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NATIONAL DAM SAFETY PROGRAM. HOLDEN WATER SUPPLY DAM (MO 20532)--ETC(U)
APR 81 E R BURTON, H L CALLAHAN DACW43-81-C-0037

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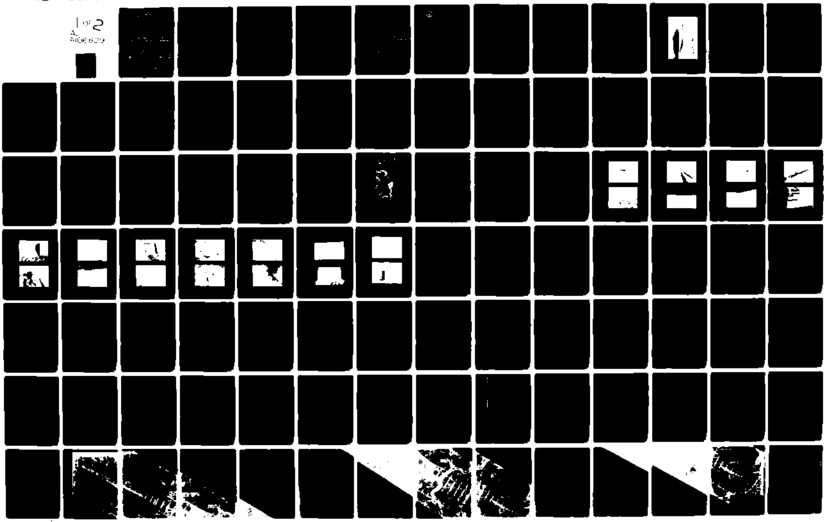

LEVEL 1

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MISSOURI-KANSAS CITY BASIN

AD A106629

HOLDEN WATER SUPPLY DAM

JOHNSON COUNTY, MISSOURI

MO 20532

PHASE 1 INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM

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20. ABSTRACT (Continue on reverse side if necessary and identify by block number)

This report was prepared under the National Program of Inspection of Non-Federal Dams. This report assesses the general condition of the dam with respect to safety, based on available data and on visual inspection, to determine if the dam poses hazards to human life or property.

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MISSOURI-KANSAS CITY BASIN

HOLDEN WATER SUPPLY DAM
JOHNSON COUNTY, MISSOURI
MO 20532

PHASE 1 INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM

United States Army
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St. Louis District

PREPARED BY: U.S. ARMY ENGINEER DISTRICT. ST. LOUIS

FOR: STATE OF MISSOURI

APRIL 1981

DEPARTMENT OF THE ARMY
ST. LOUIS DISTRICT, CORPS OF ENGINEERS
210 TUCKER BOULEVARD, NORTH
ST. LOUIS, MISSOURI 63101

REPLY TO
ATTENTION OF

SUBJECT: Holden Water Supply Dam MO 20532

This report presents the results of field inspection and evaluation of the Holden Water Supply Dam. It was prepared under the National Program of Inspection of Non-Federal Dams.

SUBMITTED BY: SIGNED — Chief, Engineering Division — Date: 28 JUL 1981

APPROVED BY: SIGNED — Colonel, CE, Commanding — Date: 31 JUL 1981

3

HOLDEN WATER SUPPLY DAM

JOHNSON COUNTY, MISSOURI

MISSOURI INVENTORY NO. 20532

PHASE I INSPECTION REPORT

NATIONAL DAM SAFETY PROGRAM

PREPARED BY:

BLACK & VEATCH
CONSULTING ENGINEERS
KANSAS CITY, MISSOURI

UNDER DIRECTION OF

ST. LOUIS DISTRICT CORPS OF ENGINEERS

FOR

GOVERNOR OF MISSOURI

APRIL 1981

PHASE I REPORT

NATIONAL DAM SAFETY PROGRAM

Name of Dam	Holden Water Supply
State Located	Missouri
County Located	Johnson County
Stream	Tributary of South Fork Blackwater River
Date of Inspection	2 April 1981

The Holden Water Supply Dam was inspected by a team of engineers from Black & Veatch, Consulting Engineers for the St. Louis District, Corps of Engineers. The purpose of the inspection was to make an assessment of the general condition of the dam with respect to safety, based upon available data and visual inspection, in order to determine if the dam poses hazards to human life or property.

The guidelines used in the assessment were furnished by the Department of the Army, Office of the Chief of Engineers and developed with the help of several Federal and state agencies, professional engineering organizations, and private engineers. Based on these guidelines, this dam is classified as an intermediate size dam with a high downstream hazard potential. According to the St. Louis District, Corps of Engineers, failure would threaten lives and property. The estimated damage zone extends approximately twelve miles downstream of the dam. Within the estimated damage zone are sixteen buildings, fourteen dwellings, a church, State Highway 131 and U.S. Highway 50. Contents of the estimated downstream damage zone were verified by the inspection team.

T ~~Our~~ inspection and evaluation indicates the spillways do not meet the criteria set forth in the guidelines for a dam having the above size and hazard potential. The spillways will not pass the probable maximum flood without overtopping the dam but will pass 65 percent of the probable maximum flood. The spillways will pass the flood which has a one percent chance of occurrence in any given year (100-year flood). The spillway design flood recommended by the guidelines is the probable maximum flood. The probable maximum flood is defined as the flood discharge which may be expected from the most severe combination of critical meteorologic and hydrologic conditions which are reasonably possible in the region.

Based on visual observations, this dam appears to be in satisfactory condition. Deficiencies visually observed by the inspection team were wave action erosion and erosion gullies on the upstream face and animal burrows on both faces of the embankment. The lake level was observed to

Cont'd

be considerably below the rip rap elevation at the time of the inspection. Seepage and seismic stability analyses required by the guidelines were not available.

There were no observed deficiencies or conditions existing at the time of the inspection which indicated an immediate safety hazard. Future corrective action and regular maintenance will be required to correct or control the described deficiencies. In addition, detailed seepage and stability analyses of the existing dam, as required by the guidelines, should be performed. A detailed report discussing each of these deficiencies is attached.

Edwin R. Burton

Edwin R. Burton, PE
Missouri E-10137

Harry L Callahan

Harry L. Callahan, Partner
Black & Veatch

OVERVIEW OF DAM

PHASE I INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM
HOLDEN WATER SUPPLY DAM

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APPENDICES

SECTION 1 - PROJECT INFORMATION

1.1 GENERAL

a. Authority. The National Dam Inspection Act, Public Law 92-367, authorized the Secretary of the Army, through the Corps of Engineers, to initiate a program of safety inspection of dams throughout the United States. Pursuant to the above, the District Engineer of the St. Louis District, Corps of Engineers, directed that a safety inspection of the Holden Water Supply Dam, be made.

b. Purpose of Inspection. The purpose of the inspection was to make an assessment of the general condition of the dam with respect to safety, based upon available data and visual inspection, in order to determine if the dam poses hazards to human life or property.

c. Evaluation Criteria. Criteria used to evaluate the dam were furnished by the Department of the Army, Office of the Chief of Engineers, in "Recommended Guidelines for Safety Inspection of Dams." These guidelines were developed with the help of several Federal agencies and many state agencies, professional engineering organizations, and private engineers.

1.2 DESCRIPTION OF PROJECT

a. Description of Dam and Appurtenances.

(1) The dam is an earth structure located in the valley of a tributary to South Fork Blackwater River. (see Plate 1). The watershed is an area of low hills with mildly sloping terrain consisting of about 40 percent cropland, 20 percent timber and 40 percent grassland. Approximately 1 mile upstream of the Holden Water Supply Dam reservoir pool is Dam Mo. 20073. The Holden Water Supply dam is approximately 3,350 feet long along the crest and 58 feet high. The dam crest is 12 feet wide and has a straight alignment. The dam has berms on the upstream and downstream faces, rip rap on the upstream face, an internal toe drain system, a gravel access road across the crest and a rock lined drainage control ditch along the downstream toe.

(2) The principal spillway, located about 600 feet from the right abutment, consists of a 36-inch diameter concrete pipe with a 3 x 9-foot concrete box drop inlet installed in the embankment. Angle bars were placed periodically along the anti-vortex piers of the drop inlet structure to act as a trash rack. Seepage collars have been placed periodically along the spillway pipe under the embankment. The box-type drop inlet has a depth of 30 feet with a variation in wall thickness of 10 inches at the crest to 21 inches at the invert. The principal spillway pipe remains underground for 345 feet then discharges to a concrete

stilling basin. The stilling basin increases in width from 3 feet-6 inches to 7 feet and has a length of 17 feet. It contains 16 inch high by 14 inch wide baffle blocks to retard excessive velocities. The spillway stilling basin discharges to a rock lined excavated channel which discharges to the natural stream below the dam.

The emergency spillway consists of a 150 feet wide open channel with trapezodial section excavated through natural material in the right abutment. The spillway has rip rap cover on the side slopes through the approach and control sections and is grass lined to the natural stream below the dam. The grass lined approach channel curves perpendicular to the dams axis at the control section.

(3) Located upstream of the dam near the left abutment is a raw water supply intake structure. The structure has an adjustable intake pipe which is connected by a 14-inch ductile iron pipe that runs under the dam to a pump house located downstream of the embankment.

(4) Pertinent physical data are given in paragraph 1.3.

b. Location. The dam is located in northwest Johnson County, Missouri, as indicated on Plate 1. The lake formed by the dam is located in an area shown on the United States Geological Survey 7.5 minute series quadrangle map for Elm, Missouri in Section 29 of T46N, R28W (see Plate 2).

c. Size Classification. Criteria for determining the size classification of dams and impoundments are presented in the guidelines referenced in paragraph 1.1c above. Based on these criteria, the dam and impoundment are in the intermediate size category. An intermediate size dam is classified as having a height less than 100 feet, but greater than or equal to 40 feet and/or a storage capacity less than 50,000 acre-feet, but greater than or equal to 1,000 acre-feet.

d. Hazard Classification. The hazard classification assigned by the Corps of Engineers for this dam is as follows: The Holden Water Supply Dam has a high hazard potential, meaning that the dam is located where failure may cause loss of life, and serious damage to homes, agricultural, industrial and commercial facilities, and to important public utilities, main highways, or railroads. For the Holden Water Supply Dam the estimated flood damage zone extends approximately twelve miles downstream of the dam. Within the estimated damage zone are 16 buildings, 14 dwellings, a church, State Highway 131 and U.S. Highway 50. Contents of the estimated downstream damage zone were verified by the inspection team.

e. Ownership. The dam is owned by the City of Holden, Missouri, Attention Water Superintendent, Tony Lerden, 110 W. 3rd, Holden, Missouri 64040.

f. Purpose of Dam. The dam will form a 302-acre lake to be used for water supply and recreation.

g. Design and Construction History. The dam was designed by E.T. Archer and Company, Consulting Engineers, Kansas City, Missouri. It was constructed by Gibson and Bowles Inc., Lee's Summit, Missouri. The construction of the dam began in June 1979 and the final inspection was made in December 1980.

h. Normal Operating Procedure. Normal rainfall, runoff, transpiration, evaporation, flow through the uncontrolled principal spillway and water supply withdrawals will all combine to maintain a relatively stable water surface elevation.

1.3 PERTINENT DATA

a. Drainage Area - 2,650 acres, 2,225 acres uncontrolled.

b. Discharge at Damsite.

(1) Normal discharge at the damsite is through an uncontrolled 3 x 9-foot concrete box drop inlet with a 36-inch concrete pipe.

(2) Estimated experienced maximum flood at damsite - Unknown.

(3) Estimated ungated spillway capacity at top of dam elevation - 8,532 cfs.

c. Elevation (Feet above m.s.l.).

(1) Top of dam - 850.2 (see Plate 3)

(2) Principal spillway drop inlet crest - 840.5

(3) Emergency spillway crest - 844.5

(4) Toe of dam - 814.3

(5) Maximum tailwater - Unknown.

d. Reservoir.

(1) Length of maximum pool - 10,200 feet ± (Probable maximum flood pool level)

(2) Length of normal pool - 8,600 feet ± (Principal spillway crest)

(3) Length of observed pool - 2,700 feet ±

e. Storage (Acre-feet).

(1) Top of dam - 6,516

(2) Emergency spillway crest - 4,590

(3) Principal spillway crest - 3,413

(4) Observed pool - 255

(5) Design surcharge - 5,520 (Based on design analysis, 11.58 inches of runoff, 12 hour storm)

f. Reservoir Surface (Acres).

(1) Top of dam - 460

(2) Emergency spillway crest - 360

(3) Principal spillway crest - 302

(4) Observed pool - 57

g. Dam.

(1) Type - Earth embankment

(2) Length - 3,350 feet

(3) Height - 58 feet ±

(4) Top width - 12 feet

(5) Side slopes - upstream face between 1.0 V on 2.6 H and and 1.0 V on 3.6 H, downstream face between 1.0 V on 2.7 H and 1.0 V on 3.3 H (see Plate 3)

(6) Zoning - Zone 1 - Core CL material; Zone 2 - upstream and downstream embankment CL or ML material; Zone 3 - toe drain, crushed rock; Zone 4 - upstream face rip rap. See Geotechnical and Soils report, page 11.

(7) Impervious core - CL Material, See Appendix C, "As-Built" drawing No. 11.

(8) Cutoff - Impervious core trench, 12-foot bottom width, 1 H on 1 V side slopes.

(9) Grout curtain - None.

h. Diversion and Regulating Tunnel - None.

i. Principal Spillway.

(1) Type - Drop Inlet with Concrete pipe, 36-inch diameter.

(2) Inlet crest elevation - 840.5 feet m.s.l.

(3) Inlet invert elevation - 810.5 feet m.s.l.

(4) Outlet invert elevation - 800.0 feet m.s.l.

(5) Gates - None.

(6) Upstream channel - None.

(7) Downstream channel - Discharges to a rock lined channel leading to the natural stream downstream of dam.

j. Emergency Spillway.

(1) Type - Grass lined channel with control section.

(2) Crest elevation - 844.5 feet m.s.l.

(3) Gates - None.

(4) Upstream channel - Grass lined approach channel.

(5) Downstream channel - Discharges to a channel leading to the natural stream downstream of the dam.

k. Regulating Outlets - Raw water adjustable intake structure, 14 inch ductile iron pipe with 400 gpm pump capacity.

SECTION 2 - ENGINEERING DATA

2.1 DESIGN

Design data in the form of a detailed geologic site investigation report and "As-Built" drawings were made available by E.T. Archer, Consulting Engineers. E. T. Archer hydrologic/hydraulic design calculations were provided by the Missouri Department of Natural Resources, Dam Safety Office. The geology report and design memorandum are included herein as Appendix B. Drawing numbers 3, 11, 13 and 14 of the "As-Built" drawings are included as Appendix C.

2.2 CONSTRUCTION

Construction records in the form of "As-Built" drawings were provided for the dam and spillways. The dam was constructed by Gibson & Bowles Construction Co., Inc. The construction began in June 1979 and was completed in December 1980. Sections of the reservoir just upstream of the dam were used for borrow areas. The construction of the embankment was accomplished in phases with the center section being completed last. Located inside the drop inlet structure at elevation 810.5 is an ungated low level inlet with a flange plate bolted over it. During construction, this inlet was used for diversion.

2.3 OPERATION

Construction of the dam was completed only three months before this inspection so there was no operational records available nor was there any available documentation of past floods at the dam site. At the time of inspection the lake level was 21 feet below principal spillway crest and the raw water intake was located above the water level. No withdrawals for water supply have been made. The watershed area has been under severe drought conditions since construction of the dam began. Rainfall for the area has been below normal for the past two years. For average annual rainfall and runoff conditions, it is estimated to require approximately 5 years to fill the reservoir.

Normal operation of the water supply facilities include withdrawals through a 14-inch raw water intake pipe by the pumping station below the dam. The water is then pumped to the water treatment plant near Holden.

2.4 GEOLOGY

The site of the dam and reservoir is located in a broad shallow valley and across an intermittent tributary to the South Fork of the Blackwater River. The land surface around the reservoir is dissected into gently rolling topography.

The soils in the area of the dam and reservoir are classified as Haig, Weller, Gorin, Deepwater, Sampsel, Snead and Nodaway soil series. The Haig, Weller and Gorin soil series developed in loess on uplands and side slopes of hillsides. The Haig and Gorin soils are poorly drained, consist of clay and are classified for engineering purposes as CL or CH materials. The Weller soils are moderately well-drained, consist of silt and clay and are classified for engineering purposes as ML, CL, or CH materials. The Deepwater and Sampsel soils developed on uplands in residuum weathered from calcoreous shale, are deep and moderately well-drained, consist of silty clay and are classified for engineering purposes as ML, CL or CH materials. The Snead soils are developed on upland slopes in residuum weathered from calcoreous shales and thin limestones, are somewhat poorly drained, consist of silty clay and are classified for engineering purposes as CL or CH materials. The Nodaway soils are developed in silty alluvium on bottom lands, are moderately well drained, consist of silty clay and are classified for engineering purposes as CL or CL-ML materials.

The bedrock in the area of the dam and reservoir consist of interbedded thick shales, thin coal and thin limestone beds of the Marmaton Group of the Des Moinesion Series of the Pennsylvanian System.

Appendix B contains a copy of the geotechnical report for the design and construction of the dam and reservoir. The data in the report indicate the subsurface materials are silty clay overlying interbedded limestone and shale. The rock units dip slightly to the northwest.

2.5 EVALUATION

a. Availability. Engineering data were obtained from E.T. Archer, Consulting Engineers as noted in Section 2.1.

b. Adequacy. Engineering data were available from which to make an assessment of the design, construction and operation. Seepage and seismic stability analyses comparable to the requirements of the "Recommended Guidelines for Safety Inspection of Dams" were not available, which is considered a deficiency. These seepage and seismic stability analyses should be performed for appropriate loading conditions (including earthquake loads) and made a matter of record.

c. Validity. The available engineering data on the design, construction and operation were determined to be valid. The design, however, does not meet the Corps of Engineers design criteria with respect to the hydrologic design conditions and stability analyses load conditions.

SECTION 3 - VISUAL INSPECTION

3.1 FINDINGS

a. General. A visual inspection of Holden Water Supply Dam was made on 2 April 1981. The inspection team consisted of Edwin Burton, team leader; Bob Pinker, geologist; Gary Van Riessen, geotechnical engineer; Harvey Coppage, hydrologic/hydraulic engineer; and Anthony C. Davis, civil engineer. The dam appeared to be in satisfactory condition. Specific observations are discussed below. No observations were made of the condition of the upstream face of the dam below the pool elevation at the time of the inspection.

b. The inspection team observed the following conditions at the dam. The general condition of the structure was good. The water level was low, at the time of inspection, exposing the upstream face above the berm level. Borrow areas from the reservoir were visible. No cracking, sliding, sloughing or other signs of settlement or instability were observed.

Wave action is eroding the unprotected upstream face of the dam at the observed pool level (Photo 18). Erosion gullies were observed on the upstream face below the rip rap protection, due to runoff from the embankment slope. The lake level was observed to be 21 feet below the principal spillway crest elevation. The lake has not filled to principal spillway elevation since construction.

The rip rap along the upstream face appeared to have been randomly dumped. The rip rap was contained at the lower edge by a lip which also ponded water on the slope. The weight of the rip rap ranged from 50 to 200 pounds. The downstream slope had a grass cover and appeared uniform above and below the berm.

A rock-lined ditch was observed along the toe of the dam to accommodate surface runoff. Erosion and displacement of rock in the ditch were observed at two locations (Photo 14).

The left abutment was observed to have a low area approximately 275 feet in length with a minimum elevation of 847.4. This section appeared to be an area that was not filled to be consistent with the "As-Built" top of dam elevation (850.0). The majority of discharge around the section will flow into a drainage basin adjacent to the area downstream of the dam.

There was no evidence that a maintenance program was in effect. Unmowed grass was observed on both faces of the dam. No trees were observed on the embankment. An 8-inch outlet to the toe drain system was observed to be dry. Many small animal burrows were observed in the embankment. There was no evidence of seepage through the embankment at the time of inspection.

The old stream bed below the dam was wet and soft, however, it serves as a drainage point for the downstream face of the dam and some adjacent areas. The latest rain had occurred four days earlier.

c. Appurtenant Structures. The inspection team observed the following items pertaining to the appurtenant structures. The principal spillway is a 36-inch diameter concrete pipe connected to a 3 x 9-foot concrete box drop inlet in the embankment. The principal spillway discharge exits onto a concrete stilling basin followed by an open trapezodial channel with rip rap cover. Standing water approximately 1 foot deep was observed in the stilling basin. The spillway was considered to be in good condition. A view through the downstream invert, showed tight joints in the pipe. Some dirt and/or debris were also observed near the inlet invert.

The emergency spillway consists of 150 feet wide trapezoidal section cut in the right abutment. The spillway channel has a good unmowed grass protective cover and no evidence of erosion was observed. Rip rap was observed on both side slopes of the spillway at the control section. It should be noted that abnormally large spillway discharges would probably not damage the embankment since riprap protection of the embankment was provided adjacent to the spillway.

There was no development in the spillway areas which would suffer damage due to flow through the spillways. The raw water intake structure (Photo 21) consists of an adjustable dual intake connected to a 14 inch ductile iron pipe which passes under the dam and leads to the pump house (Photo 22) below the dam. The intake level was located above the observed water surface. The water level, at the time of inspection, registered at approximately 21 feet below the principal spillway crest by the staff gage at the intake structure.

d. Geology.

The soils in the area surrounding the dam and reservoir consist of silty clay developed in loess and in residuum weathered from shales. Thin limestone beds are exposed in the streambed downstream from the dam. The limestone contains two sets of widely-spaced open vertical joints intersecting at approximately 90° to each other and oriented at 45° to the embankment. Weathered shale was observed at the waterline on the upstream face of the embankment (the reservoir was only partially filled).

Samples of the near-surface material in the embankment were taken with an Oakfield sampler near the center of the downstream crest. The material-samples consisted of silty clay and were visually classified as CL materials. Based on these samples, it is surmised that the remainder of the embankment consists of similar CL materials. Erosion areas observed indicate that the embankment material is susceptible to erosion.

e. Reservoir Area. No slumping or slides of the reservoir banks were observed. The lake was noted to be clean with little or no siltation. The reservoir area was cleared of trees and brush up to normal pool level. Borrow areas were visible. The lake was very muddy at the time of inspection.

Located approximately three miles upstream of the Holden Water Supply Dam is Dam Mo. 20073.

f. Downstream Channel. The principal and emergency spillways discharge onto the natural stream bed which flows in an easterly direction approximately parallel to and 300 feet downstream of the dam.

3.2 EVALUATION

The various deficiencies observed at the time of the inspection are not believed to represent an immediate safety hazard. They do, however, warrant monitoring and control.

The erosion of the upstream face is due to the lack of slope protection below the rip rap level. Due to the expected prolonged filling time this erosion will continue unless protection is provided. Erosion could also eventually cause minor slope stability problems if the water level stayed near the observed elevation (819.6) for an extended period of time.

Burrowing animals will continue to damage the embankment if a program is not undertaken to eliminate them. Animal burrows loosen the embankment soils and can cause general deterioration and erosion of the embankment. Piping failure of embankments have resulted from damage caused by burrowing animals.

SECTION 4 - OPERATIONAL PROCEDURES

4.1 PROCEDURES

The pool is primarily controlled by rainfall, runoff, evaporation, transpiration, water supply withdrawal and capacity of the uncontrolled spillways.

4.2 MAINTENANCE OF DAM

There is no evidence that a maintenance program is in effect at this dam. Grass on the embankment and in the emergency spillway channel was uncut. There was evidence of a large population of burrowing animals living in the embankment.

4.3 MAINTENANCE OF OPERATING FACILITIES

Operating facilities do exist. A raw water intake structure in the reservoir connected by a 14 inch ductile iron pipe to a pump station exists near the left abutment.

4.4 DESCRIPTION OF ANY WARNING SYSTEM IN EFFECT

There is no existing warning system or preplanned scheme for alerting downstream residents for this dam.

4.5 EVALUATION

There was no evidence to indicate any efforts have been made since completion of the dam to control erosion and burrowing animals. Although the dam is new and in good condition, it will be necessary to develop a program of regular maintenance for maintaining the dam in safe condition over its expected useful life.

SECTION 5 - HYDRAULIC/HYDROLOGIC

5.1 EVALUATION OF FEATURES

a. Design Data. Design data in the form of "As-Built" drawings and a geologic site investigation report were provided by E.T. Archer Consulting Engineers. Hydrologic/hydraulic design analysis performed by E. T. Archer were provided by the Missouri Department of Natural Resources, Dam Safety Office. The analyses were based on SCS design criteria. Independent calculations were performed by the dam inspection team for the evaluation of this data in accordance with the guidelines referenced in Section 1.1c and the St. Louis District Hydrologic/ Hydraulic Standards, Phase I Safety Inspections of Non-Federal Dams, 22 August 1980. The design data provided for an emergency spillway design based on 8.5 inches of rainfall of 6 hours duration on 4.14 square miles drainage area. This rainfall amount for the specified duration is 32 percent of the probable maximum precipitation (PMP). The design resulted in an emergency spillway maximum discharge of 31 cubic feet per second at a critical velocity of 1.9 feet per second and a maximum water surface elevation of 844.8. The freeboard design considered a 14.2 inch rainfall of 6 hours duration to produce a maximum emergency spillway discharge of 1589 at a critical velocity of 6.9 feet per second. It should be noted that the hydrologic design does not meet the Corps of Engineers guidelines. The resulting maximum water surface elevation was 847.3.

b. Experience Data. The drainage area and lake surface area are from the "As-Built" data and from the U.S.G.S. Elm and Kingsville Quadrangle Maps. The dam crest profile and embankment cross section are from a survey made during the inspection.

c. Visual Observations.

(1) The principal spillway appears to be in good condition. The lake level at the time of the inspection (El. 819.6) was significantly below the principal spillway inlet crest level (840.5). There were no obstructions to flow in the downstream channel.

(2) The emergency spillway appeared to be in good condition. The lake level at the time of inspection was below the crest elevation (844.5). The spillway has a control section approximately 50 feet long followed by a grass lined trapezoidal open channel with critical slope. There were no obstructions to flow in the spillway channel.

(3) During a PMF flood, flow velocity through the emergency spillway will be about 8 feet per second. The embankment adjacent to the emergency spillway is adequately protected from erosion due to spillway flows by riprap. The spillway floor has a good grass cover which provides borderline protection at PMF flows.

c. Overtopping Potential. The spillways will not pass the probable maximum flood without overtopping the dam. The probable maximum flood is defined as the flood discharge that may be expected from the most severe combination of critical meteorologic and hydrologic conditions that are reasonably possible in the region. The spillways will pass 65 percent of the probable maximum flood without overtopping the dam. The spillways will pass the one percent chance flood (100-year flood) developed from a 48-hour, one percent chance rainfall. According to the recommended guidelines from the Department of the Army, Office of the Chief of Engineers, a high hazard dam of intermediate size should pass 100 percent of the probable maximum flood. The portion of the estimated peak discharge of the probable maximum flood overtopping the dam would be 3,992 cfs of the total discharge from the reservoir of 17,730 cfs. The estimated duration of overtopping is 3.0 hours with a maximum height over the dam of 1.1 feet. Considerable erosion damage could occur as a result.

The hydraulic analysis for Holden Lake includes the results of a breach analysis for the upstream impoundment.

According to the St. Louis District, Corps of Engineers, the effect from rupture of the dam could extend approximately twelve miles downstream of the dam. Sixteen buildings, fourteen dwellings, a church, State Highway 131 and U.S. Highway 50 could be severely damaged and lives could be lost should failure of the dam occur. Contents of the estimated downstream damage zone were verified by the inspection team.

SECTION 6 - STRUCTURAL STABILITY

6.1 EVALUATION OF STRUCTURAL STABILITY

a. Visual Observations. Visual observations of conditions which affect the structural stability of this dam are discussed in Section 3, paragraph 3.1b.

b. Design and Construction Data. Design data and "As-Built" drawings relating to the structural stability of the dam were available from E.T. Archer & Company, Consulting Engineers, Subject: Contract Documents and Specifications for Section I - Water Reservoir Dam, Spill-Intake and Appurtenances, Section II - Pumping Station, Raw Water Pipeline and Appurtenances.

As reported in the available data, samples for testing were obtained from borings located within four proposed borrow areas and the dam foundation area. The subsurface investigation, laboratory testing and slope stability analysis were performed by General Testing Laboratories, Inc. of Kansas City, Missouri.

Results of field compaction tests performed during construction of the embankment indicate conformance to the recommended minimum design density of 100 percent of Standard Proctor Density (ASTM D 698). Moisture contents at the required density were generally at or very near optimum moisture.

Laboratory tests performed by General Testing Laboratories for the dam design included:

(1) Foundation Area:

(a) Unconfined Compression Test

(b) Consolidated Undrained Triaxial Shear Test

(c) Direct Shear Test

(d) Atterberg Limit Test

(e) Moisture Content

(2) Embankment Area:

(a) Standard Proctor Test

(b) Atterberg Limit Test

(c) Consolidated Undrained Triaxial Shear Test

(d) Unconfined

c. Stability Loading Conditions

Stability analyses conducted by General Testing Laboratories for design of the dam included consideration of two loading conditions.

(1) End of Construction

(2) Steady Seepage (Post Construction)

d. Stability Analysis

(1) End of Construction: The end of construction loading condition was analyzed for the upstream and downstream embankment slopes. The slope stability analysis was made on a 3 H:1 V embankment slope using the Modified Swedish Circle Method. Soil properties used for this analysis were determined from consolidated undrained triaxial shear tests, direct shear tests and unconfined compression tests, and were representative of embankment and foundation materials. The downstream and upstream embankment slopes for the stability analyses considered a 20-foot berm located at elevation 820.0.

The minimum factor of safety reported for the end of construction loading conditions was 1.40.

(2) Steady Seepage: The steady seepage loading condition was analyzed for the downstream slope. The stability analysis was made on a 3.0 H:1 V embankment slope with a full phreatic line (no drain) considered. A stability analysis considered the presence of a 20-foot berm at Elevation 820.0. The soil properties of the embankment and foundation materials were obtained from consolidated undrained triaxial shear tests, direct shear tests, and unconfined compression tests. Stability determinations were conducted using the prescribed soil properties.

A minimum factor of safety of 1.40 was reported for the steady seepage loading condition.

e. Evaluation. The available stability analyses included soil properties, parameters, and resulting factors of safety for steady seepage and end of construction loading conditions. A factor of safety equal to 1.40 was reported by General Testing Laboratories as the minimum factor of safety on both the upstream and downstream slopes. The test of the analysis did not indicate whether this factor of safety applied to either the end of construction case or the steady state seepage case. It was assumed that the minimum reported value was applicable to both cases.

The stability analyses for the end of construction loading condition for the upstream and downstream slopes indicated a minimum factor of safety equal to 1.40. There are no guidelines in Appendix D governing the end of construction condition, however, the calculated factors of safety appear to represent an adequate design.

The factor of safety reported for the steady seepage loading condition was 1.40, which is less than the suggested value of 1.5 as per Appendix D of the guidelines. Based upon our review of the soil strength properties and assumptions used in the stability analysis as described by General Testing Laboratories, in our opinion, the embankment is adequately designed for the steady seepage condition. It appears that the phreatic surface used for the steady seepage condition conforms to the downstream profile of the dam. If this assumption was used, and no benefit was assigned to the interior drainage control system, then the actual factor of safety for this condition may be higher than the value reported.

Stability analyses for the partial pool loading condition were not available.

An analysis considering a rapid drawdown condition was not performed on the embankment. This is acceptable in view of the fact that there are not facilities present to provide for a rapid drawdown.

Stability analyses for the earthquake loading condition were not available.

Seepage analyses for this dam were not available. A cutoff trench was constructed to reduce the potential for seepage. The embankment was constructed of low permeability CL materials.

f. Operating Records. No operational records were available for review by the inspection team.

g. Postconstruction Changes. No known post construction changes exist.

h. Seismic Stability. The dam is located in Seismic Zone 1 which is a zone of minor seismic risk. A properly designed and constructed earth dam using sound engineerng principles and conservatism should pose no serious stability problems during earthquakes in this zone.

However, an assessment of the seismic stability should be made as required by the guidelines.

SECTION 7 - ASSESSMENT/REMEDIAL MEASURES

7.1 DAM ASSESSMENT

a. Safety. Several conditions observed during the visual inspection by the inspection team should be monitored and controlled. These are erosion gullies on the upstream slope, erosion of upstream slope due to wave action, and animal burrows in the embankment. Seepage and seismic stability analyses comparable to the requirements of the "Recommended Guidelines for Safety Inspection of Dams" were not available, which is considered a deficiency.

b. Adequacy of Information. The conclusions in this report were based only on visual conditions and the available engineering design data. The inspection team considers that these data are sufficient to support the conclusions herein. Seepage and seismic stability analyses comparable to the requirements of the "Recommended Guidelines for Safety Inspection of Dams" were not available, which is considered a deficiency.

c. Urgency. It is the opinion of the inspection team that a program should be developed as soon as possible to implement remedial measures recommended in paragraph 7.2b. If the safety deficiencies listed in paragraph 7.1a are not corrected, they will continue to deteriorate and lead to a potential of failure.

d. Necessity for Phase II. The Phase I investigation does not raise any serious enough questions relating to the safety of the dam nor does it identify any serious dangers which would require a Phase II investigation. However, the additional analyses noted in paragraph 2.5b are necessary for compliance with the guidelines.

e. Seismic Stability. This dam is located in Seismic Zone 1. An assessment of the seismic stability should be included as required by the guidelines.

7.2 REMEDIAL MEASURES

a. Alternatives. The spillway capacity is considered inadequate to meet the guidelines. The spillways should be capable of safely passing the 100 percent probable maximum flood without overtopping the dam. The spillway size and/or height of the dam would need to be increased to effectively pass the spillway design flood. A highly reliable flood warning system should be developed and implemented to warn occupants of the downstream hazard zone.

b. Recommendations. The following remedies to deficiencies should be carried out under the direction of a professional engineer experienced in the design, construction, and maintenance of earth dams.

(1) Seepage and seismic stability analysis should be performed to conform with the guidelines.

(2) The embankment should be monitored closely during filling to check for evidence of seepage and instability.

(3) Erosion protection should be provided for the overflow section at the left abutment for protection against the probable maximum storms.

c. Operation and Maintenance Procedures. The following operation and maintenance procedures are recommended and should be carried out under the direction of a professional engineer experienced in the design, construction, and maintenance of earth dams.

(1) A maintenance program to establish and maintain slope protection and control the growth of brush and trees on the embankment should be developed. Grass/weed cover on the embankments should be cut periodically.

(2) The erosion gullies on the upstream slope of the embankment should be repaired and protected from further erosion with suitable materials.

(3) The animal burrows in the embankment should be corrected since they can lead to piping. Control measures should be implemented to discourage this type of animal activity. The embankment slope should be monitored by a qualified engineer during the repair of the embankment.

(4) A detailed inspection of the dam should be made periodically and documented. More frequent inspections may be required if additional deficiencies are observed or the severity of the reported deficiencies increase.

