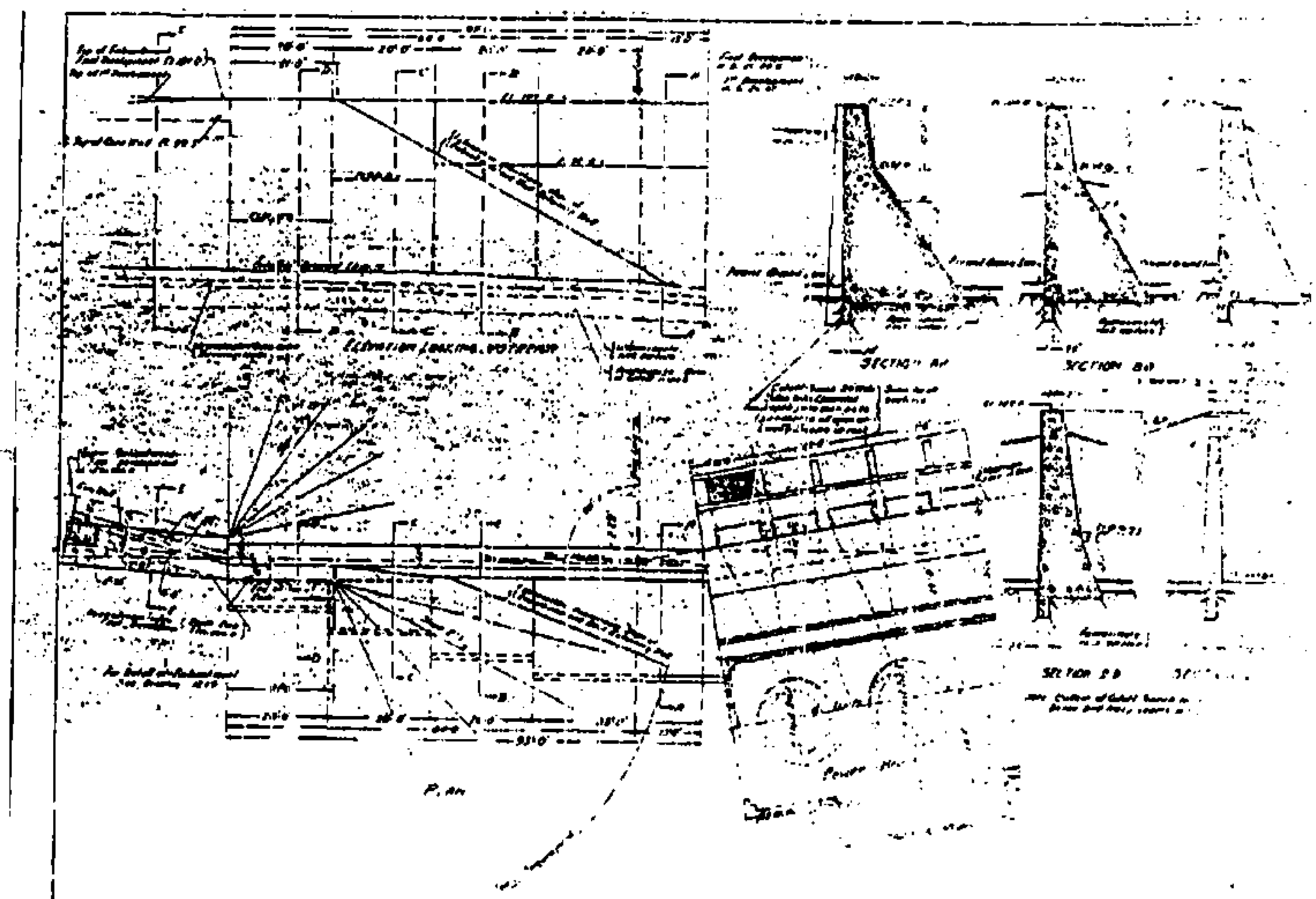


Exhibit A
Top of Embankment Final Development Elev 104.0
Top of Embankment 1st Development Elev
El 99.5
El 95
El 90
El 85
50-0
100-0
150-0
Slope of Embk
West Retaining Wall
Core Wall 24" on top 1:24 batter both sides
Core Wall 24" thick no batter
REBUILT COREWALL
Omit core wall where seam in rock is above Elev. 85.0
Assumed Bedrock Line
Bottom of core wall to be below bad or leaky seams in rock
110
100
90
80
70
60
0+00
1+00
2+00
3+00
4+00
5+00
6+00
7+00
8+00
Profile from Sta 0+00 to Sta
MEAD PAPER BONEY FALLS DAM
WEST EMBANKMENT DAM ORIGINAL DESIGN of STEPPED COREWALL
HARZA ENGINEERING CO., CHICAGO
APPROVED
DATE

Exhibit 5



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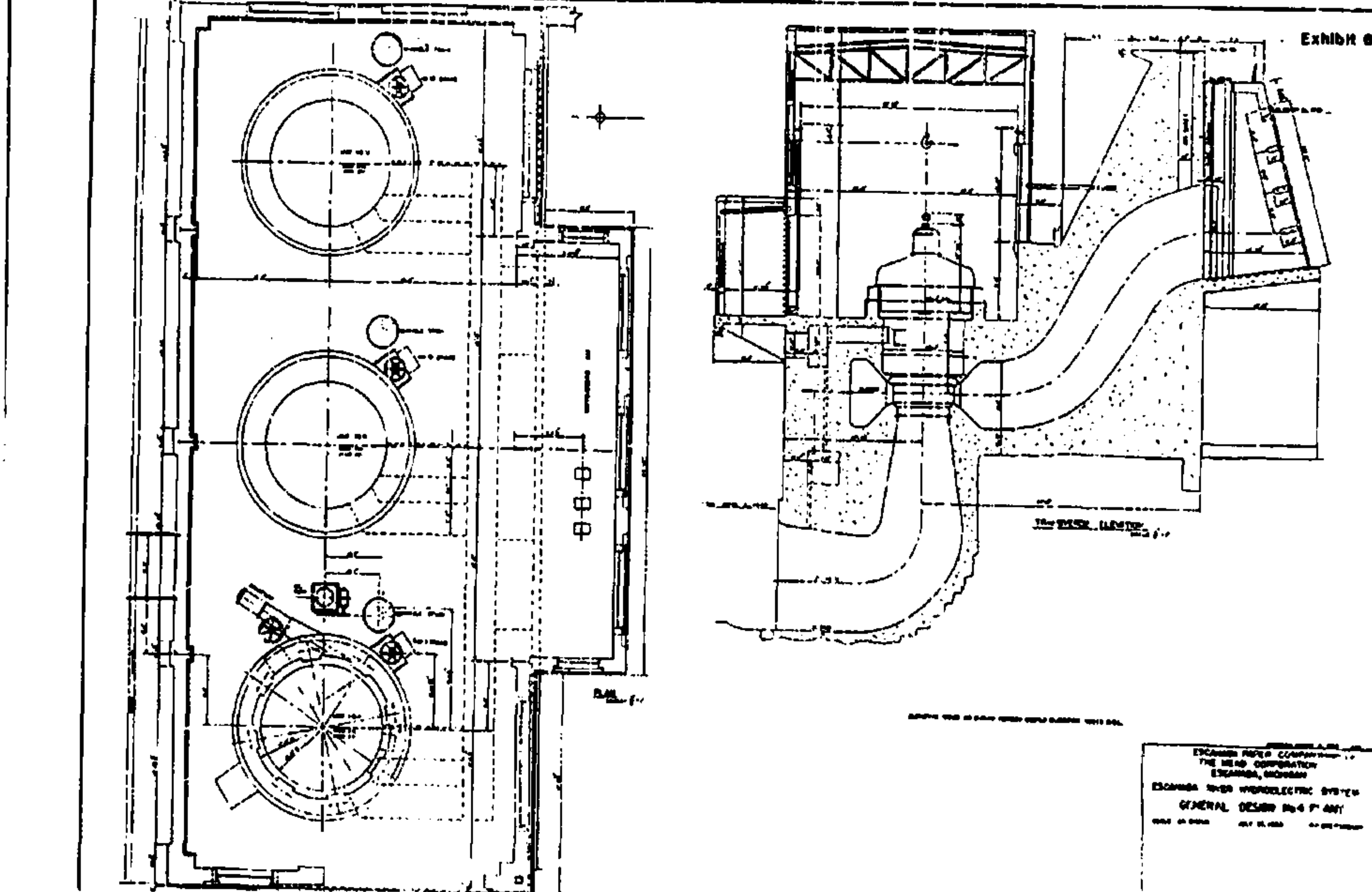
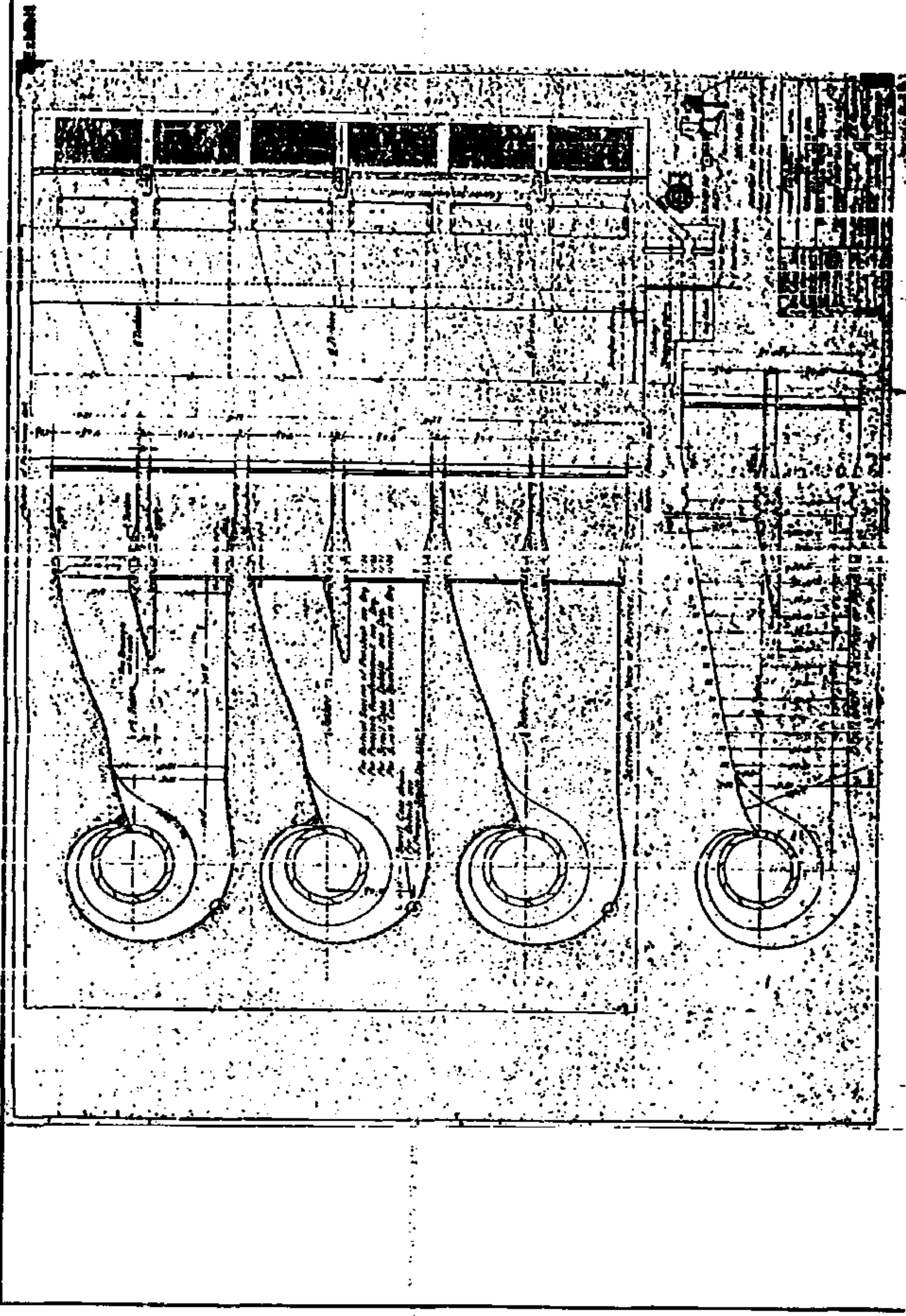