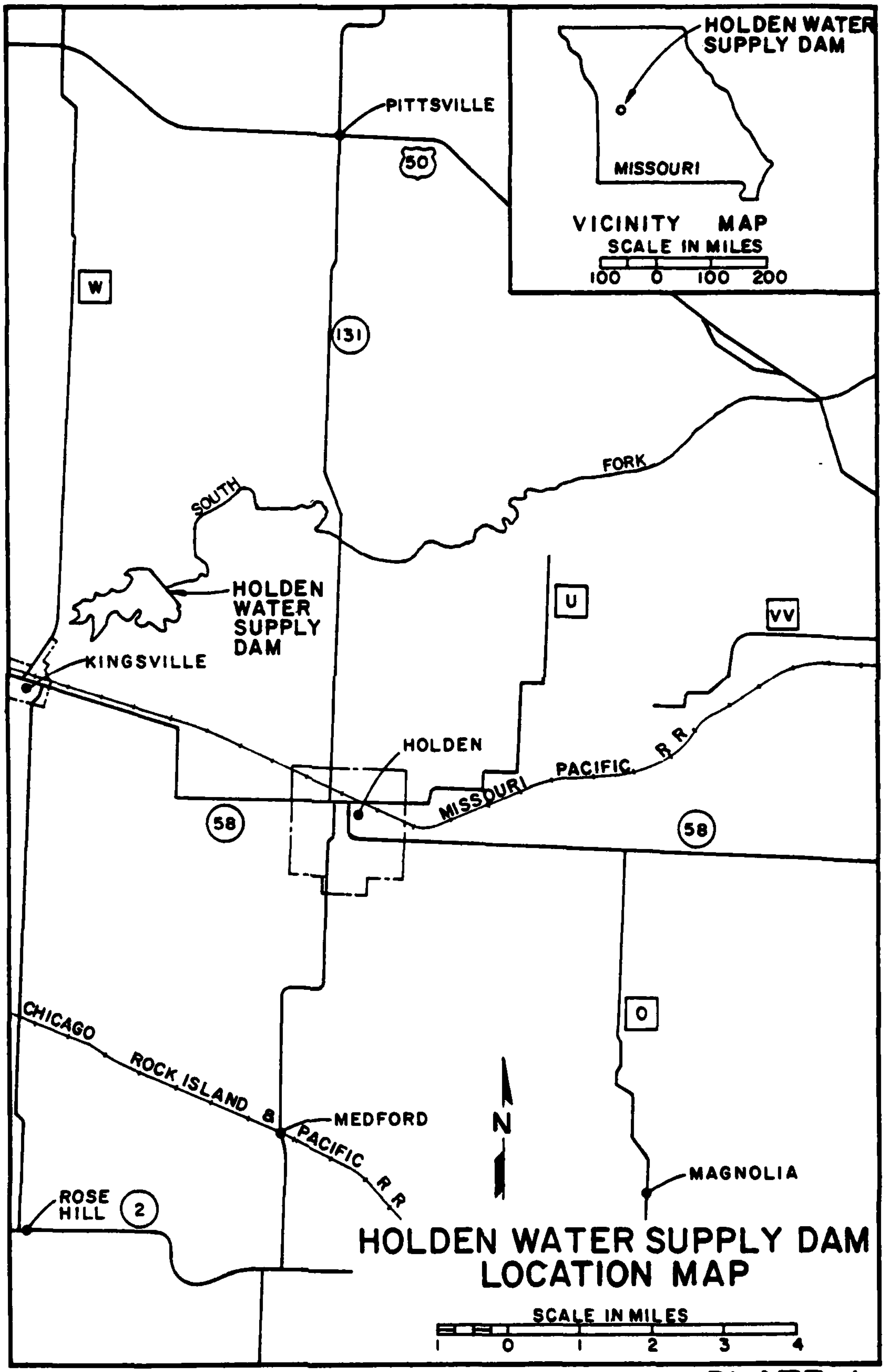

HOLDEN WATER SUPPLY DAM
MISSOURI
VICINITY MAP
SCALE IN MILES
100 0 100 200
PITTSVILLE
50
W
131
SOUTH
FORK
HOLDEN
WATER
SUPPLY
DAM
KINGSVILLE
U
VV
HOLDEN
MISSOURI PACIFIC RR
58
58
O
CHICAGO ROCK ISLAND & PACIFIC RR
MEDFORD
N
MAGNOLIA
ROSE
HILL
2
HOLDEN WATER SUPPLY DAM
LOCATION MAP
SCALE IN MILES
1 0 1 2 3 4

PLATE 1

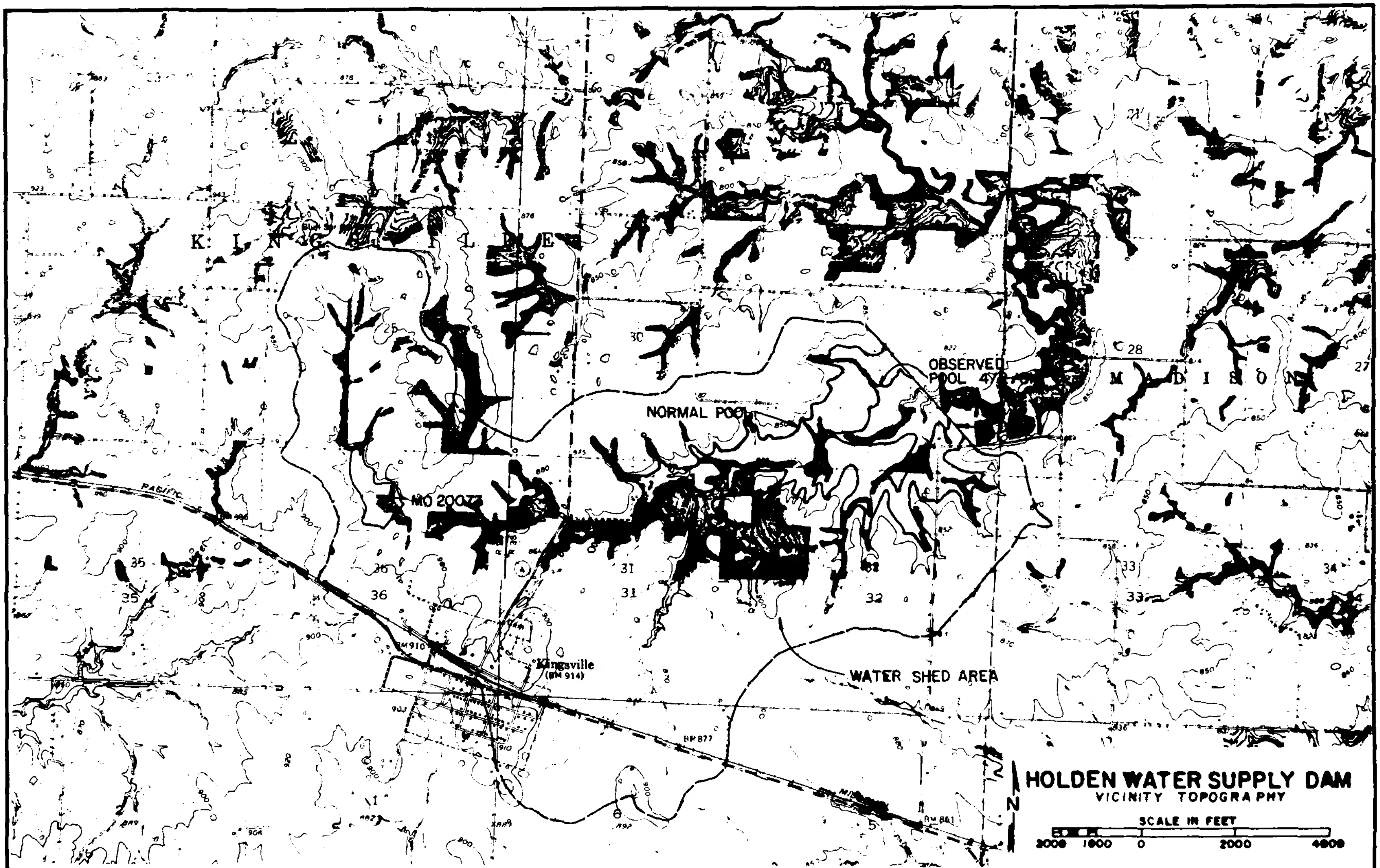
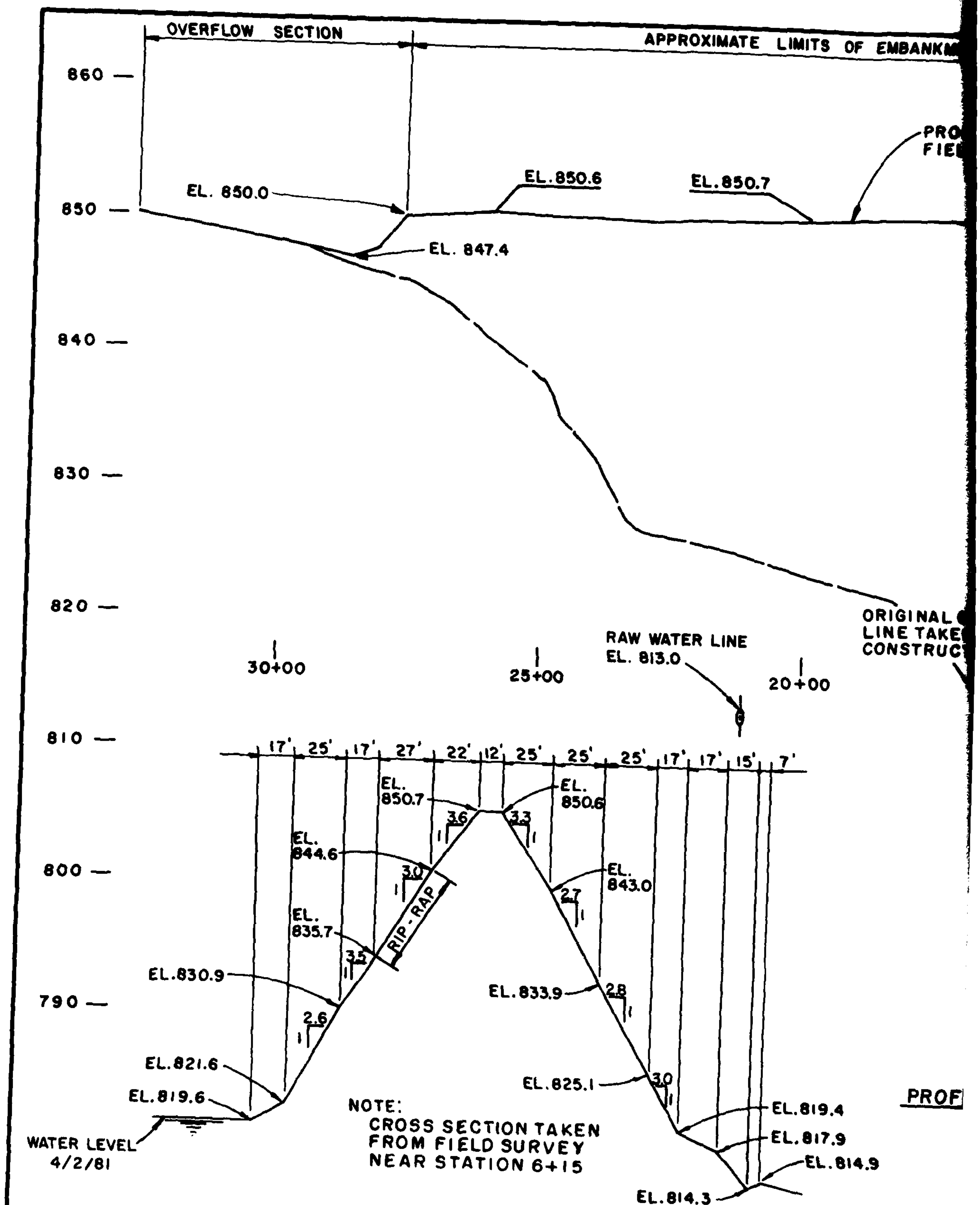
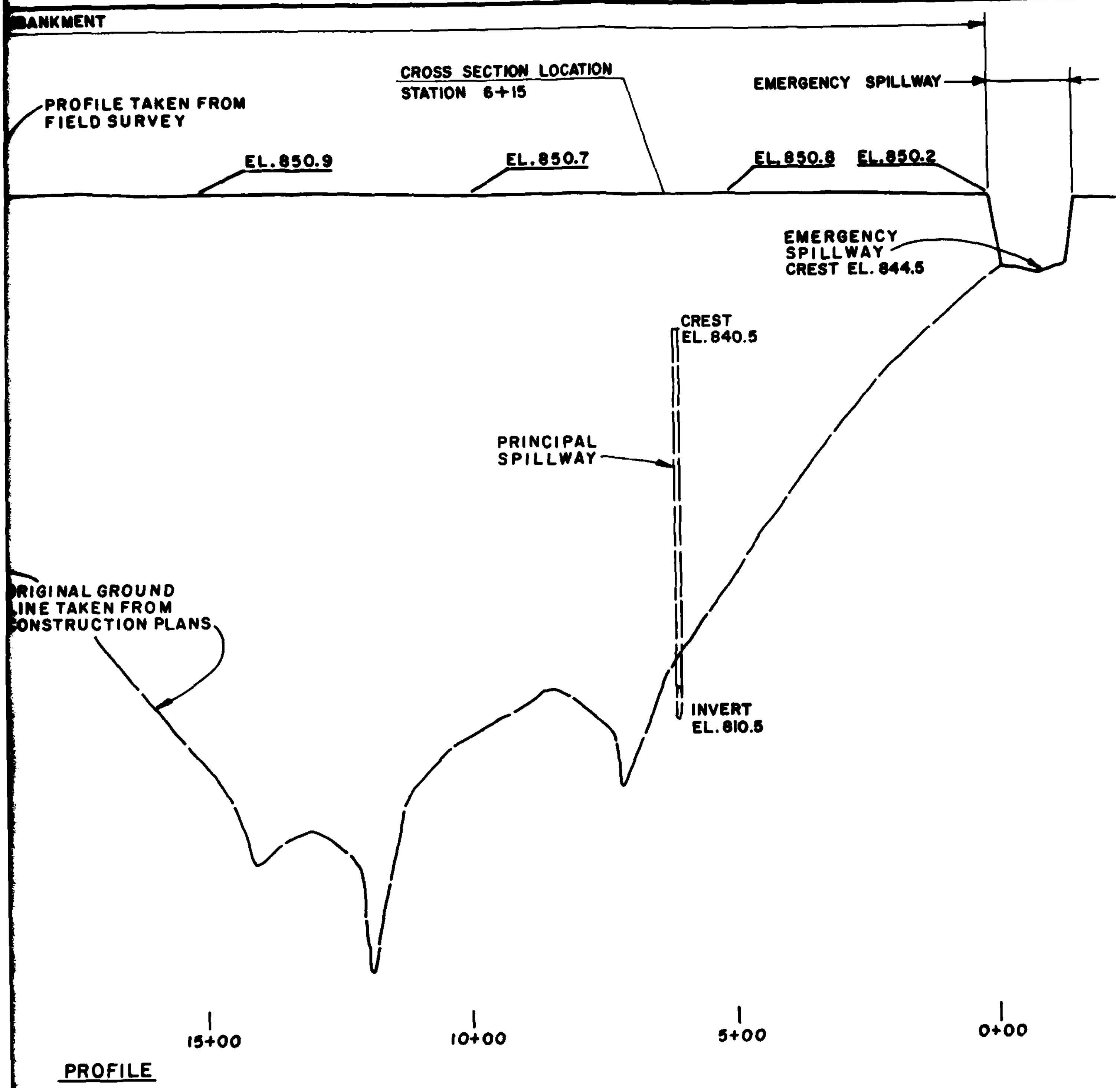

KINGSVILLE
MADISON
OBSERVED POOL
NORMAL POOL
WATER SHED AREA
MO 20077
Kingsville
PACIFIC
HOLDEN WATER SUPPLY DAM
VICINITY TOPOGRAPHY
SCALE IN FEET
2000 1000 0 2000 4000

PLATE 2

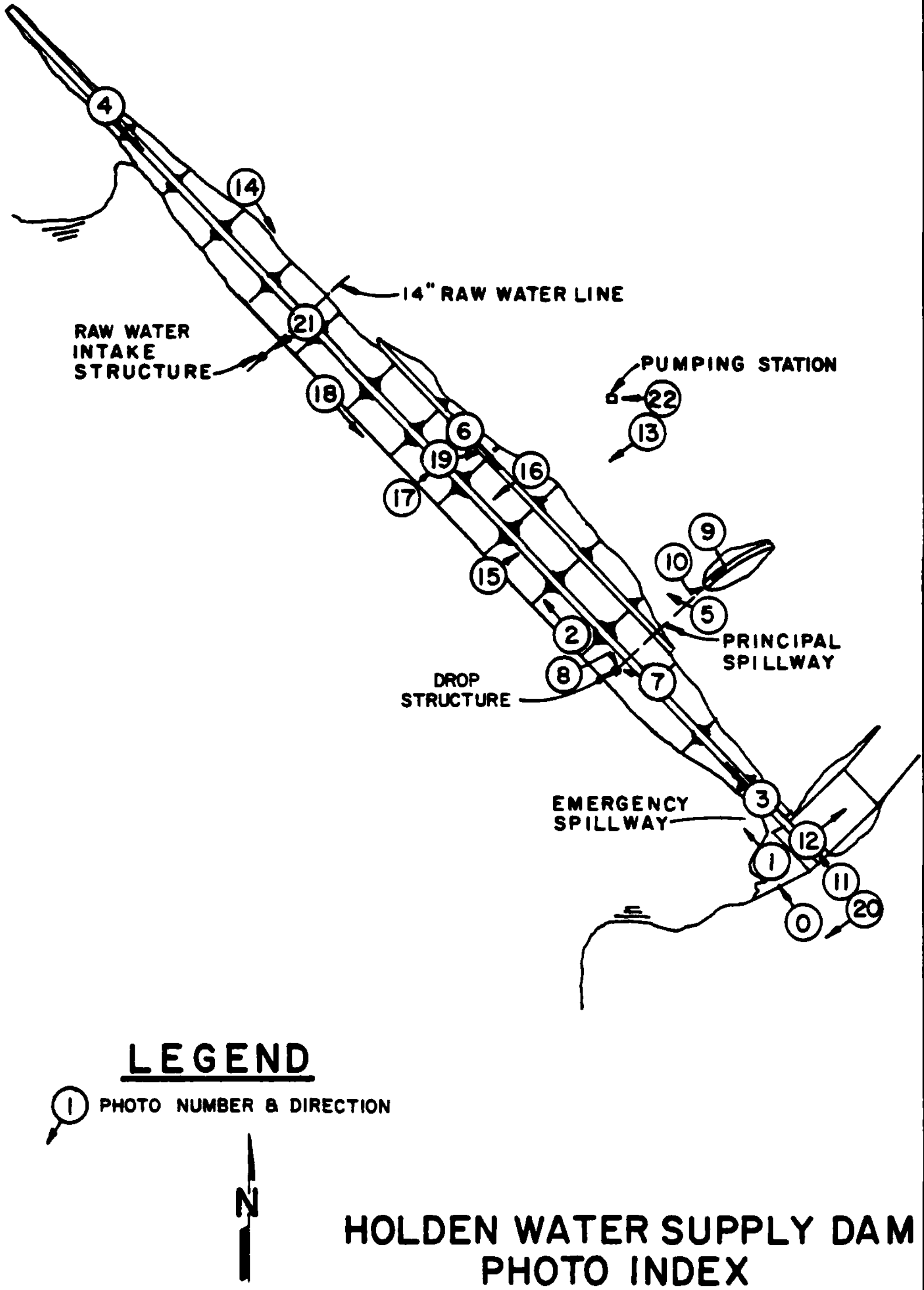


CROSS SECTION



HOLDEN WATER SUPPLY DAM
DAM CROSS SECTION
DAM CREST PROFILE

PLATE 3



HOLDEN WATER SUPPLY DAM
PHOTO INDEX

PLATE 4

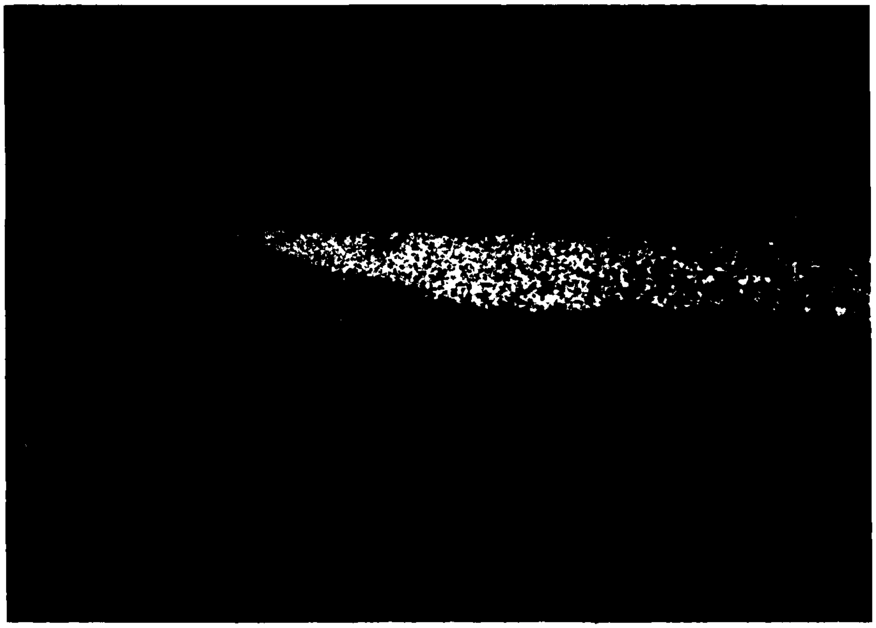


PHOTO 1: UPSTREAM FACE OF DAM

PHOTO 2: UPSTREAM RIPRAP

PHOTO 3: CREST OF DAM LOOKING WEST

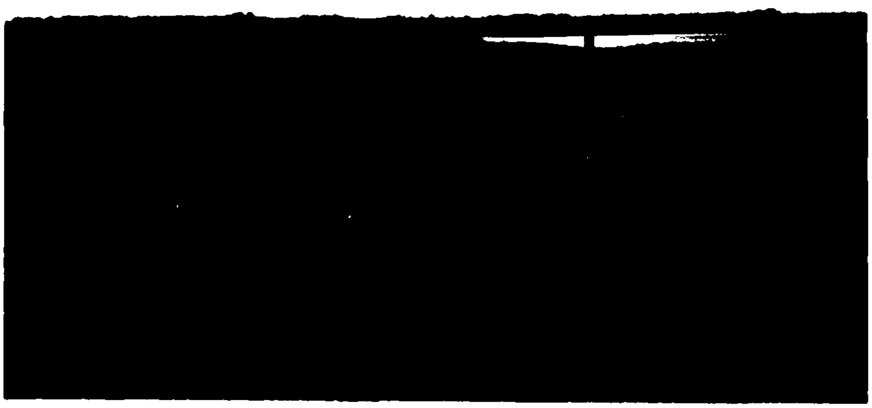


PHOTO 4: CREST OF DAM LOOKING EAST

PHOTO 5: DOWNSTREAM FACE OF DAM

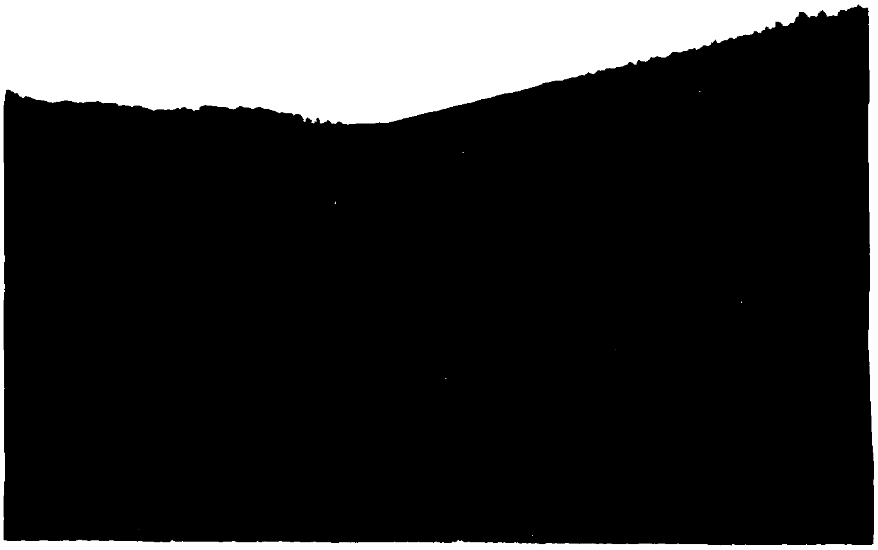


PHOTO 6: DOWNSTREAM FACE OF DAM AT BERM

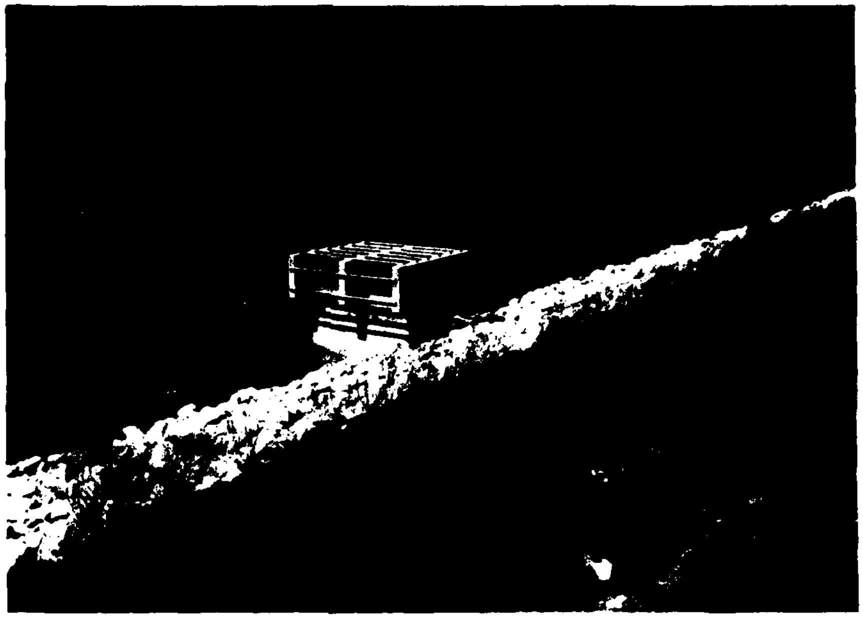


PHOTO 7: PRINCIPAL SPILLWAY DROP INLET STRUCTURE

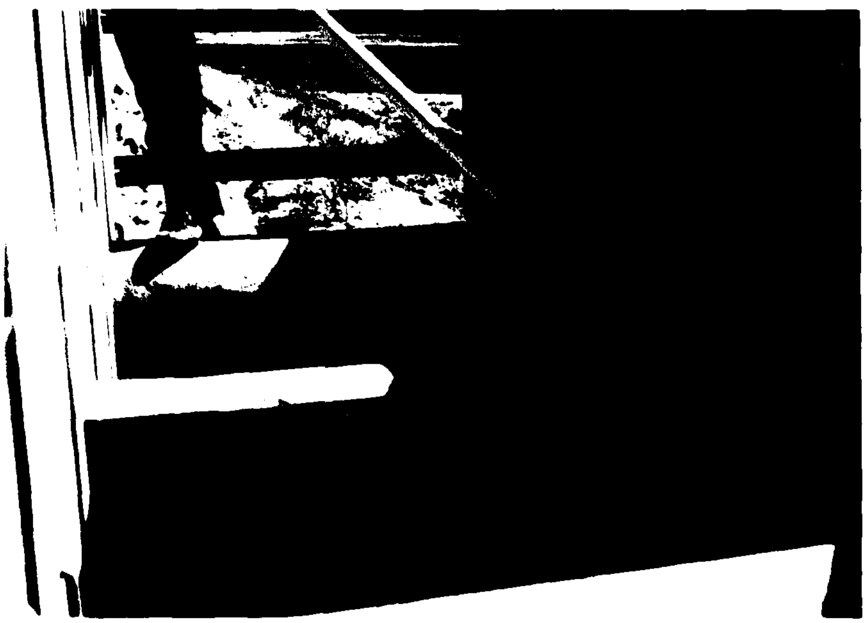


PHOTO 8: DROP INLET WEIR

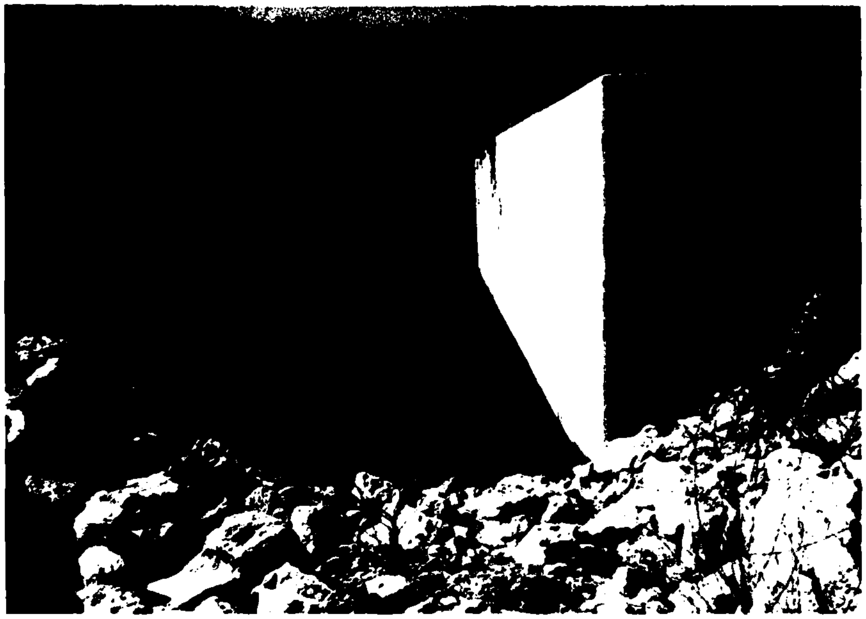


PHOTO 9: PRINCIPAL SPILLWAY PIPE OUTLET AND STILLING BASIN

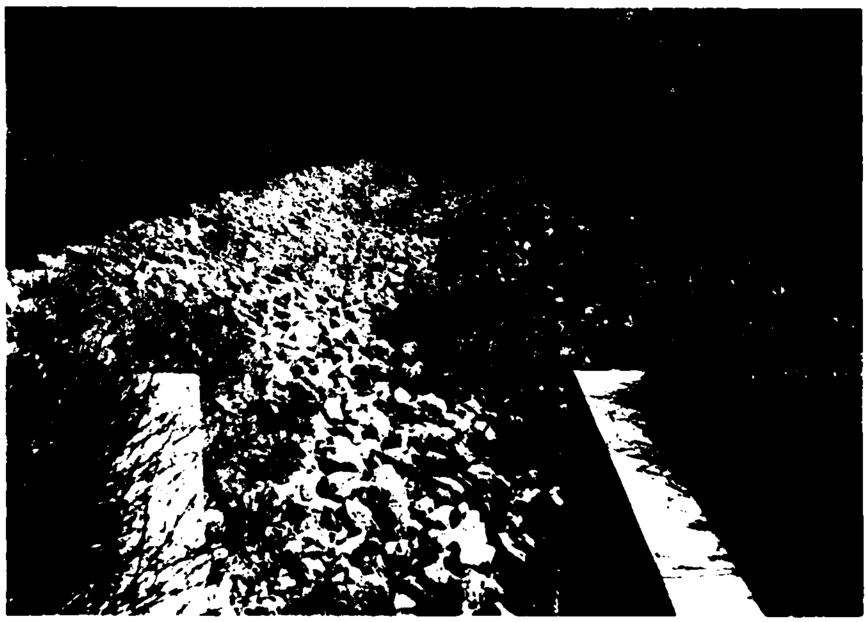


PHOTO 10: CHANNEL DOWNSTREAM OF PRINCIPAL SPILLWAY STILLING BASIN

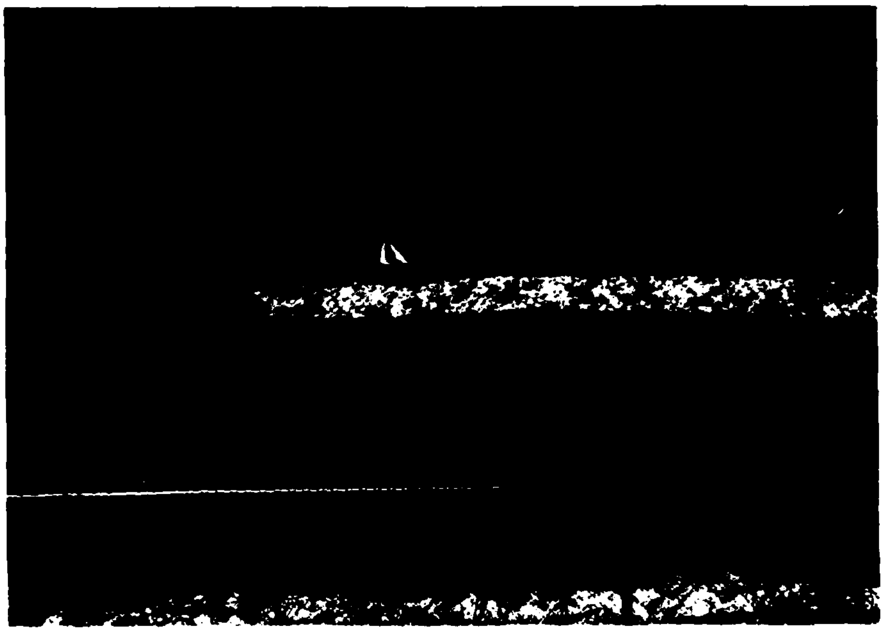


PHOTO 11: EMERGENCY SPILLWAY AT CENTERLINE OF DAM

PHOTO 12: EMERGENCY SPILLWAY CHANNEL

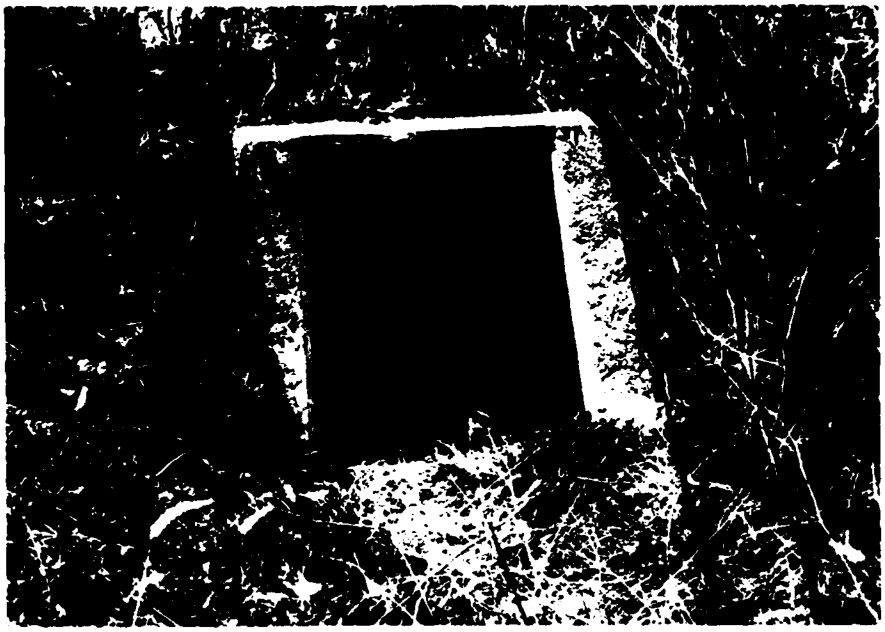


PHOTO 13: OUTLET FROM TOE DRAIN SYSTEM

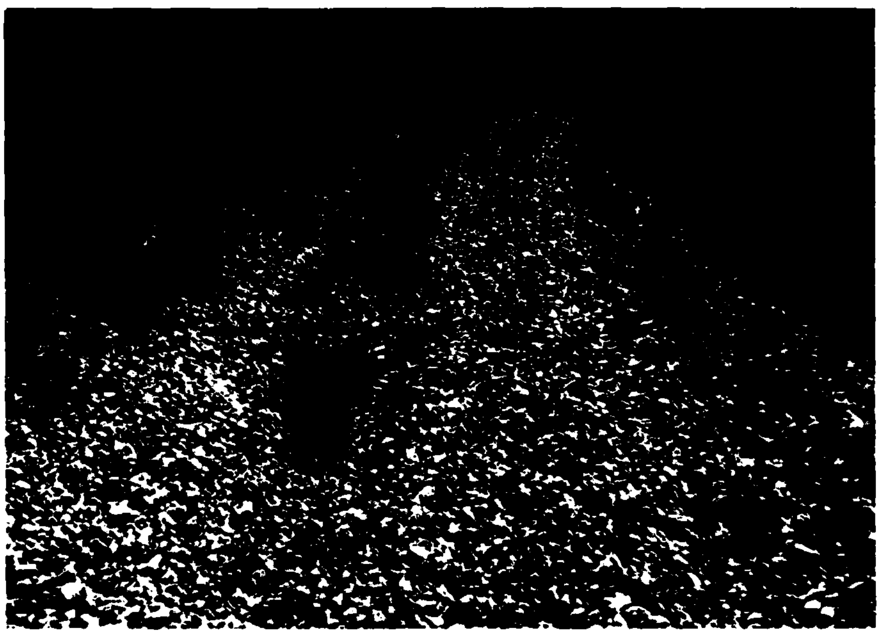


PHOTO 14: EROSION OF DITCH AT DOWNSTREAM TOE OF DAM

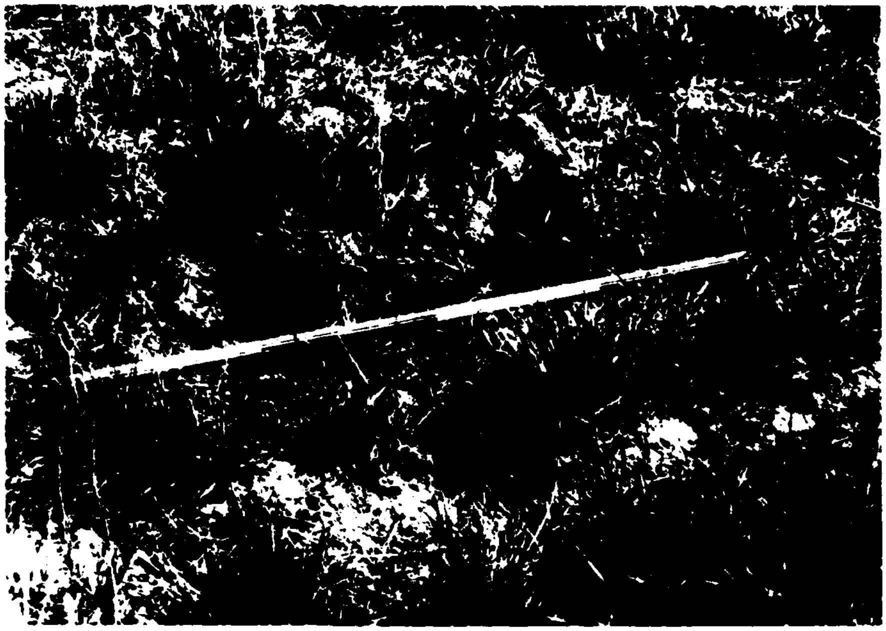


PHOTO 15: ANIMAL BURROWS ON UPSTREAM FACE

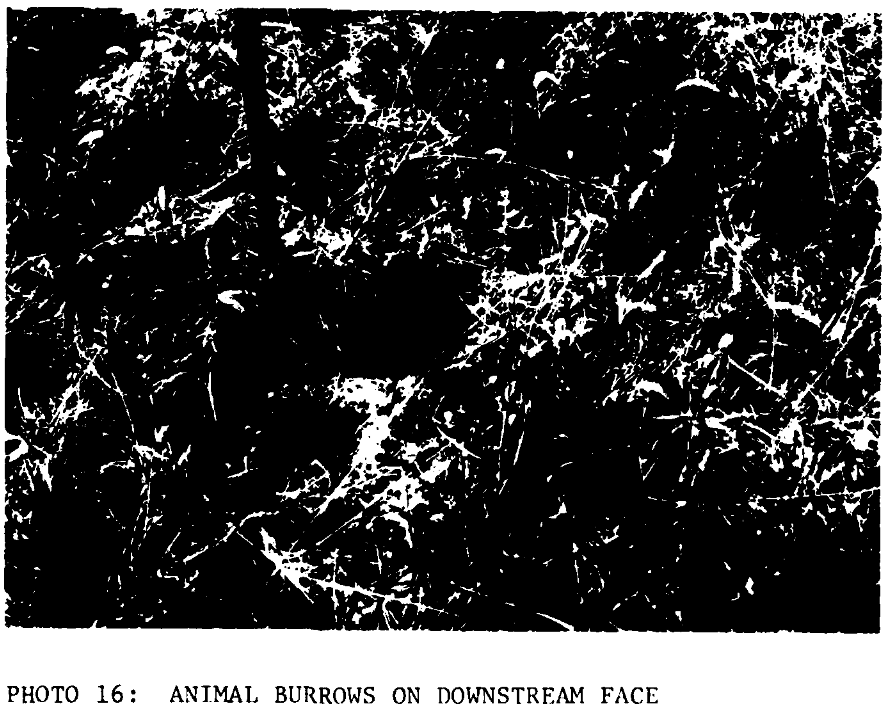


PHOTO 16: ANIMAL BURROWS ON DOWNSTREAM FACE

PHOTO 17: EROSION OF UPSTREAM FACE BELOW RIPRAP

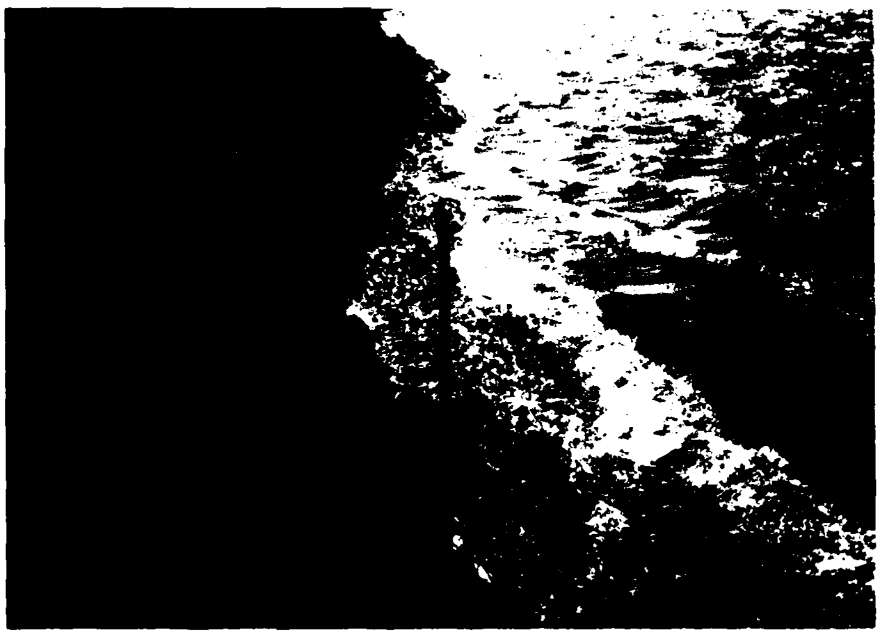


PHOTO 18: EROSION OF UPSTREAM FACE AT WATERLINE

PHOTO 19: AREA DOWNSTREAM OF DAM

PHOTO 20: PERTINENT DATA SIGN

PHOTO 21: RAW WATER INTAKE STRUCTURE

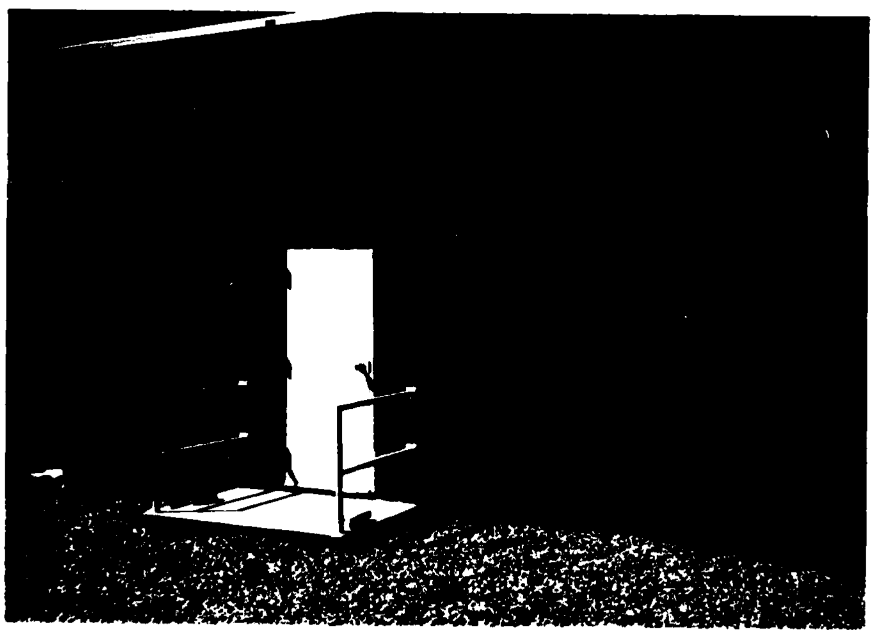


PHOTO 22: RAW WATER PUMPING STATION

APPENDIX A

HYDROLOGIC AND HYDRAULIC ANALYSES

HYDROLOGIC AND HYDRAULIC ANALYSES

To determine the overtopping potential, flood routings were performed by applying the Probable Maximum Precipitation (PMP) to synthetic unit hydrographs to develop the inflow hydrographs for Holden Water Supply Dam and the upstream reservoir (Mo. 20073). The inflow hydrographs were then routed through the reservoirs and spillways. The overtopping analysis was determined using the computer program HEC-1 (Dam Safety Version) (1).

The PMP was determined from regional charts prepared by the National Weather Service in "Hydrometeorological Report No. 33" (HMR-33) (2). Reduction factors were not applied. The rainfall distribution for the 24-hour PMP storm was determined according to the procedures outlined in HMR-33 and EM 1110-2-1411 (3). The Kansas City, Missouri rainfall distribution (10 min. interval - 48 hours duration), as provided by the St. Louis District, Corps of Engineers, was used when the one percent chance probability flood was routed through the reservoirs and spillways.

The synthetic unit hydrographs for the watersheds were developed by the computer program using the Soil Conservation Service (SCS) method (1, 5). The parameters for the unit hydrographs are shown in Table 1. Lag time and time of concentration were verified by two different methods. The results used in the analyses were obtained by using Soil Conservation Service (SCS) design criteria provided by the Missouri Department of Natural Resources (DNR).

The SCS curve number (CN) method was used in computing the infiltration losses for rainfall-runoff relationship. The CN values used and the result from the computer output are shown in Table 2.

Storms were routed through the two reservoirs noted above. Routing through the reservoirs was performed using the modified Puls Method. The initial reservoir pool elevation for the routing of the one percent probability storm was determined to be equivalent to the crest elevation of the principal spillway (840.5) in accordance with antecedent moisture condition (AMC) II preceding the storm as outlined by the U.S. Army Corps of Engineers,St. Louis District (4). Storms of 25 percent and 50 percent of the PMF were treated as antecedent storms preceding the 50 percent and 100 percent PMF storms respectively. The initial reservoir pool elevation for the routing of the probable maximum storms was determined to be 842.7 in accordance with antecedent storm condition AMC III preceding the storms. The hydraulic capacity of the spillways and the storage capacities of the reservoirs were defined by the elevation, surface area, storage, and discharge relationships shown in Table 3.

The rating curves for the spillways are shown in Table 4. The flow over the crests of the dams was determined using the nonlevel dam crest option ($L and $V cards) of the HEC-1 program. The program assumes critical flow over a broad-crested weir. The flow through the principal and emergency spillways were verified by nomographs for pipe culverts with outlet control (6) and broadcrested weirs (7) respectively. The results used in the analyses were taken from E. T. Archer design calculations and based on SCS design criteria.

Where routings through the upstream reservoir resulted in overtopping of that structure, breaching analysis on the Dam (MO 20073), located upstream of the Holden Water Supply Dam, was performed based on hydraulic parameters from the previous inspection on June 1980. The breaching parameters are noted in Table 5. The upstream dam was assumed to breach and degrade thereby releasing essentially all stored water to the downstream structure.

The results of the routing and breach analyses indicate that a flood equivalent to a maximum of 65 percent of the PMF will not overtop Holden Water Supply Dam.

A summary of the routing analysis for different ratios of the PMF is shown in Table 6.

The computer input data and a summary of the output data are presented at the back of this appendix.

"As-Built" drawings and hydrologic-hydraulic design data were made available by E.T. Archer Consulting Engineers and the Department of Natural Resources respectively.

TABLE 1

SYNTHETIC UNIT HYDROGRAPH

Parameters:	Upper Dam	Lower Dam**
Drainage Area (A)	425 acres	2,225 acres
Lag Time (L_g)	0.32 hours	1.40 hours
Time of Concentration (T_c)	0.54 hours	2.33 hours
Duration (D)	4.3 minutes	18.6 minutes
	(use 10 minutes in each case)	

Unit Hydrograph Ordinates
Discharge (cfs)*

Time (Min.)*	MO 20073 (Upper Dam)	Holden Water Supply (Lower Dam)
0	0	0
10	264	98
20	754	286
30	719	586
40	404	992
50	204	1,317
60	104	1,492
70	53	1,501
80	27	1,405
90	14	1,240
100	7	1,030
110	4	774
120	0	592
130		463
140		370
150		292
160		234
170		178
180		140
190		109
200		84
210		67
220		53
230		41

*From HEC-1 Computer Output.
**Excludes Controlled Drainage Upstream.

Unit Hydrograph Ordinates
Discharge (cfs)*

Time (Min.)*	MO 20073 (Upper Dam)	Holden Water Supply (Lower Dam)
240		32
250		25
260		20
270		16
280		13
290		10
300		8
310		5
320		3
330		1

* From HEC-1 Computer Output.

FORMULAS USED:

T_c was obtained from SCS watershed design data provided by DNR.

$L_g = 0.6\ T_c$

$D = 0.133\ T_c$

TABLE 2

RAINFALL-RUNOFF VALUES

Selected Storm Event	Storm Duration (Hours)	Rainfall (Inches)	Runoff (Inches)	Loss (Inches)
Upper A-26 Dam PMP	48	35.00	33.70	1.30
Holden Water Supply Lower Dam PMP	48	35.00	33.84	1.16
Upper Dam 100 yr.	48	8.78	6.12	2.66
Holden Water Supply Dam 100 yr.	48	8.78	6.36	2.42

Additional Data:

1) 100 Percent of Drainage Area in Hydrologic Soil Group B(7).
   40 Percent of the Land Use was Cropland.
   40 Percent of the Land Use was Grassland.
   20 Percent of the Land Use was Timberland.

2) SCS Runoff Curve CN = 91 (AMC III) Lower Lake Dam
   90 (AMC III) Upper Lake Dam
   for the PMF (5).

3) SCS Runoff Curve CN = 80 (AMC II) Lower Lake Dam
   78 (AMC II) Upper Lake Dam
   for the one percent probability flood (5).

TABLE 3

ELEVATION, SURFACE AREA, STORAGE, AND DISCHARGE RELATIONSHIPS

Elevation (feet-MSL)	Lake Surface Area (acres)	Lake Storage (acre-ft)	Spillway Discharge (cfs)
Holden Water Supply Dam			
*840.5	302	3,413	0
**844.5	366	4,590	151
***850.2	460	6,516	8,332
Upper Dam			
*894.0	18.6	98	0
**897.3	28.2	174	42
***899.6	36.0	248	316

*Principal spillway crest elevation
**Emergency spillway crest elevation
***Top of dam elevation

The relationships in Table 3 were developed from the Elm, Missouri. 7.5 minute quadrangle map, field measurements, and engineering documents provided by E.T. Archer Consulting Engineers.

TABLE 4

SPILLWAY RATING CURVES

Reservoir Elevation (ft-msl)	Principal Spillway Discharge (cfs)	Emergency Spillway Discharge (cfs)	Overflow Section Discharge (cfs)	Total Spillway Discharge (cfs)
Holden Water Supply Dam				
*840.5	0	-	-	0
842.0	103	-	-	131
**844.5	151	0	-	151
847.4	189	1,710	0	1,899
***850.2	196	5,704	2,632	8,532
851.3	198	8,480	5,060	13,738
Upper Dam				
*894.0	0	-		0
895.0	29	-		29
896.0	41	-		41
**897.3	42	-		42
898.3	43	40		83
***899.6	46	270		316

*Principal Spillway Crest Elevation
**Emergency Spillway Crest
***Top of Dam Elevation

METHOD USED:

Principal and Emergency Spillway Release Rates were determined from SCS Watershed design data which utilized the weir flow and pipe flow equations.

TABLE 5

BREACHING PARAMETERS

	Upper Dam (MO 20073)
Bottom Width of Breach (BRWID)	10 feet
Side Slope of Breach (z) (In feet horizontal to 1.0 feet vertical)	0.5
Elevation of Breach Bottom at Maximum Size of Breach (ELBM)	879.9 ft. m.s.l.
Time for Breach to Develop to Maximum Size (TFAIL)	1.0 hour
Elevation of Water Surface Which Will Cause Dam to Fail (FAILEL)	899.6 ft. m.s.l.

TABLE 6

RESULTS OF FLOOD ROUTINGS

Ratio of PMF	Peak Inflow (CFS)	Peak Lake Elevation (ft.-MSL)	Total Storage (AC.-FT.)	Peak Outflow (CFS)	Depth (ft.) Over Top of Dam	Duration of Over-topping (Hrs)
-	0	*840.5	3,413	0	-	-
0.50	73	849.1	6,137	4,952	-	-
0.65	226	850.1	6,478	7,870	0	-
0.70	658	850.4	6,572	9,041	0.2	1.2
1.00	1,302	851.3	6,901	17,730	1.1	3.0

* Principal Spillway Crest Elevation

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(3) EM-1110-2-1411, Standard Project Flood Determinations, U.S. Army Corps of Engineers, 26 March 1952.

(4) U.S. Army Corps of Engineers, St. Louis District, Hydrologic/Hydraulic Standards, Phase I Safety Inspection of Non-Federal Dams, 22 August 1980.

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(6) U.S. Department of Commerce, Bureau of Public Roads, Hydraulic Charts for the Selection of Highway Culverts, December 1965.

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APPENDIX B

GEOTECHNICAL AND SOILS REPORT

CITY OF HOLDEN, MISSOURI

CONTRACT DOCUMENTS AND SPECIFICATIONS

FOR

SECTION I
WATER RESERVOIR DAM, SPILLWAY,
INTAKE AND APPURTENANCES

SECTION II
PUMPING STATION, RAW WATER PIPELINE
AND APPURTENANCES

BIDDER

ADDRESS

E. T. ARCHER & COMPANY
CONSULTING ENGINEERS
KANSAS CITY, MISSOURI

1978

T A B L E   O F   C O N T E N T S

TABLE OF CONTENTS (CONT'D)

ILLUSTRATIONS

APPENDICES

INTRODUCTION:

Presented in this report are the results of our investigation of the soil and foundation conditions, results of laboratory testing, location of suitable borrow materials for the embankment and a discussion of slope stability analysis for the proposed Holden Dam to be constructed in a site located in Johnson County, MO. It is our understanding that the following approximate elevations will apply to the dam: top of embankment, elevation 850'; and normal pool water level, elevation 840'.

The purpose of this investigation were to provide information for design of a safe and economical structure and to establish embankment slopes with an adequate factors of safety. This information was obtained through a three-phase study program which included the following:

A. Determination of the soil condition at the site by means of visual inspection, sample and auger borings;

B. Determination of strength and physical characteristics of the foundation and embankment soils by laboratory and field testing; and

C. Engineering analysis of the data developed from the field and laboratory studies with recommendations for design criteria.

SOIL EXPLORATION:

The soils conditions at the site of the proposed dam were explored and samples of the soil strat. were obtained by means of sample borings drilled on or adjacent to the center of the proposed dam (Borings 1 through 18) and in the area of the emergency spillway (Borings 101 through 106), and in the area of the primary spillway (Borings 107 through 108). Potential borrow areas located upstream from the dam centerline were investigated by a combination of auger and sample borings (Borings 201 through 505). The location of all borings are shown on Plates 1 and 1A. Logs of the borings presenting descriptions of the various soils encountered and with results of laboratory tests were presented in the Appendix.

From the borings along and adjacent to the dam centerline, samples of cohesive soils were obtained using a 3 inch diameter Shelby tube sampler which was forced into the soil by hydraulic cylinders on a rotary drilling rig. Samples of shaley soils, too hard for the thin wall sampler, were obtained using a 2 inch diameter split spoon sampler which was forced into the soil by blows of a 140 pound hammer dropped 30 inches. The number of blows required to drive the standard split spoon the final 12 inches of 18 inch drives, or portion thereof, is recorded on the boring logs under the blows per foot column. Where hard shale and limestone were encountered, continuous cores were obtained using a NX size core barrel with diamond bits. Representative samples were obtained at the borrow areas from auger cuttings and thin wall sampling techniques. All samples were placed in appropriate containers for protection and transferred to our laboratory for testing and evaluation.

In general, sample borings were drilled along the proposed centerline at a spacing of approximately 150 feet, except where topographic features precluded movement of equipment. Additional borings were drilled in the areas of the primary and emergency spillways.

LABORATORY AND FIELD TEST:

A variety of laboratory and field tests were performed to evaluate the condition of the foundation soils in the area of the embankment and to develop strength parameters of the existing soils and of the soils from the borrow area for use in constructing the dam. The overall testing program for the foundation and embankment soils is discussed in the following paragraphs.

In the laboratory, the foundation soils and remolded embankment soils were evaluated by performing the Unconfined Compression Test, Triaxial Shear Test and Direct Shear Test. Classification tests were performed on selected specimens of both the foundation soils and samples from the borrow areas.

Selected shear test specimens were saturated prior to testing. Shear strength parameters were developed for both unconsolidated and consolidated conditions. Results of the various strength tests performed on specimens from the foundation borings are presented on the respective boring logs at appropriate depths. Unconfined and triaxial compression test results are presented on small open circles and triangles, respectively, plotted to the cohesion scale.

SAMPLE PREPARATION:

Embankment soils were investigated as indicated above by compacting remolded specimens of the borrow soils in the laboratory. These soil specimens were compacted using a Harvard miniature compaction device for the triaxial test and static compaction procedures for the direct shear test. The exact weight of soil and water was mixed and the material placed in the respective molds and compacted to the density of approximately 95 percent of Maximum Density at Optimum Moisture Content as determined by the standard compaction procedure (ASTM D 698).

STRENGTH TESTING:

Following preparation of the specimen, unconsolidated-undrained, and consolidated-undrained triaxial shear tests were performed to develop strength parameters. In a triaxial shear tests, a specimen is enclosed in a rubber membrane, a confining pressure is applied and the sample is loaded axially while recording the stress-strain relationship. In the unconsolidated-undrained test no drainage of pore water was permitted during the test. In the consolidated-undrained test, the specimen is consolidated under selected confining pressures with drainage being allowed. Following consolidation of the specimen, the specimen is loaded axially to pending shear failure. Results of triaxial tests are presented on Plates 58 through 60 in Appendix C.

Direct shear tests were performed on compacted specimens of selected embankment soils. For direct shear, a one inch high X two inch square specimen is placed in a split ring and a normal load is applied. The specimen is allowed to consolidate under the normal load and then is sheared horizontally at a constant and uniform rate. The rate of shear is selected to provide either drained or undrained conditions during the test. Results of direct shear tests are presented on Plate 62 in Appendix C.

CLASSIFICATION TESTS:

Classification and compaction tests were performed on selected representative soil samples obtained from the centerline borings and from the potential borrow areas. These tests included Plastic and Liquid Limit Tests, Sieve Analyses and Moisture-Density Tests. Results of the classification tests are plotted on the boring logs and summarized on Plates 7 through 58 in the Appendix . Results of the standard moisture density tests are presented on Plates 50 through 58 in Appendix C.

GENERAL SOIL CONDITIONS:

Geologically, the soils at the site are residual soils resulting from weathering of the underlying Marmaton Group of the Desmoinesian Series of the Pennsylvanian System. This geologic formation consists of silty shales with limestone and thin coal layers.

Generally the upper portion of shales are weathered, becoming less weathered at depths of 5 to 6 fe... Limestone layers are generally massive with few fractures and bedding planes. A slight dip in the bedding of the observed strata was observed from south to north.

To aid in visualizing soil conditions, a generalized soil profile was prepared and is shown on Plate 2 . As can be seen, the upper stratum, which varies in thickness from about 2 feet to more than 20 feet is silty clay, CL, according to the Unified Soil Classification System and was found to have cohesive shear strengths varying from 0.7 TSF to more than 2 TSF. The upper 6 inches to 1.0 foot of this stratum contains significant amounts of organic material. The stratum is a residual soil resulting from the weathering of the underlying shale stratum. Below this silty clay, a silty shale was observed. This stratum was found to be moderately soft to moderately hard, generally increasing in consistency with depth. At the southeast end of the proposed dam, two layers of limestone separated by a shale stratum, were found. The shale above and below the limestone was observed to have significant amounts of calcium deposits. The limestone observed was not found to be cavernous and no evidence of water loss was observed during the exploratory drillings. Along the north and west portion of the dam structure, shale was observed to the maximum depth explored. The shale generally became dark gray with depth and moderately hard to hard.

Soils observed in the area proposed for emergency and primary spillways were found to be consistent .ith those below the dam centerline. Generalized soil profiles illustrated in the soils encountered are shown on Plate 2. In the area which was proposed for excavation of the emergency spillway, shales were encountered at depths of two to three feet and became progressively harder with depth. It is believed that these shales can be excavated with conventional earth moving equipment. However, at lower depths the shales may need to be ripped with moderately sized dozers with single tooth rippers.

Four potential areas were investigated to determine the availability of fill for use in embankment construction as indicated on the boring plans. Borrow areas A, C and D were found to have 8 to 10 feet of silty clay to shaley clay while area B was found to have hard rock at shallow depths along the western edge of the area with 6 to 8 feet of clay soils along the eastern edge. It is anticipated that areas C and D could be expanded to the west and south if additional fill is required.

The significant features of the soils encountered which are considered pertinent to design and construction of the proposed dam are: (a) the moderate to high shear strength of the soils below the proposed embankment; (b) the relative impermeability of the soils below the embankment; (c) the availability of adequate quantities of silty clay for construction of an impervious dam; (d) the presence of approximately one foot of organic matter which will need to be undercut prior to placing fill; and (e) the possibility of some free water being encountered at depths of 6 to

10 feet in the borrow areas. The relationship of these factors to design and construction of the proposed embankment is discussed in the following paragraphs.

ANALYSIS AND RECOMMENDATIONS:

Presented in the following paragraphs is a discussion of the selection of shear strength parameters, method of slopes stability analysis, recommended embankment cross section, discussion of probable embankment settlement, location of borrow materials, and compaction and field control procedures.

GENERAL DESIGN CRITERIA:

In analysis of dam embankment design, the following criteria should be considered. First, the slopes of the embankment must be stable during construction and during all conditions of reservoir operation. Second, the embankment must be designed so that it does not impose excess stresses on the underlying foundation soils. Third, seepage flow through the embankment must be controlled and the amount limited to that tolerable by proposed usage of the reservoir. Finally, the slopes and dam crest must be protected against erosion by waves, wind and rainfall. These factors have been considered in selection of shear strength parameters for analysis of the slopes stability and in embankment design.

SHEAR STRENGTH PARAMETERS:

Selection of shear strength parameters for use in slope stability analysis was made following a review of the results of laboratory testing performed on representative soils specimens both from the foundation soils and the embankment materials. This testing program has been discussed in previous sections of this report. The minimum shear parameters determined from a variety of testing conditions were selected for use in the stability analysis. Generally these minimum parameters represent the unconsolidated-undrained, or consolidated-drained conditions. The unconsolidated-undrained conditions simulate the stress conditions in embankment during and immediately following construction. Consolidated-drained conditions simulate the post construction conditions.

The parameters selected for use in slope stability analysis for the construction condition are as follows:

Period	Area	c	ϕ
Construction	Embankment	0.50 TSF	5.0°
Construction	Foundation Soils	0.80	3.5
Post Construction	Embankment	0.27	11.0
Post Construction	Foundation Soils	0.80	3.5

SLOPE STABILITY ANALYSES:

Slope stability analyses were performed on both the upstream and the downstream slopes. The analyses were performed using the circular arc method and the strength parameters outlined above. In the computations no rapid drawdown was considered since no provisions for drawdown was made in the design. However, steady-state seepage was considered in the analyses. An illustration of the method of analysis used for slope stability computations is shown on Plates 3 and 4.

The results of the slope stability analyses indicate a minimum factor of safety on both the upstream and downstream face of 1.4, using a slope of 3.0 horizontal to 1.0 vertical and having a 20 foot wide berm at elevation 820'. These minimum factors of safety are considered adequate in view of the usage of the structure, conservative selection of shear strength parameters, and assumed seepage conditions.

RECOMMENDED EMBANKMENT DESIGN:

A section showing the recommended embankment design for the proposed dam is presented on Plate 5. The various features of this embankment were selected based on stability computations, available borrow materials and maintenance considerations. Four separate zones are delineated in the embankment. Each of these zones is discussed separately in the following paragraphs.

Zone 1 - Zone 1 should contain available silty clay and sandy clay soils found in all borrow pits. These soils are generally encountered at depths of one to two feet and extend to depths of six to twelve feet. Soils used in this zone should be classified as CL according to the Unified Soil Classification System and should have Liquid Limits ranging from 32 to 44.

Zone 2 - Zone 2, the upstream and downstream, random fill portion of the embankment, should contain silty clays and shaley clays grading to clay shales. Soils in this portion of the embankment should be selectively placed, in so far as possible, so that the more shaley material will be in the ou-. r portion of the embankment while the finer grained silty clays will be in the inner portion of the embankment.

Zone 3 - Zone 3, is a near vertical sand chimney drain, placed immediately downstream of the clay core to aid in lowering the phreatic line, thereby increasing the stability of the downstream face and preventing water from existing on the downstream slope. The chimney drain will collect flow from seepage through the clay core and allow all seepage to drain through the chimney drain to the collector system at the bottom of the drain. Seepage water should be piped a safe distance downstream of the embankment. The gradation of the sand used in the chimney drain should be controlled so as to prevent movement of fines from the embankment or foundation soils into the drain system. The collector drain should consist of a perforated pipe surrounded by crushed rock or gravel, graded to prevent movement of sand into the drain pipe. Recommended gradations of the sand and gravel are shown on Plate 6. The toe drain and chimney drain should extend the entire length of the embankment. The outflow of the drain pipe downstream should be protected against vegetation growth and rodent entry.

Zone 4 - Zone 4 should consist of a blanket of riprap or crushed rock, approximately 18 inches to 2 feet thick and extending from the top of the dam to the minimum of 5 feet below the anticipated low water level. Riprap so placed should prevent erosion from wave, wind and rain action on the upstream face of the dam.

BORROW AREAS:

In general, the valley floor of the lake bed contains 3 to 12 feet of clay underlain by shaley clays and clayey shales as indicated in the above section of the report. The upper soils may be used in Zones 1 and 2. The clayey shales or shaley clays may be used in the outer portions of Zone 2. Use of all of the available borrow material from areas A, B, C and D should produce the required 380,000 cubic yards of fill for the embankment. However, it is anticipated that area D may need to be extended to the west and area A to the south as required during construction. It is recommended that excavation in the borrow areas not extend closer than 200 to 250 feet to the upstream toe of the embankment.

EMBANKMENT SETTLEMENT:

Due to the relatively low height of the dam, no consolidation tests were performed on borrow materials. However, it is anticipated that settlement will occur during and following completion of embankment compaction. It has been our experience that settlements occurring approximately 1 ft may be possible. For this reason it is recommended that the finish elevation of the dam be adjusted upward so that final elevation of the am will be no less than that currently planned.

COMPACTION CONTROL:

It is recommended that all embankment fill be compacted to a minimum of 100 percent of Maximum Density as determined by the standard compaction procedure (ASTM D 698). It is further recommended that the moisture content at compaction be at or slightly wet of Optimum Moisture content. Compaction of the soils in the recommended range will result in some added flexibility of the compacted soils allowing some differential movement due to consolidation of the underlying soils.

During construction, fill should be spread in 8 inch (maximum) loose layers and compacted. If during compaction, smooth surfaces are created by pneumatic rollers, or haul traffic, these surfaces should be roughened with a disc so that the next layer of fill will bond and thus prevent the creation of a seepage plane in the embankment. Water stops should be provided at frequent intervals surrounding any pipes or conduits through the embankment. Soil fill in areas of structures should be compacted by hand or by other suitable means to obtain the required density. Field control should be exercised over the fill placement to insure adequate compaction. This will require full-time inspection by a qualified soil technician under the supervision of the soils engineer.

STRIPPING FOUNDATION SOILS:

The areas to receive fill including the foundation area, the dam, and the areas to contain sand chimney drains for seepage control should be stripped of all organic top soil. Where stumps are not removed, the disturbed areas should be cleaned of major roots and debris and back filled with proper compaction.

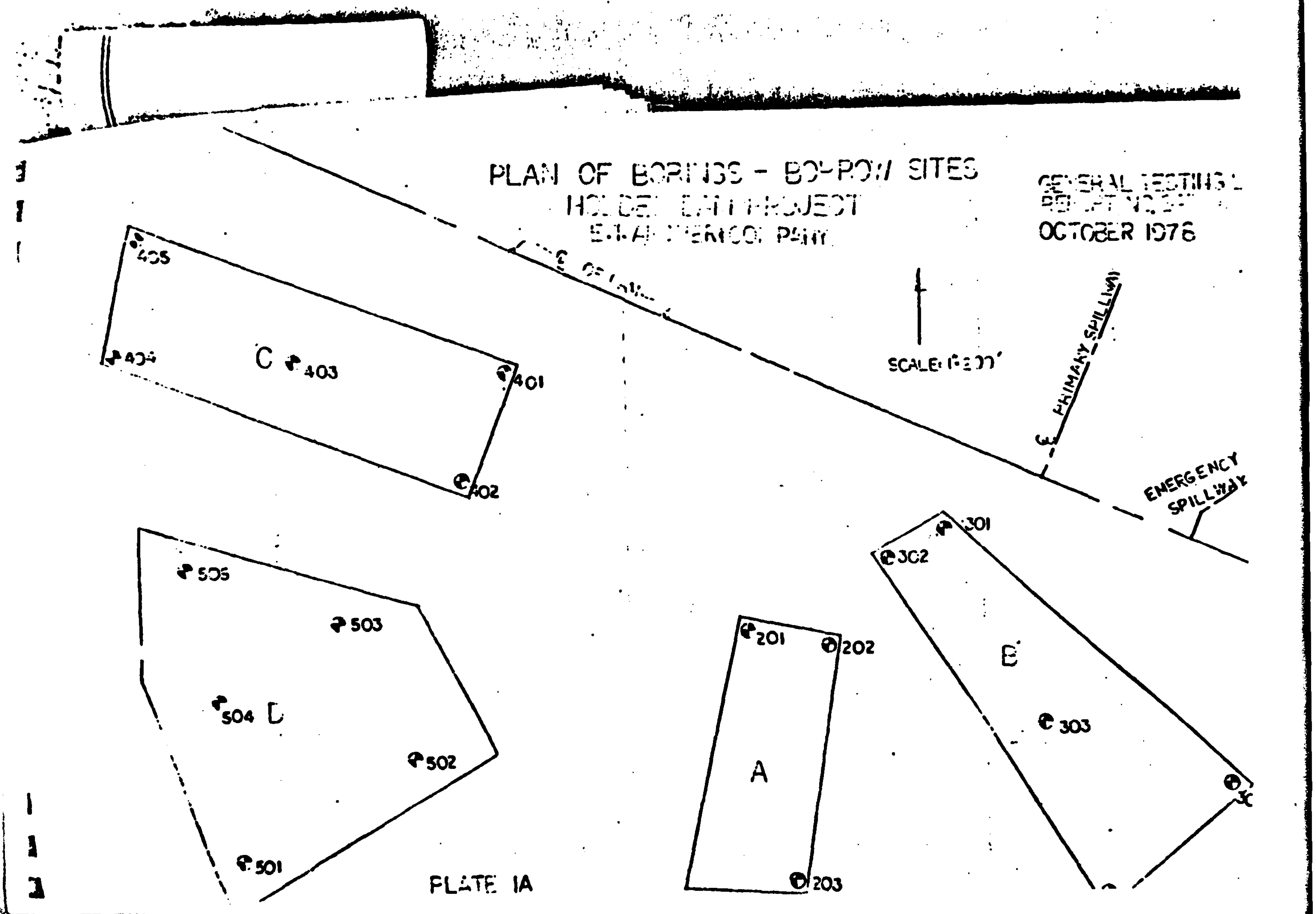
CONSTRUCTION WATER PROBLEMS:

As noted previously, some seepage may be encountered in borrow areas with depth. It is anticipated that the amount of seepage can be handled with proper terracing of borrow areas during excavation. If borrow areas are inadvertently inundated, they should be drained and the soils dried before being used in the embankment.

LIMITATIONS:

Recommendations in this report are based on the observations from the soils borings and are contingent on the assumption that soil conditions do not differ extensively from those which we encountered. If deviations from reported soil conditions are noted during construction, GENERAL TESTING LABORATORIES, INC. should be advised promptly for inspection. This may necessitate interrupting construction activity at the area in question.

ILLUSTRATIONS


PLAN OF BORINGS - BORROW SITES
OCTOBER 1976
SCALE
PRIMARY SPILLWAY
EMERGENCY SPILLWAY
C
405
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403
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402
D
505
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A
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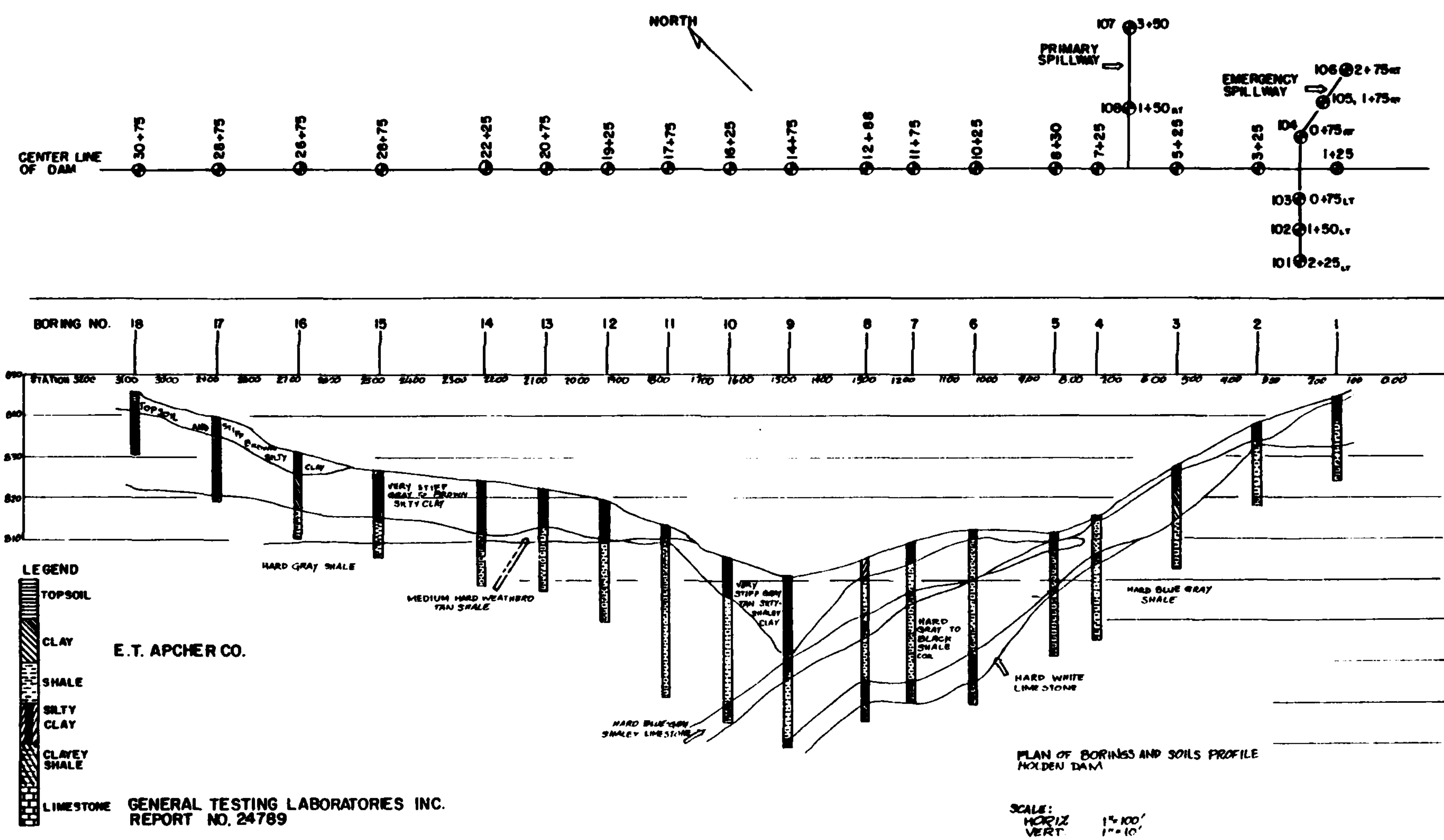
303

PLATE IA



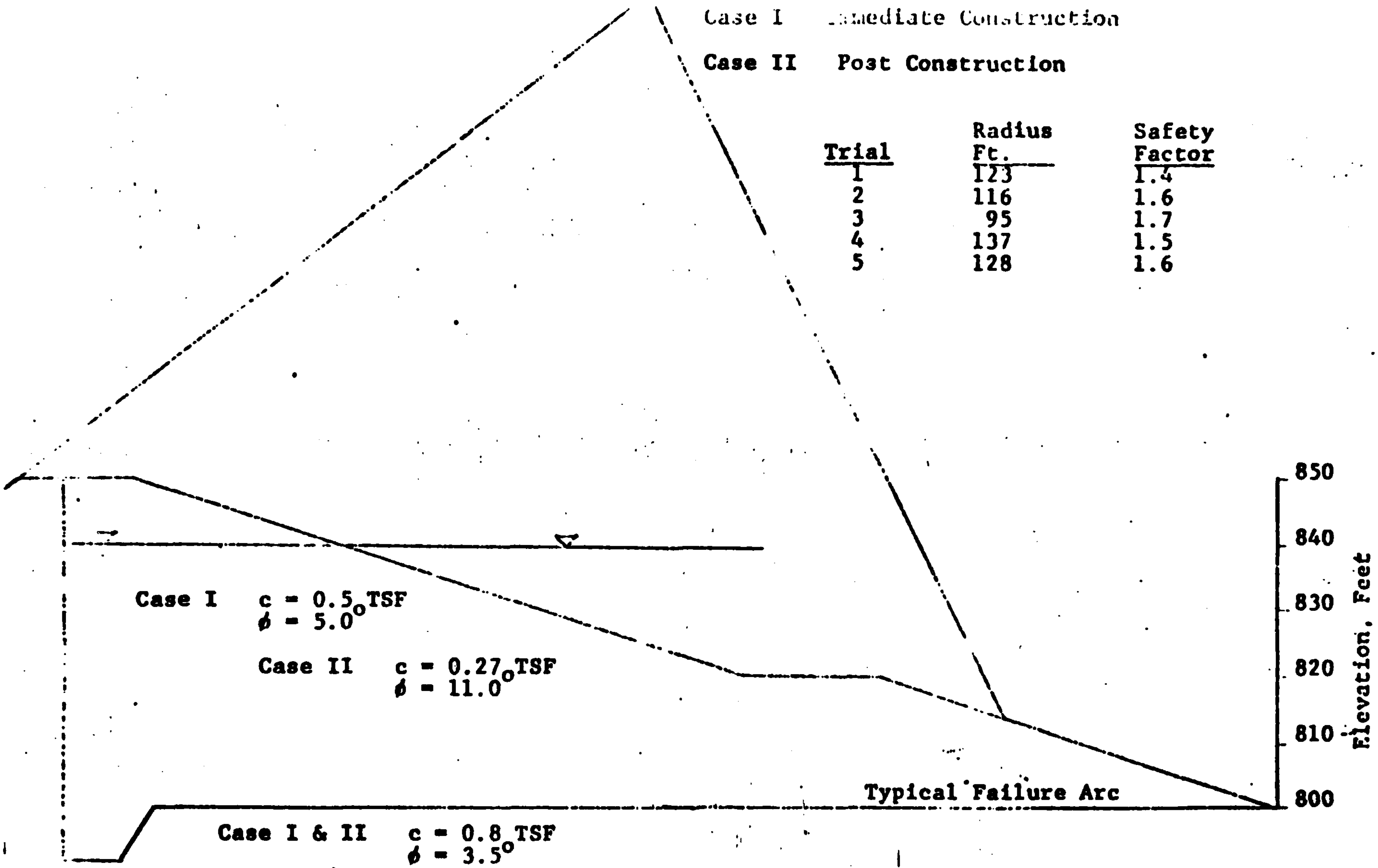
E.T. APCHER CO.