TRANSVERSE ELEVATION
PLAN
THE MEAD CORPORATION
HYDROELECTRIC SYSTEM
GENERAL DESIGN



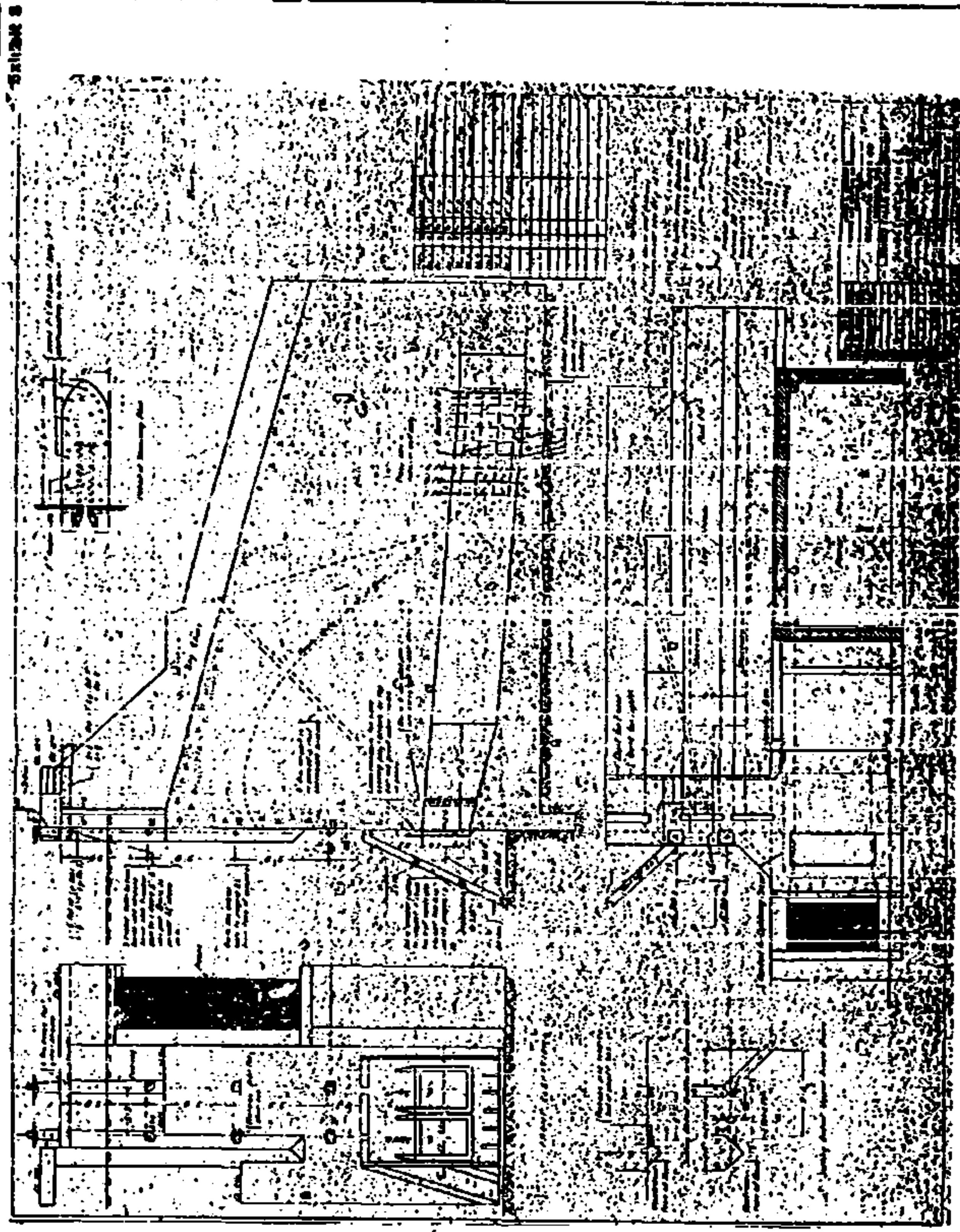


Exhibit 8

Exhibit 6

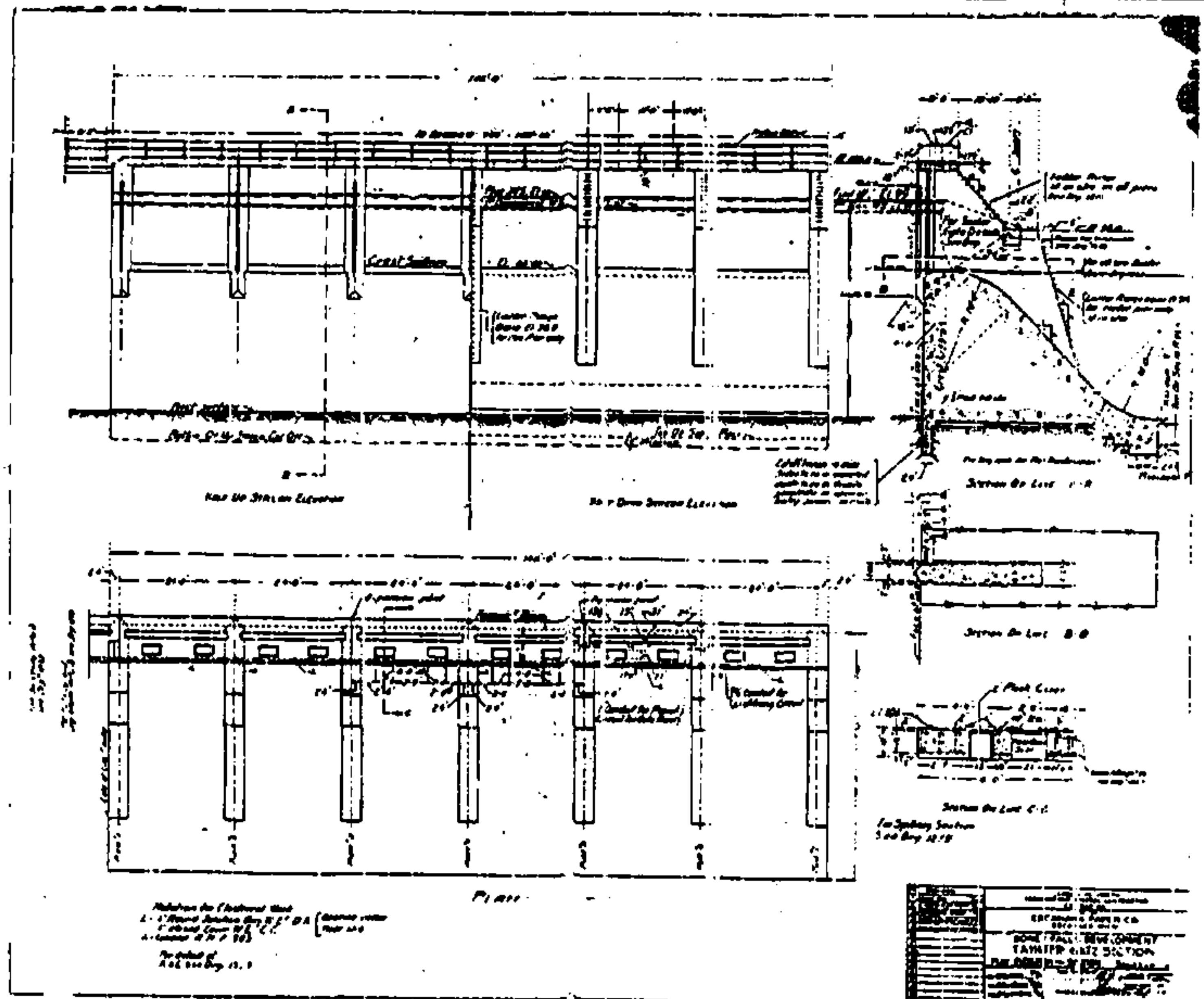


Exhibit 10

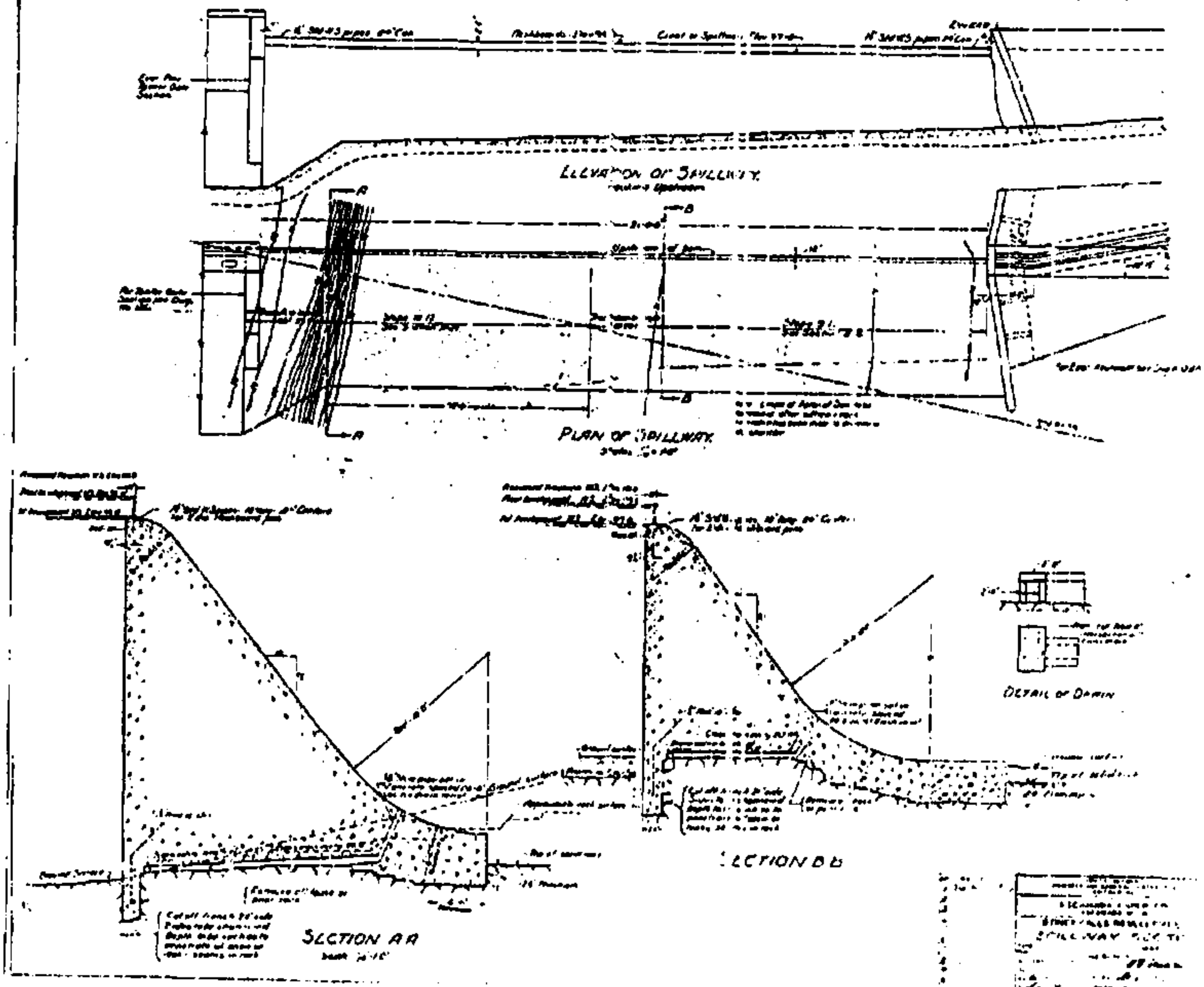

ELEVATION OF SPILLWAY
PLAN OF SPILLWAY
SECTION A-A
SECTION B-B
DETAIL OF DRAIN
A
B

Exhibit 11

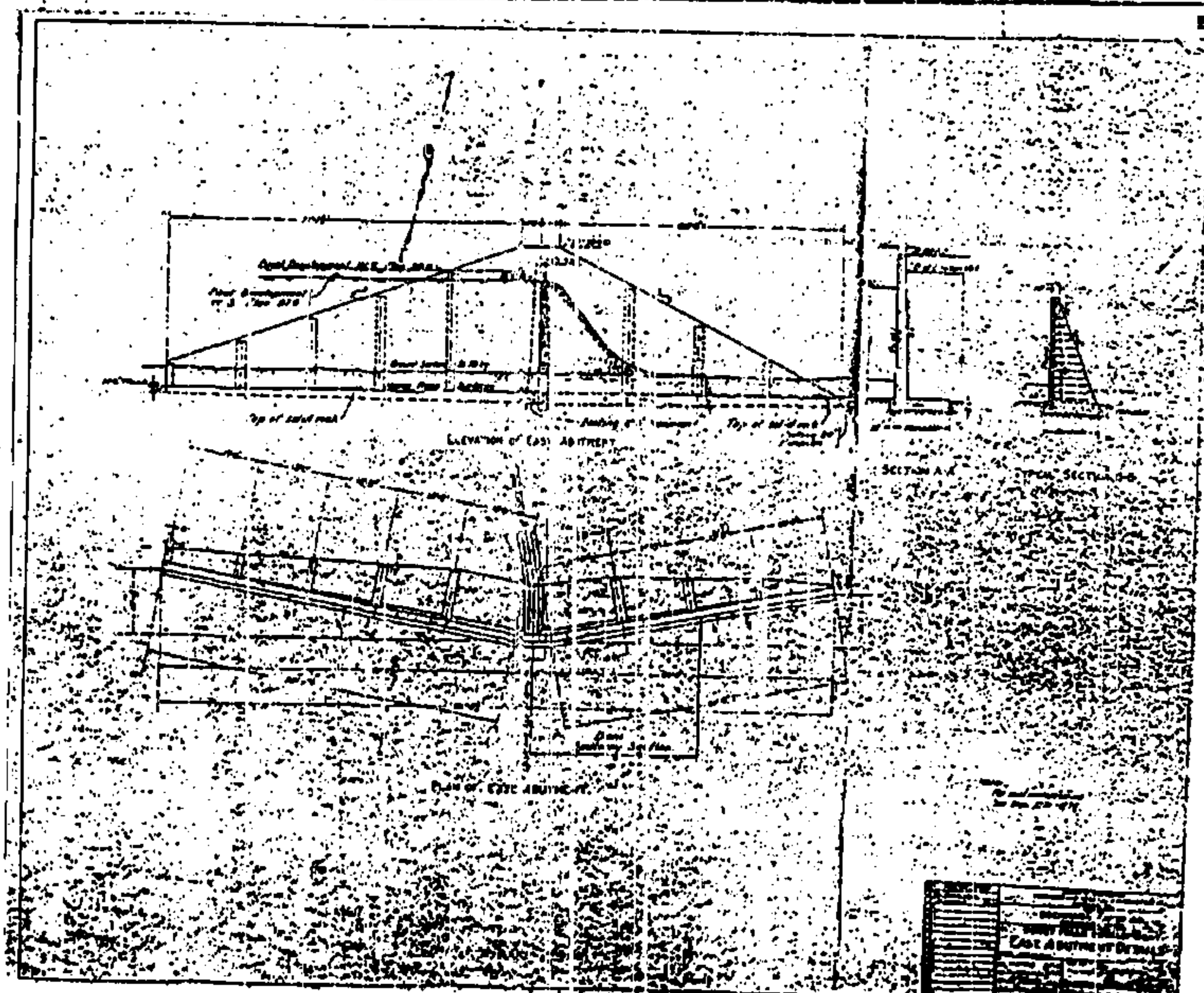
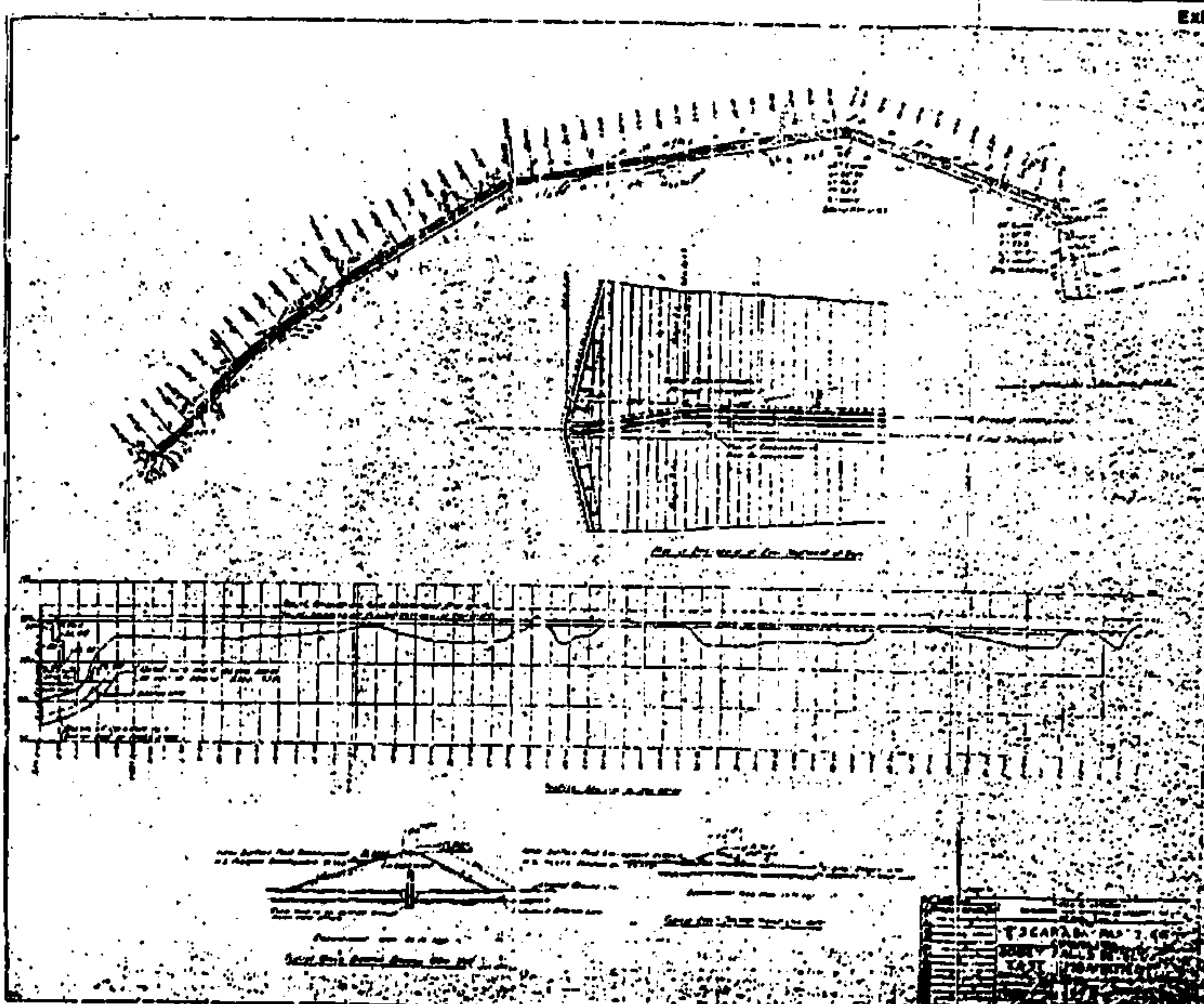

Top of solid rock
ELEVATION OF EAST ABUTMENT
SECTION A-A
PLAN OF EAST ABUTMENT
EAST ABUTMENT DETAILS

Exhibit 12



Appendices

APPENDIX A
Geophysical Investigations Report-
Summary and Conclusions

HARZA ENGINEERING COMPANY CONSULTING ENGINEERS

October 7, 1987

Mead Corporation
Publishing Paper Division
P.O. Box 757
Escanaba, MI 49829

Attention: Mr. Wayne LaFave
Superintendent of Outside Facilities

Subject: Report of Geophysical Survey Results
Boney Falls Dam, FERC Project No. 2506

Dear Mr. LaFave:

Harza is pleased to present the subject report of geophysical investigations at Boney Falls Dam. These investigations were conducted in response to FERC's June 4, 1987 directive to "determine the integrity of the embankments and the condition of the limestone foundation under the embankments" by conducting a geophysical survey of the dam and foundation. A discussion of the key survey findings and conclusions, along with Harza's recommendations for Mead's follow-up activities, follows.

Conclusions

Based on ground penetrating radar (GPR) and seismic refraction survey data, there is no evidence of any voids or slumping in the dam embankment. The embankment central concrete core wall shown on design drawings and construction photographs was detected in the right embankment, but not in the left. There may, however be a core wall in the left embankment that could not be detected. From data on the right embankment, the core wall top is well below the dam crest and could be a very short structure in the left embankment where the dam is low. This would make the core wall difficult to detect.

Foundation conditions beneath the embankments were evaluated using seismic refraction. The survey data indicate that bedrock is shallow, and that it is a high-velocity, competent rock. Notable exceptions are two areas of what appear to be either fractured/weathered rock, or a deeper bedrock surface than in other areas of the dam. One area was identified under the left embankment 200 to 260 feet east of the ungated overflow spillway section, and another was observed under the right embankment 175 to 265 feet west of the powerhouse. The low-velocity area on the left abutment roughly coincides with a northeast-trendin terrace that likely formed the original river valley wall at the damsite.

Embankment and foundation seepage were assessed using electrical resistivity methods, supplemented by dye tracing and review of piezometer and flow data. The reservoir level was between El. 895.5 and El. 898.4 during the survey. On the left abutment, the majority of the seepage passes to the east beneath the low portion of the embankment and then southward until it daylights about one-quarter to one-half mile downstream of the dam at the springs previously identified. The seepage path appears to be along two of the three primary joint sets, oriented 290° and 345°. From the resistivity results, the seepage under the east low embankment appears to be occurring at depth below the bed-rock surface, within the rock mass. Dye injected into sinkholes in the reservoir in this area appeared in the spring 1600 feet downstream within four hours of injection. Although some weathered rock was evident under the high portion of the left embankment, only a low rate of seepage was detected through the area. No large solution caverns were detected in the foundation rock.

Under the right embankment, it appears that the majority of the seepage enters the foundation at the western end of the main embankment, near the crib cofferdam where the embankment turns northward. Some of this seepage may travel eastward under the main embankment, exiting in the collector ditch at the down-stream toe of the embankment. However, most of the seepage appears to flow southward, where it is intercepted by the collector ditch at the downstream toe of the dam. Dye injected into a depression near the crib cofferdam was observed in the embankment toe ditch within 45 minutes of injection. No dye was observed in the small creek just downstream of the dam.

In summary, it is Harza's conclusion that, although the dam embankments and foundation rock are structurally sound, a signi-ficant seepage problem exists at this dam. However, we do not believe that the observed seepage conditions pose an immediate threat to the safety of the dam.

Recommendations

Harza recommends that Mead adopt the following plan to confirm the results of the geophysical work, to determine the need for remedial work, and to develop designs and cost estimates for any remedial work that might be required.

- Perform a limited embankment and foundation drilling and testing program at selected locations to confirm embankment integrity and foundation rock quality. Three to four boreholes should probably be drilled in each of the three seepage areas identified by the resistivity survey, with three to four additional holes in the area of weathered bedrock identified by seismic methods under a portion of the left embankment. Holes should be drilled from the embankment crest. Core and soil samples should be recovered from all holes and piezometers should be installed. Borehole water pressure testing should be included in the program.

- Arrange to have an FERC geologist visit the site at the initiation of the drilling program.

- Authorize Harza to identify and evaluate alternative measures for treating the seepage problem.

We are confident that the information gained from the geophysical survey, supplemented by confirming field drilling and testing, will lead to the identification of an appropriate course of action at Boney Falls Dam.

Very truly yours,

David R. Baier

David R. Baier, P.E., P.G.
Project Manager

Attachments: Geophysical Investigation Report, October 1987

APPENDIX B
Geotechnical Investigations Report-
Summary and Conclusion

A comprehensive geotechnical exploration program was undertaken by Harza Engineering Company at Boney Falls Dam in 1988. The following is a summary of findings.

General

- East-west embankment sections are comprised of a relatively pervious gravelly silty sand (SM) with some gravels and cobbles and occasional boulders to 1.5 feet in diameter.

- In-situ overburden is a glacial till deposit consisting primarily of a gravelly silty sand (SM) which is overlain by a sandy silt (ML) to silty clay layer about 7 feet thick on the left abutment; total overburden thickness ranges from 6 to 15 feet.

- Bedrock is a thin to very thinly bedded limestone with shale interbeds and partings; slightly weathered with locally very thin zones of moderately weathered rock; occasional very thin vuggy layer and occasional vugs locally; bedding is subhorizontal.

East Embankment - Geologic Conditions

- From the results of geophysical surveys, multiple seepage paths within the rock foundation pass beneath the embankment between stations 7+50L to 10+50L and (primary) 1+75L to 2+75L (secondary) and exit as springs in an area approximately 1000 to 2000 feet downstream of the dam as features referred to locally as the North and South Creeks.

- The foundation underseepage zones identified during the geophysical surveys are characterized by an upper zone of slightly to locally moderately weathered rock approximately 7 to 13 feet thick. Within this zone thin vuggy layers, occasional vugs and solution enlarged fractures, and bedding planes were encountered. Beneath this zone the rock is generally fresh, solid and of good quality.

- The underseepage zones are characterized by high rock mass permeability indicative of open conditions; permeability ranges from about 40 to 220 lugeons.

East Embankment - Concrete Core Wall

- The core wall was encountered in a trench beneath the center of the embankment at Station 0+36L at El. 907.3, within 0.8 feet of the original design elevation; no attempt was made to verify the extent of the wall to the east.

East Embankment - Instrumentation

- Piezometers installed along the primary underseepage path indicate that responses to reservoir fluctuations are negligible below El. 896.6.

- Seepage flows in North and South Creeks are also negligible below reservoir El. 897.0.

- Artesian conditions were encountered in the poorly drained area at the downstream toe of the dam in BH-2 (Sta. 1+25L) and BH-W38 (Sta. 2+25L) with the reservoir at El. 903; pore pressures resulted in piezometric heads up to 0.7 feet above the ground surface.

East Embankment - Seepage Evaluation

- Based on the length of the primary seepage path (min. 1150 feet), the low hydraulic gradient (i=0.005) and the fact that seepage is mostly through the rock foundation, these conditions do not appear to present a hazard to the safety of the dam at this time; no remedial work for long-term safety is anticipated if the high portion of the embankment is replaced by a new gravity structure for added spillway capacity as presently planned. Replacement of the dam with a gravity structure on rock would eliminate the significance of potential piping of embankment fill into the foundation.

- Seepage along the secondary path exits the rock foundation as seeps beneath the poorly drained, organic-rich material at the downstream toe of the dam.

- An instrumentation monitoring program is proposed to determine long term pressure and flow characteristics.

West Embankment - Geologic Conditions

- From geophysical resistivity surveys, seepage passes beneath the embankment between stations 7+10R to 7+60R and through the embankment between stations 2+35R and 3+75R.

- The underseepage zones are characterized by an upper zone of high rock mass permeability about 7 to 13 feet thick (permeability ranges from 70 to 150 lugeons), and a lower zone of moderate permeability (generally 10-40 lugeons).

West Embankment - Concrete Core Wall

- The core wall exists within the east-west embankment section beneath the downstream side of embankment; the core wall height at stations 1+23R to 1+40R and 5+40R are within 0.8 feet of design height, El. 907.9 and El. 908.9 respectively.
- The core wall top is at El. 893.5+, 12.5 feet below normal pool, between about stations 2+40R to 5+40R (and > 3 feet between stations 1+50R to 2+40R). This allows seepage to migrate downstream over the core wall into the pervious embankment fill.

West Embankment - Instrumentation

- Relatively high embankment pore pressures are indicated in piezometers BH-W3 and W5 in the area of the dam crest with the low core wall, BH-W6 upstream of the core wall and BH-W21, in the downstream slope. Lower pressures were recorded in the crest piezometer BH-W4, in the area of the low core wall, which may be a result of relief into underlying bedrock foundation.
- Uplift pressures within the rock foundation are less than embankment pore pressures.

West Embankment - Seepage Evaluation

- From geophysical tests, the primary seepage path passing beneath the embankment occurs between stations 6+85R and 7+35R which is characterized by high rock mass permeability and open conditions.
- Underseepage along the primary seepage path appears to be intercepted by the buried collector ditch at the toe of the dam. Recent flows observed in "Barney's Creek", downstream of the embankment are only in part a function of underseepage; generally less than 40 gpm.
- A wet area in the downstream embankment slope exists between stations 2+35R to 3+75R in the area with the low core wall. This zone is characterized by high embankment pore pressures and a high gross hydraulic gradient (i=0.2).

Borrow Sources for Proposed Embankment Raising

- A source of semi-impervious construction material comprised of medium dense silty sand (SM) material with a trace of gravel and slightly plastic fines was studied for use in the embankment raising proposed as part of project spillway capacity expansion.

- Compaction tests indicate maximum densities between 127-133 pcf at 8 percent optimum moisture content; natural moisture content is 10-11 percent but some moisture loss is anticipated during handling.

Concrete Structures - Geologic Conditions

- Core drilling operations encountered locally thin vuggy layers and scattered vugs in the gated spillway foundation.

Concrete Structures - Instrumentation

- Uplift pressures beneath the non-overflow gravity structure indicate piezometric levels approximately 22 feet above top of rock beneath the center of the dam with the reservoir at El. 903.

- Relatively high uplift pressures exist beneath the central and eastern portions of the ungated spillway apron; artesian conditions indicate piezometric levels up to 3.2 feet above the apron for reservoir El. 903.

- Uplift pressures beneath the eastern end of the ungated spillway are higher beneath the apron than the crest; this may be a function of the depth of the piezometer in BH-W26.

Concrete Structures - Foundation Parameters

- From laboratory tests on samples recovered during drilling, rock foundation shear strength parameters recommended for use in the stability analyses are an angle of internal friction of 23° and a cohesion factor of 0.5 MPa (70 psi).

RCC Spillway Foundation Preparation

- A portion of the existing east embankment will be removed down to bedrock for construction of a new RCC overflow spillway at the project.

- Dental excavation of the weathered and solutioned rock will be required particularly in underseepage areas. The foundation should be hydraulically cleaned and all open fractures slush grouted.

- Consolidation grouting will likely be required beneath the RCC spillway foundation in the underseepage zones (Station 1+75L to 2+75L; 7+25L to 10+50L) to locally strengthen the foundation and to increase the depth of the seepage path beneath the RCC structure; following an inspection of the exposed spillway foundation more extensive consolidation grouting may be required.