GENERAL TESTING LABORATORIES INC.
REPORT NO. 24789

PLAN OF BORINGS AND SOILS PROFILE
HOLDEN DAM

SCALE:
HORIZ 1" = 100'
VERT 1" = 10'

PLATE 2



Case I Immediate Construction

Case II Post Construction

Trial	Radius Ft.	Safety Factor
1	123	1.4
2	116	1.6
3	95	1.7
4	137	1.5
5	128	1.6

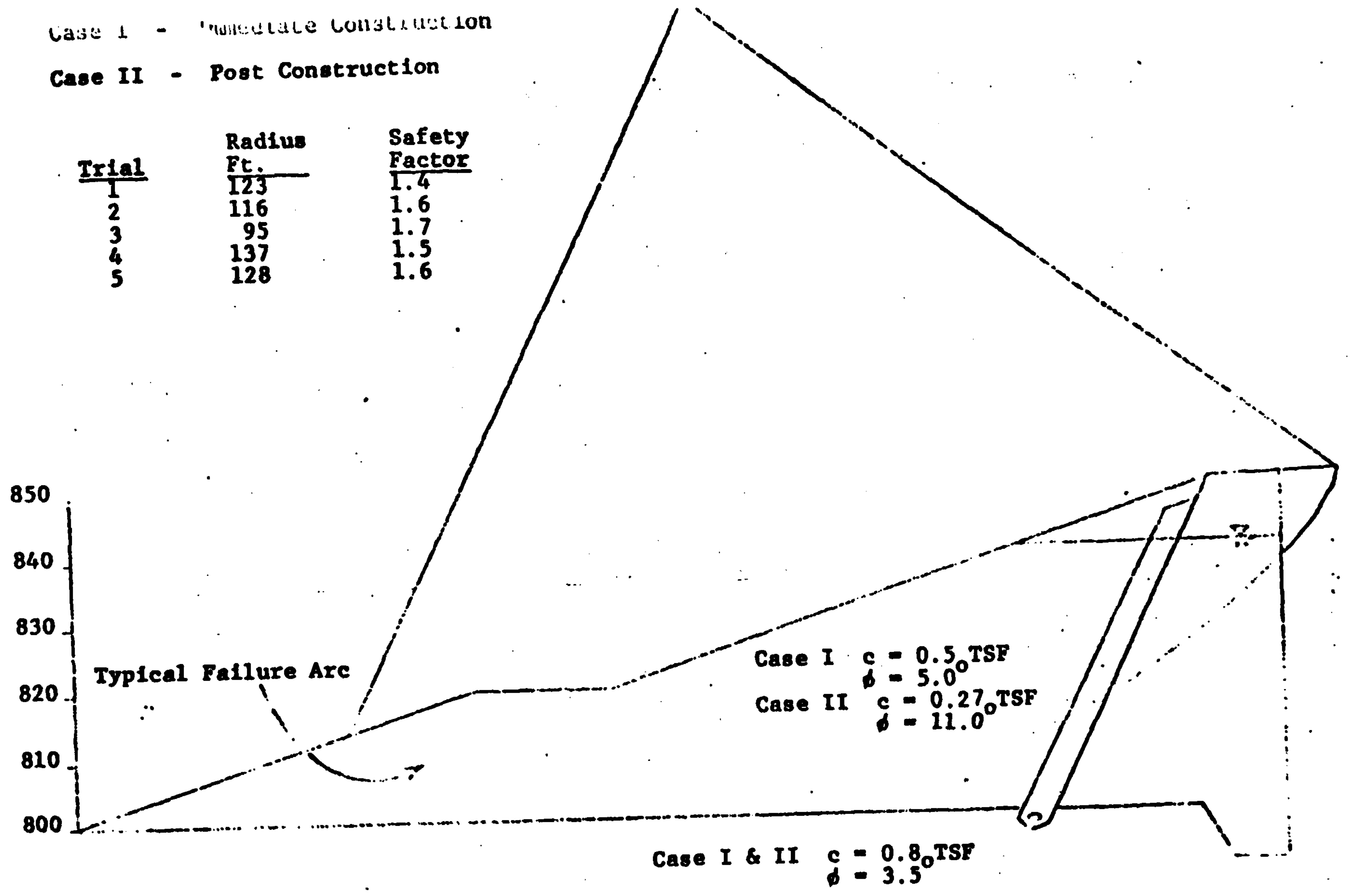
STABILITY ANALYSIS - UPSTREAM SLOPE

PLATE 3

Case I - Immediate Construction

Case II - Post Construction

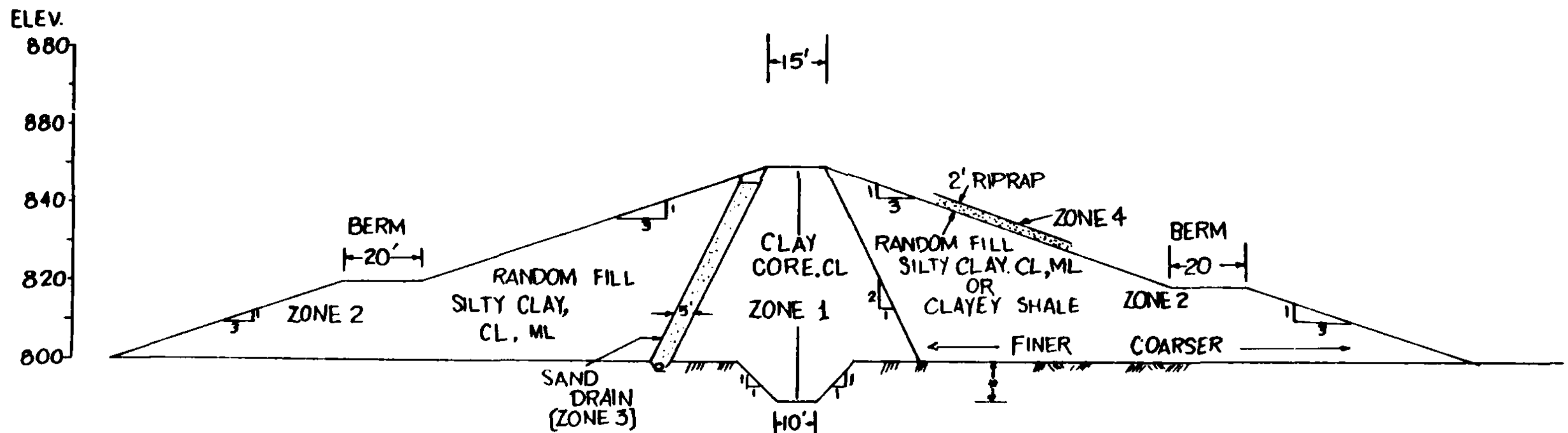
Trial	Radius Ft.	Safety Factor
1	123	1.4
2	116	1.6
3	95	1.7
4	137	1.5
5	128	1.6



STABILITY ANALYSIS - DOWNSTREAM SLOPE

PLATE 4

TYPICAL SECTION OF EMBANKMENT



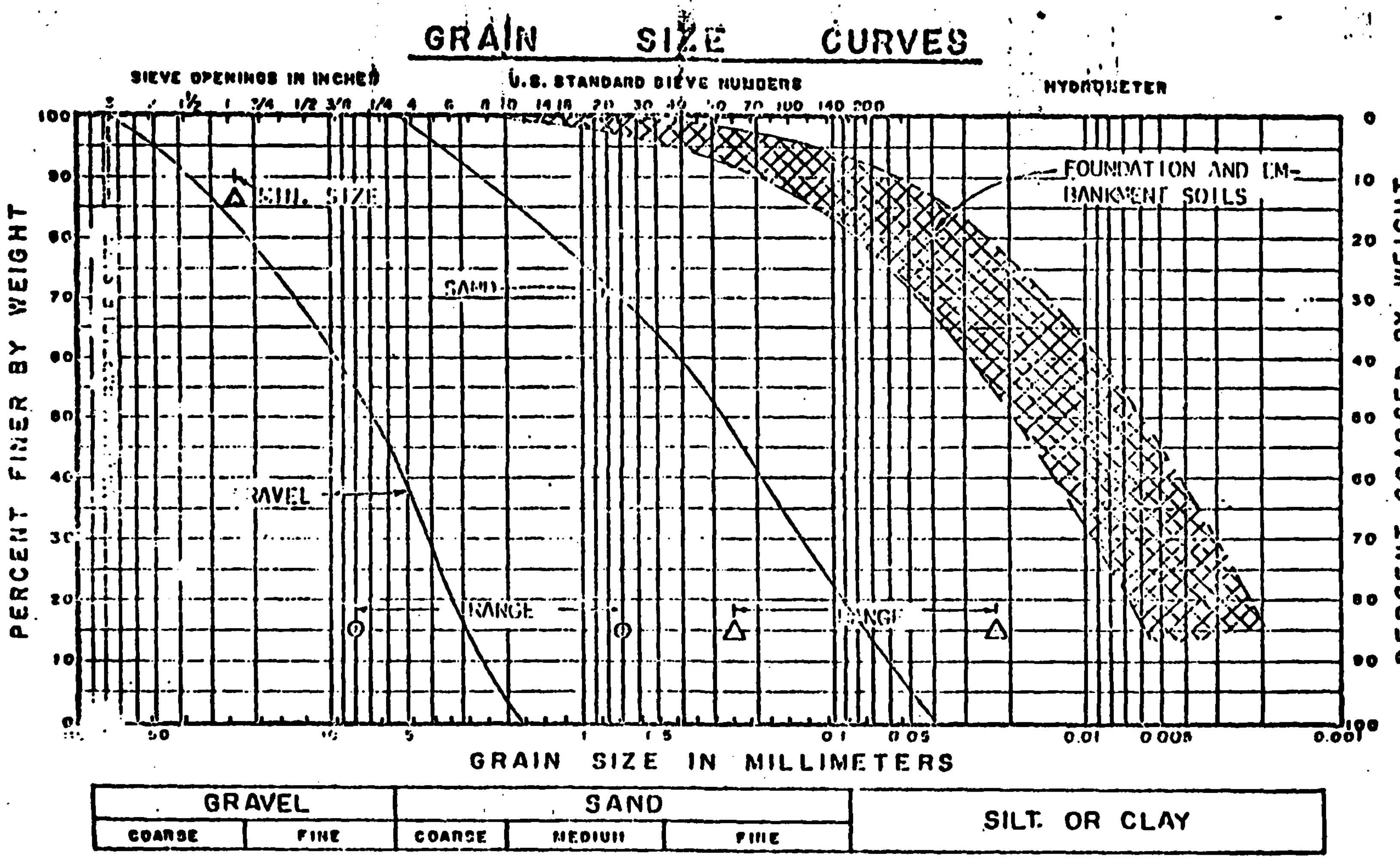
SCALE: 1" = 30'

E.T. ARCHER & CO.
HOLDEN DAM PROJECT
HOLDEN, MO.

DEPTH OF CORE TRENCH:
CORE TRENCH SHOULD EXTEND A MINIMUM OF 0.2 TIMES THE HEIGHT OF THE DAM OR 6 FEET INTO SHALE, WHICHEVER IS LESS.

GENERAL TESTING LABORATORIES, INC.
REPORT NO. 24789

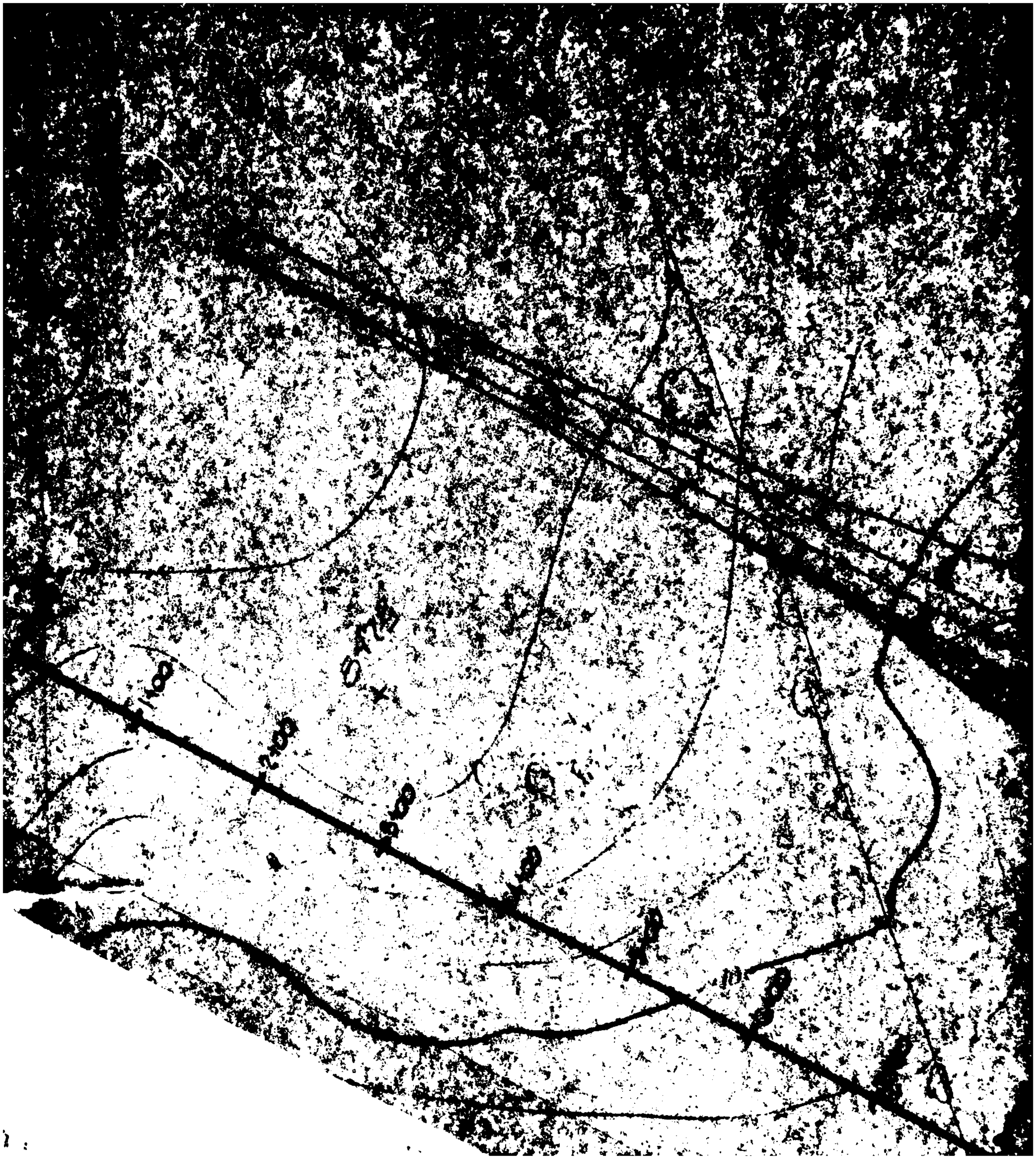
PLATE 5

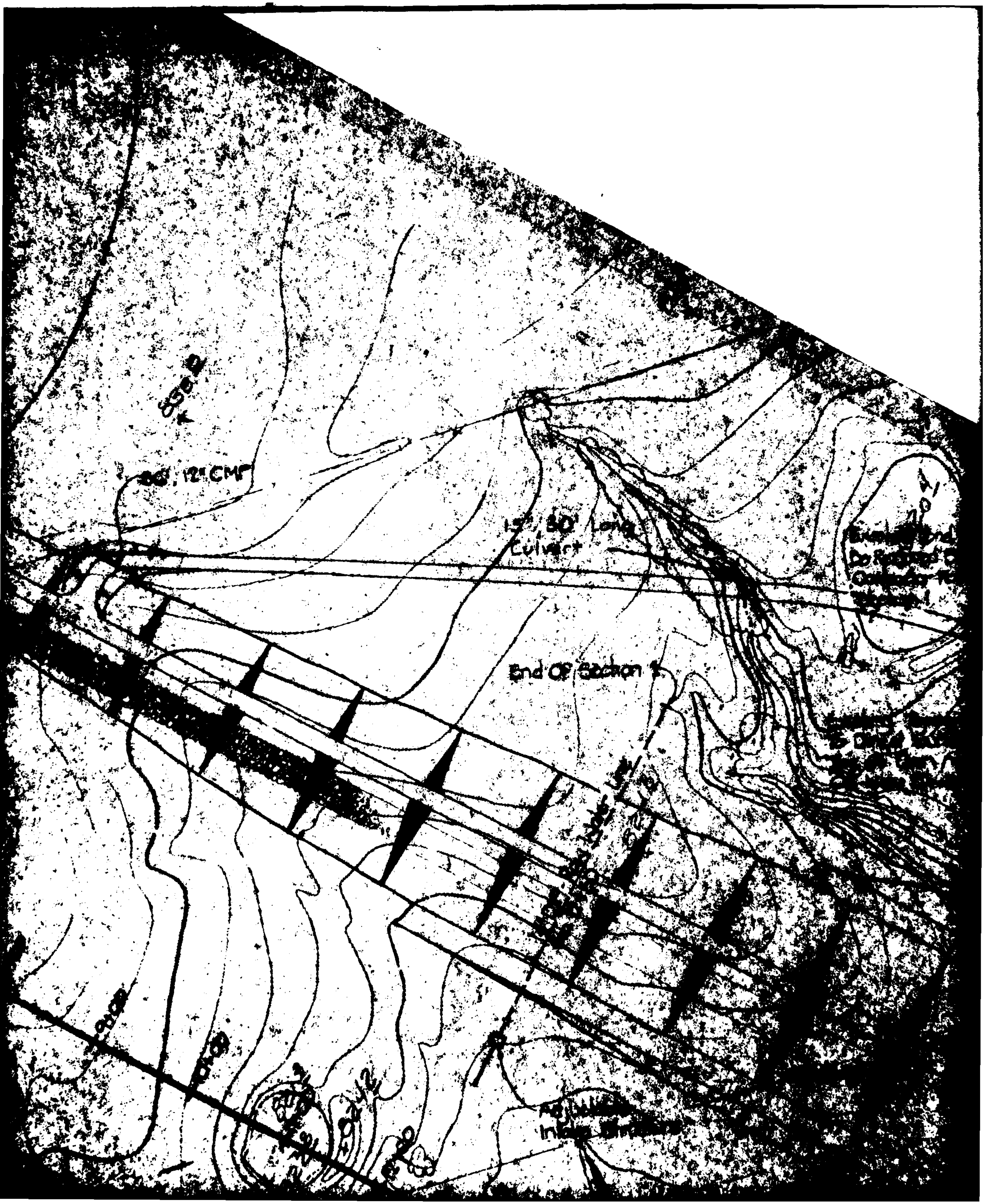


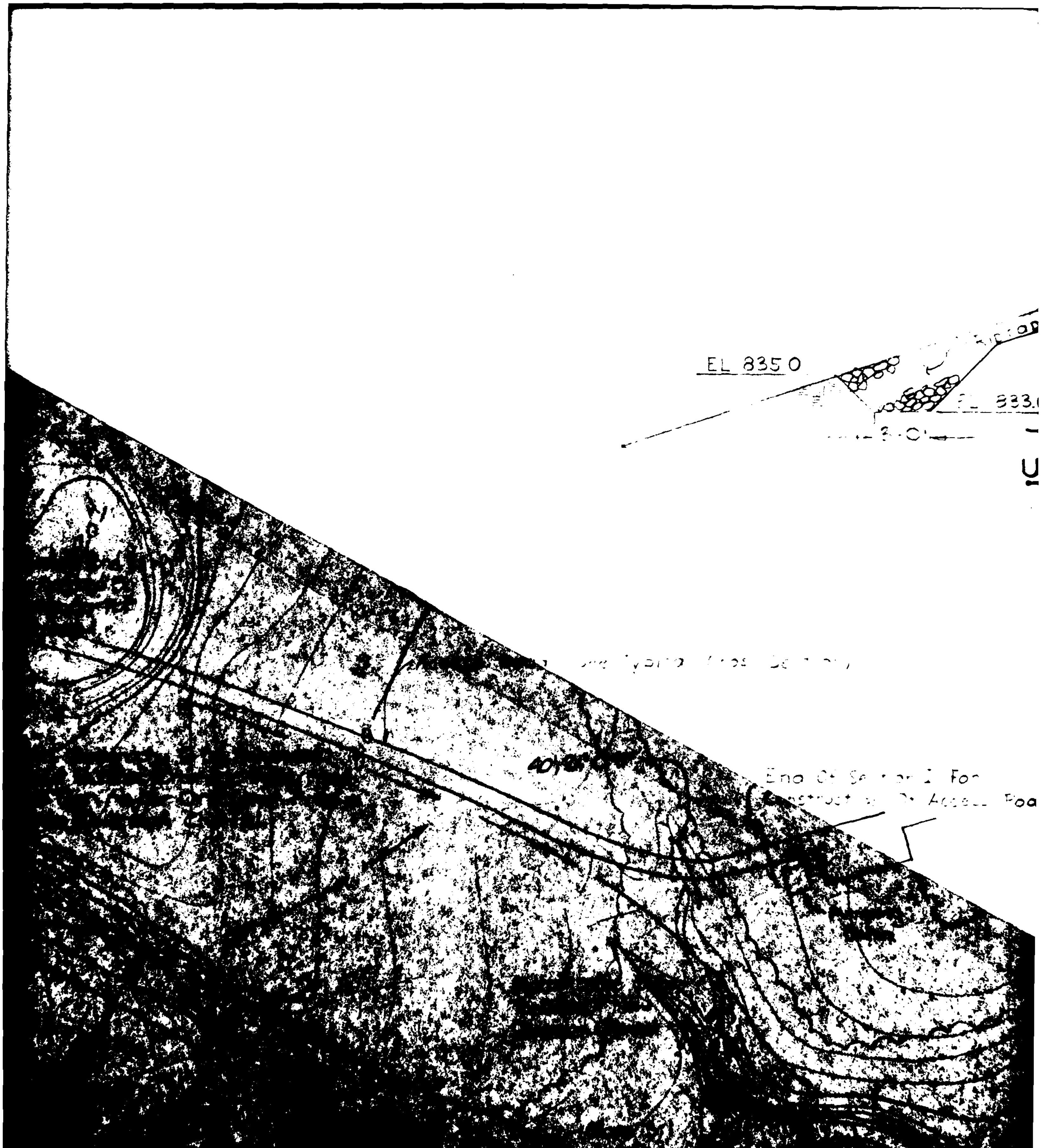
FOUNDATION DRAIN-FILTER GRADATIONS

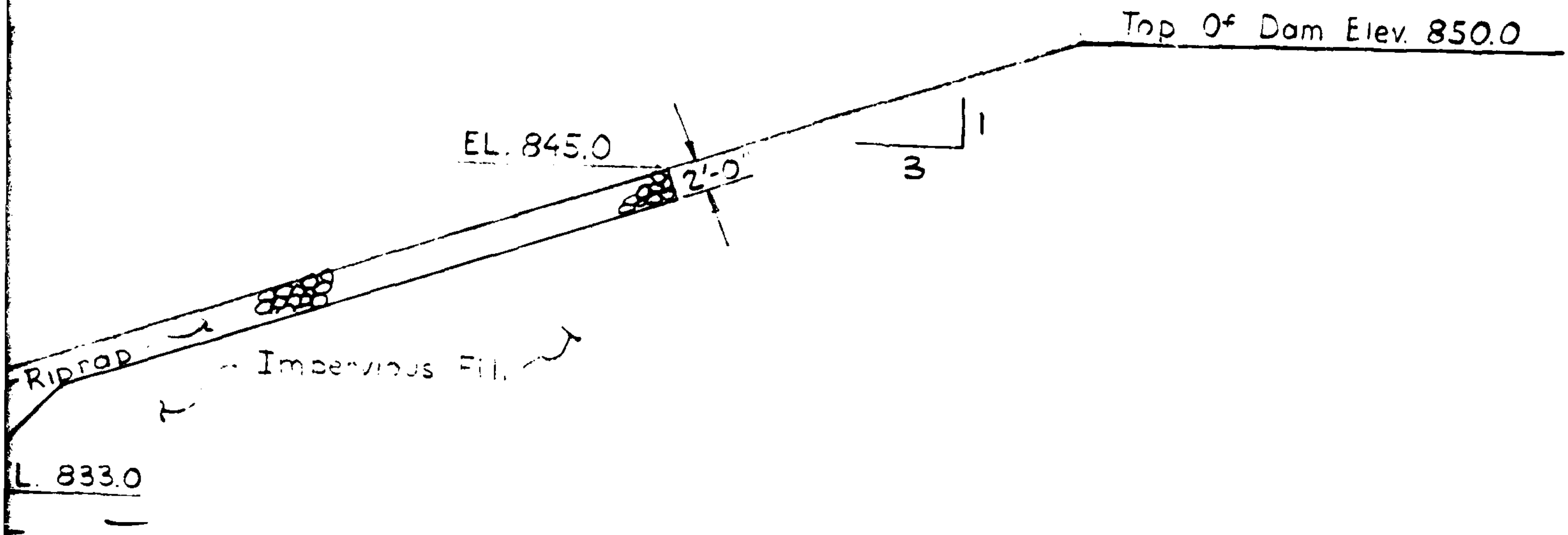
APPENDIX C

"AS-BUILT" DRAWINGS




12" CMP
Culvert
End Of Section 1

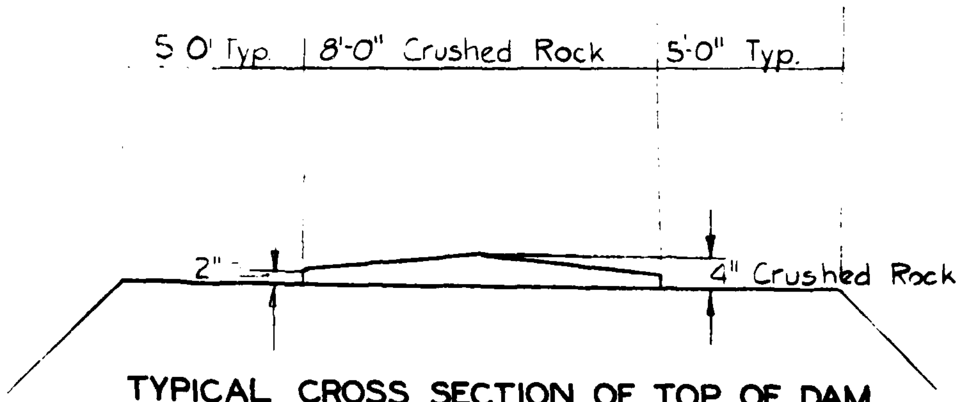

EL 835.0



UPSTREAM SLOPE RIPRAP DETAIL

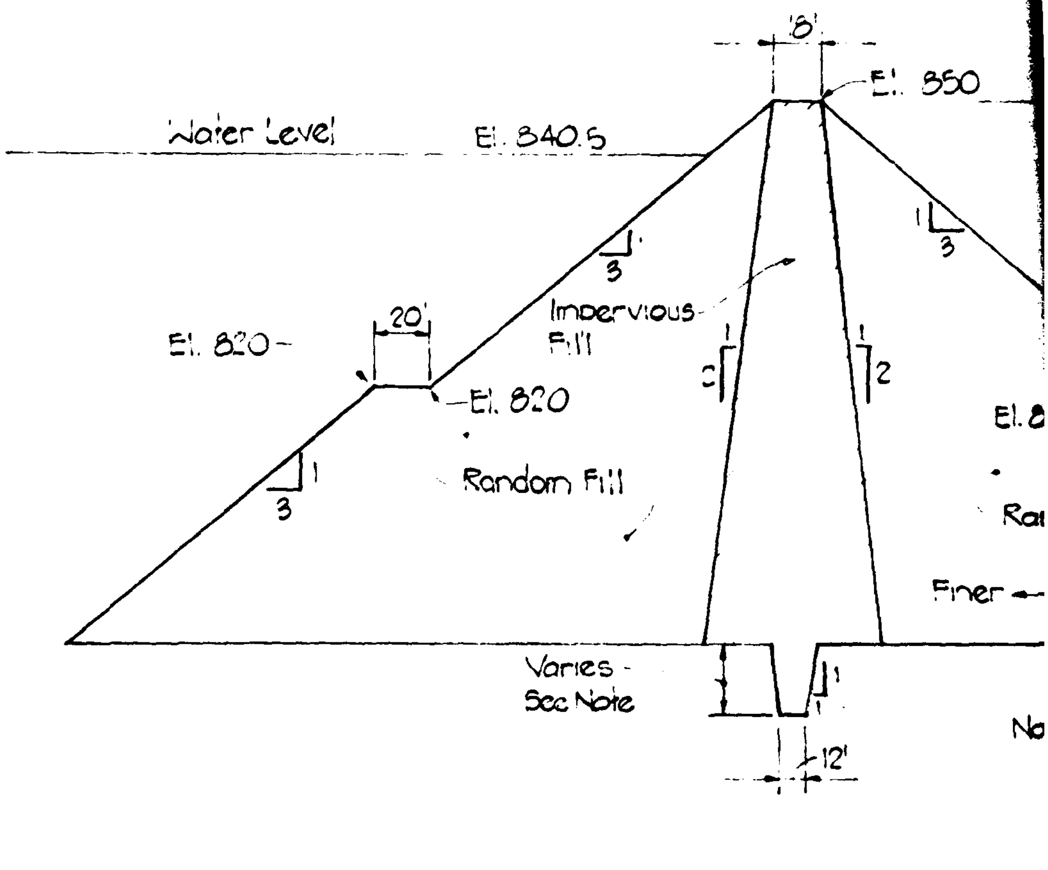
Not To Scale

s Road



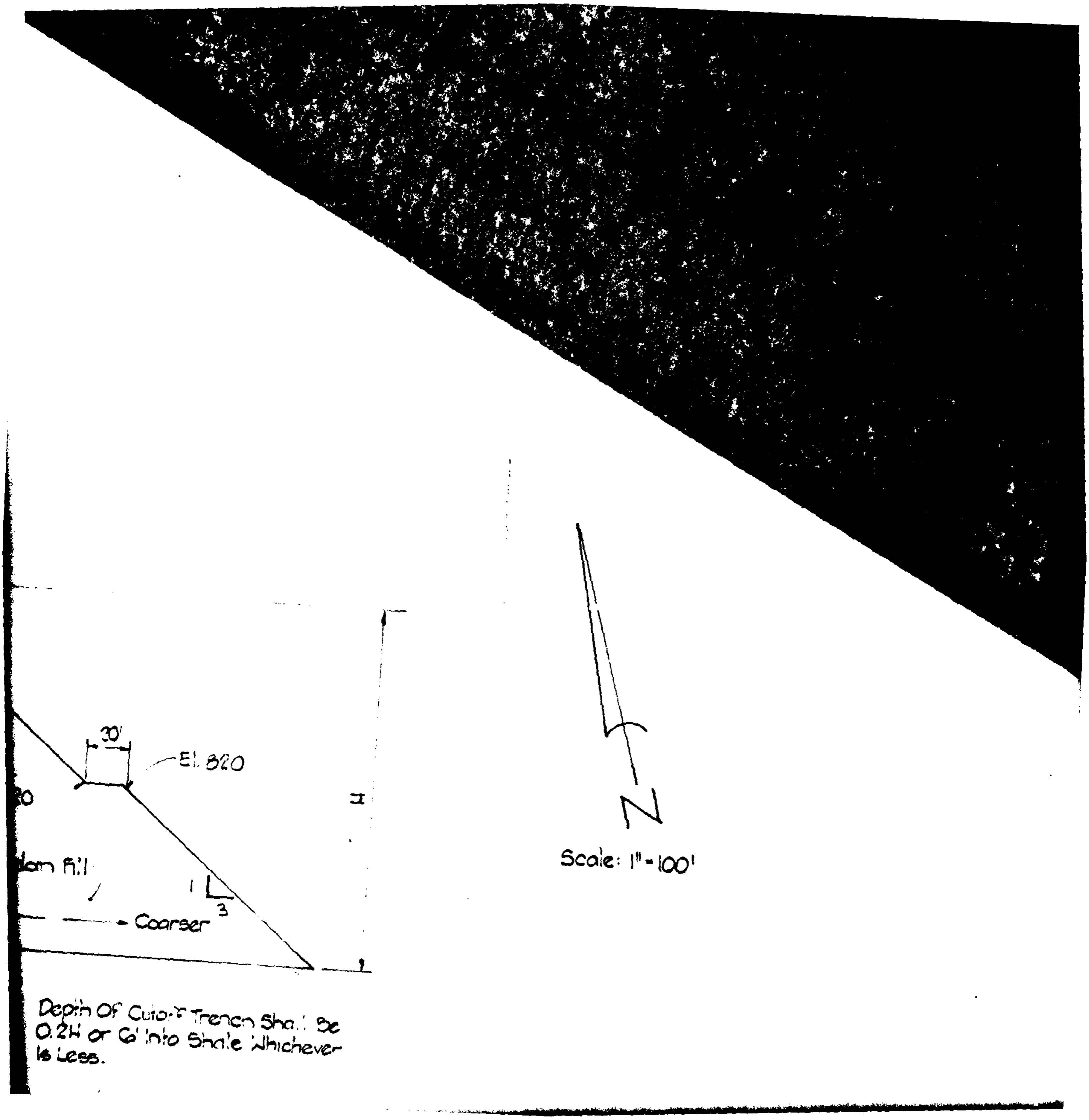
TYPICAL CROSS SECTION OF TOP OF DAM

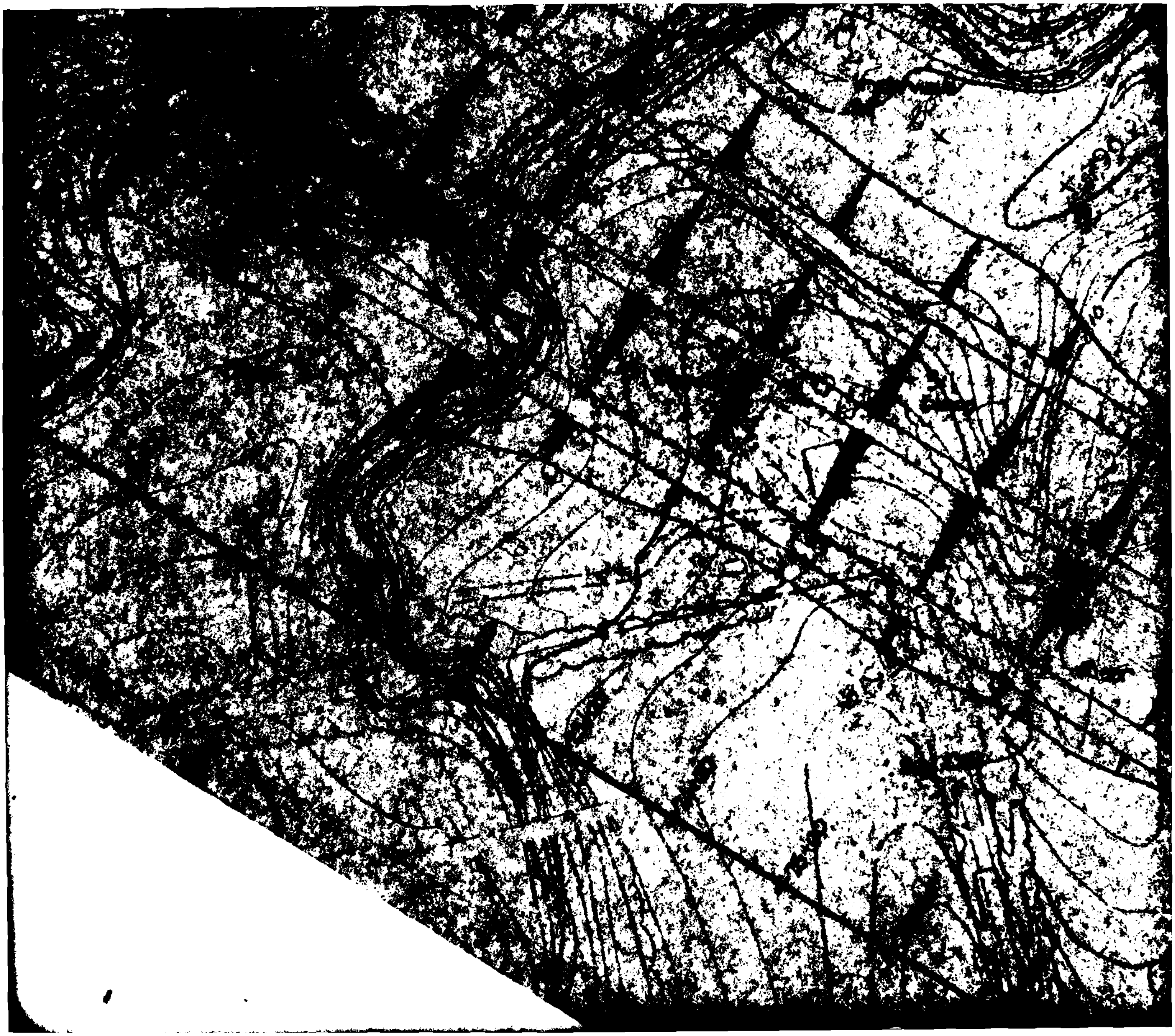
Not To Scale

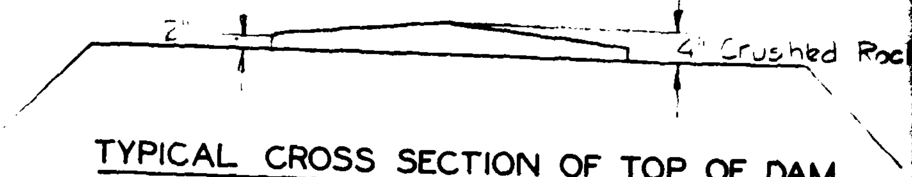


TYPICAL CROSS SECTION OF D

Not To Scale

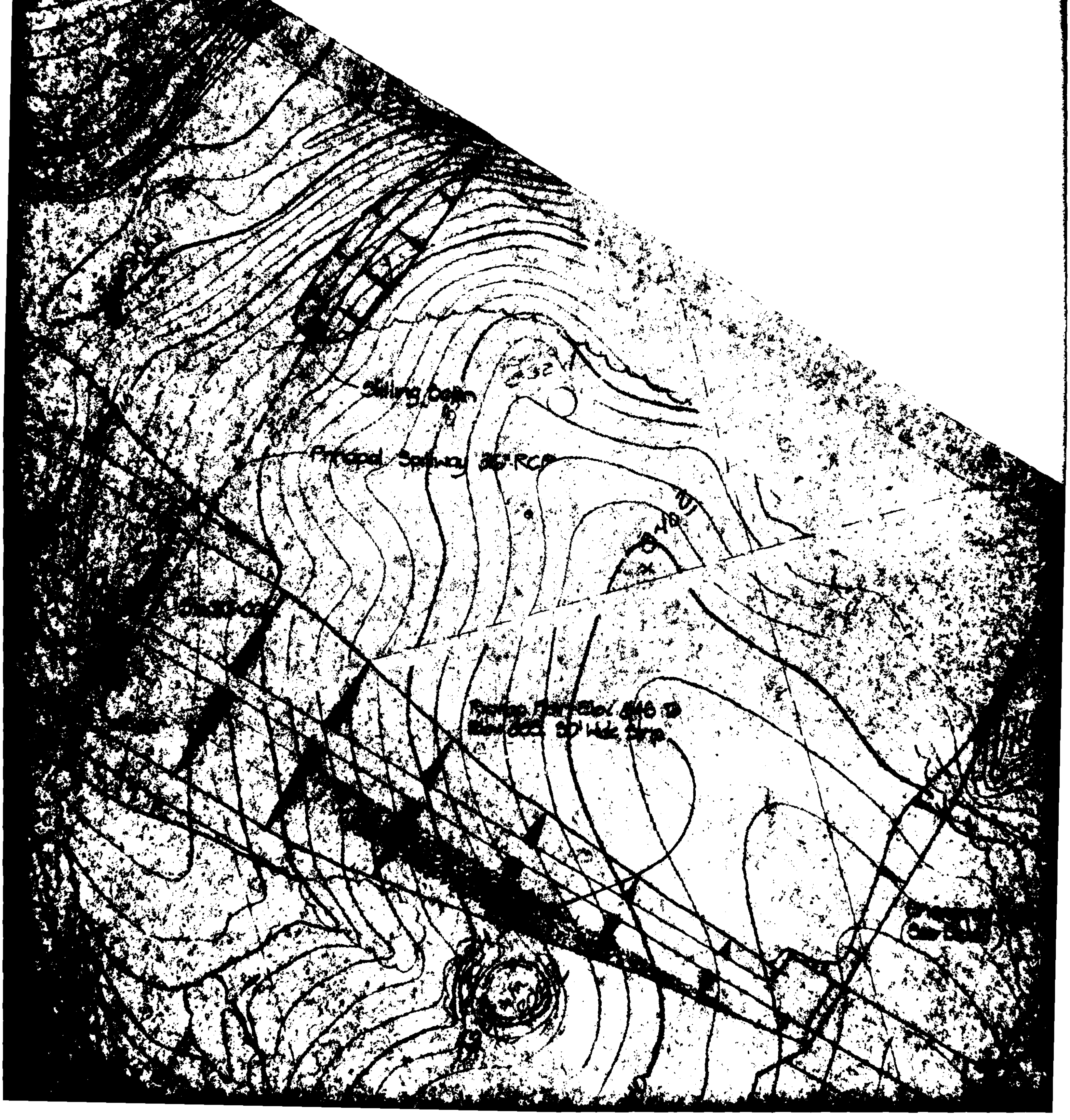

30'
El. 820
20
H
dom Fill
1
3
Coarser
Depth Of Cutoff Trench Shall Be 0.2H or 6' Into Shale Whichever Is Less.
N
Scale: 1" = 100'

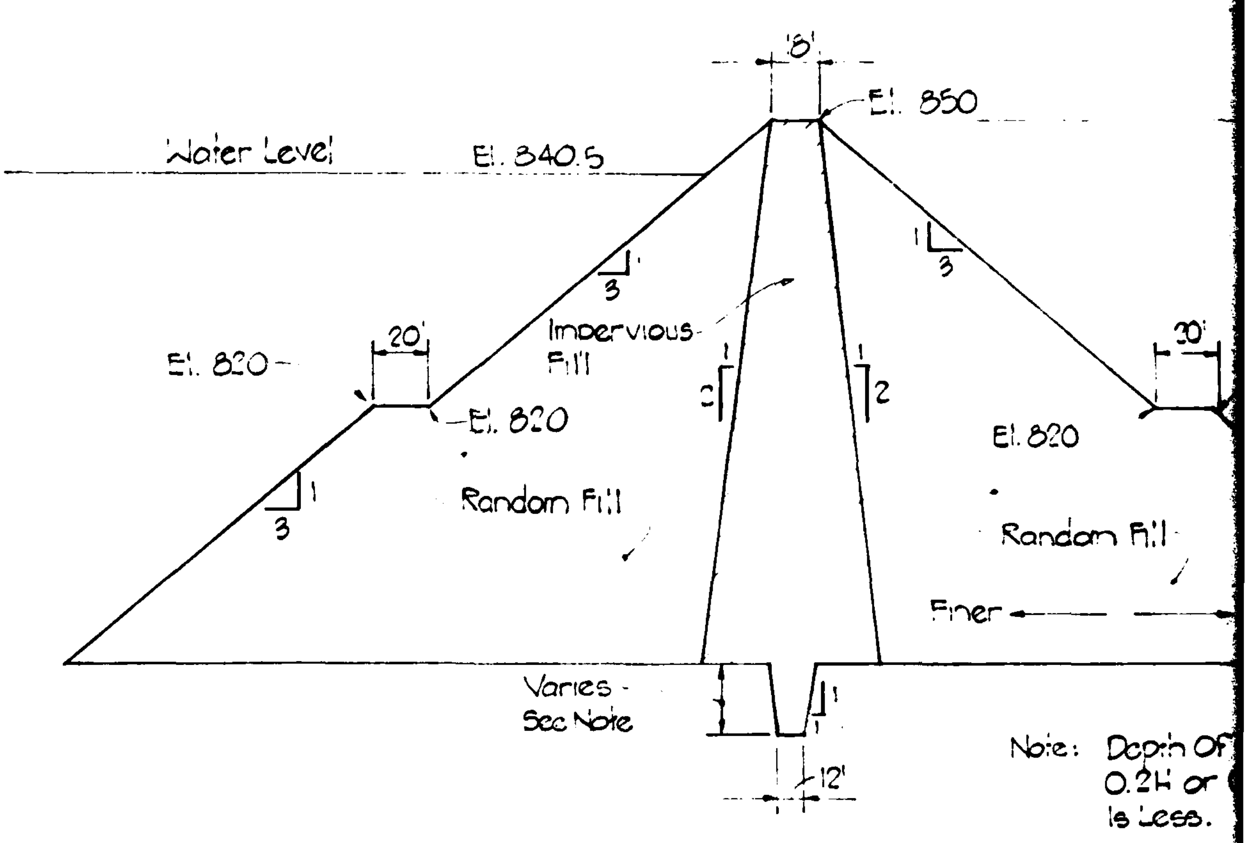




TYPICAL CROSS SECTION OF TOP OF DAM

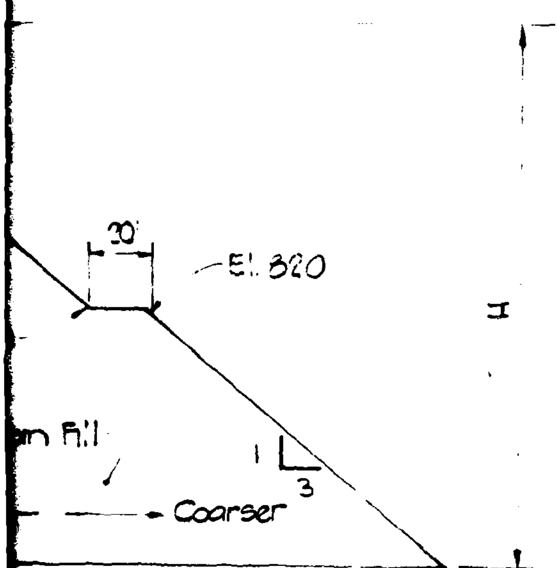
Not To Scale



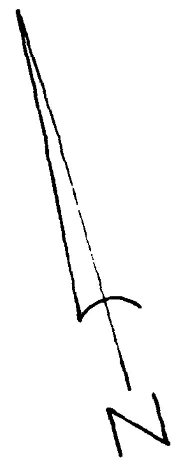


TYPICAL CROSS SECTION OF DAM

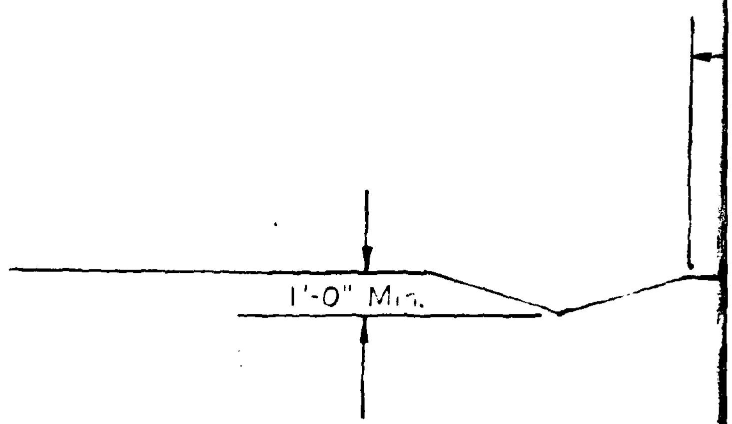
Not To Scale



Depth Of Cut-off Trench Shall Be 0.2H or 6' Into Shale Whichever Is Less.



Scale: 1" = 100'



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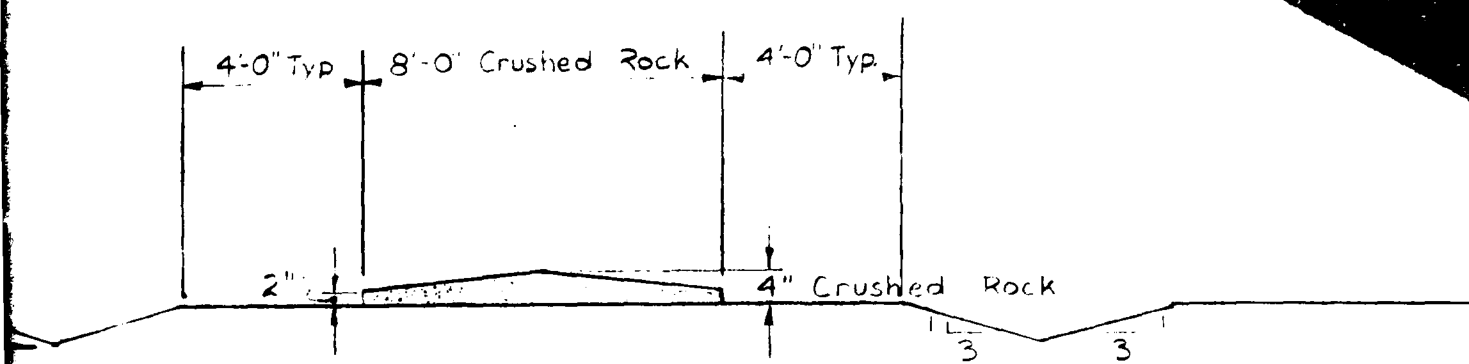
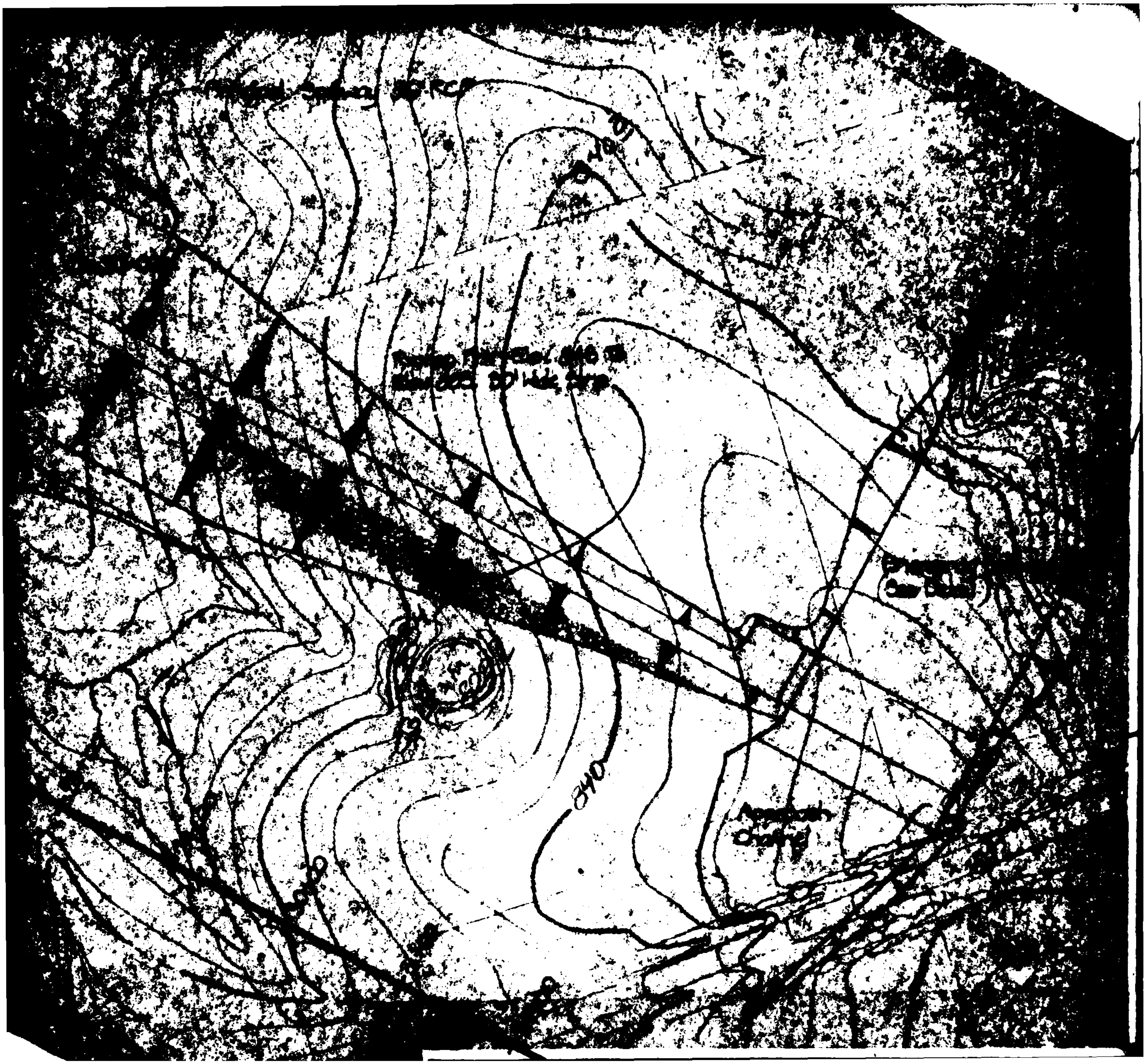


TYPICAL CROSS SECTION OF ACCESS ROAD

Not To Scale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PLAN

NOTE BOOK

No.

SURVEYED

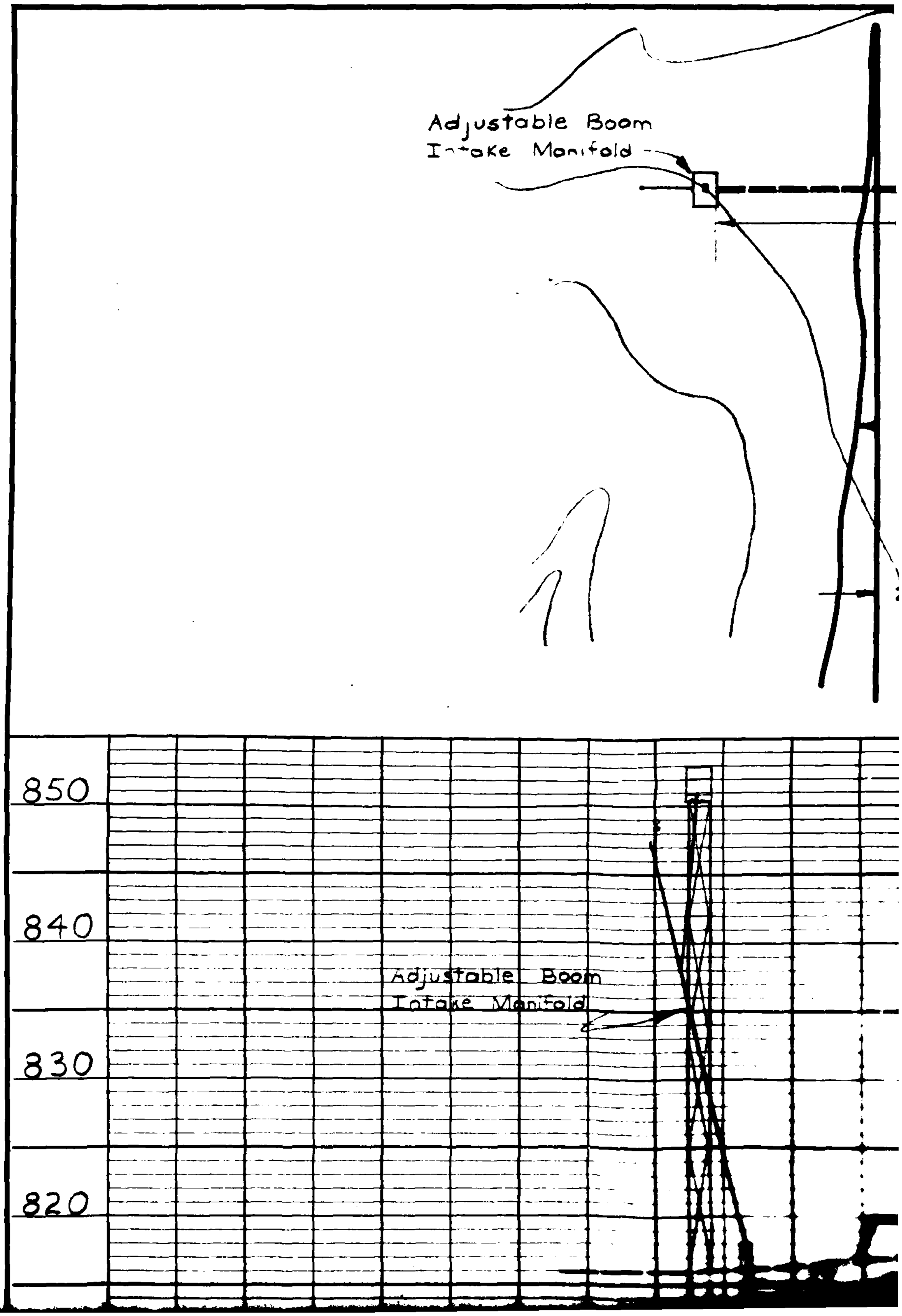
PLOTTED

ALIGNMENT CHECKED

RT. OF WAY CHECKED

BY

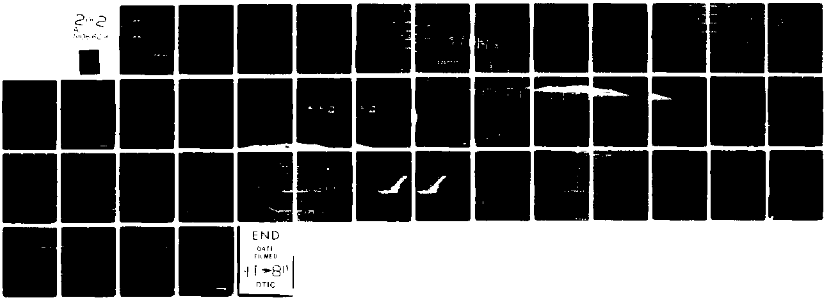
DATE



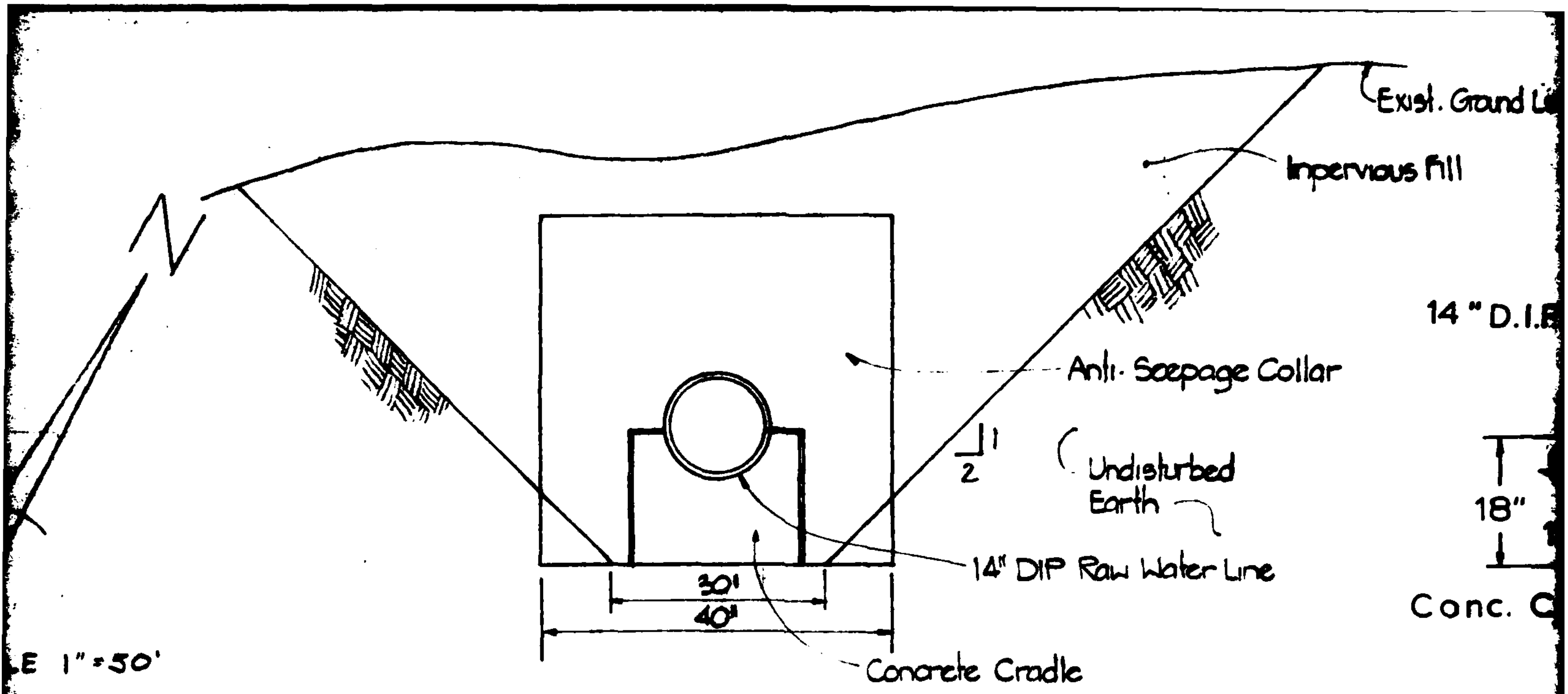
AD-A106 629 BLACK AND VEATCH KANSAS CITY MO F/G 13/13
NATIONAL DAM SAFETY PROGRAM. HOLDEN WATER SUPPLY DAM (MO 20532)--ETC(U)
APR 81 E R BURTON, H L CALLAHAN DACW43-81-C-0037

UNCLASSIFIED NL

2 of 2

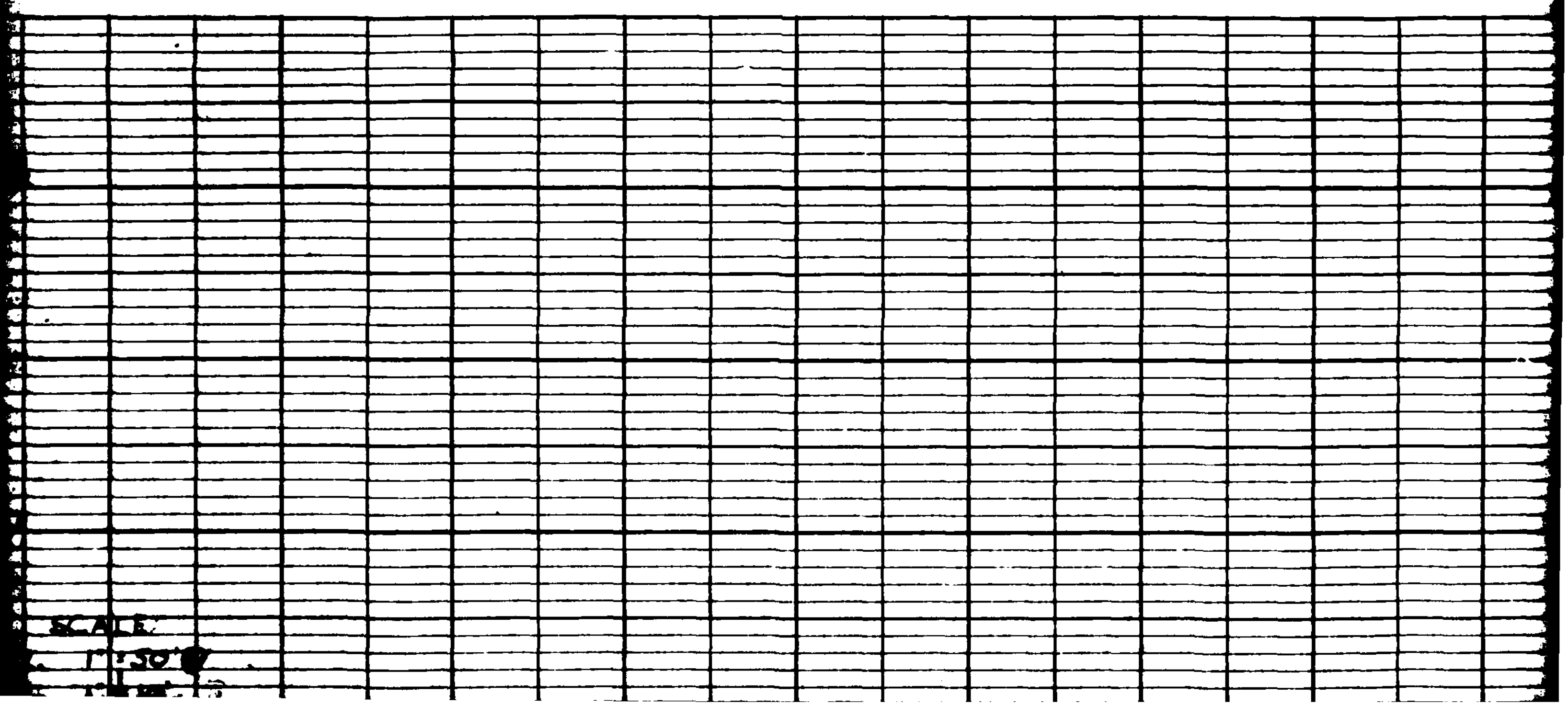


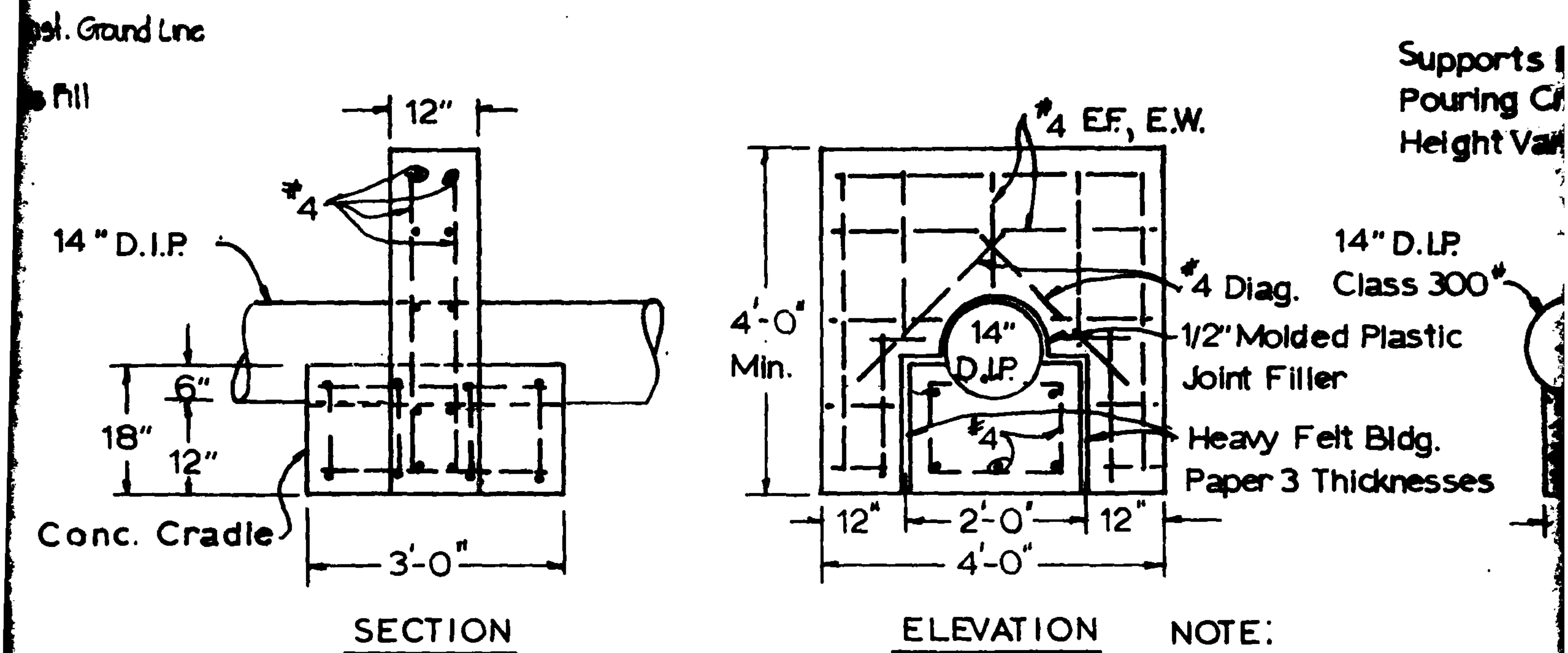

S-a 12+50
End Of Section I.
14" DIP
Row Water Line
180'
175'
824.8
Slope 3:1
€ Dam
SCALE 1" =
20'
820
18'
Slope 3:1
RAW WATER LINE
PLAN AND PROFILE THROUGH DAM
Original Groundline
End Of Section I
Anti-Seepage Collar & Pipe Cradle (See Detail)
HORIZ.
VERT.



TYPICAL CROSS SECTION OF TRENCH
FOR INTAKE STRUCTURE PIPING

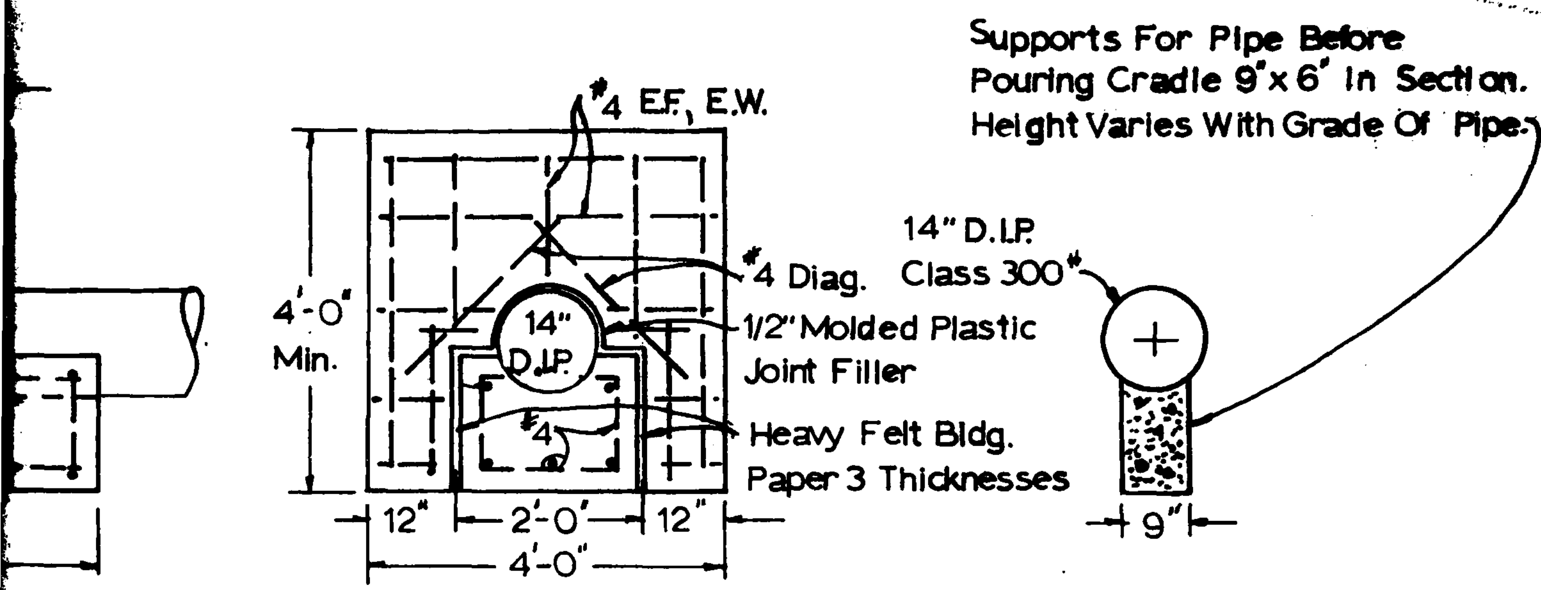
1" = 2' 0"





NOTE:
Seepage Collars To Exte...
Cradle A Min. Of 12" In A...
Bedrock.

ANTI-SEEPAGE COLLAR & PIPE CRADLE
FOR INTAKE STRUCTURE PIPING 1/2" = 1'-0"



ON

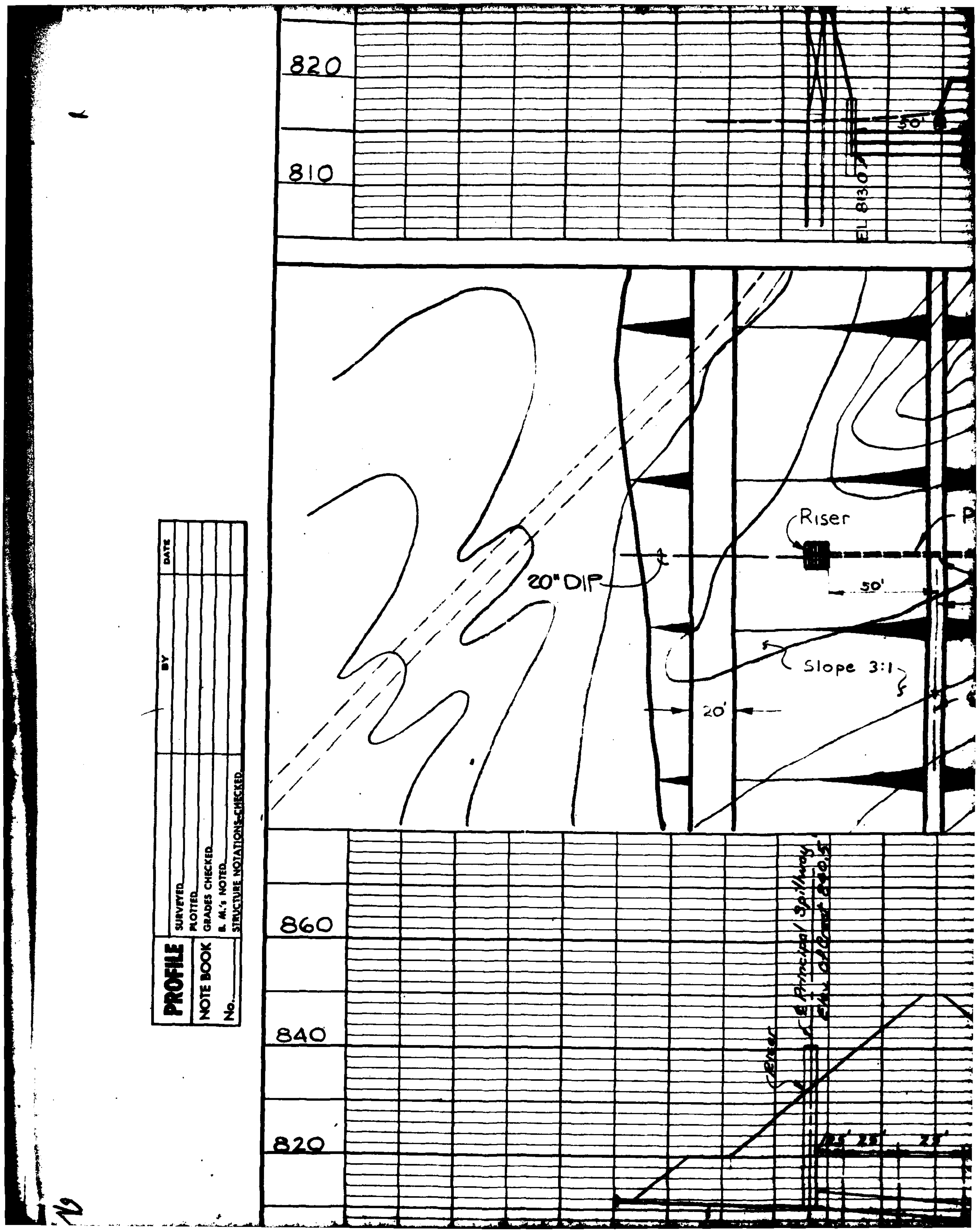
ELEVATION

NOTE:

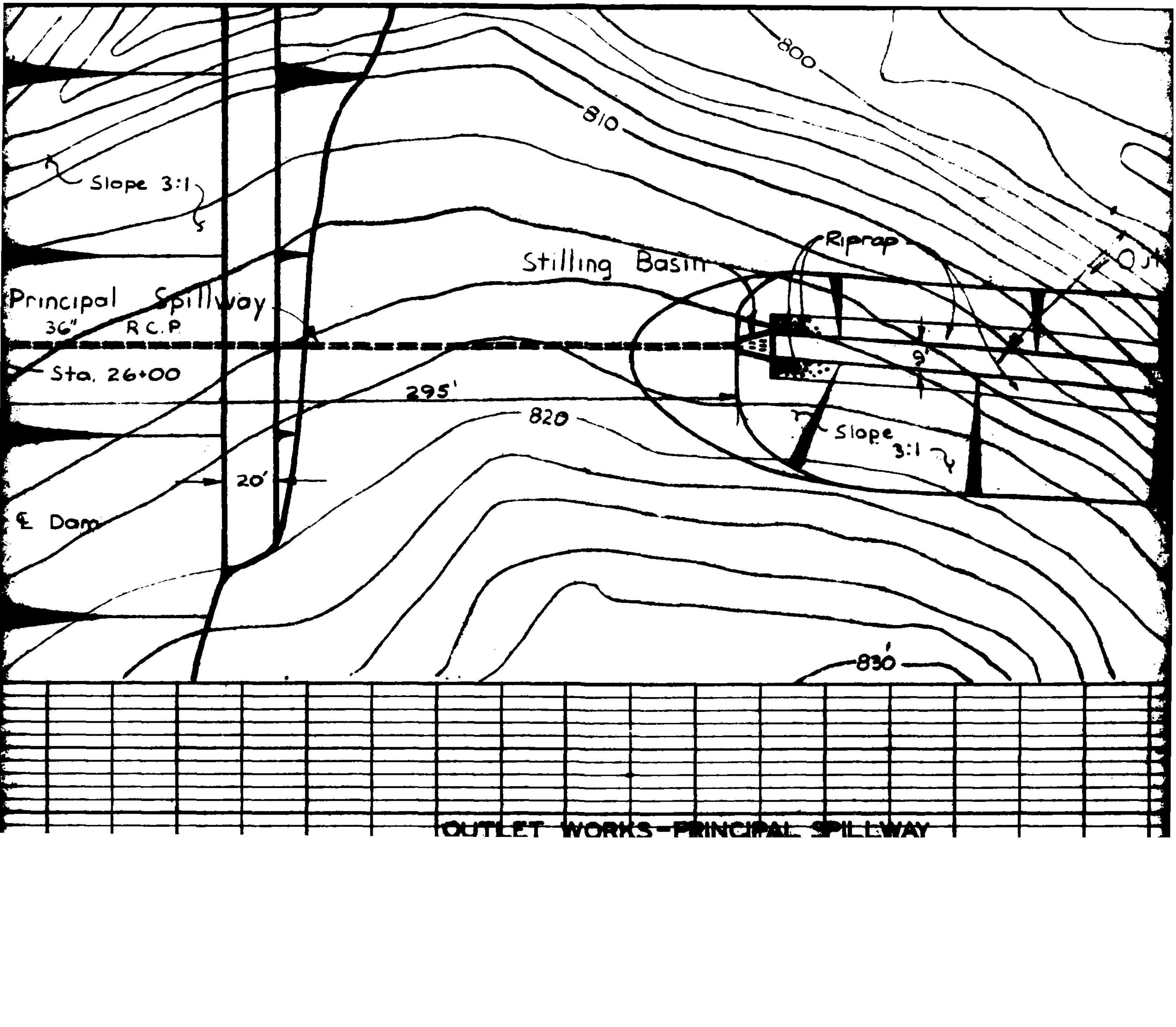
Seepage Collars To Extend Outside Of Pipe & Cradle A Min. Of 12" In All Directions Including Bedrock.