APPENDIX C
FERC Third 5-Year Inspection and Safety Report-
Summary and Conclusions

CHAPTER I

SUMMARY

A. Report History and Content

A.1 Previous Reports. This is the Five-Year Inspection and Safety Report for the Boney Falls Project prepared persuant to Order No. 122, Subpart D of Part 12 of the Federal Energy Regulatory Commission (FERC) regulations. Mead Corporation, Publishing Paper Division owns and operates the project under FERC License No. 2506(4) granted on 31 January 1978.

Previous Part 12 Reports were prepared in 1980 and 1985. The 1985 Report was resubmitted in 1986. Both reports were prepared by Barr Engineering Company, Minneapolis, Minnesota. This is the first report prepared by Harza Engineering Company, Chicago, Illinois. By FERC letter dated 12 December 1985, the FERC rejected the 1980 and 1985 Part 12 Reports, along with the 1983 Dam Break and Stability Analysis Supplement to the 1980 Report. The 1985 Part 12 Report was resubmitted in 1986 incorporating modifications requested in FERC's 12 December 1985 letter.

By FERC letter dated 10 June 1988, the FERC rejected the 1986 Part 12 Report and ordered that an addendum be submitted by 1 October 1988 to address a series of specific concerns. By Mead Paper letter dated 14 September 1988, a deadline extension for submission of the required addendum to 1 December 1988 was requested. By FERC letter dated 16 September 1988 the extension was granted.

By letter dated 11 November 1988, Mead Paper requested that a new Part 12 Report be prepared and submitted in January 1989 in lieu of an addendum to previous rejected reports. Mead had undertaken extensive geologic and hydraulic studies of the project since the 1986 Part 12 Report was submitted. In view of the new information made available by these studies, it was felt that an all new Part 12 Report was justified. By letter dated 23 November 1988 the FERC approved the preparation of a new Part 12 Safety

Inspection Report with the understanding that earlier FERC concerns regarding the 1985 and 1986 submittals be specifically addressed. The next Part 12 Report will be due in 1994.

Copies of the above pertinent correspondence, including the FERC approval of Mr. David R. Baier as the Independent Consultant, are attached as Appendix A.

A.2 Response to FERC Comments on Previous Reports. The FERC raised a series of specific concerns relative to the 1985 and 1986 Part 12 submittals in their 10 June 1988 letter to Mead Paper. These concerns included the involvement of the prior Independent Consultant in the 1985 and 1986 work, the stability analyses of project structures, and comments on the dam break/flood wave studies previously submitted.

A statement regarding the involvement of the prior Independent Consultant in the preparation of the 1985 and 1986 Part 12 Reports is attached as Appendix B.

Stability of the concrete structures and earth embankments have been completely re-analyzed since the 1985 and 1986 submittals. These new analyses are based on newly acquired information from field investigations conducted in 1987 and 1988 by Harza.

Dam break/flood wave analyses also have been revised by Harza since the 1985 and 1986 submittals using a more current version of the DAMBRK model than was used for the earlier submittals. Specific FERC comments on the prior analyses have been incorporated into the current analyses. Results of these analyses have been presented to Mead Paper and forwarded to the FERC in Harza's Inflow Design Flood Report [6] (Note: References are attached to end of this report ordered by date), and Inflow Design Flood Report Addendum No. 1 [7] submitted to the FERC in April and October 1988 respectively

A.3 Scope of Work. The scope of work for the preparation of this Inspection and Safety Report is attached as Appendix C.

A.4 Independent Consultant's Resume. By letter dated 14 September 1988, Mead Paper requested that Harza's Mr. David R. Baier be approved as an Independent Consultant for the Boney Falls Project to facilitate preparation of the addendum by Harza. By FERC letter dated 26 September 1988 Mr. Baier was approved as the Independent Consultant. This letter is included in Appendix A. The resume of the Independent Consultant, Mr. David R. Baier is attached as Appendix D.

B. Field Inspection

A field inspection was performed on 11 November 1988 by the Independent Consultant with Mr. Clayton Carlson of Mead Paper. Significant findings of this inspection are:

1. The dam is in good overall condition with significantly improved appearance and condition due to recent repair and maintenance activities of the owner.
2. The west (right) embankment dam continues to show signs of a high phreatic surface near the downstream toe between stations 2+50R and 3+90R (NOTE: Station designations with "R" indicate west embankment dam stationing along right-hand side of river; Station designations with "L" indicate east embankment dam stationing along left-hand side of river). The wetness in this area, however, is not as apparent as it has been during past inspections. The reservoir water level has been kept at a minimum of about 3 feet below normal pool since 1987 per FERC orders (NOTE: Normal Pool level is El. 906.58 which is local datum El. 98.00+808.58 conversion to ft, msl). This is the likely reason for the apparent reduction in wetness. New piezometers were installed in this area in 1987 and 1988. Seepage flows that are presently collected in a buried pipe at the toe of the westerly half of the embankment have been separated from flows collected in the rock-lined ditch at the toe of the easterly portion of the embankment closest to the powerhouse. The flows have been routed through a Parshall Flume into a creek downstream of the Dam. The owner takes periodic readings of seepage flows through the flume and of the total

flow in the creek downstream of the point where the seepage has been diverted. As a result of this seepage flow re-routing there is now very little seepage flowing into the river from the west bank of the tailrace. There is now very little flow in the rock-lined ditch at the east end of the dam. Provisions should be made for more accurate measurement of flows in this ditch in the future. Consideration should be given to replacing the toe drains along the entire length of the west embankment dam with a buried filtered pipe drain.

3. The non-overflow gravity dam appears to be sound and free of distress. A new retaining wall downstream of the section has been rebuilt replacing old masonry blocks with a new concrete wall. This wall supports fill on the downstream side of the non-overflow section.

4. The powerhouse appears to be sound and free of distress. All three units are operational as verified by recent efficiency testing done in conjunction with relicensing activities. The log sluice and fishway adjacent to the powerhouse have been capped with concrete as part of the gated spillway pier replacement program which has just been completed. The fish ladder and sluice are no longer functional.

5. The gated spillway appears to be sound and free of distress. Replacement of all piers is now complete with piers 1 and 3 being replaced in 1988. The spillway chute concrete appears sound. There is no visible erosion damage of any significance.

6. The ungated spillway appears to be sound and free of distress. There is some erosion of concrete material near the crest of the ogee which should be repaired in the next five years.

7. The east embankment dam appears to be sound and free of distress. An inspection trench was excavated at the toe of a portion of the east embankment in 1987 to investigate seepage phenomena. A filtered pipe drain was installed in the trench. An exit weir has been provided for measuring flows. A detailed report [4] of these activities was prepared by Harza and transmitted to Mead Paper by letter dated

16 February 1988. Recent repairs to an oversteepended area of the downstream face of the east embankment dam and the effect of the oversteepening on embankment stability, are also described in the February, 1988 report. The east embankment dam is only accessible from the powerhouse by foot across the toe of the ungated spillway or by walking across the ogee crest. A pedestrian bridge should be constructed along the weir crest to facilitate access to the east dam for inspection during periods of high water.

8. Major springs were discovered in the area about 2000 feet downstream of the east embankment in 1986. There are two spring areas identified as the 'North' and 'South' Creeks which are monitored. Seepage flows vary with reservoir head. Measured seepage under normal pool conditions is about 5,000 gpm. This is the combined discharge from the two spring areas. While there is evidence from neighboring property owners that these springs had been in existence for many years (note Photo 2 which shows a waterfall of the north creek in 1978), 1985 was the first time that these features have ever been reported to the FERC. There is no mention of these features in the prior Part 12 reports. The owner has established an extensive seepage monitoring program relative to these springs. The seepage water is clear. The FERC staff from both the Chicago and Washington D.C. offices visited the site and inspected the springs in 1987 and 1988. A detailed geophysical investigation program of the foundation and embankments was conducted in 1987 [3] to further evaluate the significance of the observed seepage. The embankments do not appear to have been damaged by seepage flows.

C. Stability Evaluation

C.1 Concrete Structures. Stability analyses presented as a supplement to this report indicate that all structures meet FERC's stability criteria up to and including the Probable Maximum Flood (PMF) loading conditions except for the powerhouse. Stability analyses are based on the results of exploratory drilling, lab testing, and field measurements taken during 1988 by

Harza. The powerhouse stability factors against sliding are below FERC guidelines for Normal Pool, Normal Pool plus Ice, and Normal Pool plus Earthquake (0.1g) loading conditions. A cracked-base section develops for normal pool conditions. The structure fails to meet FERC stability criteria if full headwater uplift develops in the cracked portion of the base as required by FERC approved analytical procedures [i]. The foundation of Boney Falls Dam is karstic limestone. The significance of a crack at the base with respect to pore water pressure distribution in the foundation is questionable given the degree of drainage that is probably provided by the joints in the foundation. Uplift under the powerhouse should be verified by installation of piezometers within the next year. If high uplift is discovered, post-tensioned anchors should be installed for stability improvement.

C.2 Embankment Dams. An extensive drilling and testing program was conducted in the embankments in 1988. The purpose of the program was to (i) verify foundation and embankment conditions revealed by geophysical testing conducted in 1987, (ii) determine phreatic conditions in the embankments, (iii) sample embankment and foundation materials, and (iv) determine geometry of the embankment corewall. A portion of the corewall crest in the west embankment was found to be significantly below normal pool level. Stability analyses presented as a supplement to this report show that that portion of the west embankment dam where the corewall is low (with a high phreatic surface) does not meet FERC established stability criteria. However, where the corewall is high (with a lower phreatic surface) the stability factors are acceptable. It is recommended that the corewall in the west embankment be raised to above the design flood level (El. 912.58).

A new, filtered pipe drain should be installed at the toe of the west embankment from station 5+00R to station 8+00R for improved seepage control. The design of the existing buried pipe toe drain along the westerly end of the west embankment is not certain.

The stability of the east embankment dam was studied at the request of the FERC in December 1987 when the existence of a hand-layed stone wall was noticed during a field inspection. The results in Harza's report [4] showed that the presence of the stone wall reduced the factor of safety. The wall was subsequently removed and additional material was placed on the embankment slope. The east embankment dam is now meets FERC stability criteria for all loading conditions.

Zones of preferred seepage under the east embankment dam identified by geophysical [3] and drilling [8] studies do not require special treatment at this time. The embankment in the two underseepage areas identified by the geophysical survey in 1987 will be replaced in 1989 with a Roller-Compacted-Concrete (RCC) gravity spillway section as part of planned spillway capacity expansion. The foundation in these areas should be grouted prior to construction of the new structures as described in Harza's Geotechnical Report [8].

D. Spillway Adequacy

The existing spillway capacity is 33,000 cfs with water at the dam crest (El. 909.08). This is significantly less than the 147,000 cfs peak PMF inflow presented in prior Part 12 Reports. Dam break/flood wave studies presented in Harza's Inflow Design Flood (IDF) Report [6,7] show that, based on insignificant incremental damages downstream, an IDF peak of approximately 100,000 cfs would be more appropriate for the Boney Falls Hydroelectric Project. Additional spillway capacity is required. Spillway capacity expansion design drawings and specifications are currently under preparation which include construction of an RCC Spillway to replace portions of the east embankment dam and raising the west embankment dam to a uniform height. It is expected that these features will be constructed in 1989.

E. Operation and Maintenance

Current owner operation and maintenance of the plant are adequate.

F. Monitoring Data

Seepage measurements are presented in this report. There has been no major change in observed flow quantities since the present monitoring program was begun in 1986. There are no survey reference benchmarks adjacent to the project structures to permit periodic survey of structure elevations and/or horizontal displacement. Mead Paper plans to establish of off-structure benchmarks in the area during spillway expansion construction in 1989.

CHAPTER X
CONCLUSIONS

A. Assessment of Dam

The dam and appurtenant structures are in good condition. The owner has just completed an extensive rebuilding program. All the gate piers have been replaced. Other recently completed work is described in Chapter IX. all project structures except the powerhouse and the portion of the west embankment dam with a low corewall meet FERC stability criteria. Stresses within the concrete sections and foundation are acceptable. Measured flows from the two spring areas discovered in 1986 have not changed. There is no evidence that seepage in the bedrock under the embankment dams is threatening public safety. Improvement in the toe ditch seepage collection system is recommended for the west embankment dam.

From recent studies, a flood with a peak discharge of 100,000 cfs is the appropriate project Inflow Design Flood. The existing spillway can pass only about 33,000 cfs with the reservoir at the top of the existing earth dams. Existing spillway capacity is therefore inadequate. Spillway expansion studies are under preparation and will be the subject of a separate report.

B. Adequacy of Instrumentation and Monitoring

The existing array of instruments at the project is adequate for evaluating the behavior of the structure except for potential vertical and horizontal movements of the structures. There is no system of survey monuments and targets provided for routine surveying of structural settlements or deflections. A suitable layout should be developed, and measurements should be taken periodically. Frequency of reading existing instruments is acceptable. No changes are recommended at this time.

C. Adequacy of Maintenance and Surveillance

Existing maintenance and surveillance practices are sound. Access for inspection and surveillance to the east embankment dam is poor. A pedestrian bridge over the controlled spillway should be constructed. Emergency alarms to initiate . Emergency Action Plan have just been installed.

D. Adequacy of Project Operation

The project is adequately operated. A new automated system has just been installed

E. Adequacy of Operation of Spillway Gates and Standby Power

The existing gate hoisting system is similar to that at many other similar projects. The adequacy of the hoisting mechanism will be review as part of the spillway expansion studies presently under preparation. No changes are necessary at this time.

CHAPTER XI
RECOMMENDATIONS

A. Corrective Measures Required for Project Structures

A.1 West Embankment Corewall. The west embankment corewall should be raised to at least El. 912.58 from station 1+00R to 8+00R to improve embankment stability up to the maximum reservoir pool level during an IDF peak inflow of 100,000 cfs.

A.2 West Embankment Toe Drain. The existing west embankment toe drain system should be replaced by a new, filtered buried pipe drain for improved seepage control and monitoring.

A.3 Powerhouse Stability. At least two piezometers should be installed in the powerhouse to check for high uplift pressures. Stability should be recomputed if actual uplift under normal pool conditions is found to be less than full theoretical uplift pressure. If the computed factor of safety for the powerhouse is found to be less than that required by the FERC, post-tensioned anchors may be necessary for stability improvement.

A.4 Uncontrolled Spillway Concrete. There is some minor concrete deterioration on the downstream face of the uncontrolled overflow spillway within about 5 feet of the crest which requires some repair within the next five years.

A.5 East Embankment Dam Underseepage. If an RCC gravity spillway section is constructed in place of the high portion of the east embankment dam from station 0+00L to 5+00L, then there should be no need for further special remedial work to control underseepage. Foundation grouting for seepage control and general foundation improvement beneath the new spillway structure as discussed in the geotechnical report [7] should be incorporated into spillway construction work.

A.6 Spillway Capacity. Spillway capacity should be expanded to pass an inflow design flood of 100,000 cfs. Current plans by the owner to construct an RCC gravity section overflow spillway in place of the existing east embankment dam with raising the west embankment dam will lead to an increase in spillway capacity sufficient to meet this requirement.

B. Corrective Measures Required for Project Maintenance and Surveillance

B.1 Uncontrolled Spillway Walkway. A pedestrian bridge should be constructed to facilitate inspection of the east embankment dam, especially during flood events.

B.2 Underseepage Monitoring. Permanent Parshall Flumes should be installed on the North and South Creeks pending the successful performance of the flume on spring N-1.

C. Corrective Measures Required for Project Operation

No changes in project operation are needed at this time.

D. Schedule to Carry Out Suggested Corrective Measures

All recommended improvements should be undertaken within the next two years or as specified above.

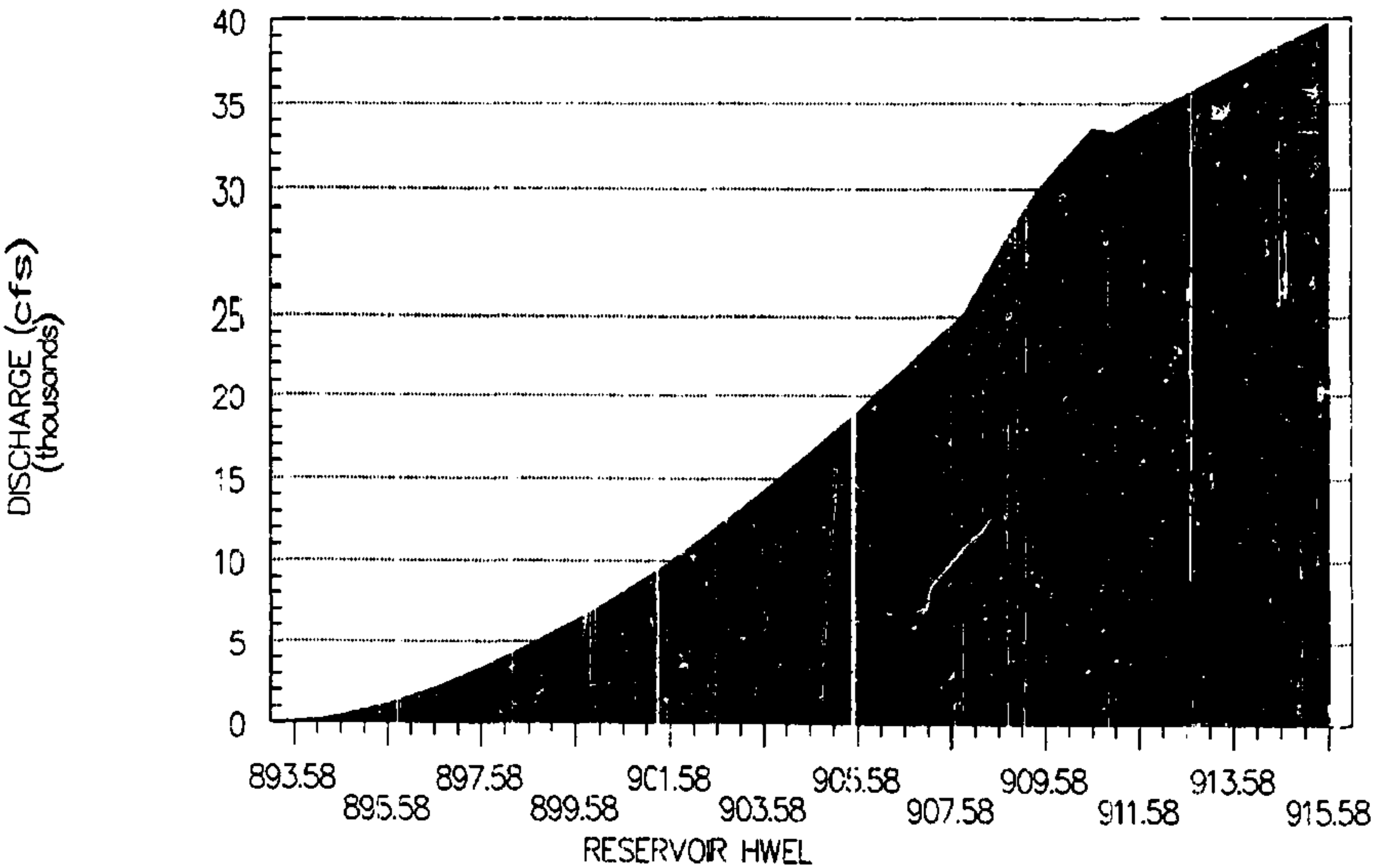
E. Additional Safety Requirements

No other additional remedial safety measures are needed at this time.