COLLAR & PIPE CRADLE

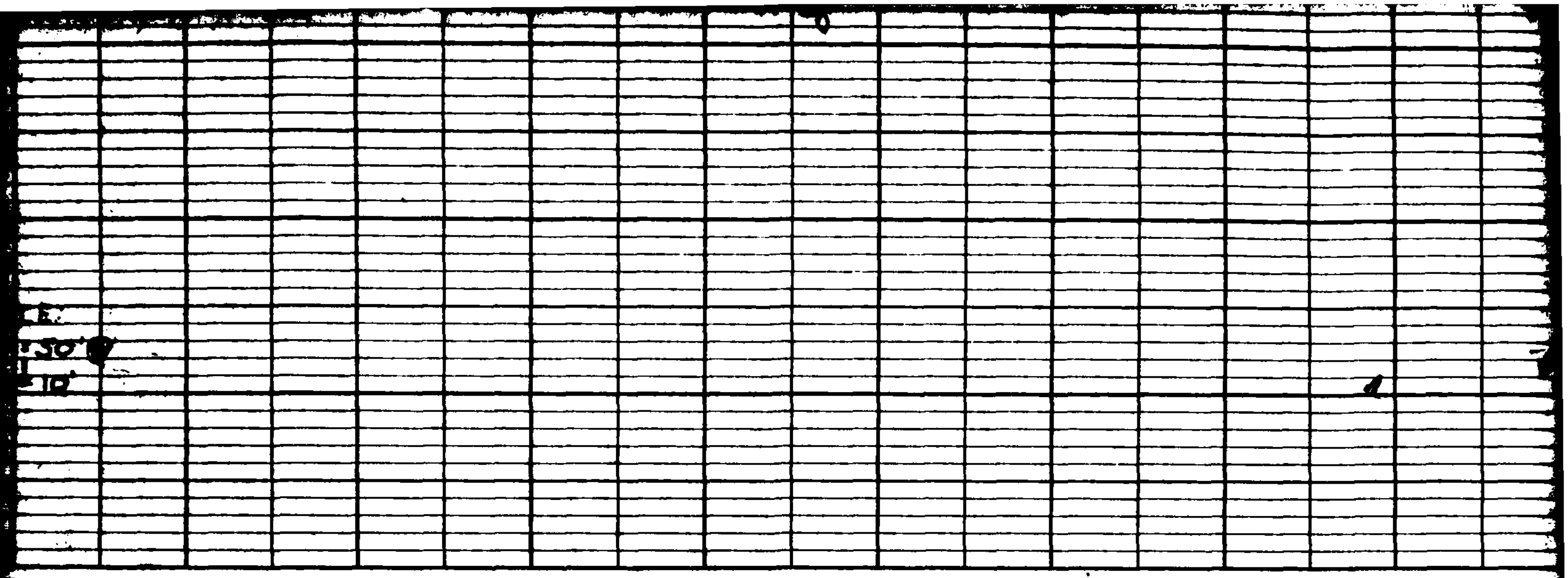
UCTURE PIPING 1/2" = 1'-0"

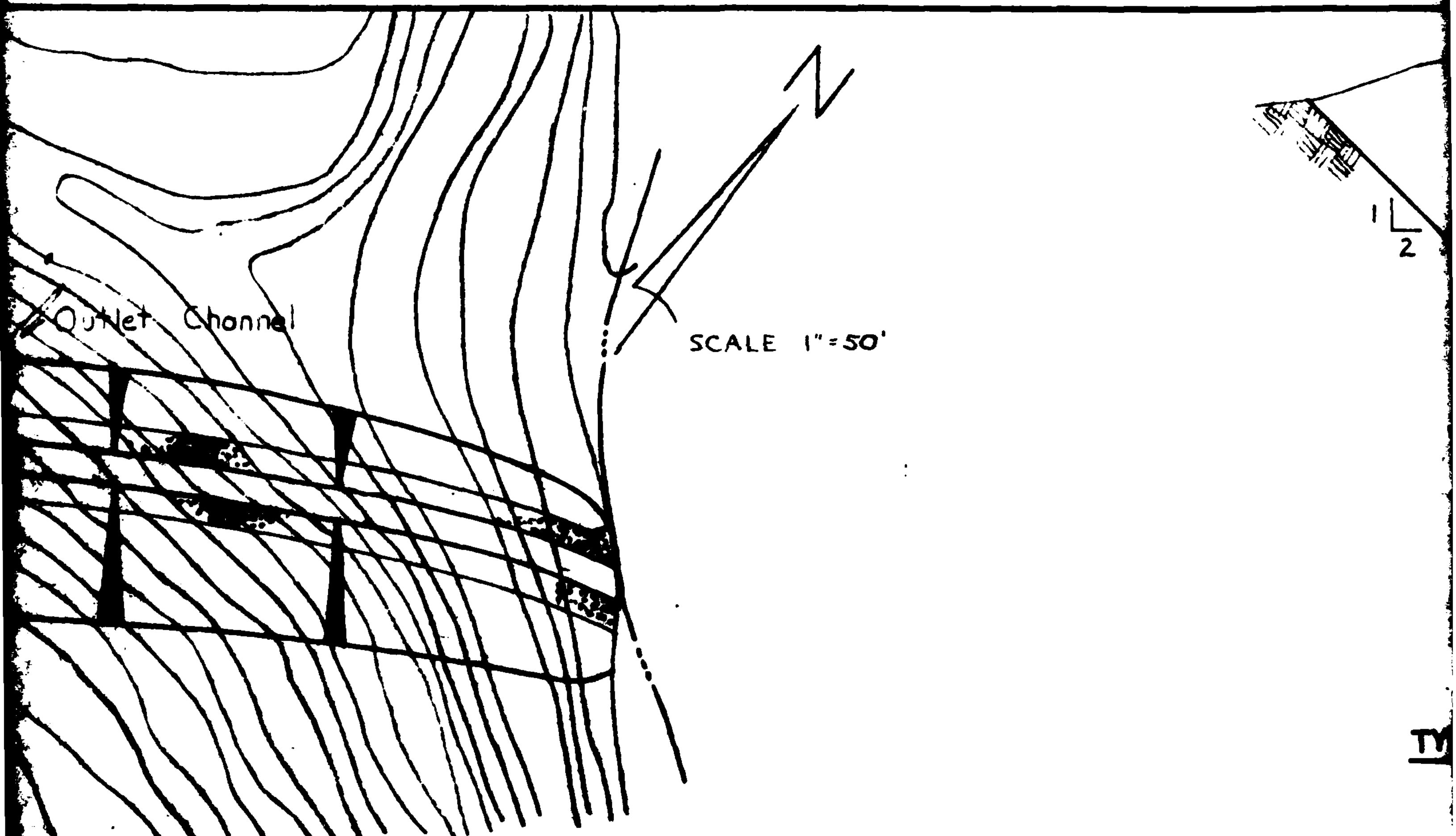


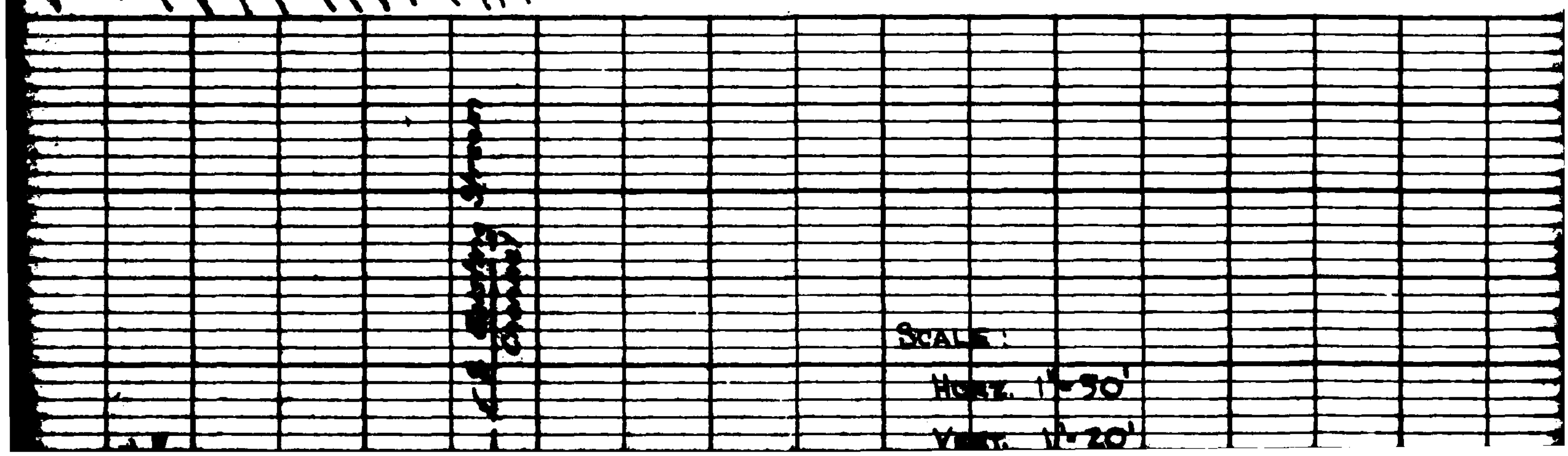
820
810
50'
EL 813.0
Riser
20" DIP
50'
Slope 3:1
20'
860
840
820
Riser
Principal Spillway
25' 25' 25'
PROFILE
NOTE BOOK
No.
SURVEYED
PLOTTED
GRADES CHECKED
B. M.'s NOTED
STRUCTURE NOTATIONS CHECKED
BY
DATE

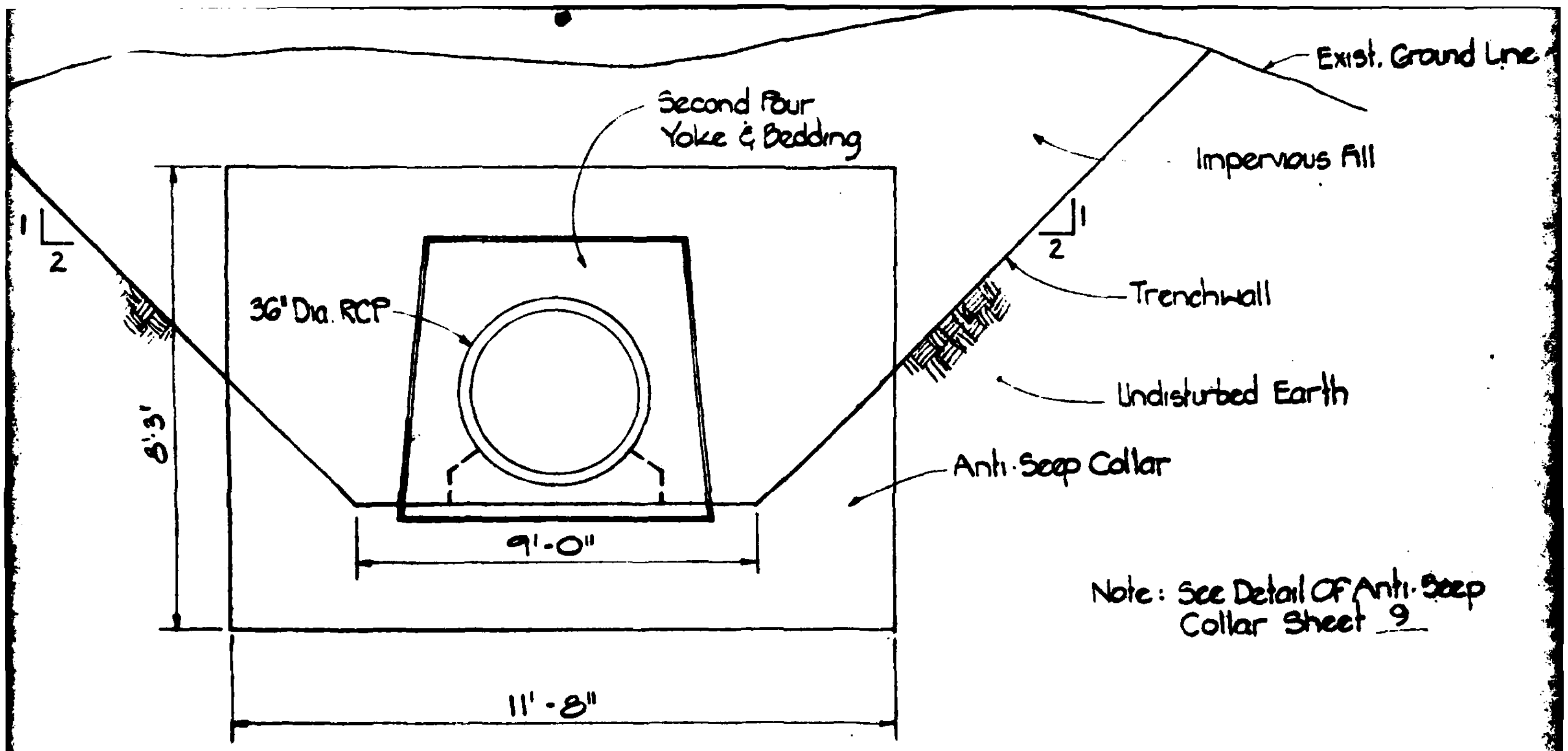


OUTLET WORKS-PRINCIPAL SPILLWAY


1"=50'
1"=10'

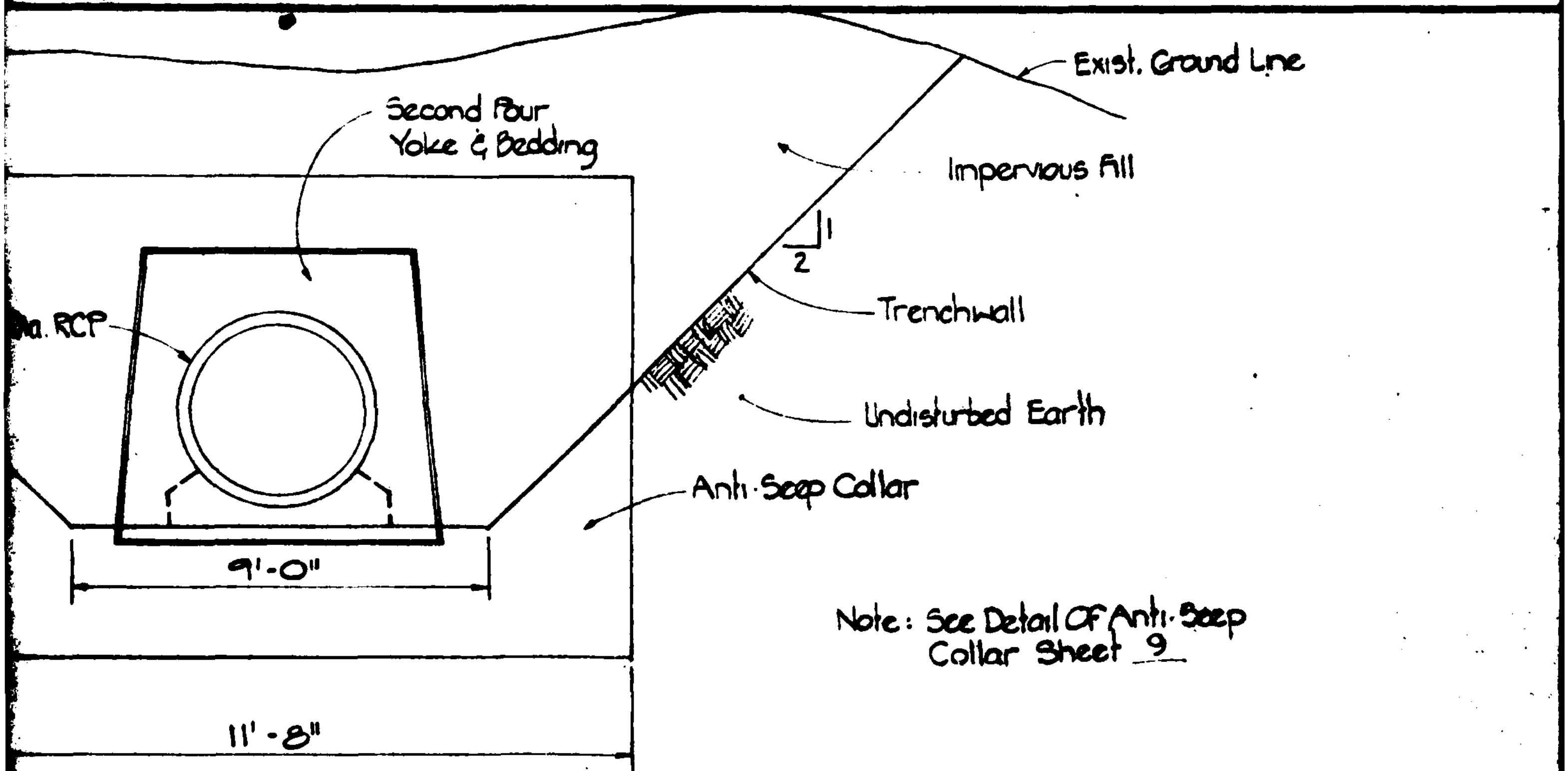

Outlet Channel
SCALE 1"=50'
1
2
TY


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



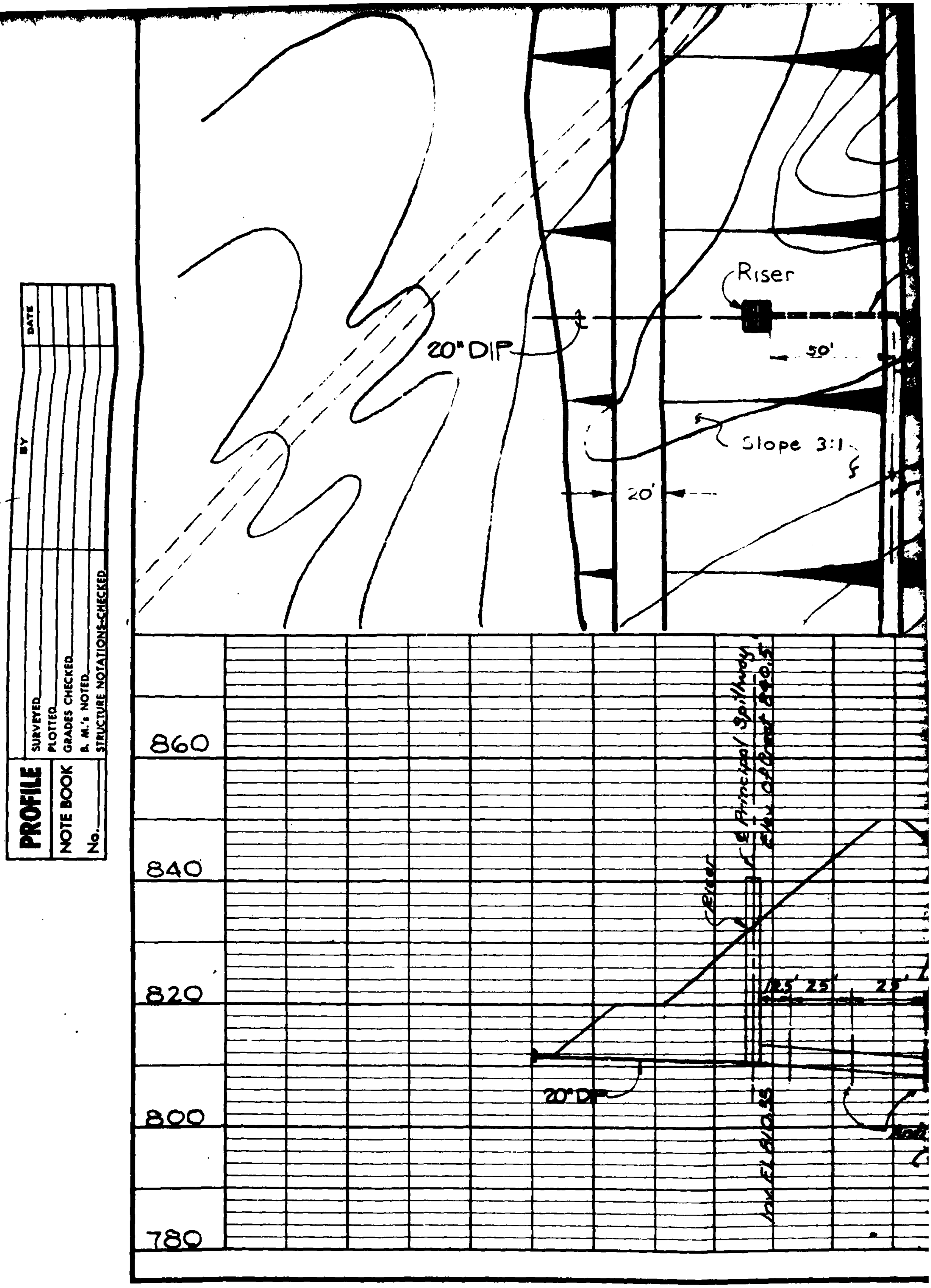
TYPICAL CROSS SECTION OF TRENCH FOR PRINCIPAL SPILLWAY

N.T.S

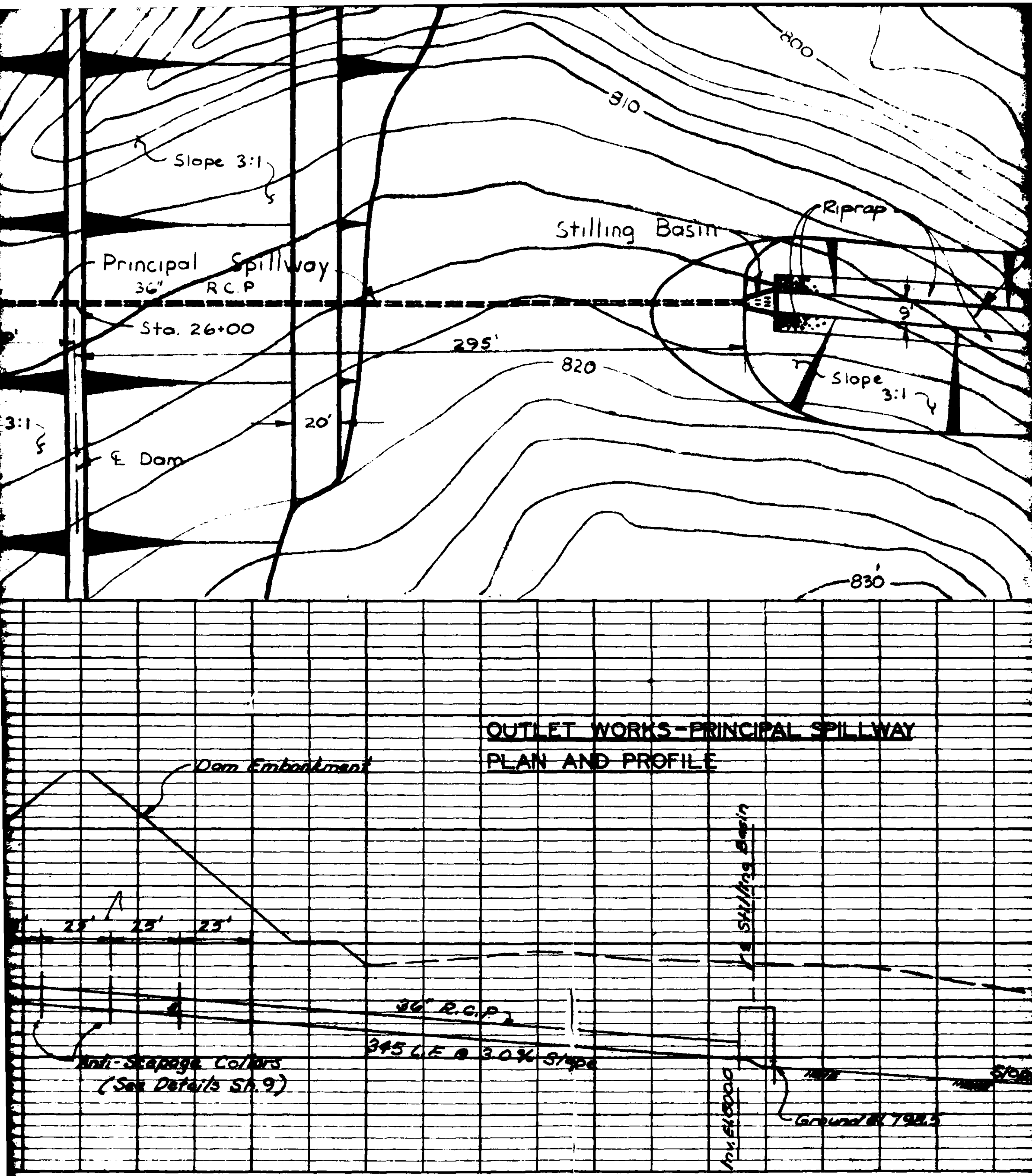


SECTION OF TRENCH FOR PRINCIPAL SPILLWAY

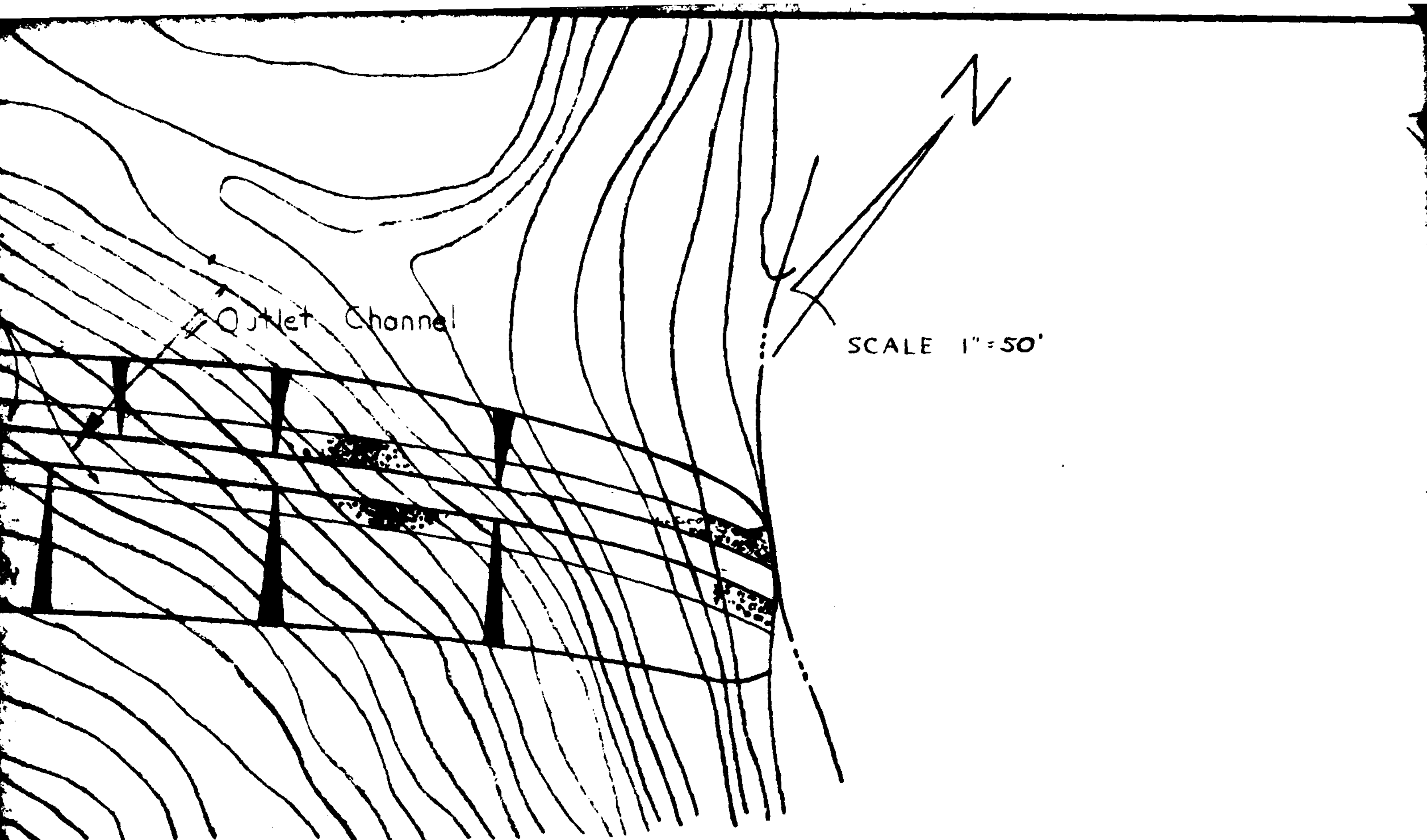
N.T.S

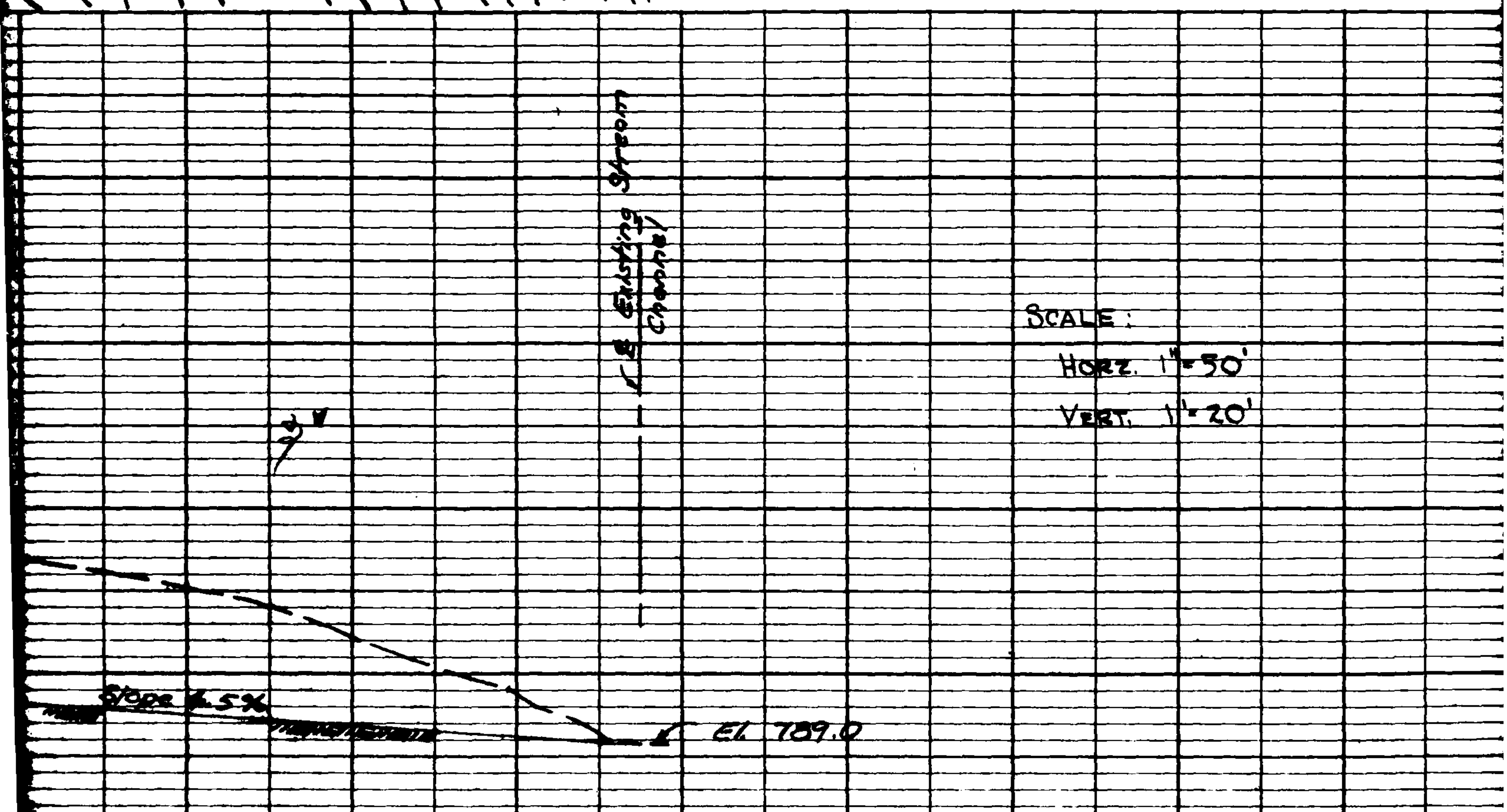


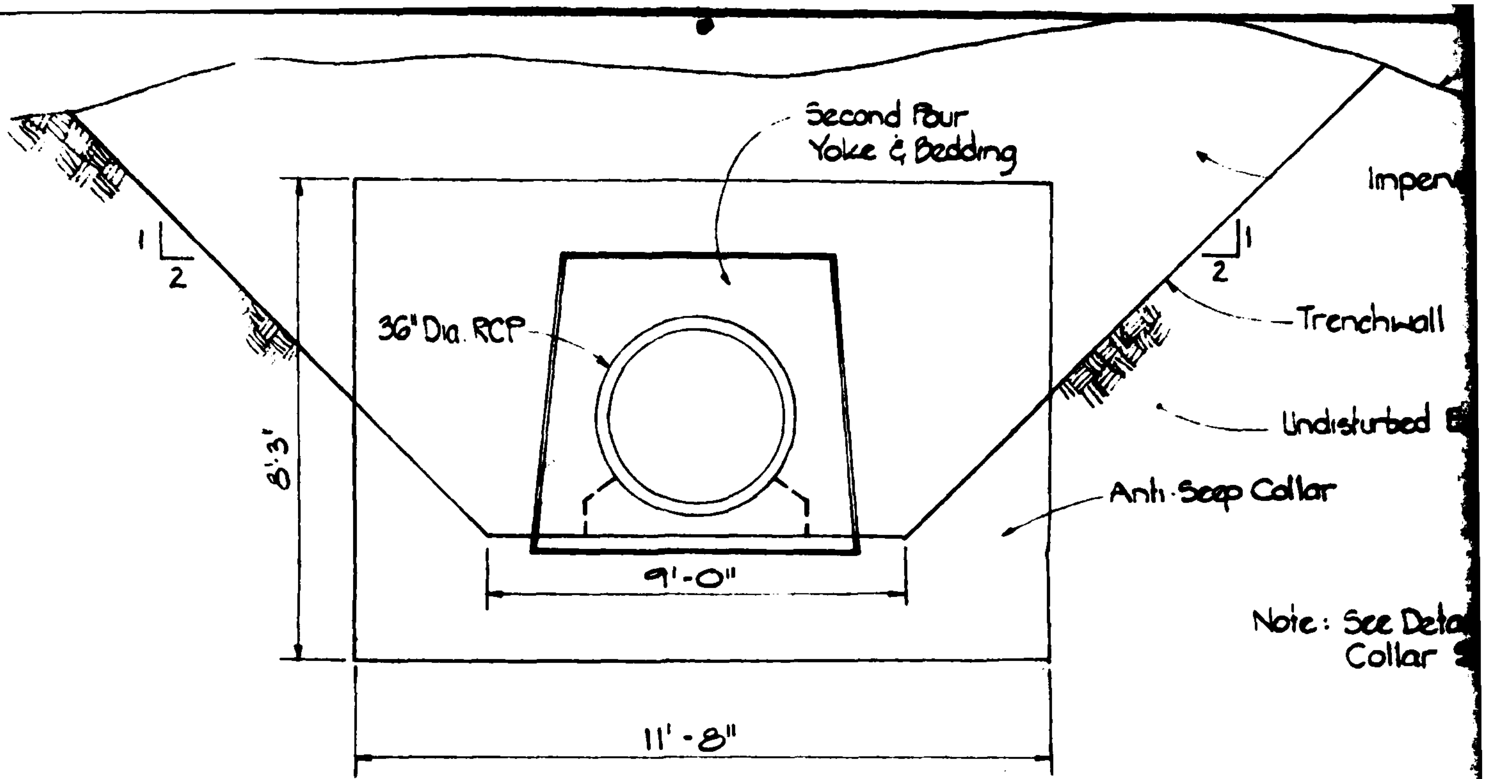
E.T. ARCHER & CO. CONSULTING EN



NG ENGINEERS — KANSAS CITY, MO.