APPENDIX D
Project Hydraulic Data

BONEY FALLS DAM

Gated Spillway Rating Curve



SUBJECT: Tainter Gated Spillway
Discharge Rating Curve
(Free Over-fall Conditions)
COMPUTED: M.F. Rogers

PROJECT: Boney Falls
NUMBER: 1979L
DATE: 25-Apr-89
PAGE ____ of ____

BAYS:
Crest EL: 893.58
Width = 20
# Bays = 6

CONTRACTIONS:
K(pier) = 0.00
K(abut) = 0.05
Piers = 5

DESIGN:
Ho = 14.2
EGLo = 907.78
Co = 3.94

RESERVOIR WEL (ft,msl)	Head, He (ft)	He/Ho	Cd/Co	Cd	Leff (ft)	Discharge (cfs)
893.08	-0.50	--	--	--	--	0
893.58	0.00	0.000	0.000	0.00	120.00	0
894.08	0.50	0.035	0.802	3.15	119.95	134
894.58	1.00	0.070	0.810	3.19	119.90	382
895.08	1.50	0.106	0.824	3.24	119.85	713
895.58	2.00	0.141	0.835	3.29	119.80	1,115
896.08	2.50	0.176	0.846	3.33	119.75	1,576
896.58	3.00	0.211	0.856	3.37	119.70	2,096
897.08	3.50	0.246	0.863	3.40	119.65	2,664
897.58	4.00	0.282	0.874	3.44	119.60	3,291
898.08	4.50	0.317	0.883	3.47	119.55	3,960
898.58	5.00	0.352	0.890	3.50	119.50	4,676
899.08	5.50	0.387	0.897	3.53	119.45	5,439
899.58	6.00	0.423	0.905	3.56	119.40	6,247
900.08	6.50	0.458	0.913	3.59	119.35	7,100
900.58	7.00	0.493	0.920	3.62	119.30	7,998
901.08	7.50	0.528	0.927	3.65	119.25	8,940
901.58	8.00	0.563	0.933	3.67	119.20	9,899
902.08	8.50	0.599	0.940	3.70	119.15	10,925
902.58	9.00	0.634	0.945	3.72	119.10	11,962
903.08	9.50	0.669	0.951	3.74	119.05	13,037
903.58	10.00	0.704	0.958	3.77	119.00	14,187
904.08	10.50	0.739	0.962	3.79	118.95	15,339
904.58	11.00	0.775	0.969	3.81	118.90	16,527
905.08	11.50	0.810	0.974	3.83	118.85	17,732
905.58	12.00	0.845	0.979	3.85	118.80	19,013
906.08	12.50	0.880	0.985	3.88	118.75	20,362
906.58	13.00	0.915	0.990	3.89	118.70	21,643
907.08	13.50	0.951	0.993	3.91	118.65	23,012
907.58	14.00	0.986	0.998	3.93	118.60	24,416
908.08	14.50	1.021	1.002	3.94	118.55	25,790
908.58	15.00	1.056	1.004	3.95	118.50	27,195
909.08	15.50	1.092	1.010	3.97	118.45	28,696
909.58	16.00	1.127	1.015	4.00	118.40	30,310
910.08	16.50	1.162	1.020	4.01	118.35	31,808
910.58	17.00	1.197	1.024	4.03	118.30	33,417
911.08	17.50	1.232	1.023	4.04	118.25	34,974

SUBJECT: Tainter Gated Spillway
Discharge Rating Curve
(Orifice Flow Conditions)
COMPUTED: M.F. Rogers

PROJECT: Boney Falls
NUMBER: 1979L
DATE: 25-Apr-89
PAGE ____ of ____

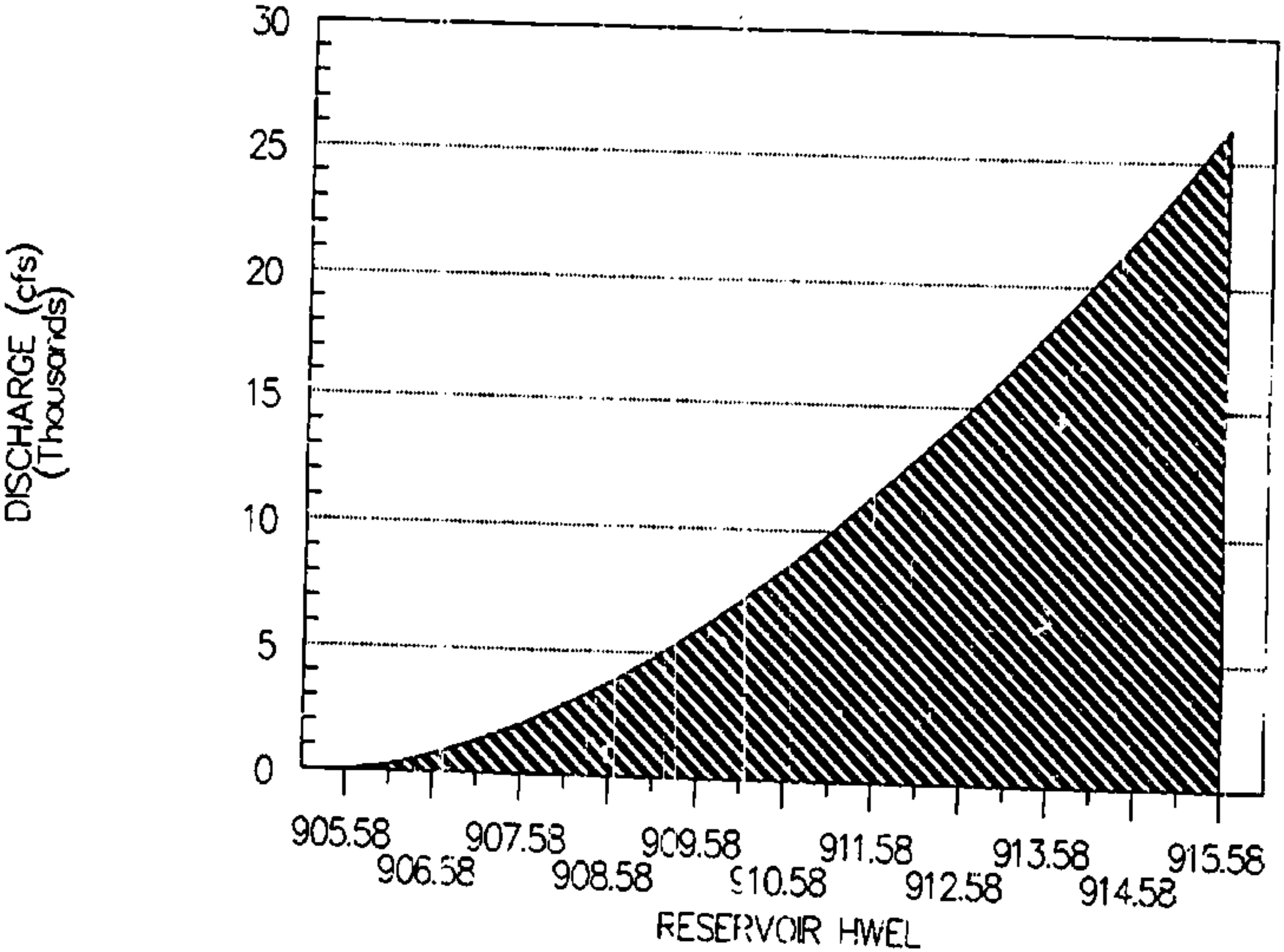
BAYS:
Crest EL 893.58
Width = 20 ft.
# Bays= 6
Cd = 0.76

MAX OPENING:
Height = 14.0 ft
Area = 280 ft^2/bay
Lip EL 907.58
Center EL 900.58

RESERVOIR WEL (ft,msl)	Head, He (ft)	Discharge (cfs) (1 Bay)	DISCHARGE (cfs) (all bays)
911.08	10.50	5,534	33,204
911.58	11.00	5,664	33,984
912.08	11.50	5,791	34,746
912.58	12.00	5,916	35,496
913.08	12.50	6,038	36,228
913.58	13.00	6,157	36,942
914.08	13.50	6,275	37,650
914.58	14.00	6,390	38,340
915.08	14.50	6,503	39,018
915.58	15.00	6,614	39,684
916.08	15.50	6,723	40,338
916.58	16.00	6,831	40,986
917.08	16.50	6,937	41,622
917.58	17.00	7,041	42,246
918.08	17.50	7,144	42,864
918.58	18.00	7,245	43,470
919.08	18.50	7,345	44,070
919.58	19.00	7,444	44,664
920.08	19.50	7,541	45,246

BONEY FALLS DAM

Ungated Spillway Rating Curve



SUBJECT: Ungated Spillway
Discharge Rating Curve
(Free Overfall Conditions)
COMPUTED: N.F. Rogers

PROJECT: Boney Falls
NUMBER: 1979L
DATE: 25-Apr-89
PAGE ____ of ____

BAYS:
Crest EL 905.58
Width = 200
# Bays = 1

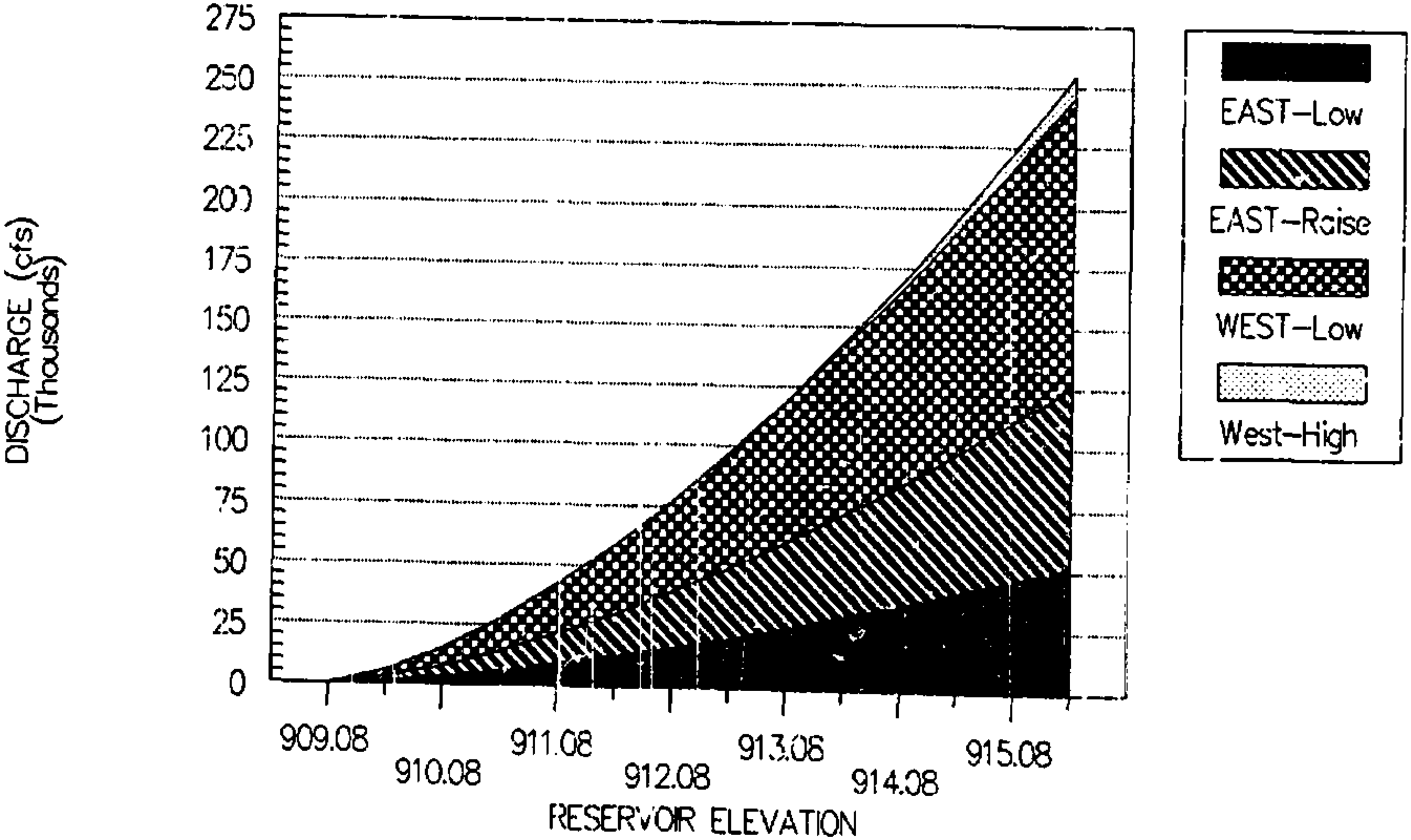
CONTRACTIONS:
K(pier)= 0.00
K(abut)= 0.05
Piers= 0

DESIGN:
Ho = 6.70
EGLo = 912.28
Co = 3.95

RESERVOIR WSEL (ft,msl)	Head, He (ft)	He/Ho	Cd/Co	Cd	Leff (ft)	Discharge (cfs)
905.58	0.00	0.000	0.000	0.00	200.00	0
906.08	0.50	0.075	0.812	3.20	199.95	226
906.58	1.00	0.149	0.838	3.30	199.90	660
907.08	1.50	0.224	0.859	3.39	199.85	1,205
907.58	2.00	0.299	0.879	3.47	199.80	1,961
908.08	2.50	0.373	0.895	3.53	199.75	2,797
908.58	3.00	0.448	0.911	3.59	199.70	3,725
909.08	3.50	0.522	0.925	3.65	199.65	4,772
909.58	4.00	0.597	0.939	3.71	199.60	5,924
910.08	4.50	0.672	0.952	3.76	199.55	7,162
910.58	5.00	0.746	0.963	3.80	199.50	8,476
911.08	5.50	0.821	0.975	3.85	199.45	9,905
911.58	6.00	0.896	0.987	3.90	199.40	11,429
912.08	6.50	0.970	0.996	3.93	199.35	12,983
912.58	7.00	1.045	1.004	3.96	199.30	14,617
913.08	7.50	1.119	1.014	4.00	199.25	16,370
913.58	8.00	1.194	1.023	4.04	199.20	18,210
914.08	8.50	1.269	1.033	4.08	199.15	20,136
914.58	9.00	1.343	1.041	4.11	199.10	22,094
915.08	9.50	1.418	1.050	4.14	199.05	24,129
915.58	10.00	1.493	1.059	4.18	199.00	26,304
916.08	10.50	1.567	1.067	4.21	198.95	28,498
916.58	11.00	1.642	1.074	4.24	198.90	30,767
917.08	11.50	1.716	1.082	4.27	198.85	33,113
917.58	12.00	1.791	1.089	4.30	198.80	35,535
918.08	12.50	1.866	1.097	4.33	198.75	38,033
918.58	13.00	1.940	1.104	4.36	198.70	40,607
919.08	13.50	2.015	1.112	4.39	198.65	43,257
919.58	14.00	2.090	1.119	4.42	198.60	45,983
920.08	14.50	2.164	1.126	4.44	198.55	48,675

BONEY FALLS DAM

Flow Over Embankment–EXISTING



SUBJECT: Embankment Overtopping Flow
Discharge Rating Curve
EXISTING CONDITIONS
COMPUTED: M.F. Rogers

PROJECT: Boney Falls
NUMBER: 1979L
DATE: 01-Jan-89
PAGE ___ of ___

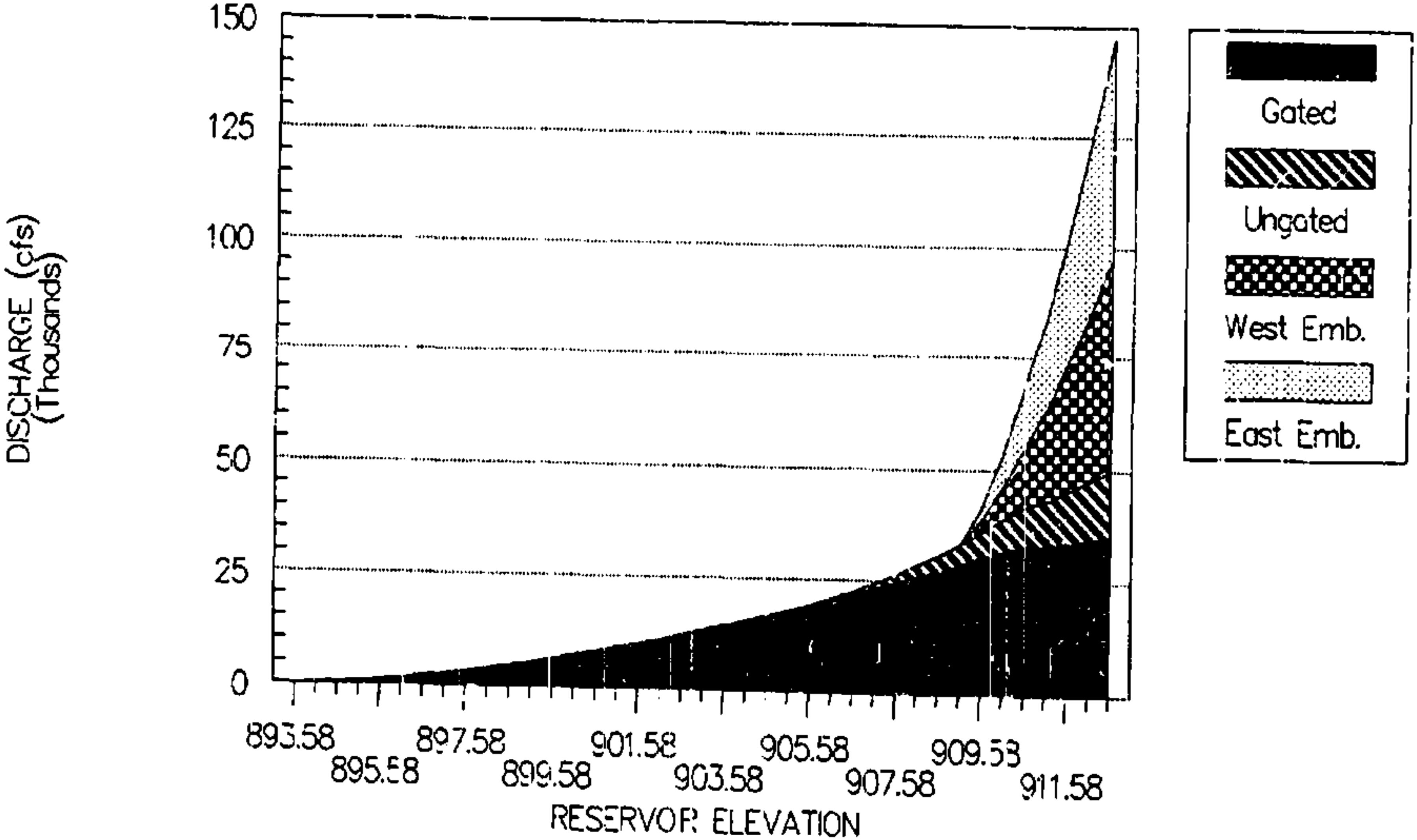
WEST EMBANKMENT DAM:
- Sta. 2+00 to 8+40: Crest EL 912.58, Length 640, Cd = 2.67
- Sta. 8+40 to 35+60: Crest EL 909.08, Length 2720 ft., Cd = 2.67

EAST EMBANKMENT DAM:
- Sta. 0+00 to 16+65: Crest EL 909.08, Length 1665, Cd = 2.67
- Sta. 16+65 to 57+00: Crest EL: 909.08, Eff.Length 1735 ft., Cd = 1.8

RESERVOIR HWEL (ft,msl)	HIGH PORTION H (ft)	HIGH PORTION Q (cfs)	LOW PORTION H (ft)	LOW PORTION Q (cfs)	COMBINED Discharge (cfs)	RAISED PORTION H (ft)	RAISED PORTION Q (cfs)	LOW PORTION H (ft)	LOW PORTION Q (cfs)	COMBINED Discharge (cfs)	TOTAL COMBINED DISCHARGE (cfs)	RESERVOIR HWEL (ft,msl)
909.08	0.00	0	0.00	0	0	0.00	0	0.00	0	0	0	909.08
909.58	0.00	0	0.50	2,568	2,568	0.50	1,572	0.50	1,104	2,676	5,244	909.58
910.08	0.00	0	1.00	7,262	7,262	1.00	4,446	1.00	3,123	7,569	14,831	910.08
910.58	0.00	0	1.50	13,342	13,342	1.50	8,167	1.50	5,737	13,904	27,246	910.58
911.08	0.00	0	2.00	20,541	20,541	2.00	12,574	2.00	8,833	21,407	41,948	911.08
911.58	0.00	0	2.50	28,707	28,707	2.50	17,573	2.50	12,345	29,918	58,625	911.58
912.08	0.00	0	3.00	37,737	37,737	3.00	23,100	3.00	16,228	39,328	77,065	912.08
912.58	0.00	0	3.50	47,553	47,553	3.50	29,109	3.50	20,449	49,558	97,111	912.58
913.08	0.50	604	4.00	58,099	58,703	4.00	35,564	4.00	24,984	60,548	119,251	913.08
913.58	1.00	1,709	4.50	69,326	71,035	4.50	42,437	4.50	29,812	72,249	143,284	913.58
914.08	1.50	3,139	5.00	81,196	84,335	5.00	49,703	5.00	34,916	84,619	168,954	914.08
914.58	2.00	4,833	5.50	93,675	98,508	5.50	57,342	5.50	40,282	97,624	196,132	914.58
915.08	2.50	6,755	6.00	106,735	113,490	6.00	65,336	6.00	45,899	111,235	224,725	915.08
915.58	3.00	8,879	6.50	120,351	129,230	6.50	73,671	6.50	51,754	125,425	254,655	915.58
916.08	3.50	11,189	7.00	134,502	145,691	7.00	82,333	7.00	57,839	140,172	285,863	916.08
916.58	4.00	13,670	7.50	149,167	162,837	7.50	91,310	7.50	64,145	155,455	318,292	916.58
917.08	4.50	16,312	8.00	164,329	180,641	8.00	100,591	8.00	70,665	171,256	351,897	917.08
917.58	5.00	19,105	8.50	179,973	199,078	8.50	110,168	8.50	77,393	187,561	386,639	917.58
918.08	5.50	22,041	9.00	196,085	218,126	9.00	120,030	9.00	84,321	204,351	422,477	918.08
918.58	6.00	25,114	9.50	212,650	237,764	9.50	130,170	9.50	91,444	221,614	459,378	918.58
919.08	6.50	28,318	10.00	229,657	257,975	10.00	140,581	10.00	98,758	239,339	497,314	919.08
919.58	7.00	31,647	10.50	247,095	278,742	10.50	151,255	10.50	106,257	257,512	536,254	919.58
920.08	7.50	35,098	11.00	264,953	300,051	11.00	162,186	11.00	113,936	276,122	576,173	920.08

BONEY FALLS DAM

EXISTING Project Discharge Rating Curve



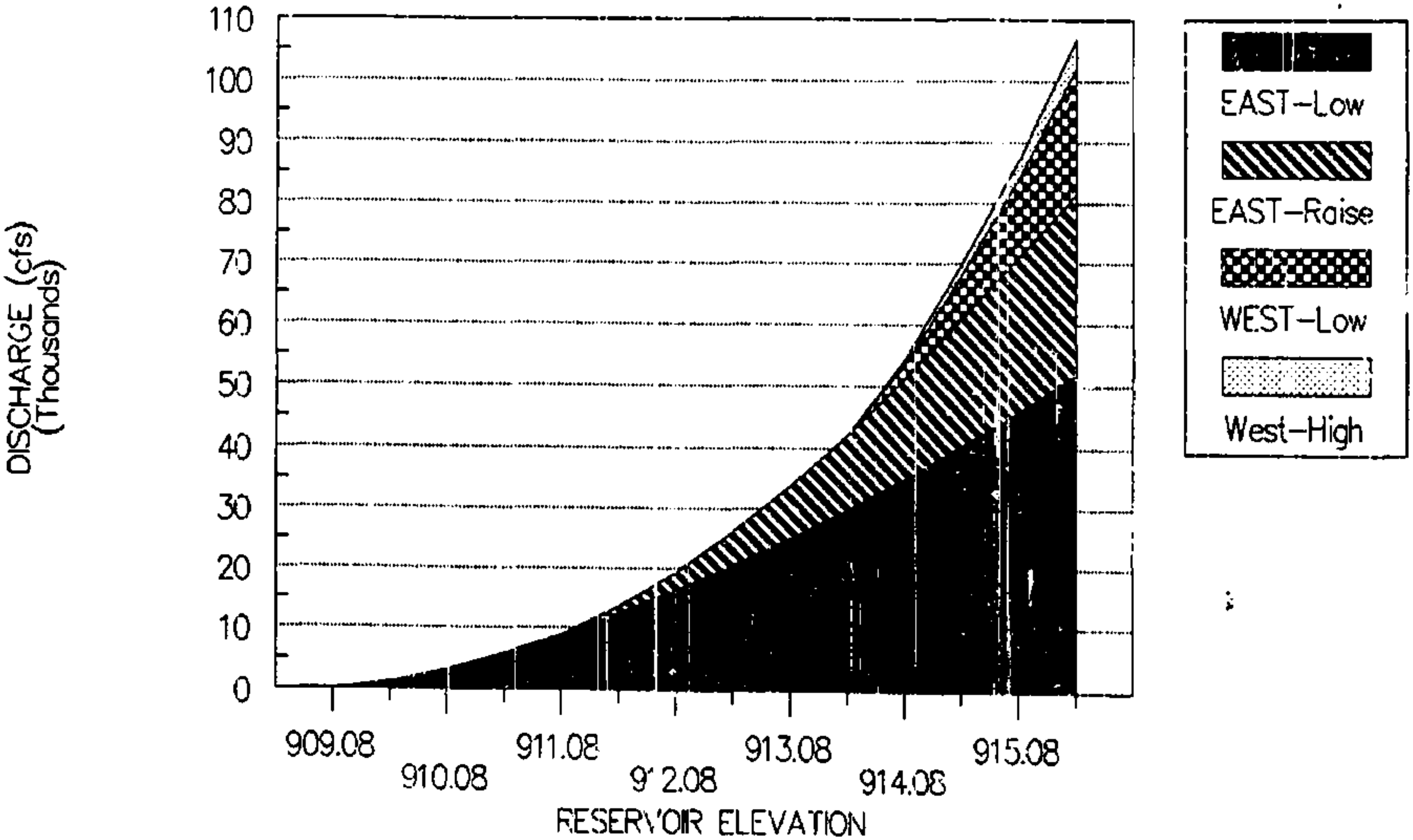
SUBJECT: Project Discharge Capacity
Headwater vs. Discharge Rating Curve
EXISTING CONDITIONS
COMPUTED: M.F. Rogers

PROJECT: Boney Falls
NUMBER: 1979L
DATE: 01-May-89
PAGE ___ of ___

RESERVOIR NWEL (ft,msl)	GATED SPILLWAY Flow Control	GATED SPILLWAY Discharge (cfs)	UNGATED Discharge (cfs)	WEST EMBANKMENT Discharge (cfs)	EAST EMBANKMENT Discharge (cfs)	EXISTING PROJECT DISCHARGE (cfs)	RESERVOIR NWEL (ft,msl)
893.08	Free	0	0	0	0	0	893.08
893.58	Free	0	0	0	0	0	893.58
894.08	Free	134	0	0	0	134	894.08
894.58	Free	382	0	0	0	382	894.58
895.08	Free	713	0	0	0	713	895.08
895.58	Free	1,115	0	0	0	1,115	895.58
896.08	Free	1,576	0	0	0	1,576	896.08
896.58	Free	2,096	0	0	0	2,096	896.58
897.08	Free	2,664	0	0	0	2,664	897.08
897.58	Free	3,291	0	0	0	3,291	897.58
898.08	Free	3,960	0	0	0	3,960	898.08
898.58	Free	4,676	0	0	0	4,676	898.58
899.08	Free	5,439	0	0	0	5,439	899.08
899.58	Free	6,247	0	0	0	6,247	899.58
900.08	Free	7,100	0	0	0	7,100	900.08
900.58	Free	7,998	0	0	0	7,998	900.58
901.08	Free	8,940	0	0	0	8,940	901.08
901.58	Free	9,899	0	0	0	9,899	901.58
902.08	Free	10,925	0	0	0	10,925	902.08
902.58	Free	11,962	0	0	0	11,962	902.58
903.08	Free	13,037	0	0	0	13,037	903.08
903.58	Free	14,187	0	0	0	14,187	903.58
904.08	Free	15,339	0	0	0	15,339	904.08
904.58	Free	16,527	0	0	0	16,527	904.58
905.08	Free	17,752	0	0	0	17,752	905.08
905.58	Free	19,013	0	0	0	19,013	905.58
906.08	Free	20,362	226	0	0	20,58C	906.08
906.58	Free	21,643	660	0	0	22,303	906.58
907.08	Free	23,012	1,245	0	0	24,257	907.08
907.58	Free	24,416	1,961	0	0	26,377	907.58
908.08	Free	25,790	2,787	0	0	28,577	908.08
908.58	Free	27,193	3,725	0	0	30,918	908.58
909.08	Free	28,696	4,772	0	0	33,468	909.08
909.58	Free	30,310	5,924	2,568	2,676	41,478	909.58
910.08	Free	31,808	7,162	7,262	7,569	53,801	910.08
910.58	Free	33,417	8,476	13,342	13,904	69,139	910.58
911.08	Orifice	33,204	9,905	20,541	21,407	85,057	911.08
911.58	Orifice	33,984	11,429	28,707	29,918	104,038	911.58
912.08	Orifice	34,746	12,983	37,737	39,328	124,794	912.08
912.58	Orifice	35,496	14,617	47,553	49,558	147,224	912.58
913.08	Orifice	36,228	16,370	58,703	60,548	171,849	913.08
913.58	Orifice	36,942	18,210	71,035	72,249	198,436	913.58
914.08	Orifice	37,650	20,136	84,335	84,619	226,740	914.08
914.58	Orifice	38,340	22,094	98,508	97,624	256,566	914.58
915.08	Orifice	39,018	24,129	113,490	111,235	287,872	915.08
915.58	Orifice	39,684	26,304	129,230	125,425	320,643	915.58
916.08	Orifice	40,338	28,498	145,691	140,172	354,699	916.08
916.58	Orifice	40,986	30,767	162,837	155,455	390,045	916.58
917.08	Orifice	41,622	33,113	180,641	171,236	426,632	917.08
917.58	Orifice	42,246	35,535	199,078	187,561	464,420	917.58
918.08	Orifice	42,864	38,033	218,126	204,351	503,374	918.08
918.58	Orifice	43,470	40,607	237,764	221,614	543,455	918.58
919.08	Orifice	44,070	43,257	257,975	239,339	584,641	919.08
919.58	Orifice	44,664	45,983	278,742	257,512	626,901	919.58
920.08	Orifice	45,246	48,675	300,051	276,122	670,094	920.08

BONEY FALLS DAM

Flow Over Embankments after Raising



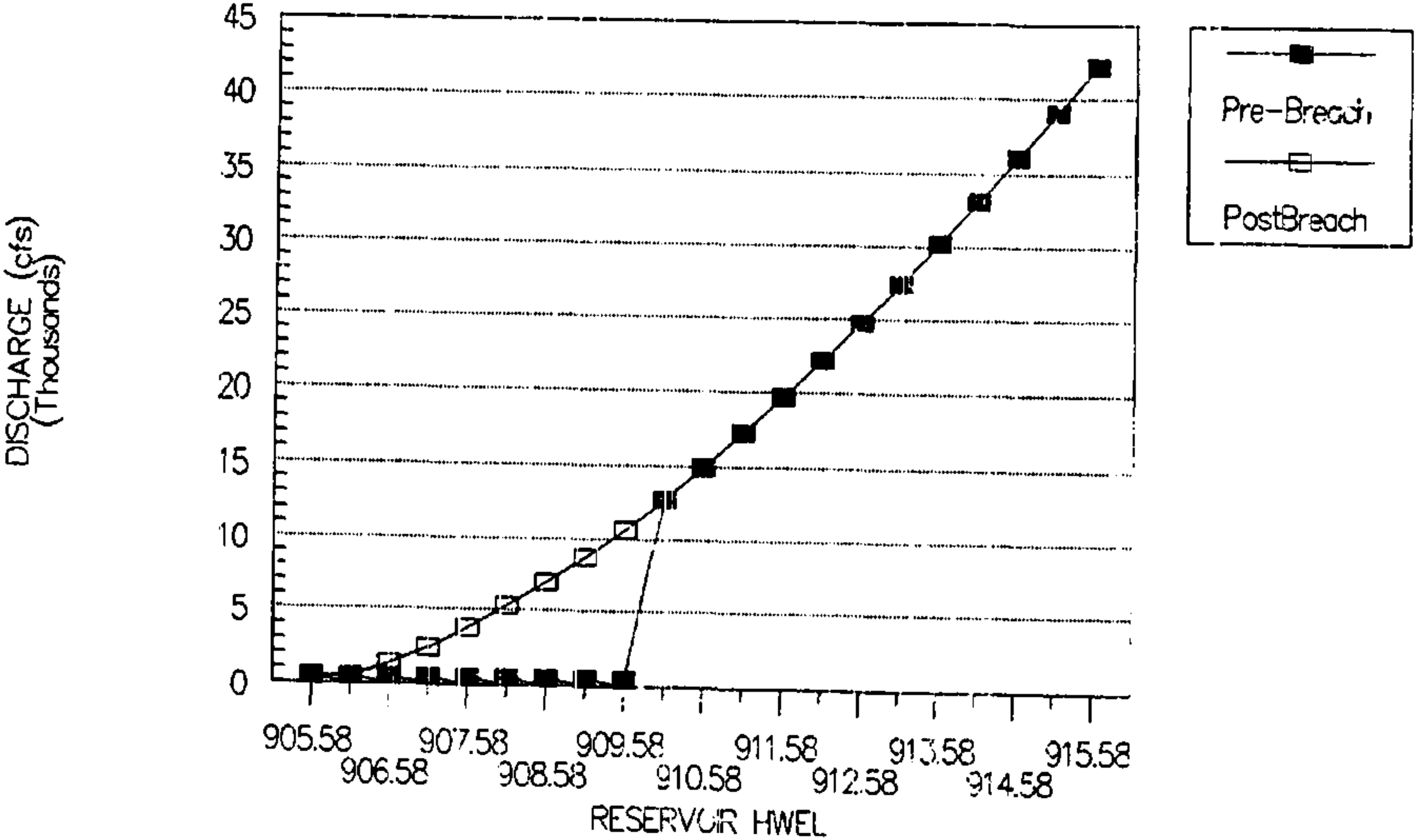
SUBJECT: Embankment Overtopping Flow
Discharge Rating Curve
AFTER SPILLWAY EXPANSION
COMPUTED: M.F. Rogers

PROJECT: Borey Falls
NUMBER: 1979L
DATE: 01-Jan-89
PAGE ___ of ___

	WEST EMBANKMENT DAM: Sta. 2+00 to 8+40, Crest EL 913.58, Length 640, Cd = 2.67		WEST EMBANKMENT DAM: Sta. 8+40 to 35+60, Crest EL 913.58, Length 2720 ft., Cd = 2.67			EAST EMBANKMENT DAM: Sta. 5+00 to 16+65, Crest EL 911.08, Length 1165, Cd = 2.67		EAST EMBANKMENT DAM: Sta. 16+65 to 37+00, Crest EL: 909.08, Eff.Length 1735 ft., Cd = 1.6				
RESERVOIR WSEL (ft,msl)	HIGH PORTION H (ft)	HIGH PORTION Q (cfs)	LOW PORTION H (ft)	LOW PORTION Q (cfs)	COMBINED Discharge (cfs)	RAISED PORTION H (ft)	RAISED PORTION Q (cfs)	LOW PORTION H (ft)	LOW PORTION Q (cfs)	COMBINED Discharge (cfs)	TOTAL COMBINED DISCHARGE (cfs)	RESERVOIR WSEL (ft,msl)
909.08	0.00	0	0.00	0	0	0.00	0	0.00	0	0	0	909.08
909.58	0.00	0	0.00	0	0	0.00	0	0.50	1,104	1,104	1,104	909.58
910.08	0.00	0	0.00	0	0	0.00	0	1.00	3,123	3,123	3,123	910.08
910.58	0.00	0	0.00	0	0	0.00	0	1.50	5,737	5,737	5,737	910.58
911.08	0.00	0	0.00	0	0	0.00	0	2.00	8,833	8,833	8,833	911.08
911.58	0.00	0	0.00	0	0	0.50	1,100	2.50	12,345	13,445	13,445	911.58
912.08	0.00	0	0.00	0	0	1.00	3,111	3.00	16,228	19,339	19,339	912.08
912.58	0.00	0	0.00	0	0	1.50	5,714	3.50	20,449	26,163	26,163	912.58
913.08	0.00	0	0.00	0	0	2.00	8,798	4.00	24,984	33,782	33,782	913.08
913.58	0.00	0	0.00	0	0	2.50	12,296	4.50	29,812	42,108	42,108	913.58
914.08	0.50	604	0.50	2,568	3,172	3.00	16,163	5.00	34,916	51,079	54,251	914.08
914.58	1.00	1,709	1.00	7,262	8,971	3.50	20,368	5.50	40,282	60,650	69,621	914.58
915.08	1.50	3,139	1.50	13,342	16,481	4.00	24,884	6.00	45,899	70,783	87,264	915.08
915.58	2.00	4,833	2.00	20,541	25,374	4.50	29,693	6.50	51,754	81,447	106,821	915.58
916.08	2.50	6,755	2.50	28,707	35,462	5.00	34,777	7.00	57,839	92,615	128,078	916.08
916.58	3.00	8,879	3.00	37,737	46,616	5.50	40,122	7.50	64,145	104,267	150,883	916.58
917.08	3.50	11,189	3.50	47,553	58,742	6.00	45,716	8.00	70,665	116,381	175,123	917.08
917.58	4.00	13,670	4.00	58,099	71,769	6.50	51,547	8.50	77,393	128,940	200,709	917.58
918.08	4.50	16,312	4.50	69,326	85,638	7.00	57,608	9.00	84,321	141,929	227,567	918.08
918.58	5.00	19,105	5.00	81,196	100,301	7.50	63,889	9.50	91,444	155,333	255,634	918.58
919.08	5.50	22,041	5.50	93,675	115,716	8.00	70,384	10.00	98,758	169,142	284,858	919.08
919.58	6.00	25,114	6.00	106,735	131,849	8.50	77,084	10.50	106,257	183,341	315,190	919.58
920.08	6.50	28,318	6.50	120,351	148,669	9.00	83,985	11.00	113,934	197,921	346,590	920.08

BONEY FALLS DAM

New RCC Spillway Rating Curve



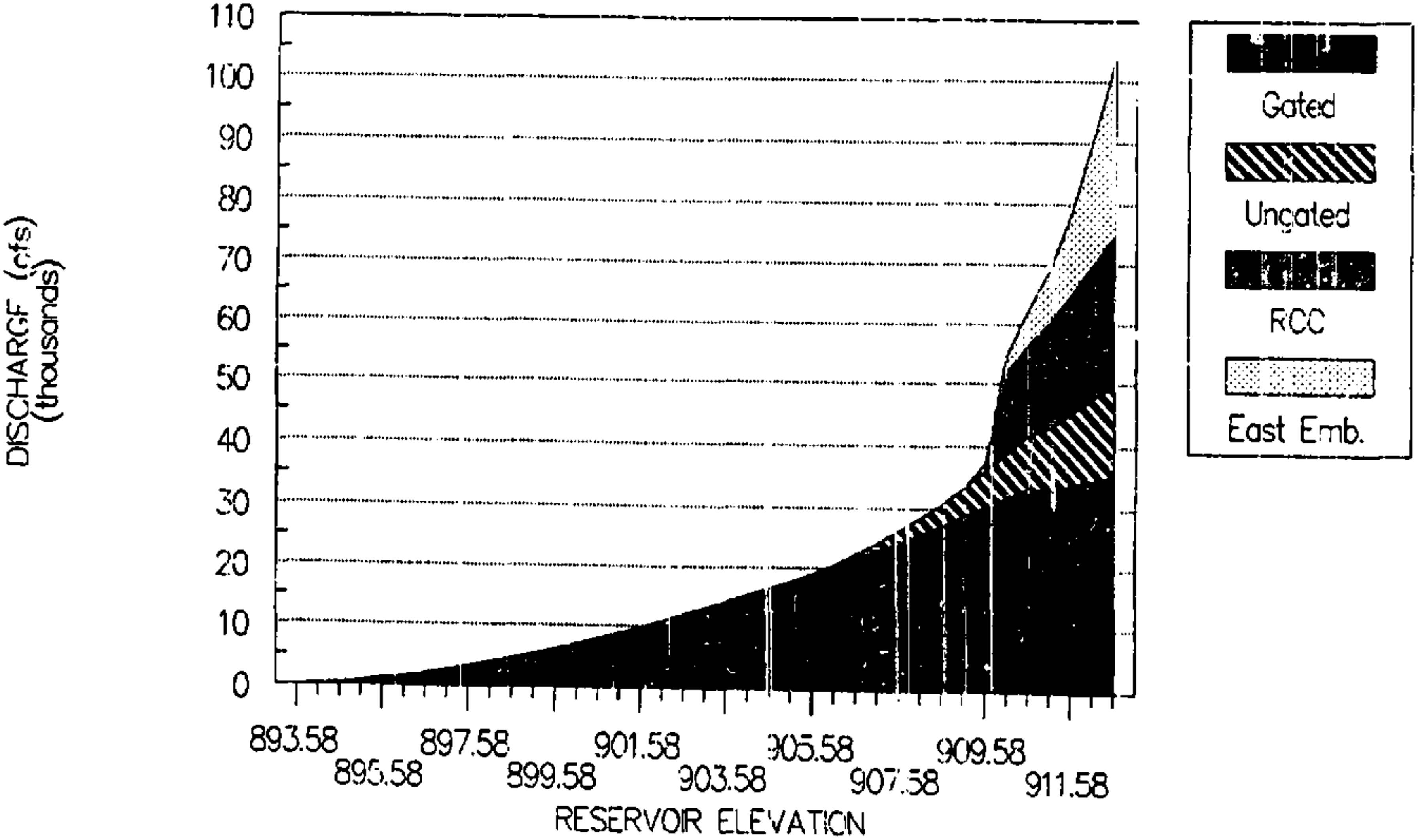
SUBJECT:	Roller Compacted Concrete Emergency Spillway Discharge Rating Curve	PROJECT:	Honey Falls
		NUMBER:	1979L
		DATE:	01-Jan-89
COMPUTED:	H.F. Rogers	PAGE ____ of ____	

EAST EMBANKMENT DAM
Sta. 0+00 to 5+00 Top of Dam: 910.08
Crest EL: 905.58
Length 500
Cd = 2.67

RESERVOIR WEL (ft,msl)	Head (ft)	Cd	DISCHARGE (cfs) Prior to Breach	After Breach
905.58	0.00	2.67	0	0
906.08	0.50	2.67	0	472
906.58	1.00	2.67	0	1,335
907.08	1.50	2.67	0	2,453
907.58	2.00	2.67	0	3,776
908.08	2.50	2.67	0	5,277
908.58	3.00	2.67	0	6,937
909.08	3.50	2.67	0	8,741
909.58	4.00	2.67	0	10,680
910.08	4.50	2.67	12,744	12,744
910.58	5.00	2.67	14,926	14,926
911.08	5.50	2.67	17,220	17,220
911.58	6.00	2.67	19,620	19,620
912.08	6.50	2.67	22,123	22,123
912.58	7.00	2.67	24,725	24,725
913.08	7.50	2.67	27,420	27,420
913.58	8.00	2.67	30,208	30,208
914.08	8.50	2.67	33,083	33,083
914.58	9.00	2.67	36,045	36,045
915.08	9.50	2.67	39,090	39,090
915.58	10.00	2.67	42,216	42,216
916.08	10.50	2.67	45,422	45,422
916.58	11.00	2.67	48,705	48,705
917.08	11.50	2.67	52,063	52,063
917.58	12.00	2.67	55,495	55,495
918.08	12.50	2.67	58,999	58,999
918.58	13.00	2.67	62,574	62,574
919.08	13.50	2.67	66,219	66,219
919.58	14.00	2.67	69,932	69,932
920.08	14.50	2.67	73,711	73,711