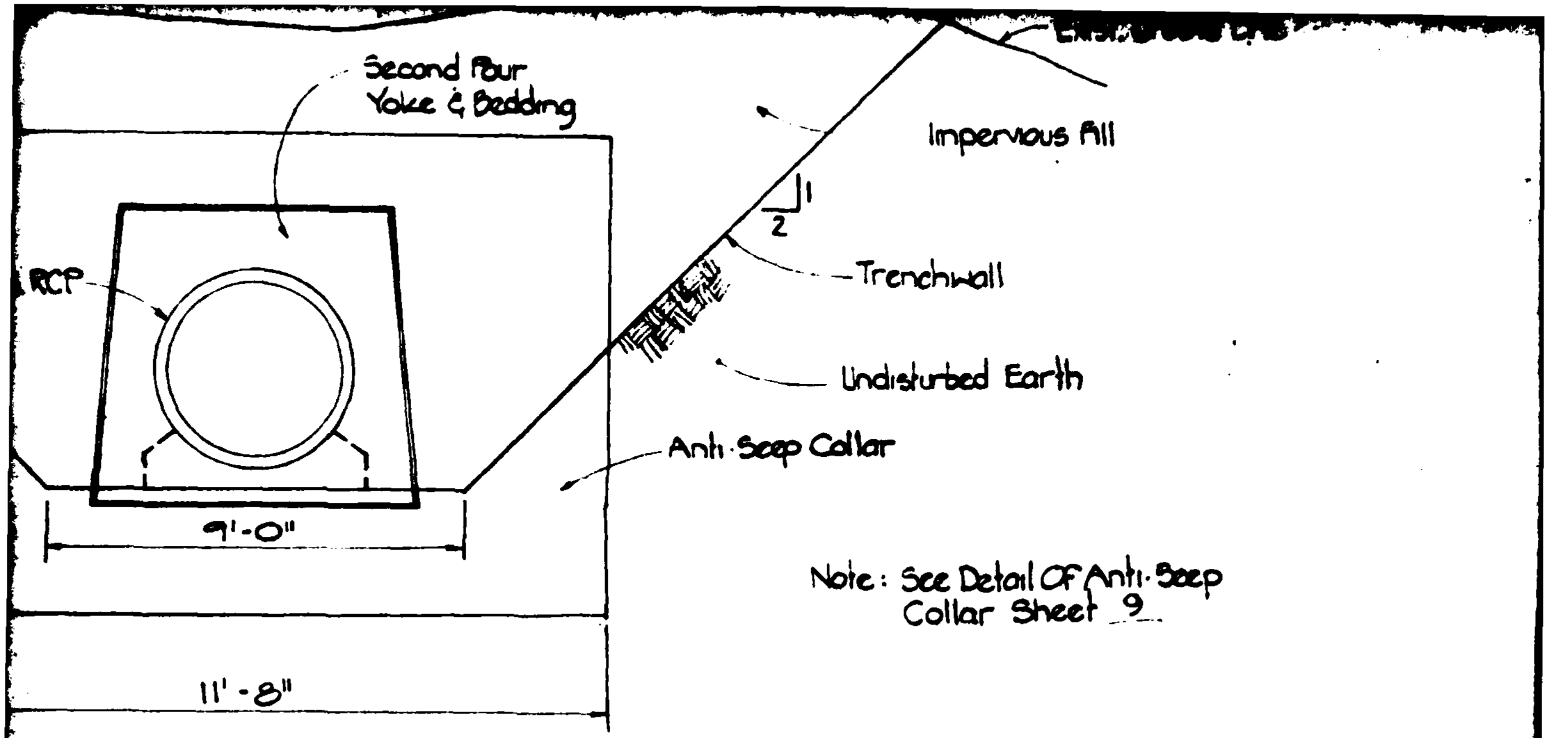

Outlet Channel
N
SCALE 1" = 50'


℄ Existing Stream Channel
SCALE:
Horz. 1" = 50'
Vert. 1" = 20'
Slope
El. 789.0



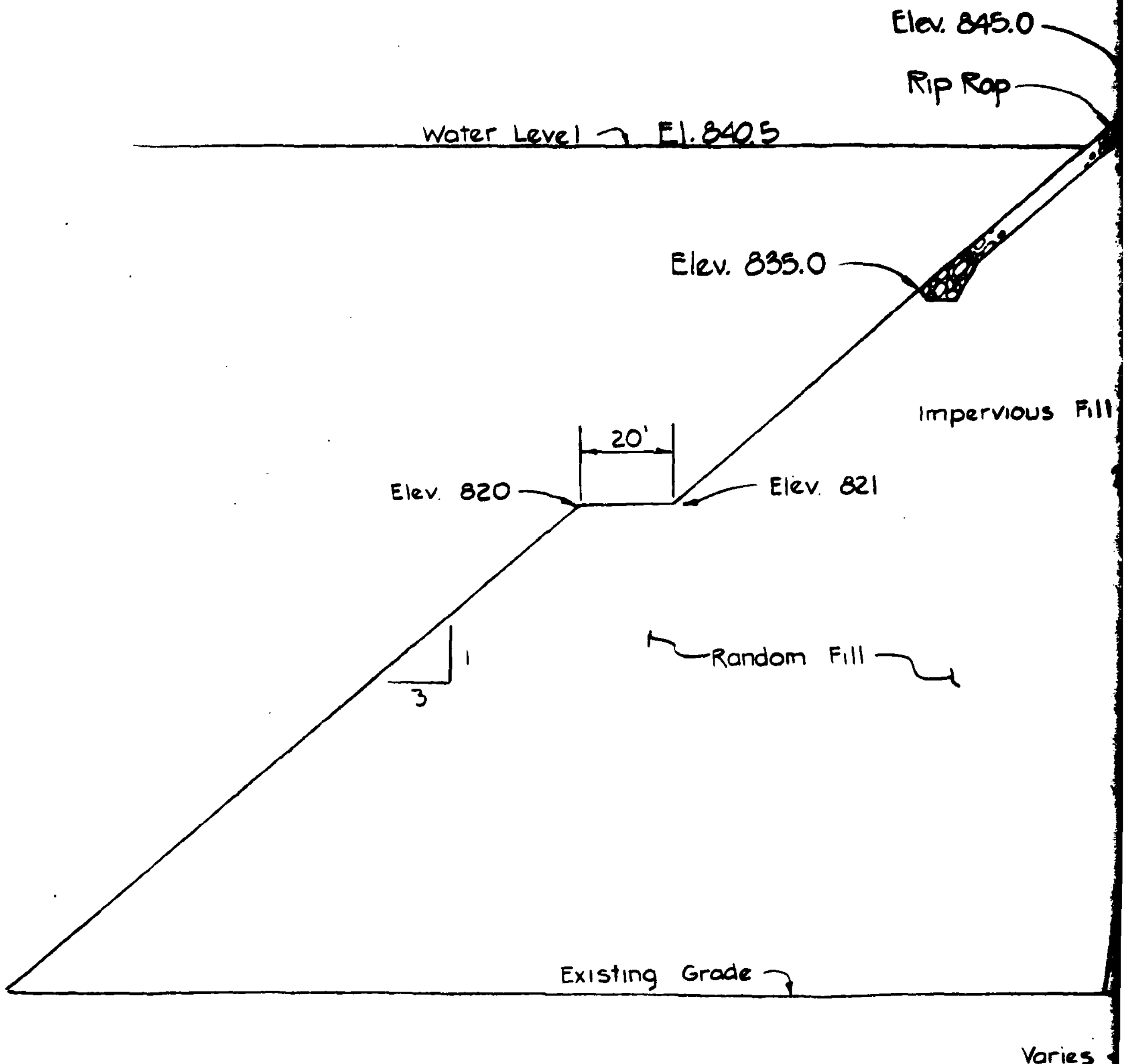
TYPICAL CROSS SECTION OF TRENCH FOR PRINCIPAL SPILLWAY

N.T.S



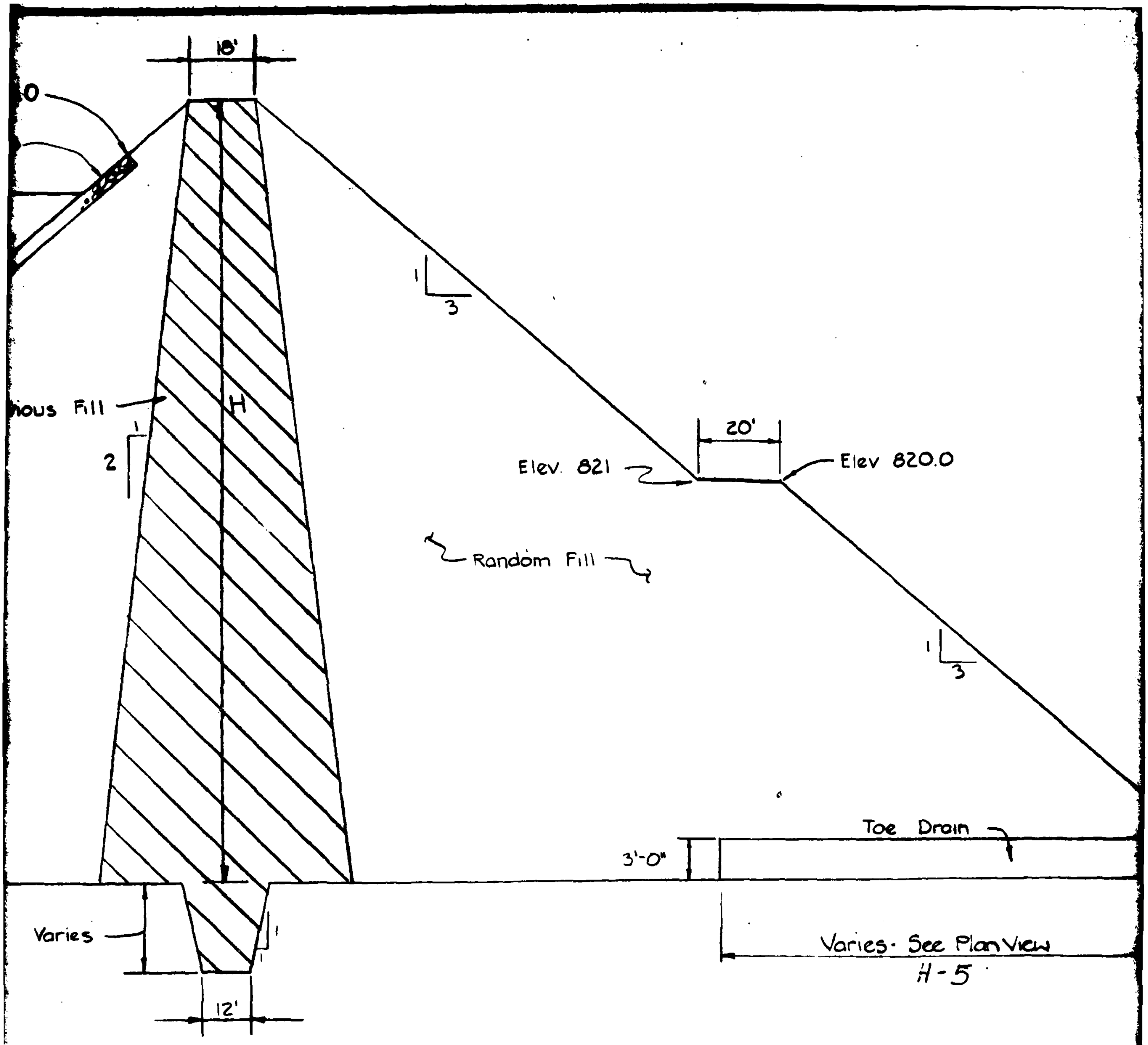
SECTION OF TRENCH FOR PRINCIPAL SPILLWAY

N.T.S



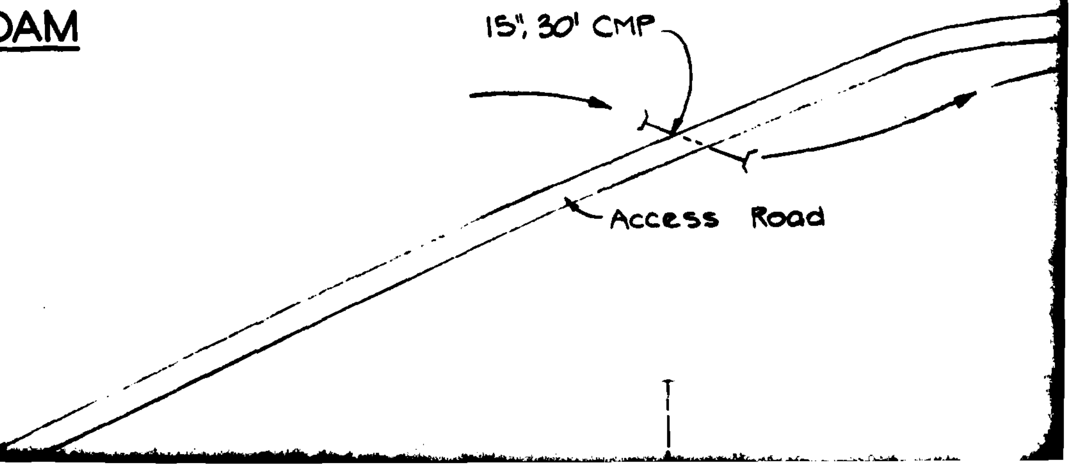
TYPICAL CRO

12"



CROSS SECTION OF DAM

NO SCALE



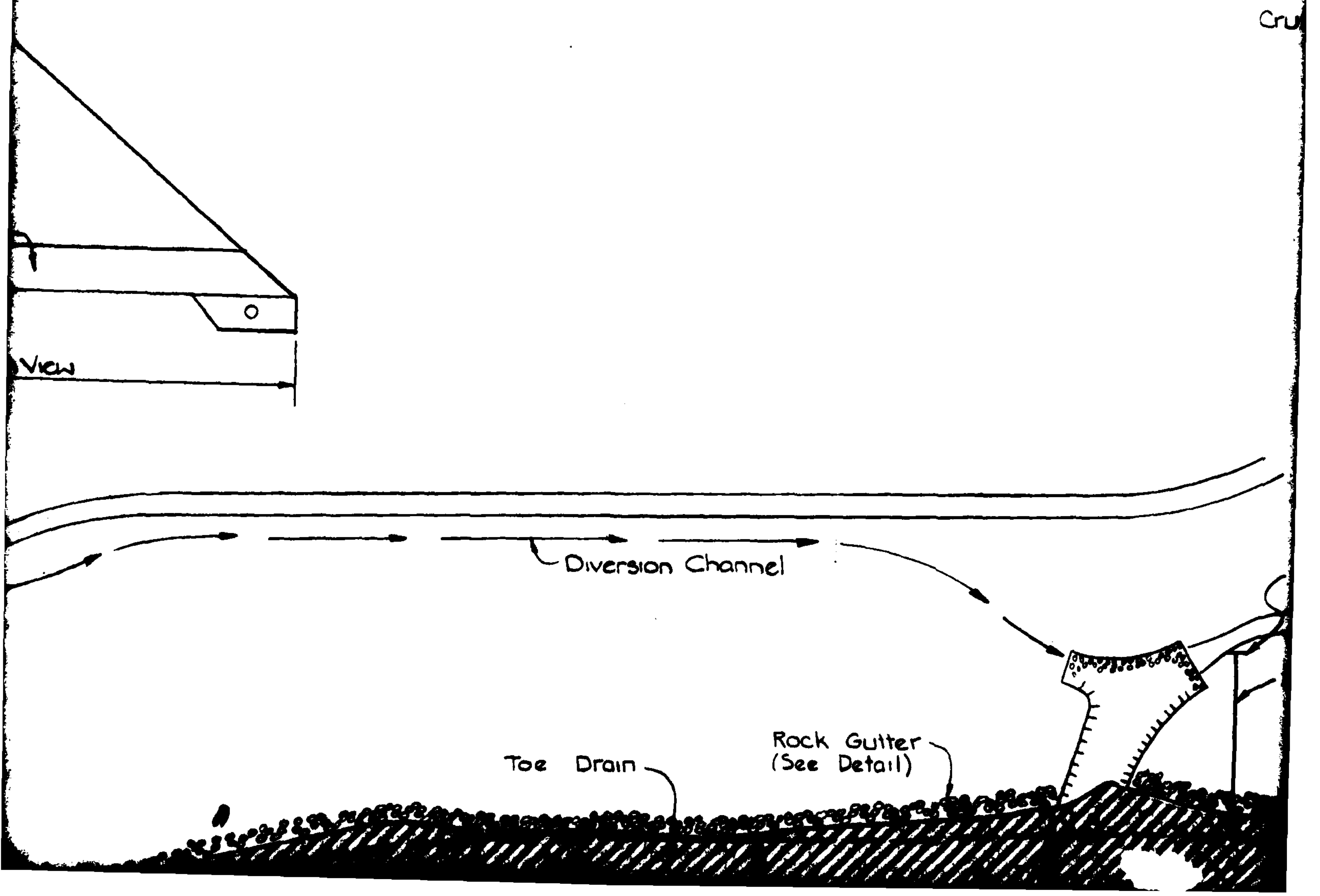
Typical Sieve Analyses For Toe Drain Materials Are As Follows:

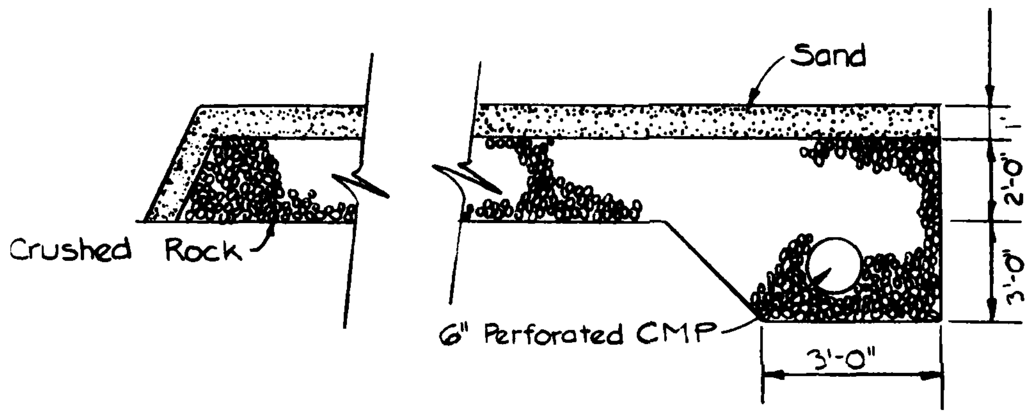
Gravel

Sieve Size	Percent Passing
1"	100
1/2"	68
3/8"	60
No 4	37
No 20	0

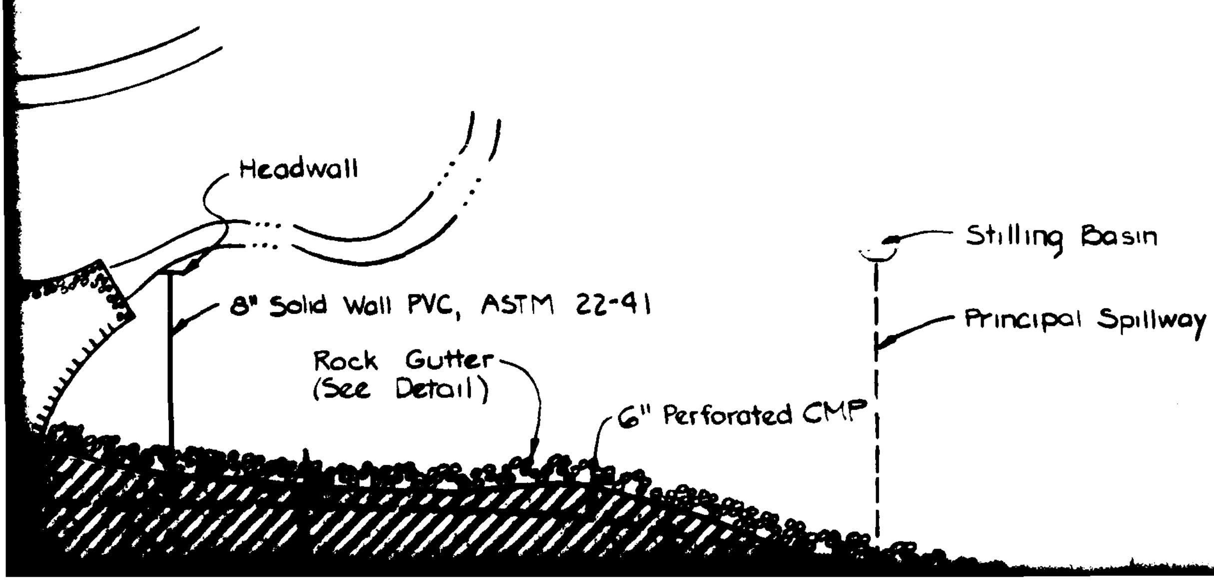
Sand

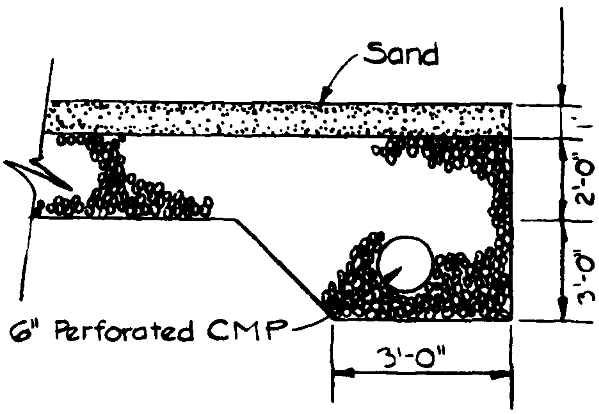
Sieve Size	Percent Passing
1/4"	100
No 6	94
No 10	86
No 16	78
No. 30	66
No. 50	52
No. 200	14



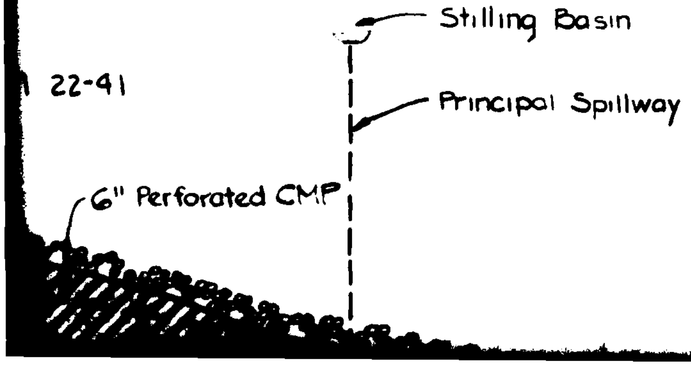


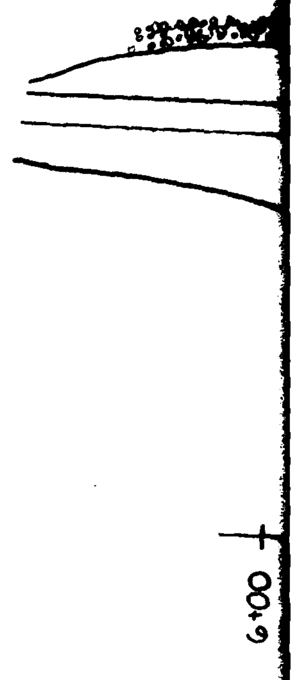
TOE-DRAIN DETAIL

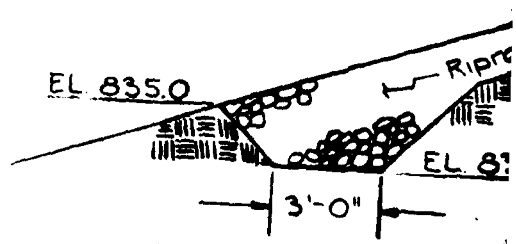


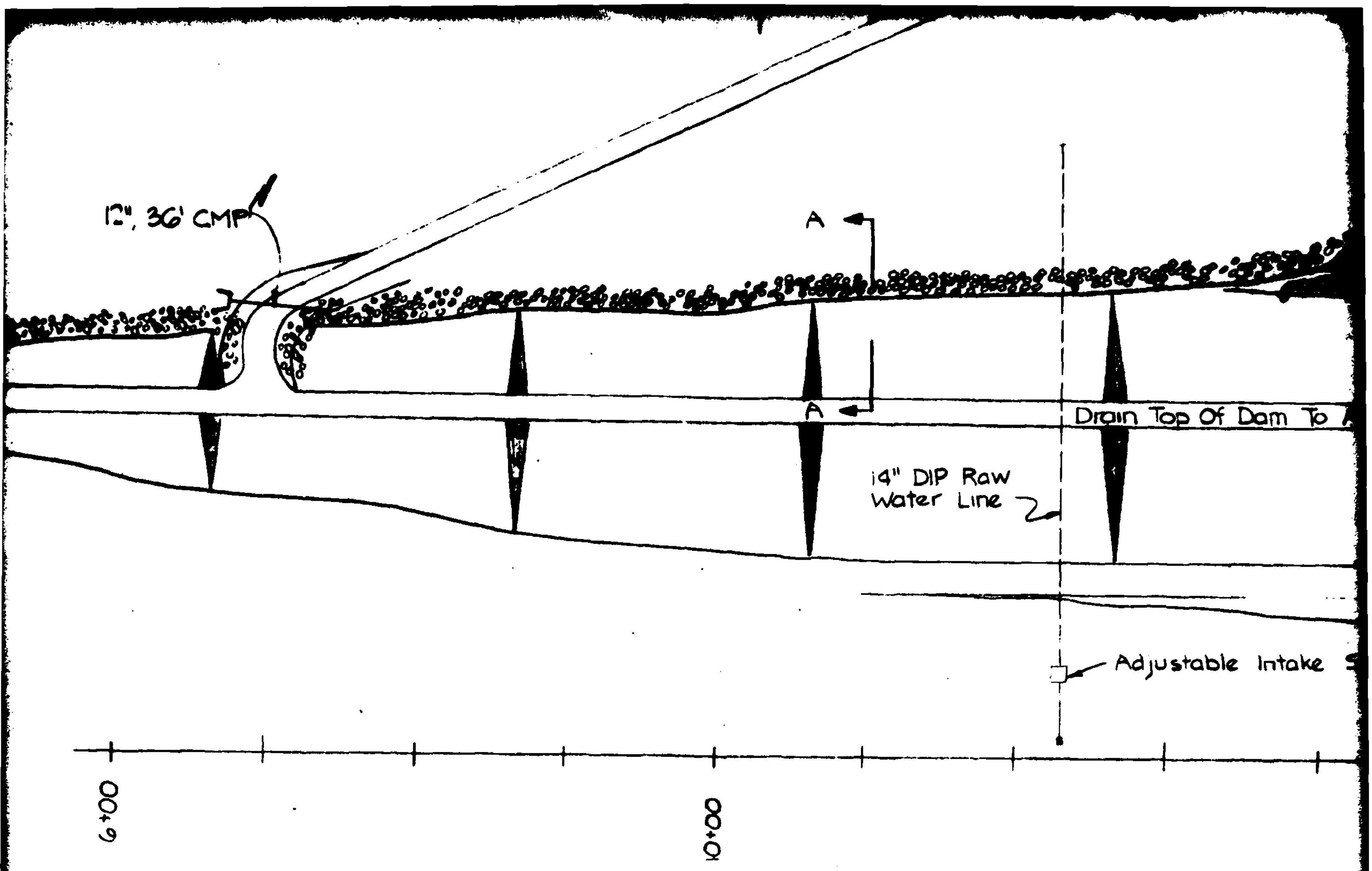


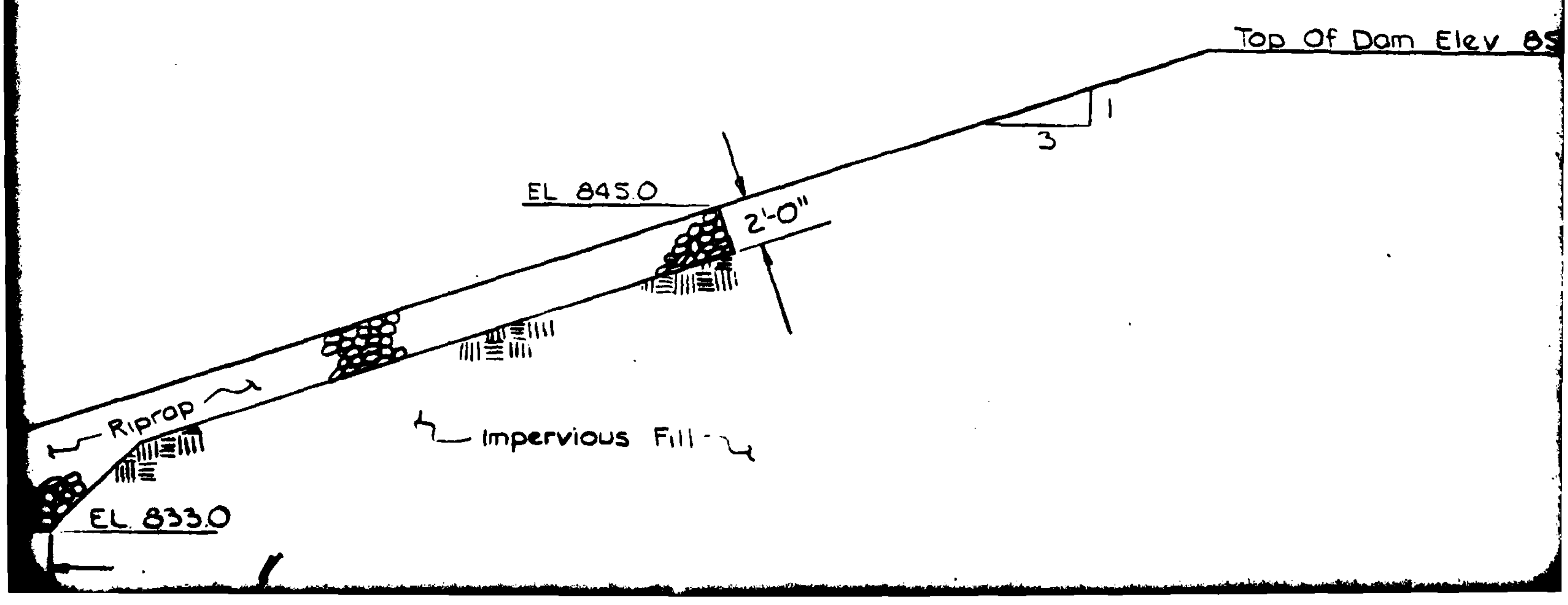
E–DRAIN DETAIL

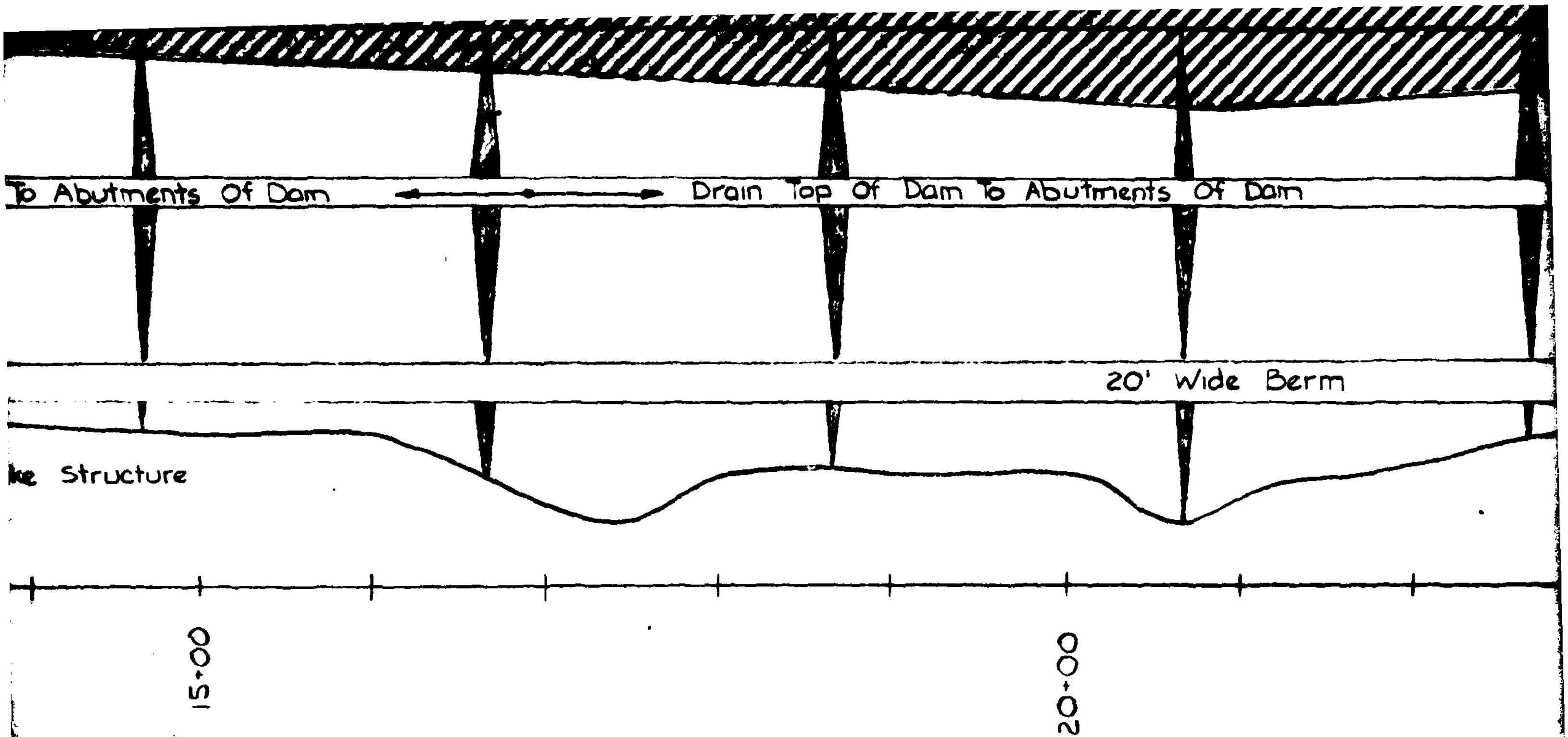



6+00


EL. 835.0
3'-0"


12", 36' CMP
A
A
Drain Top Of Dam To
14" DIP Raw Water Line
Adjustable Intake
6+00
8+00

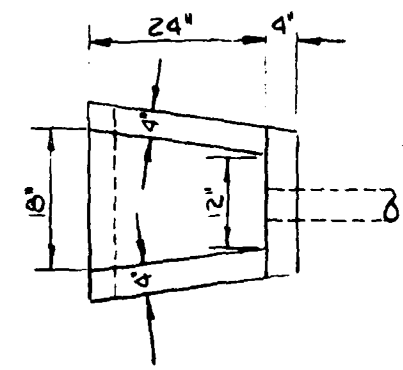

Top Of Dam Elev 85
1
3
EL 845.0
2'-0"
Riprop
Impervious Fill
EL 833.0

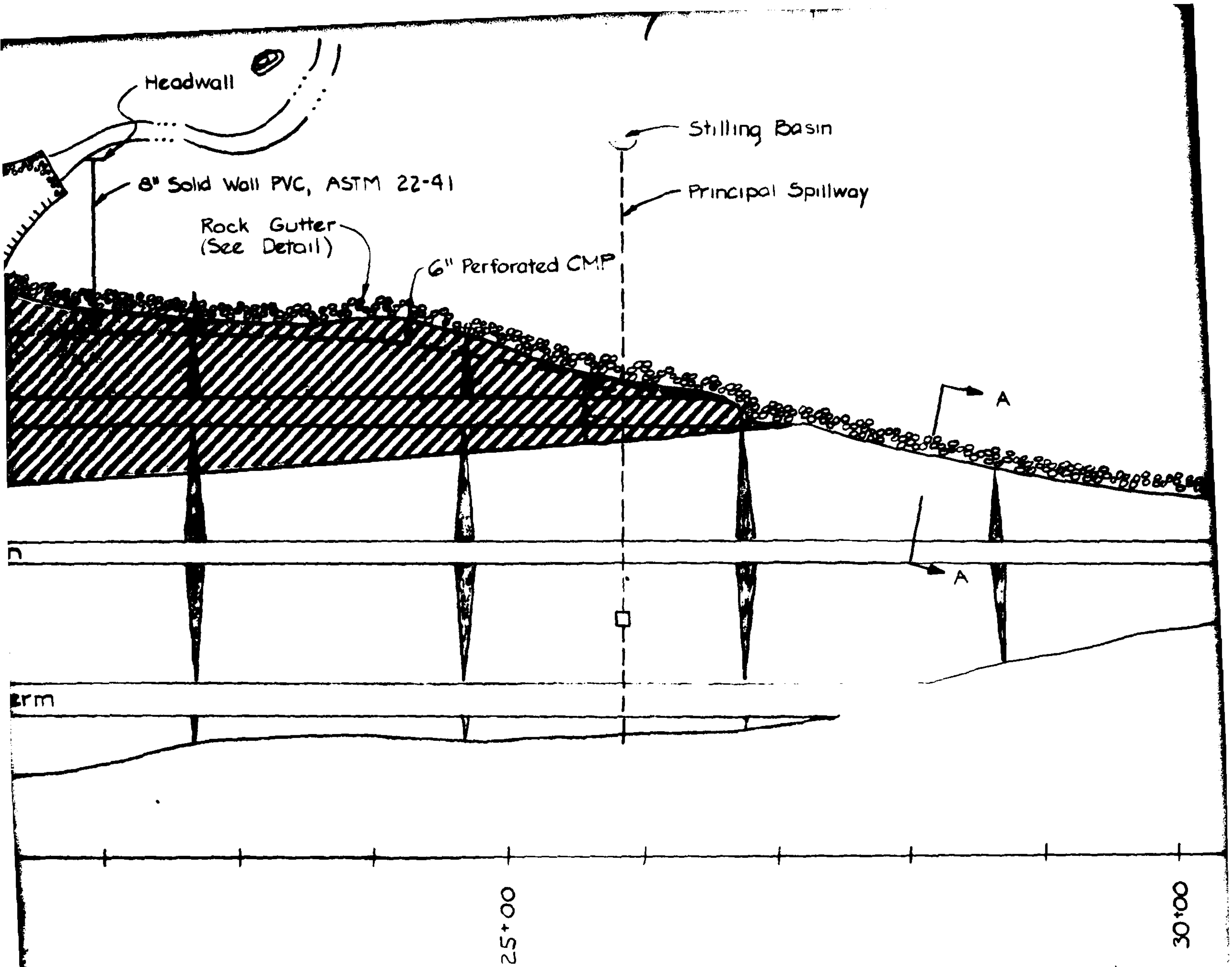


DOWNSTREAM DRAINAGE SYSTEM – PLAN VIEW

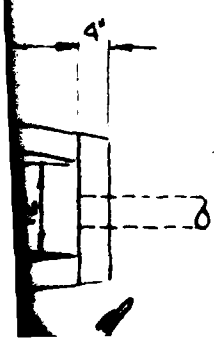
SCALE 1"=100'