BONEY FALLS DAM

New Project Discharge Rating Curve



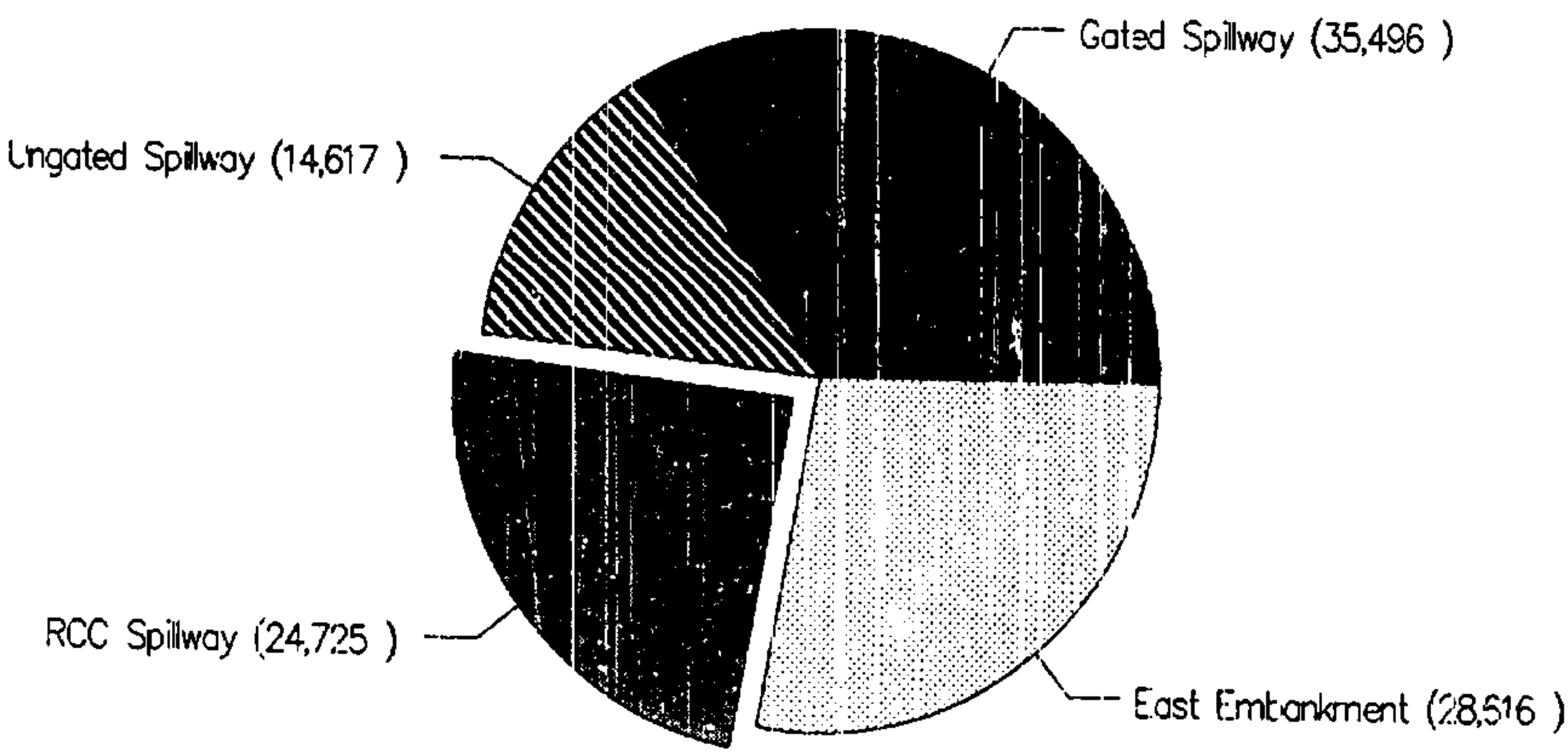
SUBJECT: Project Discharge Capacity
Headwater vs. Discharge Rating Curve
AFTER SPILLWAY EXPANSION
COMPUTED: M.F. Rogers

PROJECT: Boney Falls
NUMBER: 1979L
DATE: 01-May-89
PAGE ___ of ___

RESERVOIR NWEL (ft,msl)	GATED SPILLWAY Flow Control	GATED SPILLWAY Discharge (cfs)	UNGATED Discharge (cfs)	WEST EMBANKMENT Discharge (cfs)	EAST EMBANKMENT Discharge (cfs)	RCC Discharge (cfs)	PROJECT DISCHARGE (cfs)	RESERVOIR NWEL (ft,msl)
893.08	Free	0	0	0	0	0	0	893.08
893.58	Free	0	0	0	0	0	0	893.58
894.08	Free	134	0	0	0	0	134	894.08
894.58	Free	382	0	0	0	0	382	894.58
895.08	Free	713	0	0	0	0	713	895.08
895.58	Free	1,115	0	0	0	0	1,115	895.58
896.08	Free	1,576	0	0	0	0	1,576	896.08
896.58	Free	2,096	0	0	0	0	2,096	896.58
897.08	Free	2,664	0	0	0	0	2,664	897.08
897.58	Free	3,291	0	0	0	0	3,291	897.58
898.08	Free	3,960	0	0	0	0	3,960	898.08
898.58	Free	4,676	0	0	0	0	4,676	898.58
899.08	Free	5,439	0	0	0	0	5,439	899.08
899.58	Free	6,247	0	0	0	0	6,247	899.58
900.08	Free	7,100	0	0	0	0	7,100	900.08
900.58	Free	7,998	0	0	0	0	7,998	900.58
901.08	Free	8,940	0	0	0	0	8,940	901.08
901.58	Free	9,899	0	0	0	0	9,899	901.58
902.08	Free	10,925	0	0	0	0	10,925	902.08
902.58	Free	11,962	0	0	0	0	11,962	902.58
903.08	Free	13,037	0	0	0	0	13,037	903.08
903.58	Free	14,187	0	0	0	0	14,187	903.58
904.08	Free	15,339	0	0	0	0	15,339	904.08
904.58	Free	16,527	0	0	0	0	16,527	904.58
905.08	Free	17,752	0	0	0	0	17,752	905.08
905.58	Free	19,013	0	0	0	0	19,013	905.58
906.08	Free	20,362	226	0	0	0	20,588	906.08
906.58	Free	21,643	660	0	0	0	22,303	906.58
907.08	Free	23,012	1,245	0	0	0	24,257	907.08
907.58	Free	24,416	1,961	0	0	0	26,377	907.58
908.08	Free	25,790	2,787	0	0	0	28,577	908.08
908.58	Free	27,193	3,725	0	0	0	30,918	908.58
909.08	Free	28,696	4,772	0	0	0	33,468	909.08
909.58	Free	30,310	5,924	0	1,104	0	37,338	909.58
910.08	Free	31,808	7,162	0	3,123	12,744	54,837	910.08
910.58	Free	33,417	8,476	0	5,737	14,926	62,556	910.58
911.08	Orifice	33,204	9,905	0	8,833	17,220	69,162	911.08
911.58	Orifice	33,984	11,429	0	13,917	19,620	78,950	911.58
912.08	Orifice	34,746	12,983	0	20,674	22,123	90,526	912.08
912.58	Orifice	35,496	14,617	0	28,616	24,725	103,454	912.58
913.08	Orifice	36,228	16,370	0	37,558	27,420	117,576	913.08
913.58	Orifice	36,942	18,210	0	47,385	30,208	132,745	913.58
914.08	Orifice	37,650	20,136	3,172	58,016	33,083	152,057	914.08
914.58	Orifice	38,340	22,094	8,971	69,391	36,045	174,841	914.58
915.08	Orifice	39,018	24,129	16,481	81,463	39,090	200,181	915.08
915.58	Orifice	39,684	26,304	25,374	94,191	42,216	227,769	915.58
916.08	Orifice	40,338	28,498	35,462	107,542	45,422	257,262	916.08
916.58	Orifice	40,986	30,767	46,616	121,487	48,705	288,561	916.58
917.08	Orifice	41,622	33,113	58,742	136,001	52,063	321,541	917.08
917.58	Orifice	42,246	35,535	71,769	151,064	55,495	356,109	917.58
918.08	Orifice	42,864	38,033	85,638	166,654	58,999	392,188	918.08
918.58	Orifice	43,470	40,607	100,301	182,754	62,574	429,706	918.58
919.08	Orifice	44,070	43,257	115,716	199,349	66,219	468,611	919.08
919.58	Orifice	44,664	45,983	131,849	216,425	69,932	508,853	919.58
920.08	Orifice	45,246	48,675	148,669	233,966	73,711	550,267	920.08

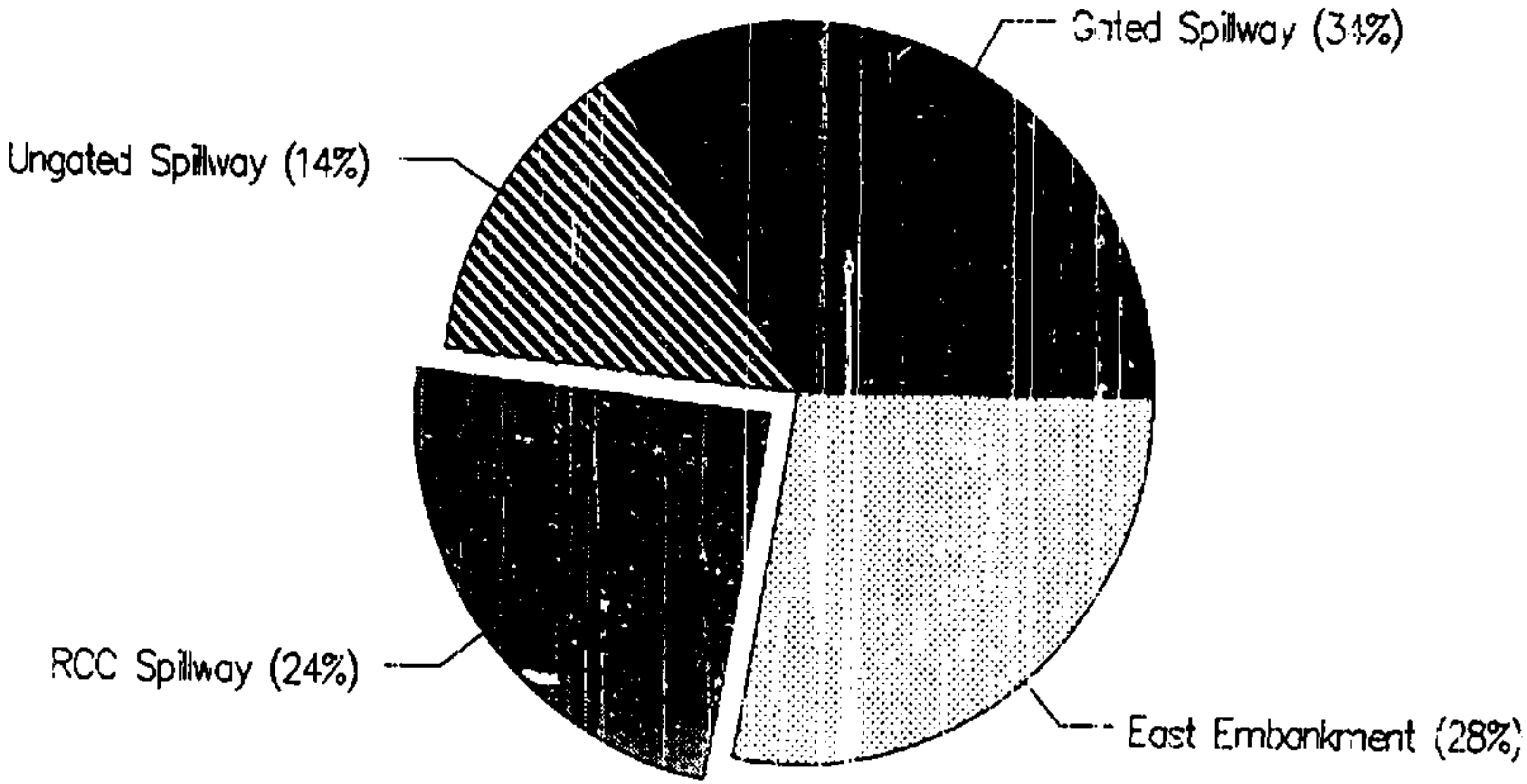
BONEY FALLS DAM

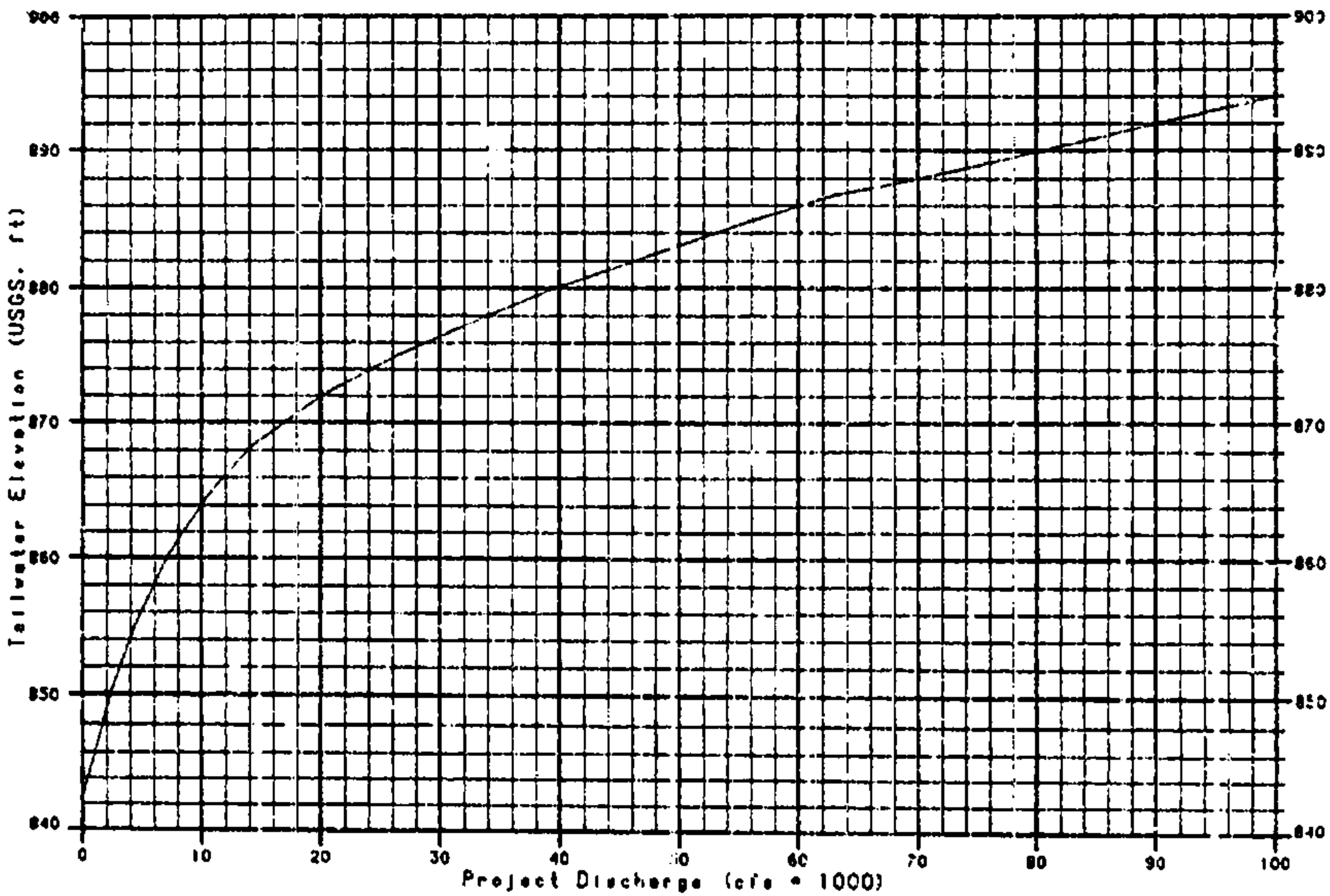
Discharge Contribution at HWEL.912.58



BONEY FALLS DAM

Discharge Contribution at HWEL.912.58





Source : Barr Engineering Co.

MEAD CORPORATION
PUBLISHING PAPER DIVISION
RCC SPILLWAY | BONEY FALLS DAM
TAILWATER RATING CURVE
HARZA ENGINEERING COMPANY

APPENDIX E
RCC Spillway Section Stability Analyses

OBJECTIVE: Verify stability of RCC spillway section.

REFERENCES: [1] Harza Design Guide, DG-16 and DG-29.

[2] "Engineering Guidelines for Evaluation of Hydropower Projects", FERC, July, 1987.

[3] "Geotechnical Investigations Report," HECo, Oct. 88

ASSUMPTIONS:

(1.) Two dimensional analysis with unit width at maximum section.

(2.) Neglect effects of ½ Cornwell RCC berm.

(3) Full uplift.

(4.) Unit Weights:

RCC	140 lbs/cf
soil (dry)	95 lbs/cf
soil (satur)	59 lbs/cf.
water	62.5 lbs/cf.

(5) Sliding Friction Parameters. (based on field tests) [3]

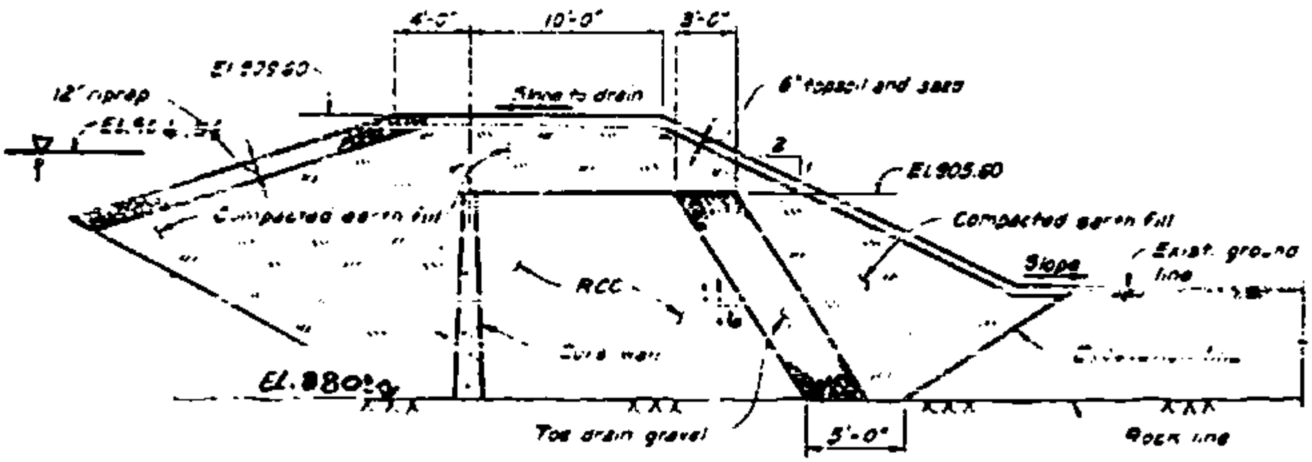
$f = \tan(22.8°) = 0.42$

$C = 70 psi$

(6) Loading Conditions

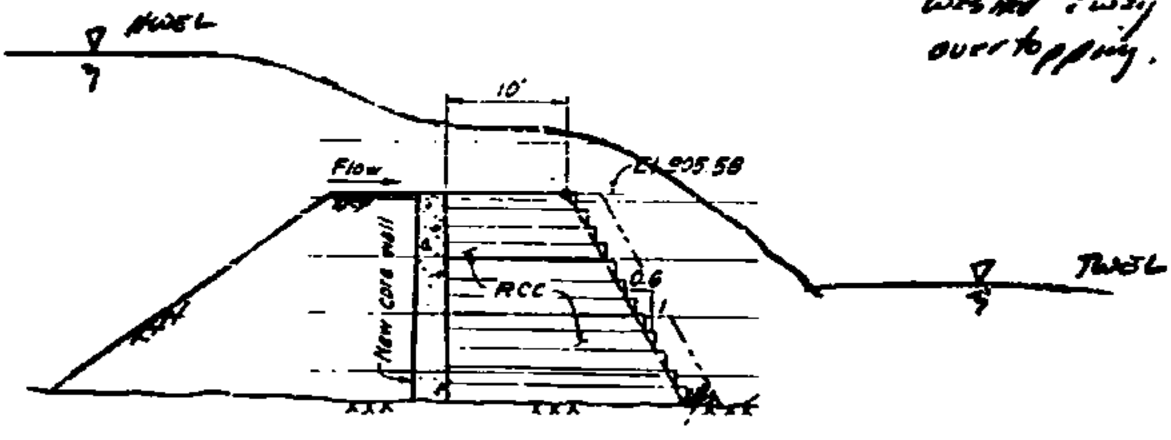
	HWEL	TWEL
I. Normal Pool	906.58	851.58
III. NP + Earthquake (0.1g)	906.58	857.58
IVa IDF = 100,000 cfs	912.58	894.58
b PMF = 147,000 cfs	915.2	905.58

TYPICAL SECTION (... breach)

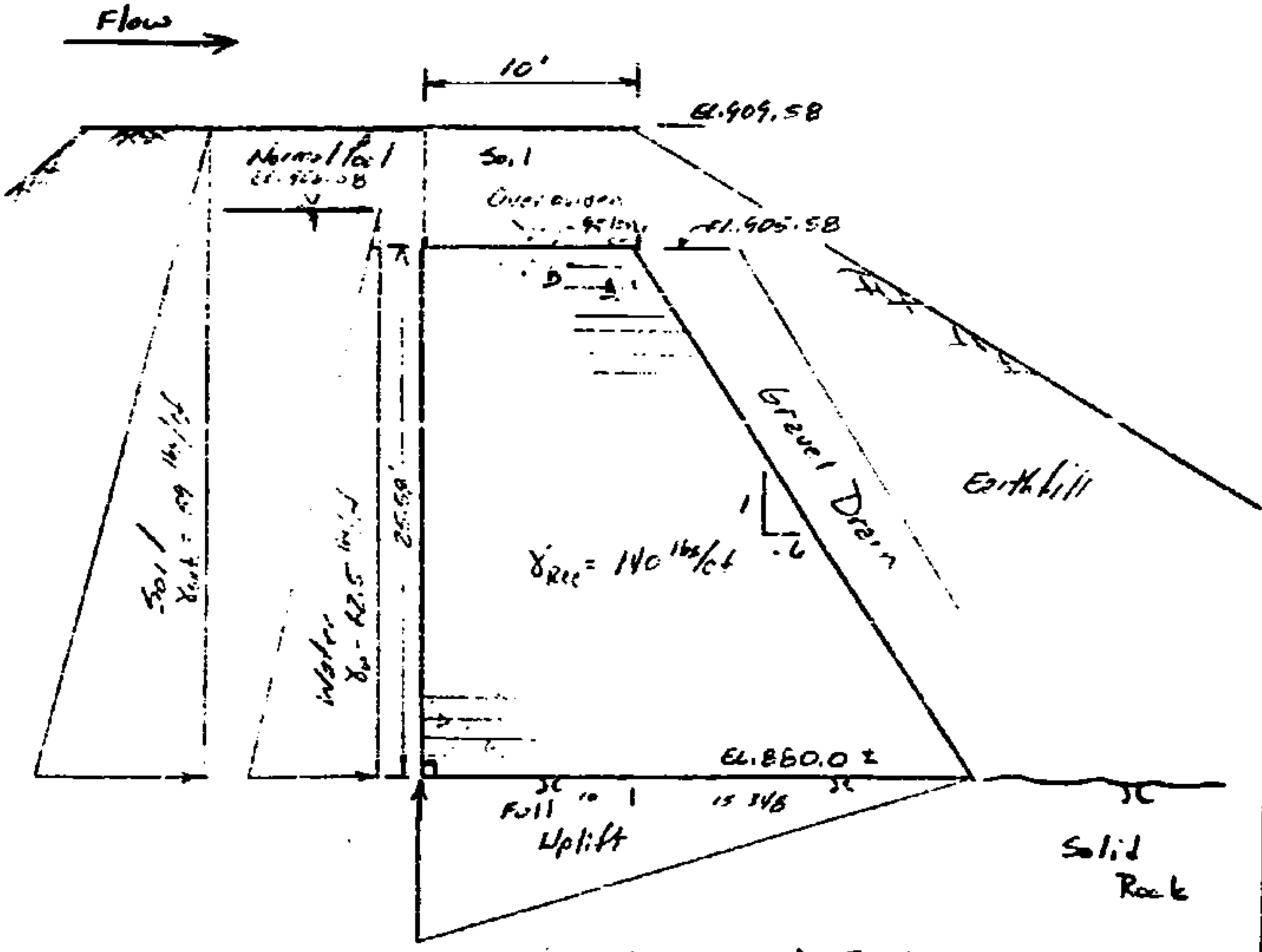


TYPICAL SECTION
Scale 0 5 Feet
1" = 5'

TYPICAL SECTION (after breach) - Covering material washed away due to overtopping.



Case I – Normal Pool Conditions



Typical (Maximum) Section

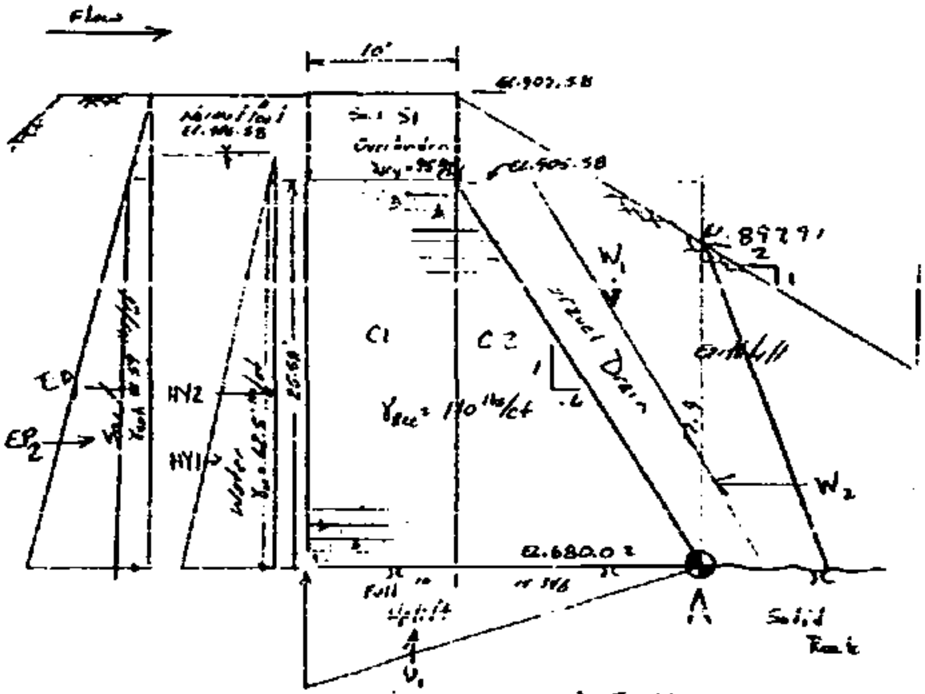
RCC SECTION

CASE I - NORMAL POOL

REQUIRED SHEAR-FRICTION FACTOR = 3.0
COMPUTED " " " = 9.36

UNIT WEIGHTS
γ_{CONC} = 0.140 K/FT³
γ_{WATER} = 0.0625 K/FT³
U/S EARTHFILL:
γ_{DRY} = 0.095 (TYP. FOR ML)
ASSUME S.G. = 2.65,
then V_s = 0.095 / (2.65)(0.0625)
V_s = 0.574 ft³
V_v = 1 − 0.574 = 0.426
∴ γ_{SUB} = 0.095 − .574(0.0625)
= 0.059 K/ft³

Computed [redacted]



Typical (Maximum) Section

Item No.	
a.	DEAD W[redacted]
	C1 = (10)[redacted]
	C2 = (½)[redacted]
	S1 = (10[redacted]
b.	HYDROS[redacted]
	HY1 = ½[redacted]
	HY2 = [redacted]
c.	STATIC [redacted]
	EP1 = (25[redacted]
	EP2 = ½[redacted]
d.	UPLIF[redacted]
	U1 = ½ (2[redacted]
e	D/S FILL[redacted]
	W1 = ½ ([redacted]
	W2 = ½ ([redacted]

Su[redacted]

f = friction factor = tan 23° = 0.42
C = Cohesion = 10 ksf (70 psi)
A = L × b = area of base under compression = 25.348 ft² (unit width)

$$Q = \text{shear-friction factor} = \frac{f(\Sigma V) + C\,A}{\Sigma H} = \frac{0.42(58.416) + 10(25.348)}{29.99} = 9.36$$

X = ΣMA / ΣV [redacted]

Pressures [redacted]
f (toe) [redacted]
f (hee[redacted]
Sliding [redacted]

ACS Date 4/28/87 Checked by CK Date 5-1-87

STABILITY ANALYSIS AT ELEVATION ______

Force	H K	V KIPS + ↓	V KIPS −	arm FT	M_A K-FT +	M_A K-FT −	Remarks
...T OF DAM							
(5.58)(1)(0.140)		35.812		20.348	128.73		
25.58)(15.348)(0.140)		27.482		10.232	281.20		
+)(0.095)		3.800		20.348	77.32		
ΣDW		67.094			1087.22		
...TIC							
(5.58)²(1)(.0625)	20.448			8.527		174.36	
1)(25.58)(0.0625)	1.599			12.79		20.45	
	22.047					194.81	
...RIZ. FILL PRESSURE							
58)(4)(1)(0.059)5	3.018			12.79		38.606	
25.58)²(1)(0.059)5	9.651			8.527		82.298	
	12.669					120.90	
.58)(25.348)(1)(.0625)			20.26	16.898		342.35	
5.58)(15.348)(0.059)		11.582		5.116	59.25		
50)²(0.059)(0.5)	−4,726			5.967	28.198		
					87.448		
Summations	ΣH = 29.99	78.676	−20.26		1174.67	658.064	
		ΣV = 58.416			ΣM_A = 516.61		

= 516.61 / 58.416 = 8.845   e = ½ − x̄ = 12.674 − 8.835 = 3.839 < L/6 = 4.226

= ΣV/L [1 ± 6e/L] = 4.9 KSF (c), 0.216 KSF (c)

$\tan\theta = \frac{\Sigma H}{\Sigma V} = \frac{29.99}{58.416} = 0.513;\ \theta = 27.2°$

BONEY FALLS DAM
STABILITY ANALYSIS
RCC SECTION
CASE I - NORMAL POOL
HARZA ENGINEERING CO., CHICAGO
APPROVED ______
DATE 28 APR 89 | 19796

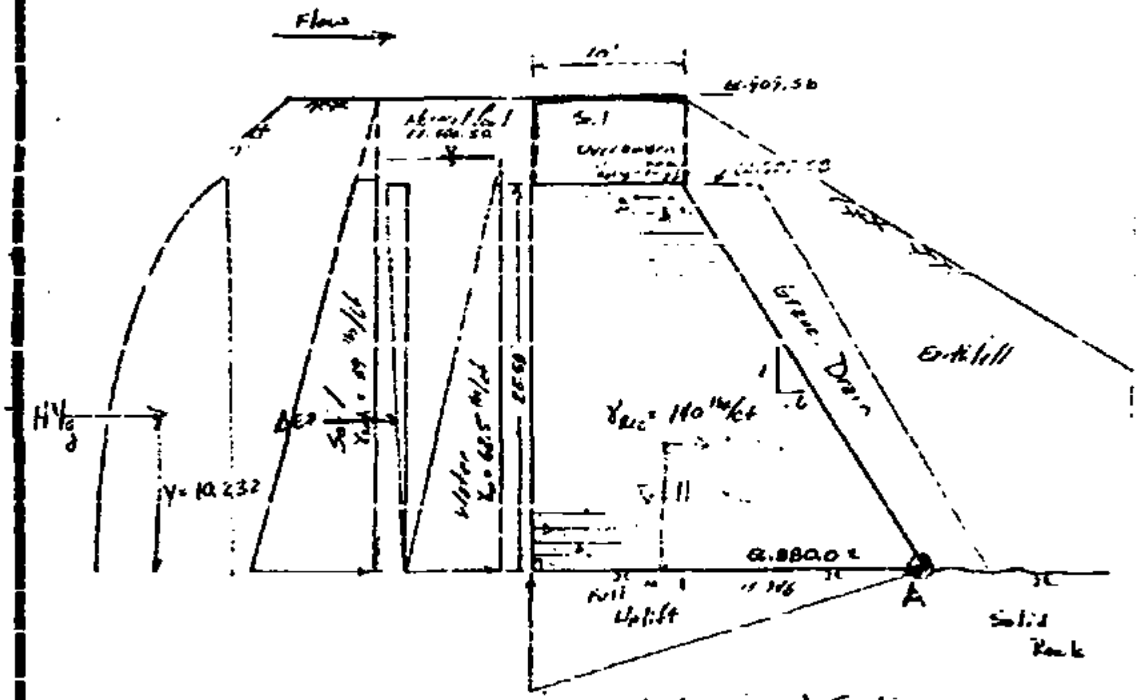
RCC SECTION

CASE III - NORMAL POOL & EARTHQUAKE

REQUIRED SHEAR-FRICTION FACTOR = 1.0
COMPUTED " " " = 6.71

ADDITIONAL EARTHQUAKE INDUCED LOAD

$DW_g = (0.1)\,\Sigma DW$
$HY_g = 0.583 H^2 \gamma_w (0.1)$



Typical (Maximum) Section

COMPRESSION LENGTH = $\frac{25.348}{6.239}(5.424) = 22.037$

C = ... FORMULA IN ... CASE ... NORMAL POOL

f = friction factor = tan 23° = 0.42
C = COHESION = 10.1 KSI (70 PSI)
A = L x b = area of base under compression = 25.348 m² (UNIT WIDTH)

Q = shear-friction factor = $\frac{f(\Sigma V) + C\,A}{\Sigma H} = \frac{0.42(58.416) + 10.1(22.037)}{39.81}$ = 6.207

Computed [redacted]

Item No.	
a	CASE [redacted]
b.	DEAD W [redacted]
	DWb - [redacted]
C.	HYDRO [redacted]
	HY_g = .0 [redacted]
d.	DYNAM [redacted]
	W_E D - [redacted]
	(1) OMAGE [redacted]
	(2) PRAVAS [redacted]
	Su [redacted]

$\bar{X} = \frac{\Sigma M}{\Sigma V}$ [redacted]

Pressures [redacted]
f (toe [redacted]
f (hee [redacted]
Sliding [redacted]

ACS Date 4/28/89 Checked by CK Date 5-1-89 Page 5 of 8

STABILITY ANALYSIS AT ELEVATION ________

Force	H K	V K ↓+	V K −	arm FT	M_A K·FT ↺+	M_A K·FT −	Remarks
... RESULTS	29.99	58.416			516.61		
... INERTIA FORCE							
0.1(67.094)	6.709			11.882		-79.716	
... FORCE							
(55)(25.58)²(.0625)(0.1)	2.384			10.232		-24.393	
SOIL PRESSURE							
(0.059)(25.58)²(0.075)							
(0.5)	0.724			8.527		-6.172	
...ANALYSIS (1929)							
... AND BASAVANNA (1969)							
Summations	ΣH= 39.81	58.416			516.61	110.28	
		ΣV= 58.416			ΣM_A= 406.33		

$\frac{406.33}{58.416} = 6.956$

$e = \frac{L}{2} - 6.956 = 5.718$

$5.718 > \frac{L}{6}$ 4.225 N/G CRACKED

$\frac{\Sigma V}{L}\left[1 \pm \frac{6e}{L}\right] = 2.305\left[1 \pm 1.355\right] = 5.424$ KSF(C)
$= -0.815$ KSF(T)

$\tan\theta = \frac{\Sigma H}{\Sigma V} = 0.681$; $\theta = 34.3°$

BONEY FALLS DAM

STABILITY ANALYSIS
RCC SECTION
CASE III - NORMAL ... + EARTHQUAKE

NOAZA ENGINEERING CO. ...
APPROVED ________
DATE 28 APR '89 | 1979L

RCC SECTION

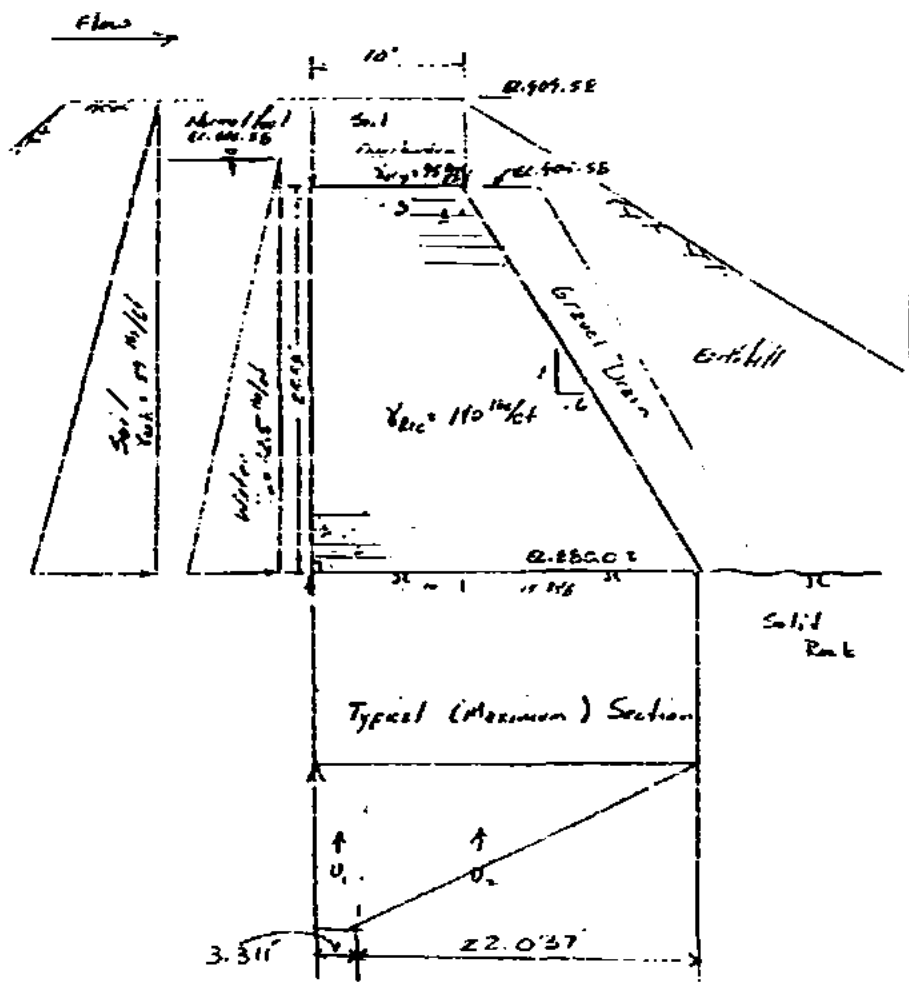
CASE 1' - NORMAL POOL - CRACKED BASE

REQUIRED SHEAR-FRICTION FACTOR = 1.0
COMPUTED " " " = 8.2

UNIT WEIGHTS
γ_{CONC} = 0.150 k/ft³
γ_{WATER} = 0.0625 k/ft³
U/S EARTHFILL:
γ_{dry} = 0.095 (TYP. FOR ML)
ASSUME: Sp. = 2.65,
then V_s = 0.095 / (2.65)(0.0625)
V_s = 0.574 ft³
V_v = 1 - 0.574 = 0.426
∴ γ_{sub} = 0.095 - .574(0.0625)
= 0.059 k/ft³

Computed [redacted]

Item No.	
a	bcA [redacted]
b	HYD [redacted]
c	STA [redacted]
e	D/s [redacted]
d	UP [redacted]
	U(26. [redacted]
	U_2 = ½ [redacted]



f = friction factor = tan 23° = 0.42
C = COHESION = 10.1 KSF (70 PSI)
A = L x b = area of base under compression = 22.037 ft² (UNIT WIDTH)

$$Q = \text{shear-friction factor} = \frac{f(\Sigma V) + C\,A}{\Sigma H} = \frac{0.42(55.767) + 10.1(22.037)}{29.99} = 8.203$$

$\bar{X}$ = Σ[redacted]

Pressur[redacted]
f (t[redacted]
f (h[redacted]
Sliding [redacted]

by ACS Date 5/1/89 Checked by CK Date 5-1-89

STABILITY ANALYSIS AT ELEVATION ______

Force	H	V ↓ +	V ↑ −	arm	M_A ↺ +	M_A −	Remarks
[illegible] DAM		67.094			1087.27		
[illegible]STATIC	22.047					-194.81	
[illegible] CAVITATING PRESSURE	12.669					-120.90	
FILL	-4.726	11.582			87.448		
[illegible]LIFT							
[illegible]8)(3.811)(0.0623)			5.293	23.693		-125.42	
(22.037)(25.58)(0.0625)			17.616	14.691		-258.79	
Summations	ΣH= 29.49	78.676	-22.909		1174.67	-699.92	
		ΣV= 55.767			ΣM_A= 474.75		

$\frac{M_A}{V} = \frac{474.75}{55.767} = 8.513$ e = $\frac{25.348}{2} - 8.513 = 4.16 < \frac{L}{6}$

[illegible]s:

[illegible]e) $\frac{\Sigma V}{L}\left[1 \pm \frac{6e}{L}\right] = 2.2\ (1 \pm 0.985)$ = 4.367 KSF (c) = 0.033 KSF

[illegible]el)

$\tan\theta = \frac{\Sigma H}{\Sigma V} = \frac{29.99}{55.767} = 0.538$; $\theta = 28.27°$

BONNY FALLS DAM

STABILITY ANALYSIS
RCC SECTION
CASE I - NORMAL POOL; CRACKED

HARZA ENGINEERING CO., CHICAGO

APPROVED ______

DATE 1 MAY 89 | [illegible] 19791

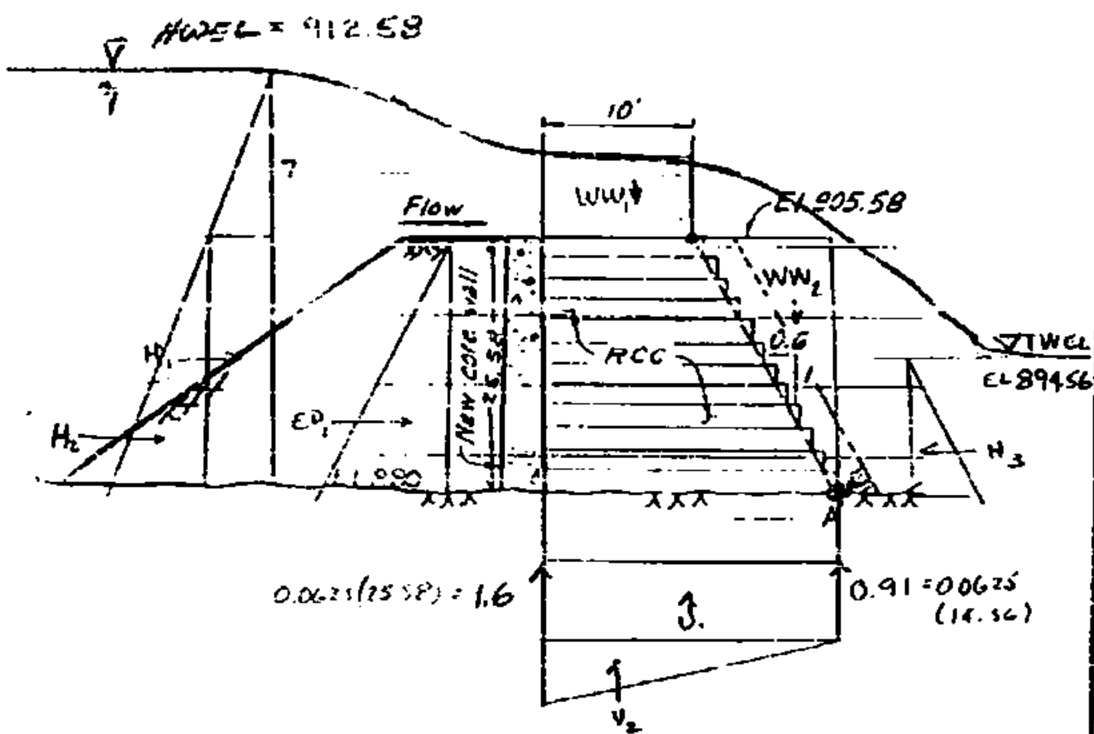
RCC SECTION

CASE IVa - IDF = 100,000 CFS

REQUIRED FACTOR OF SAFETY = 2.0

COMPUTED " " " =

Computed



Item No.	
a	DEAD...
	Ci = (1...
	C2 = ½...
b	WEIGH...
	WW1 = 1(...
	WW2 = ½...
c	HYDR...
	H1 = 7(...
	H2 = ½(...
	H3 = ½ (...
d	EARTH...
	EP1 = ½...
e	UPLIFT...
	U1 = 0....
	U2 = 0....
	Su...

f = friction factor = tan 23° = 0.42
C = COHESION = 10.1 ksf (70 psi)
A = Lx b = area of base under compression =

Q = shear-friction factor = $\frac{f(\Sigma V) + C\ A}{\Sigma H}$ = SEE CRACKED BASE

$\bar{X} = \frac{\Sigma M...}{\Sigma V}$

Pressures
f (toe...
f (hee...
Sliding

STABILITY ANALYSIS AT ELEVATION ______

Force	H k →	V kip ↓ +	V −	arm	M_A k-ft ↺ +	M_A −	Remarks
WEIGHT OF DAM							
(25.58)(1)(0.140)		35.812		20.348	728.70		
(25.58)(15.348)(0.140)		27.482		10.232	281.20		
		63.294			1009.90		
OF WATER DAM							
(...)(0.0625)		2.917		20.348	59.36		
(5.348)(25.58)(0.0625)(.6)		7.361		5.116	37.66		
		10.278			97.02		
STATIC							
(25.58)(0.0625)	11.19			12.79		-143.12	
(25.58)²(0.0625)	20.45			8.53		-174.44	
(9.56)²(0.0625)(0.6)	-3.975			4.85	19.29		
	27.67				19.29	-317.56	
PRESSURE							
(25.58)²(1)(0.059)(1.5)	9.651			8.527		-82.29	
PRESSURE							
91(25.348)			23.067	12.674		292.35	
69(25.348)½			8.745	16.899		147.78	
		73.572	-31.812		1126.21	-839.98	
Summations	ΣH = 37.32	ΣV = 41.76			ΣM_A = 286.23		

$x = \frac{286.23}{41.76} = 6.854$

$e = \frac{L}{2} - x = 12.674 - 6.854 = 5.82 > \frac{L}{6} = 4.225$ N/G

$\sigma = \frac{\Sigma V}{L}\left[1 \pm \frac{6e}{L}\right] = 1.647(1 \pm 1.378)$ = 3.918 ksf (C), = −0.623 ksf (T) CRACKED BASE ANALYSIS REQ.

$\tan\theta = \frac{\Sigma H}{\Sigma V} = \frac{37.32}{41.76} = 0.894$; $\theta = 41.8°$

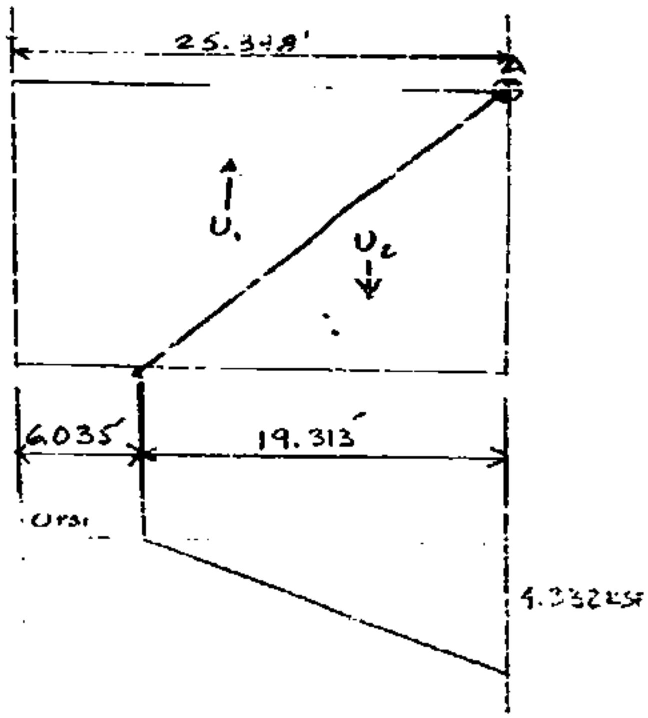
BONEY FALLS
STABILITY ANALYSIS RCC SECTION
CASE IVa - IDF 100,000 cfs
HARZA ENGINEERING CO., CHICAGO
APPROVED ______
DATE 1 MAY '89 | 19791

...ION

CASE IV a' - IDF 100,000 CFS; CRACKED ANALYSIS

REQUIRED FACTOR OF SAFETY = 2.0

COMPUTED " " " = 5.69



Computed

Item No.	
a	WEI
b	WEI
c	EAR
d	HOR
e	UP
	U_1 =
	$U_2 = \frac{1}{2}$
	21
	33
	63

S

f = friction factor = tan 23° = 0.42
C = Cohesion = 10.1 KSF (70 PSI)
A = L x b = area of base under compression = 19.313 ft²

$$Q = \text{shear-friction factor} = \frac{f(\Sigma V) + C\,A}{\Sigma H} = \frac{0.42(91835) + 10.1(19.313)}{37321} = 5.69$$

$\bar{X} = \frac{\Sigma M}{\Sigma V}$

Pressure
f (toe
f (hee
Sliding

By ACS Date 4/28/89 Checked by CK Date 5-1-89 Page 8 of 8

STABILITY ANALYSIS AT ELEVATION ______

Force	H →	V ↓ +	V −	arm	M_A +	M_A −	Remarks
Wt of Concrete		63.294			1009.70		
Wt of Water		10.278			97.02		
Pressure	9.651					-82.29	
Tailwater Water	27.67					-298.27	
UPLIFT							
(25.58)(0.0625)(25.348)			40.525	12.674		513.614	
(7.56)(.0625) T		0.455T		T/3	$0.152T^2$		
		73.572			1106.92		
Summations	ΣH = 37.321	+0.455T	-40.525		$+0.152T^2$	-894.174	
		ΣV = 33.047 + 0.455T			$\Sigma M_A = 212.75 + 0.152T^2$		
		= 41.835			= 269.45		

$$\frac{212.75 + 0.152T^2}{33.047 + 0.455T} = \frac{T}{3}$$

$$638.25 = 33.047\,T$$

$$T = 19.313$$

$$\frac{269.45}{41.835} = 6.441 \qquad e = \frac{25.348}{2} - 6.441 = 6.233$$

$$\frac{25.348}{6} - 6.233 = -2.0$$

$$\frac{2\Sigma V}{T} = \frac{2(41.835)}{19.313} = 4.332 \text{ ksf}$$

$$\tan\theta = \frac{\Sigma H}{\Sigma V} = \frac{37.321}{41.835} = 0.892, \quad \theta = 41.7^\circ$$

BONEY FALLS DAM
STABILITY ANALYSIS
RCC SECTION
CASE IVa - IDF - 109000 cfs
HARZA ENGINEERING CO. CHICAGO
APPROVED ______
DATE 1 MAY 89
JOB NO. 15796

APPENDIX F
RCC SPILLWAY DESIGN

I. OBJECTIVE: Establish proper dimensions for design of RCC emergency spillway to increase project discharge capacity to 100,000 cfs at the top of the dam.

II. ASSUMPTIONS:

1. Maximum W.S.EL. 912.58 established previously (top of gravity dam section).
2. West Embankment Dam raised to EL. 912.58 to prevent overtopping during IDF.
3. Corewall within high portion of West Embankment Dam (Sta. 2+00 to 8+80) raised to EL. 912.58.
4. RCC spillway replaces high portion of East Embankment Dam.
5. Raise East Embankment Dam 2' to EL. 911.08 from end of RCC to Sta. 16+65 to protect from overtopping until RCC spillway overtopped.
6. Keep East Embankment Dam at current height, EL. 909.08± from Sta. 16+65 to 57+00.

III. REFERENCES

Ref.: [1.] "Evaluation of Spillway Capacity Expansion Alternatives," HECO, August, 1987.

[2] "Design Flood Documentation Report," HECO, 6 May 1988.

[3] "Inflow Design Flood Report - Addendum 1, HECO, 7 Oct 88.

[4.] "Geotechnical Investigations Report", HECO, October, 1988

[5] "Third 5-Year Inspection & Safety Report," HECO, January 1989.

[6.] "Third 5-Year Inspection & Safety Report - Stability Analysis Supplement", HECO, January 1989.

[7] FERC letter to [illegible], dated 15 Mar 1989, approving IDF = 100,000 cfs.

IV. Existing Discharge Capacity

1. Top of Existing Embankment Dams

High West : 912.58 ±
Low West : 909.08 ±

High East : 909.08 ±
Low East : 909.08 ±

2. Spillway Capacity at 909.08

Gated: 28,696 cfs
Ungated: 4,772 cfs
** Existing Discharge Capacity = **33,468 cfs**

3. Necessary Increase in Capacity

100,000 cfs (approved IDF, [7])
− 33,468
66,532 cfs

V. RCC Spillway Discharge

1. Design Flood Peak Elevation, EL. 912.58
2. Gated Spillway Capacity: 35,496 cfs
3. Ungated Spillway Capacity: 14,617 cfs
4. Unimproved East Embankment Dam:
   - Crest El. 909.08 ± (1987 survey)
   - Net length: Sta. 16+65 to 57+00 = 4035 ft.
     - After review of topography & tree cover, effective length extends to about sta. 34+00

       (a) eff. length, L_e = 1735 feet
       (b) submergence on crest = 90% (reduces discharge coefficient)
   - Flow controlled as a broad-crested weir, then $C_d = 2.67$
   - Submergence: based on COE design chart 711 in Hydraulic Design Criteria,

     $C_s/C_d = 0.67 \Rightarrow C_s = 1.80$
   - Effective Head, $H_e = 912.58 - 909.08 = 3.5$ ft.
   - Discharge, $Q = C_s L_e H_e^{3/2}$
     $= (1.80)(1735)(3.5)^{3/2}$
     $= 20,449$ cfs.

HARZA ENGINEERING COMPANY CHICAGO

SUBJECT	RCC Spillway	PROJECT	Berry Falls Dam
		FILE NUMBER	1974L
COMPUTED	[illegible]	CHECKED	
DATE	26 Apr 89	Page 5 of 8 Pages	

5. Improved (raised) East Embankment Dam

- Crest El. 911.08, $H_e = 912.58 - 911.08 = 1.5$ ft
- Net Length = 1665 − (Length of RCC)
- $C_d = 2.67$
- $Q_E = 2.67\,(1665 - L_{RCC})(1.5)^{3/2}$
  $= 4.905\,(1665 - L_{RCC})$

6. RCC Spillway

- Assume crest @ El. 905.58 (Crest of ungated spillway), $H_e = 912.58 - 905.58 = 7$ ft.
- $C_d = 2.67$ (broad-crest)
- Length = L_{RCC}
- Then, $Q_{RCC} = 2.67\,L_{RCC}\,7^{3/2}$
  $= 49.449\,L_{RCC}$
- Q_{needed} = 100,000 (IDF)
  − 35,496 (Gated Spillway)
  − 14,617 (Ungated ")
  − 20,449 (Improved East E.L.)
  29,438 = $Q_{RCC} + Q_E$
  $= 49.449\,L_{RCC} + 4.905\,(1665 - L_{RCC})$
  21,271 = $44.544\,L_{RCC}$

  L_{RCC} = 478 feet, say $L_{RCC} = 500$ ft

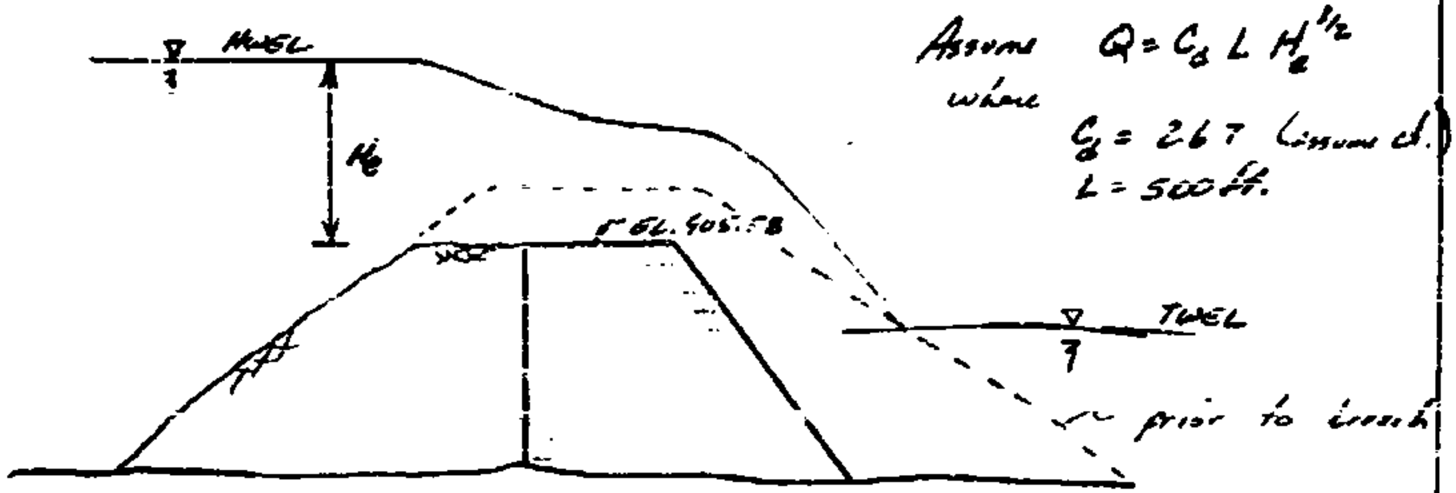
VI. Project Discharge Capacity @ El. 912.58

	Effective Length (ft)	Effective Head (ft)	Discharge Coefficient	Q (cfs)
1. Gated Spillway	120	12	.76 (orifice)	34,746
2. Ungated Spillway	199.3	7	3.96	14,617
3. Unimproved East Emb.	1135	3.5	1.8	20,449
4. Raised East Emb.	1145	1.5	2.67	5,714
5. RCC Spillway	500	7	2.67	24,725
				100,251

NEGLECTED:

1. Powerhouse flows
2. Discharge capacity of 2 - 5'x5' Mud Gates

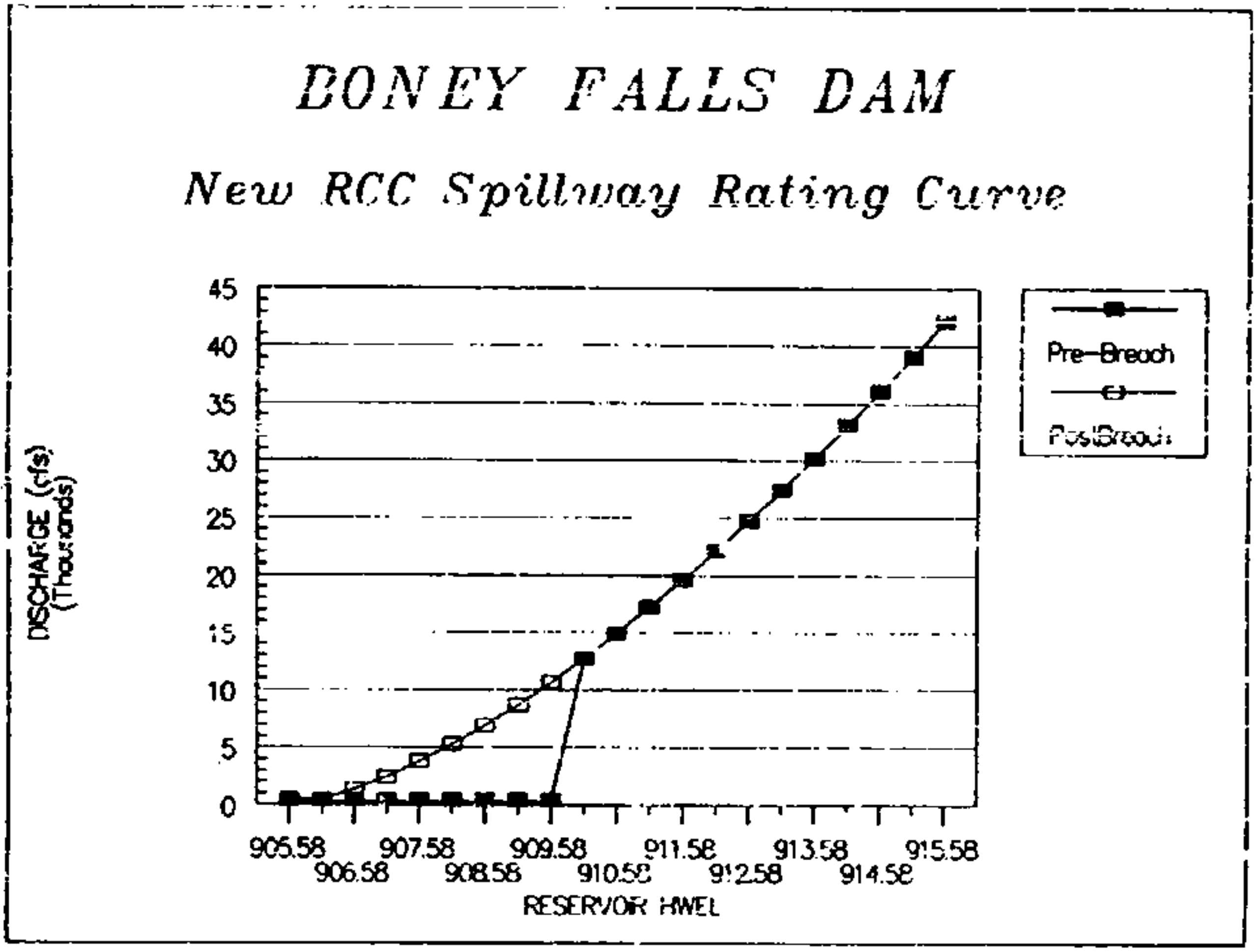
VII. RCC Spillway Rating Curve



Assume $Q = C_d L H_e^{3/2}$
where
$C_d = 2.67$ (assumed)
$L = 500$ ft.

EAST EMBANKMENT DAM
Sta. 0+00 to 5+00 ~~Topwater Elev~~: 910.08 Failure Elev!
Crest EL: 905.58
Length 500
Cd = 2.67

RESERVOIR NWEL (ft,msl)	Head (ft)	Cd	DISCHARGE (cfs) Prior to Breach	DISCHARGE (cfs) After Breach
905.58	0.00	2.67	0	0
906.08	0.50	2.67	0	472
906.58	1.00	2.67	0	1,335
907.08	1.50	2.67	0	2,453
907.58	2.00	2.67	0	3,776
908.08	2.50	2.67	0	5,277
908.58	3.00	2.67	0	6,937
909.08	3.50	2.67	0	8,741
909.58	4.00	2.67	0	10,680
910.08	4.50	2.67	12,744	12,744
910.58	5.00	2.67	14,926	14,926
911.08	5.50	2.67	17,220	17,220
911.58	6.00	2.67	19,620	19,620
912.08	6.50	2.67	22,123	22,123
912.58	7.00	2.67	24,725	24,725
913.08	7.50	2.67	27,420	27,420
913.58	8.00	2.67	30,208	30,208
914.08	8.50	2.67	33,083	33,083
914.58	9.00	2.67	36,045	36,045
915.08	9.50	2.67	39,090	39,090
915.58	10.00	2.67	42,216	42,216
916.08	10.50	2.67	45,422	45,422
916.58	11.00	2.67	48,705	48,705
917.08	11.50	2.67	52,063	52,063
917.58	12.00	2.67	55,495	55,495
918.08	12.50	2.67	[illegible]	[illegible]
918.58	13.00	2.67	62,576	62,576
919.08	13.50	2.67	66,219	66,219
919.58	14.00	2.67	69,932	69,932
920.08	14.50	2.67	73,711	73,711


BONEY FALLS DAM
New RCC Spillway Rating Curve
DISCHARGE (cfs) (Thousands)
45
40
35
30
25
20
15
10
5
0
905.58
906.58
907.58
908.58
909.58
910.58
911.58
912.58
913.58
914.58
915.58
RESERVOIR HWEL
Pre-Breach
PostBreach