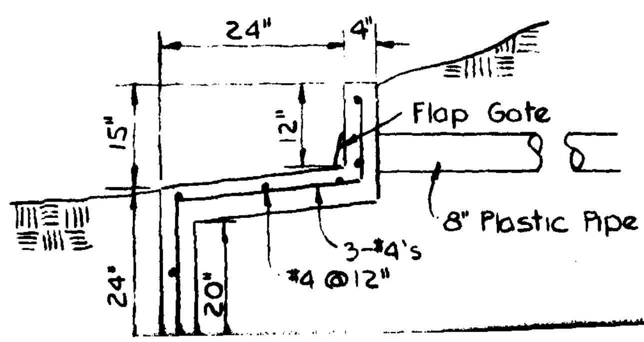
8500

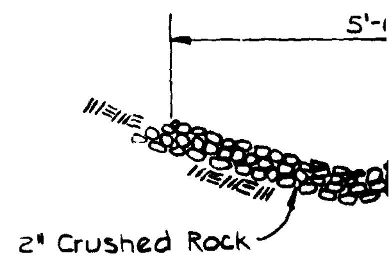




N VIEW

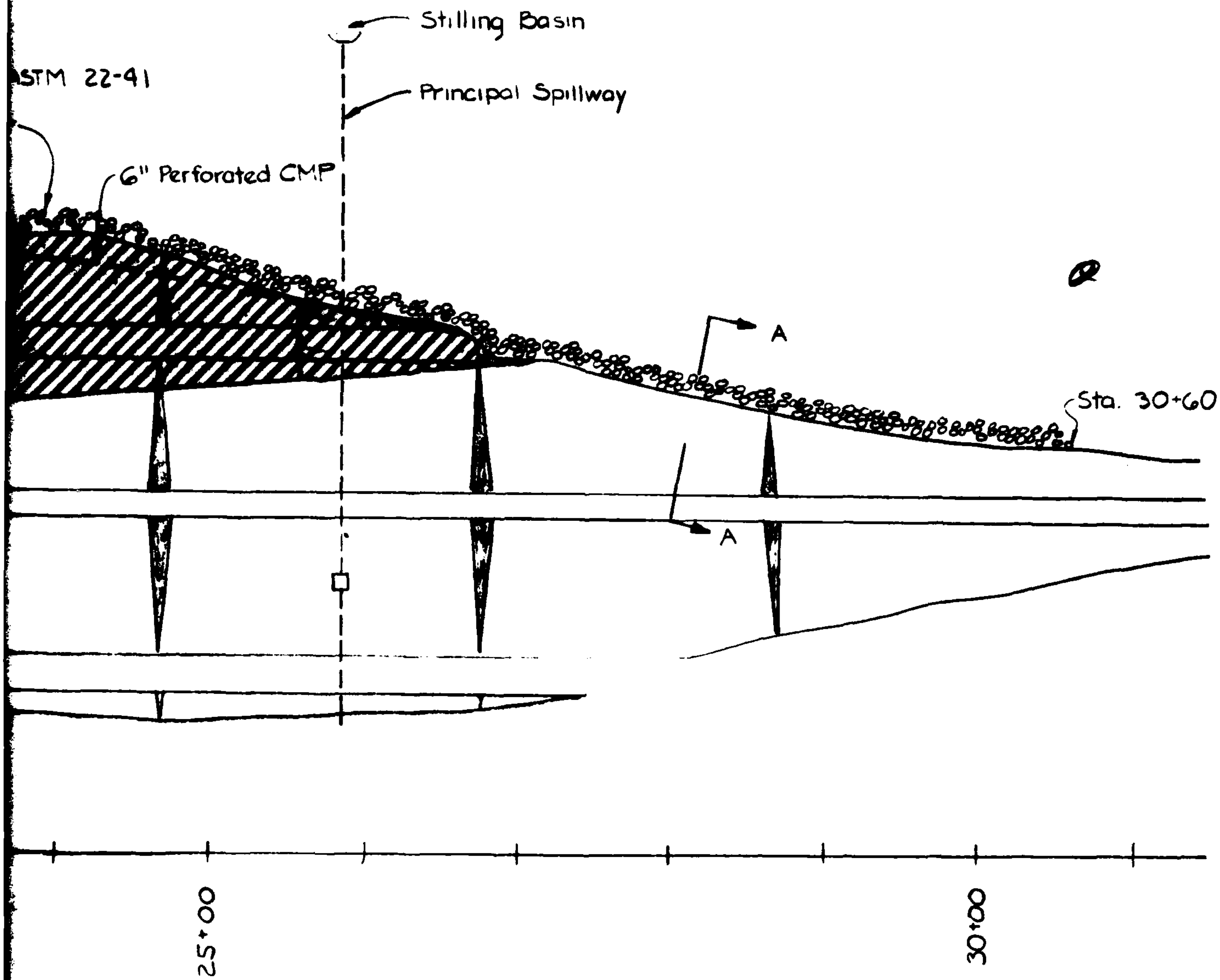


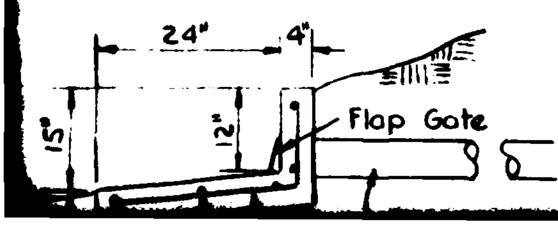


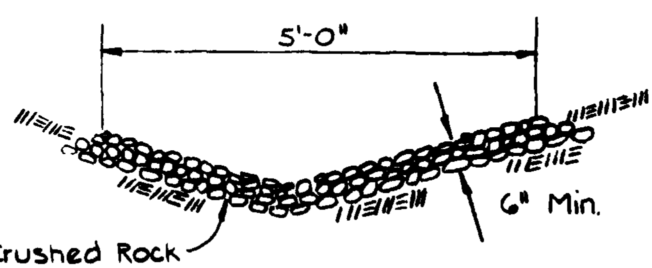


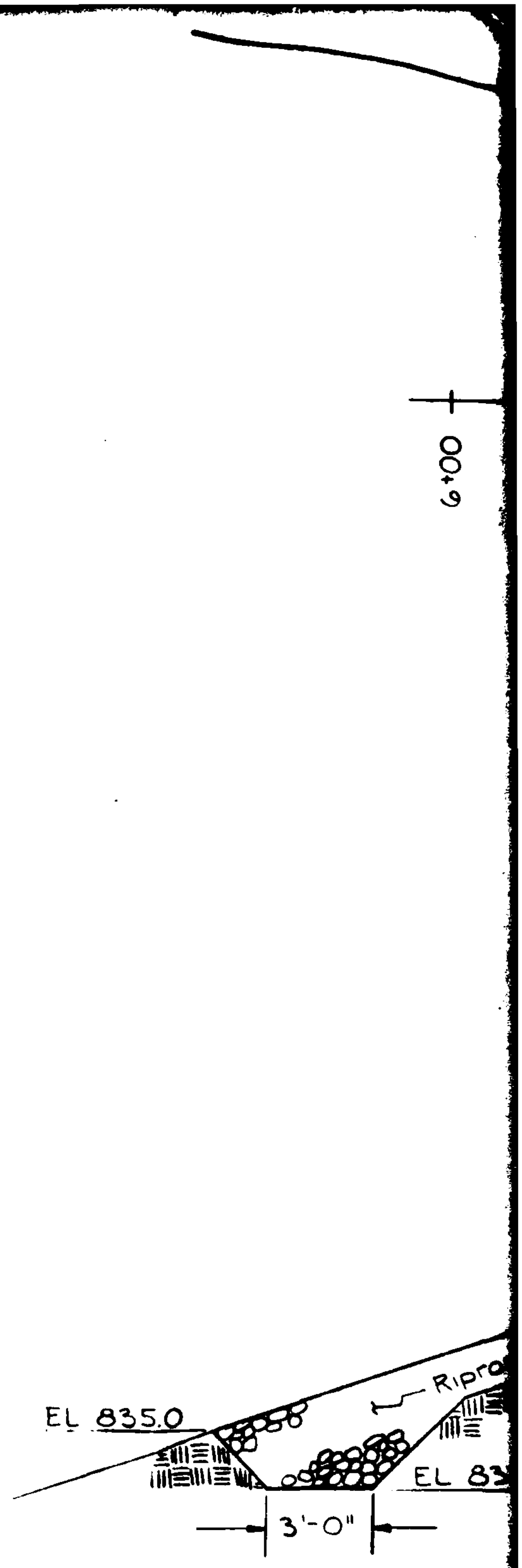
ROCK GUT

9

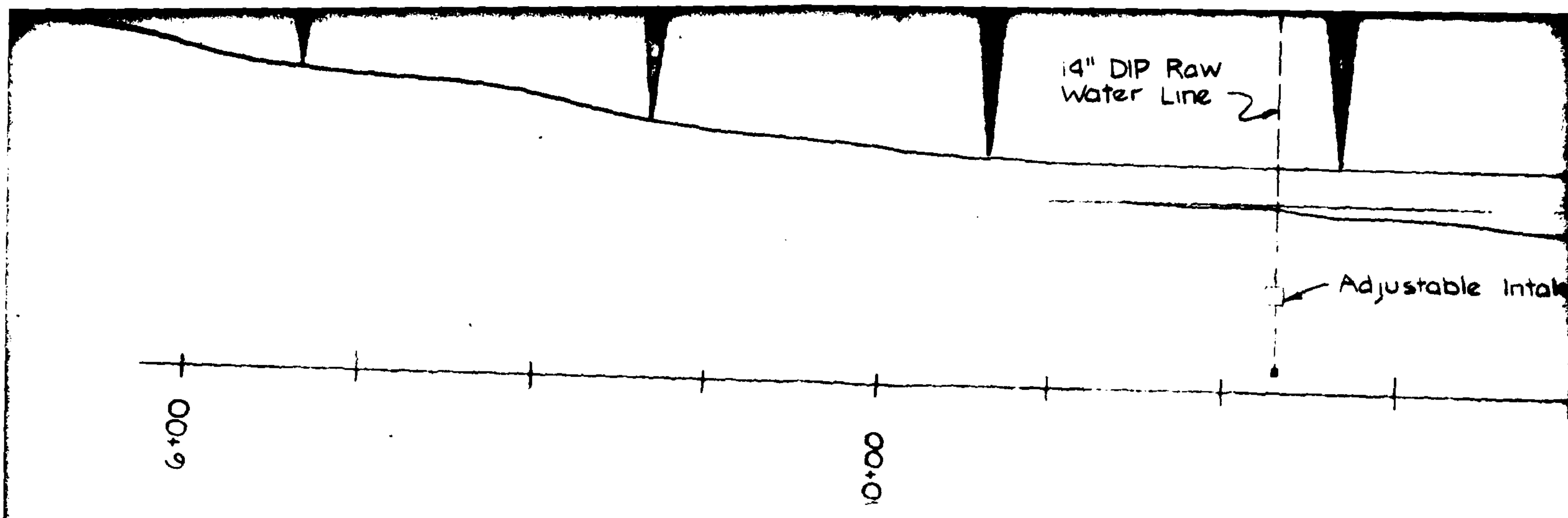

Stilling Basin
Principal Spillway
STM 22-41
6" Perforated CMP
A
A
Sta. 30+60
25+00
30+00

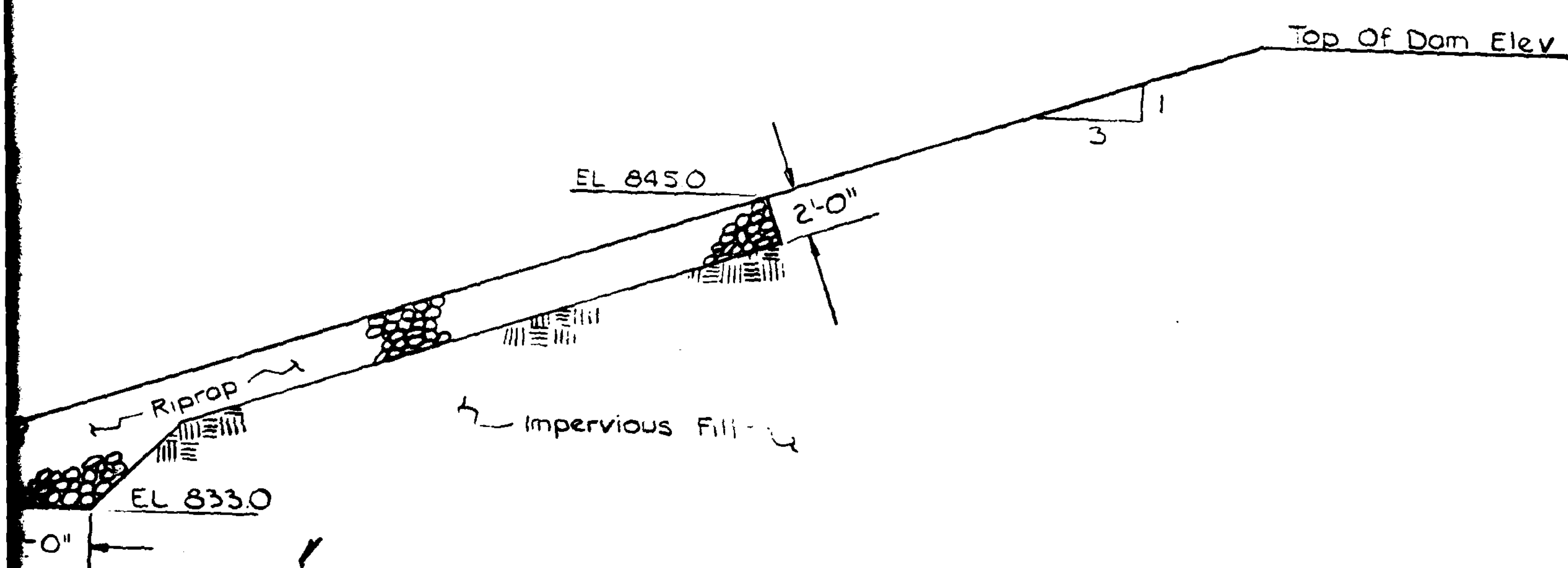

24"
4"
15"
12"
Flap Gate


5'-0"
6" Min.
2" Crushed Rock



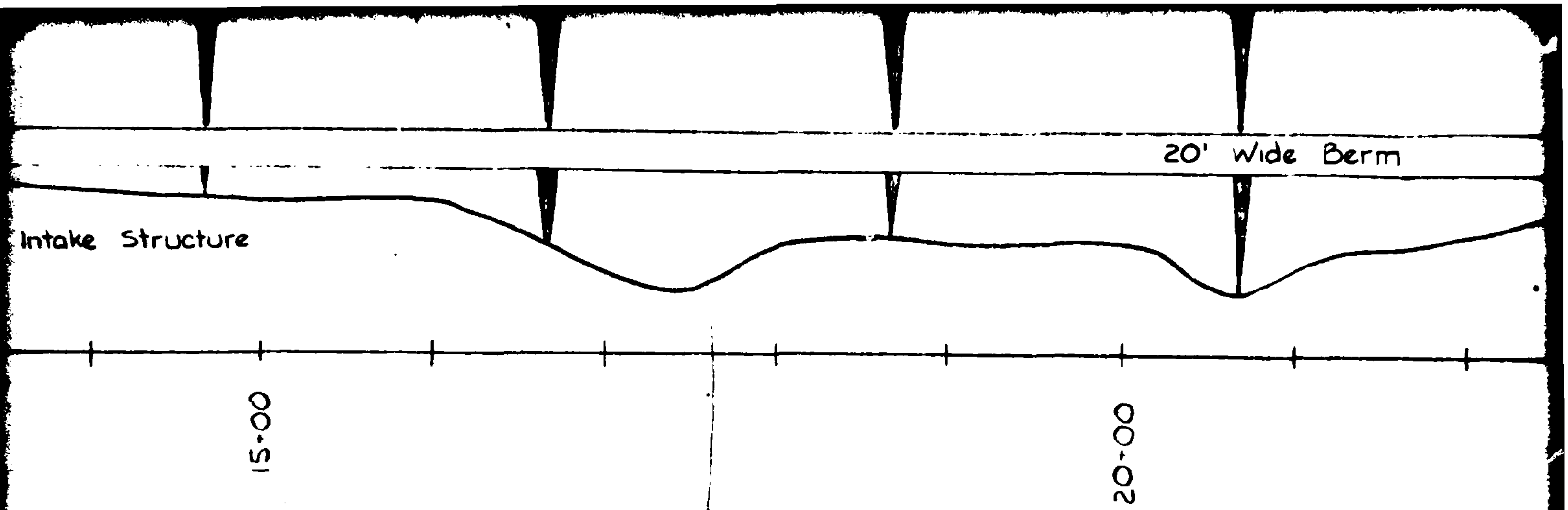
UPSTR





UPSTREAM SLOPE RIPRAP DETAIL

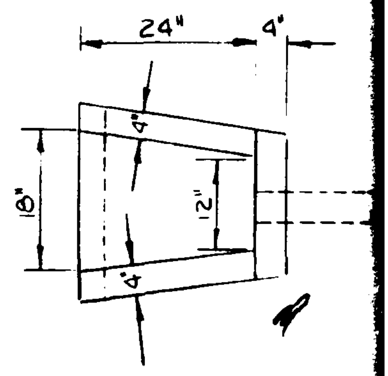
NO SCALE



DOWNSTREAM DRAINAGE SYSTEM – PLAN VIEW

SCALE 1"=100'

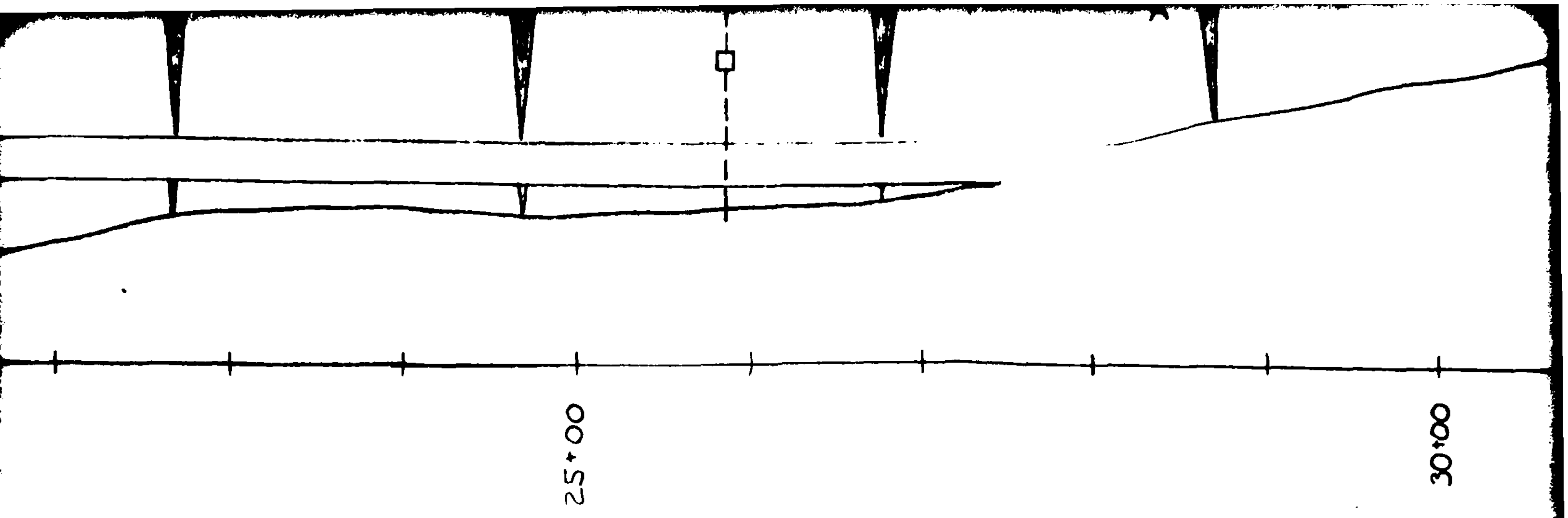
Elev 8500



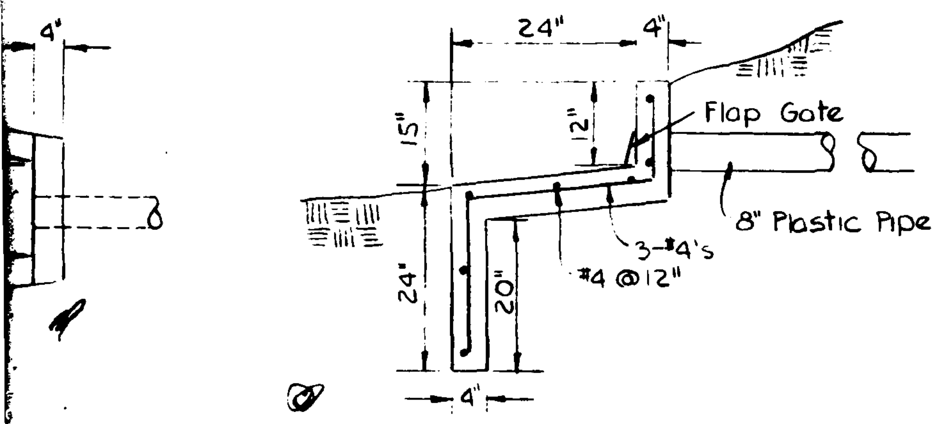
PLAN

HEADW

SC

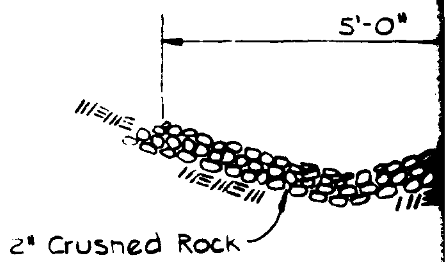


VIEW



SECTION

HEADWALL DETAIL
SCALE 1"= 2'



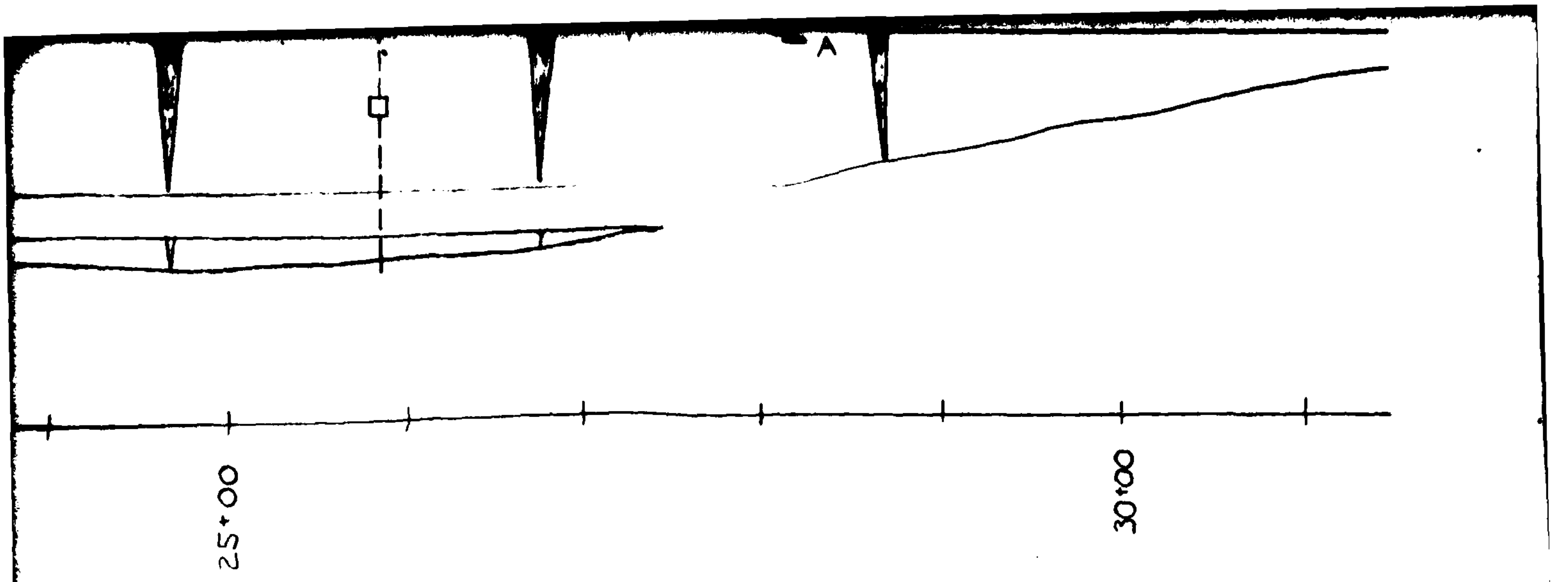
ROCK GUTTE

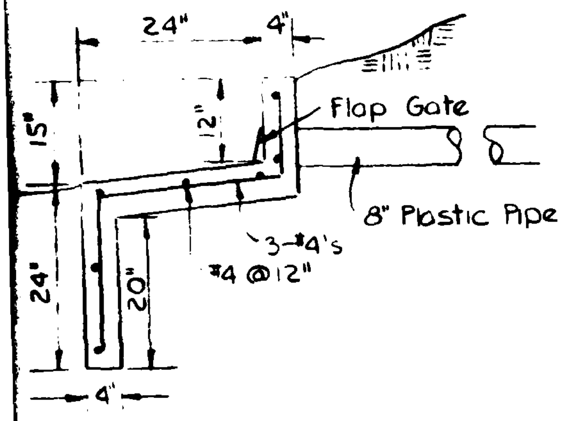
WATER SUPPLY RESERVOIR
HOLDEN, MISSOURI

E. T. ARCHER & COMPA
CONSULTING ENGINEERS
KANSAS CITY, MISSOURI

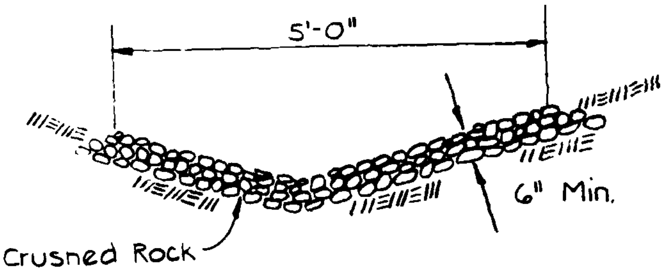
DAM TOE DRAIN & RIPRAP

DWN. BY WEB	CKD. BY GML	SHEET NO. 11 OF 35





SECTION



ROCK GUTTER DETAIL

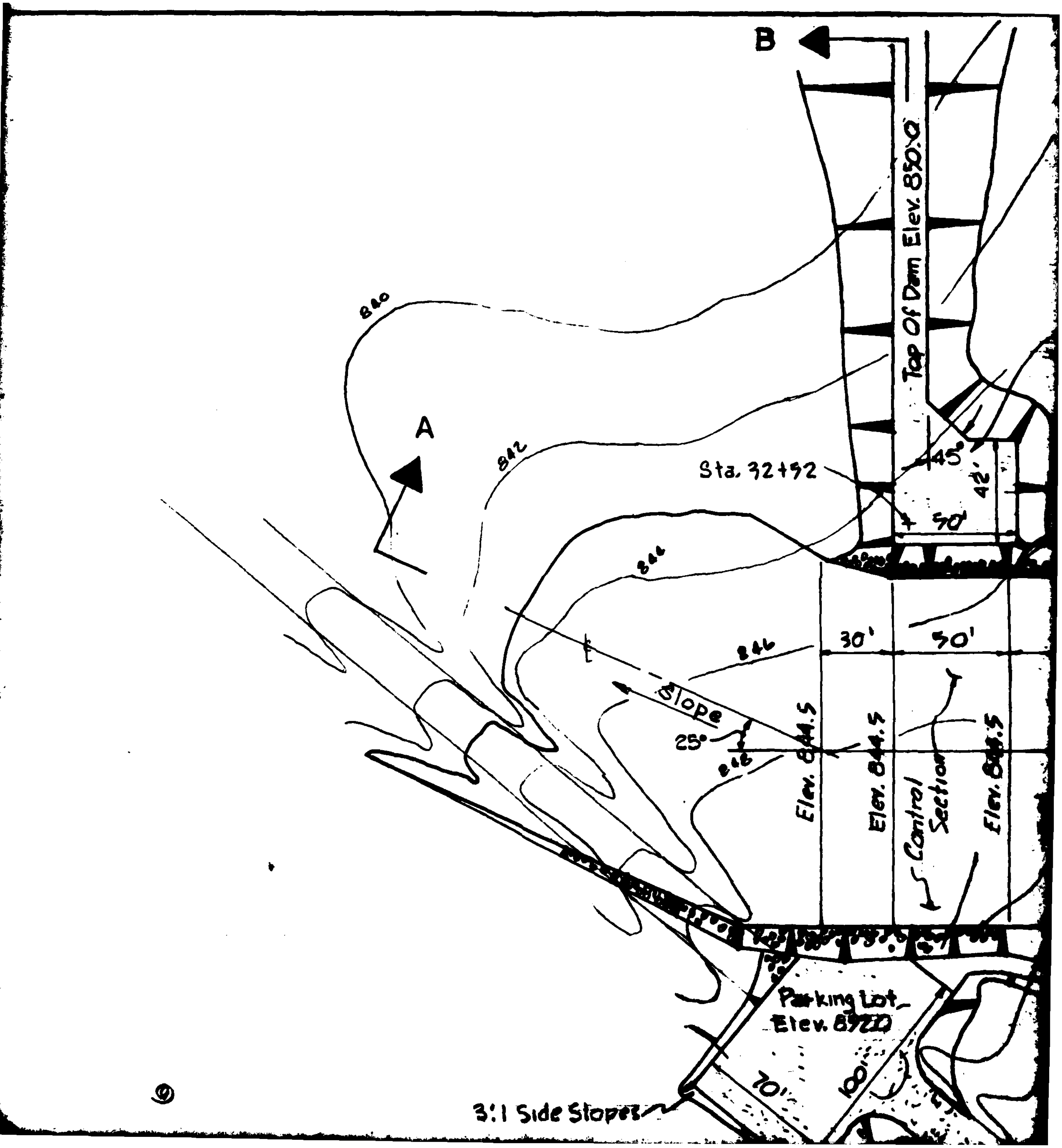
WATER SUPPLY RESERVOIR
HOLDEN, MISSOURI

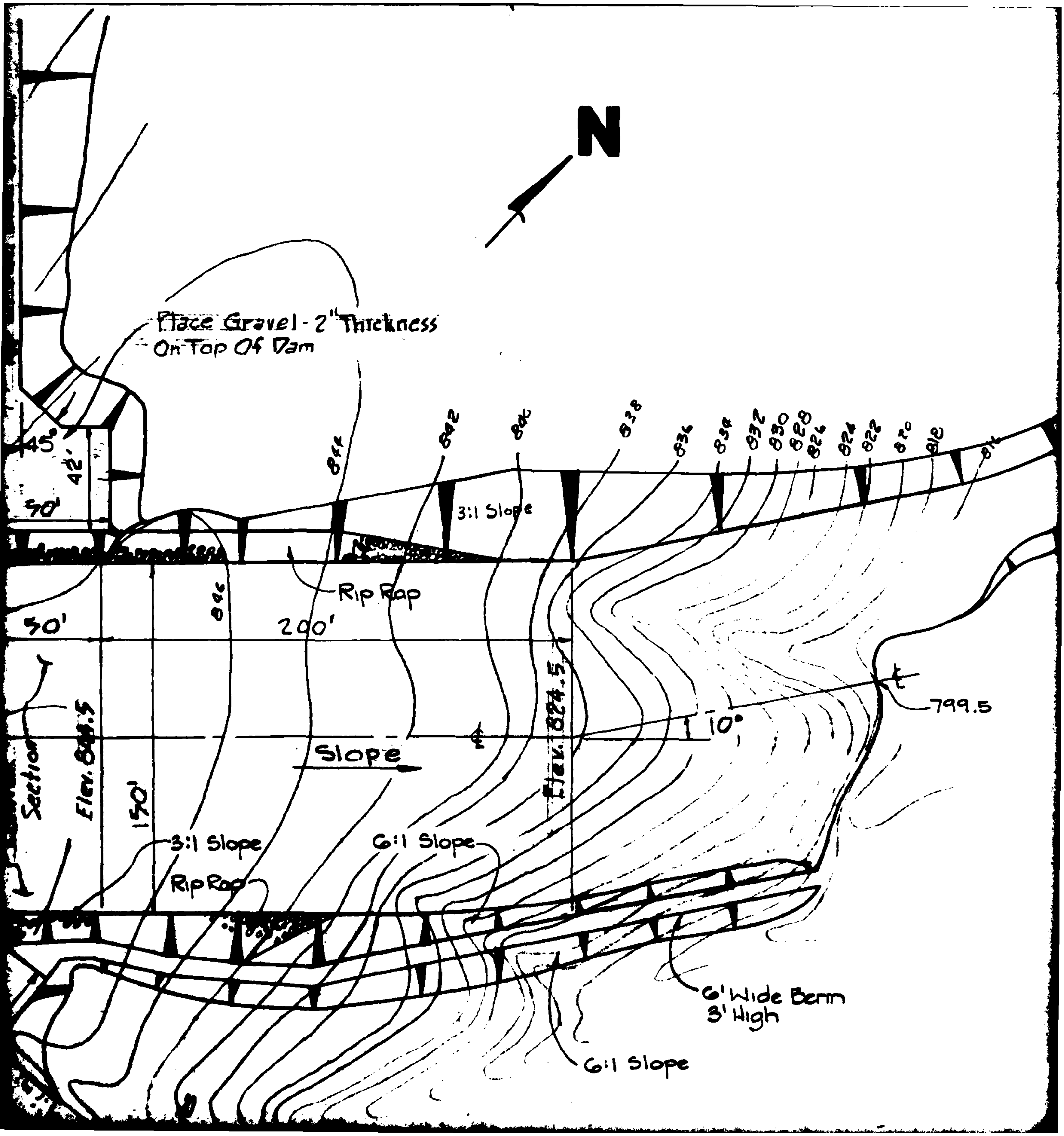
E. T. ARCHER & COMPANY
CONSULTING ENGINEERS
KANSAS CITY, MISSOURI

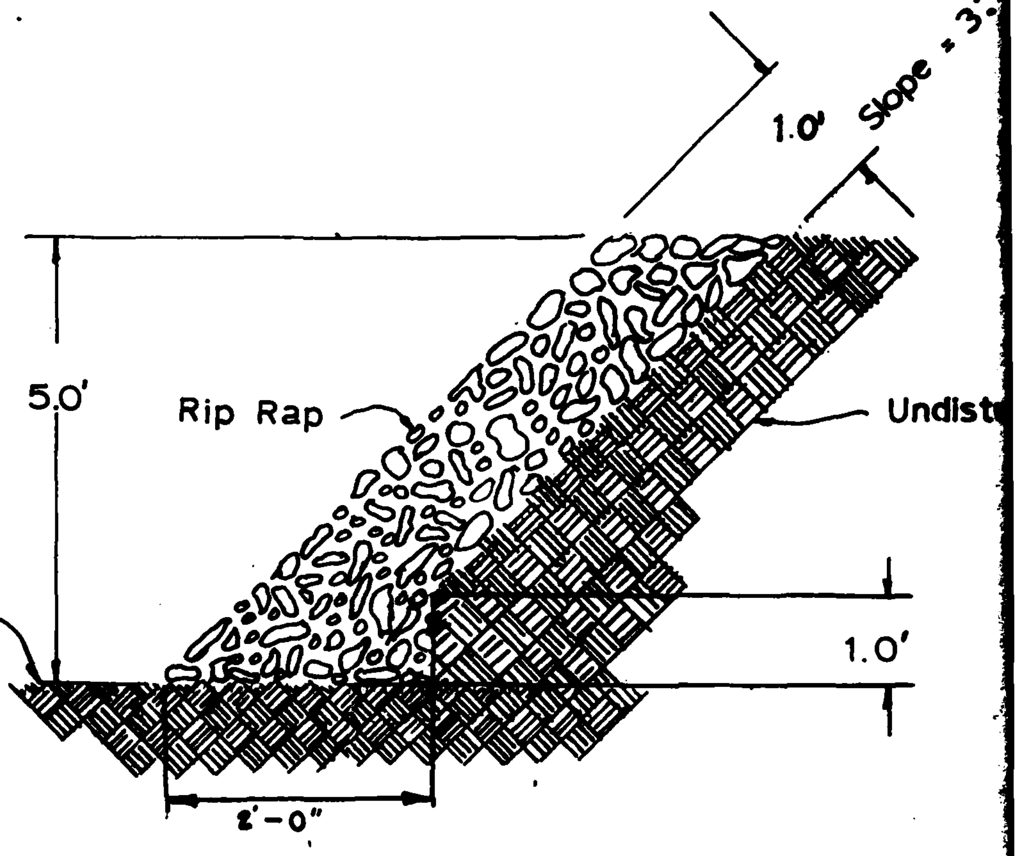
DAM TOE DRAIN & RIPRAP DETAIL

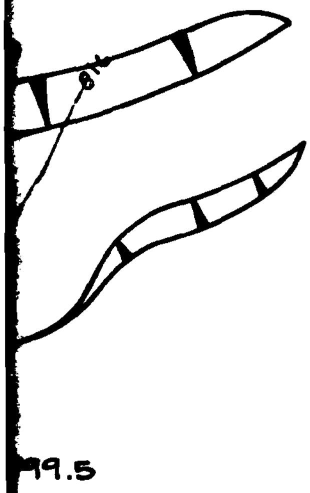
DWN. BY WEB	CKD. BY GML	SHEET NO. 11 OF 35	DWG. NO. 379-11

PLAN		BY	DATE
NOTE BOOK	SURVEYED		
	PLOTTED		
	ALIGNMENT CHECKED		
No.	RT. OF WAY CHECKED		

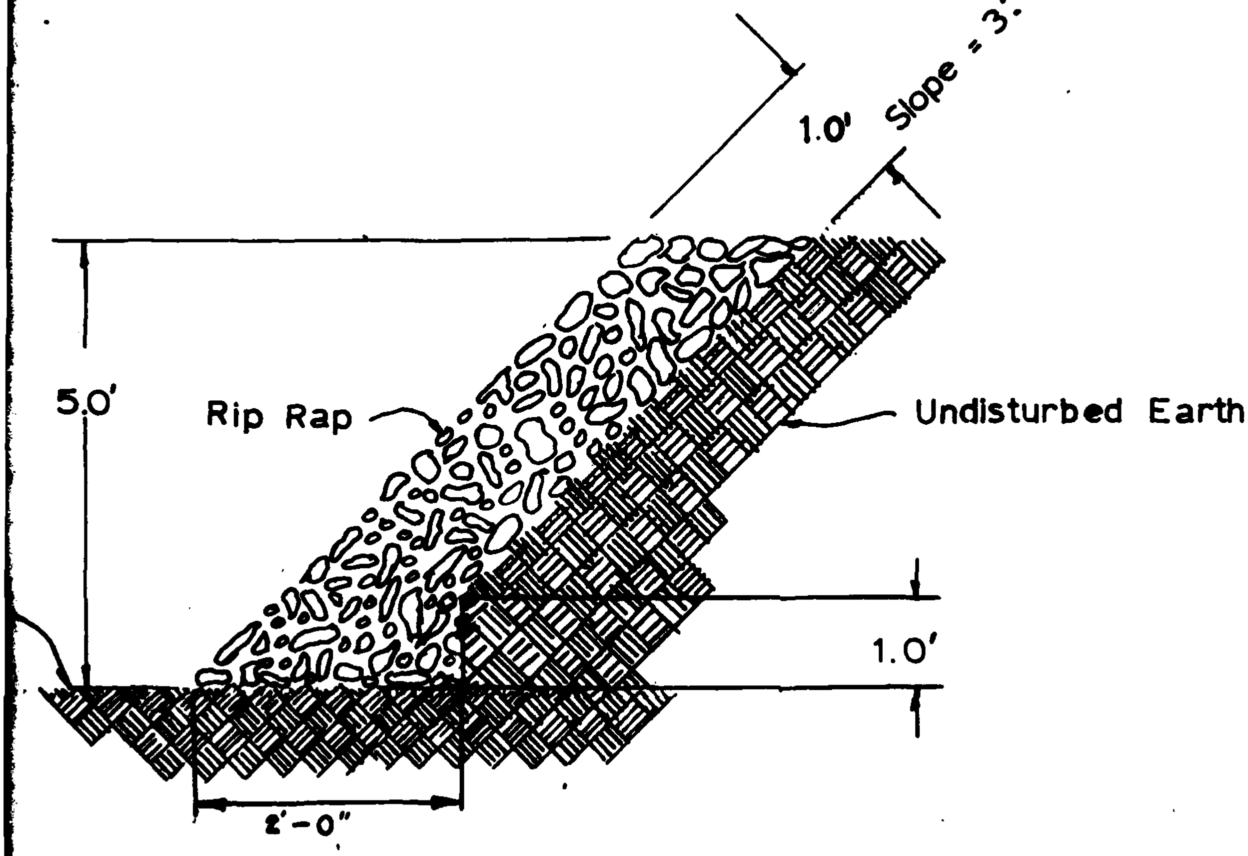

B
Top Of Dam Elev. 850.0
840
A
842
Sta. 32+52
45°
42'
50'
30'
50'
Slope
25°
Elev. 844.5
Elev. 844.5
Control Section
Parking Lot
70'
100'
3:1 Side Slopes


N
Place Gravel - 2" Thickness
On Top Of Dam
45°
42'
30'
844
842
840
838
836
834
832
830
828
826
824
822
820
818
816
3:1 Slope
Rip Rap
846
30'
200'
Section
Elev. 846.5
150'
Slope
Elev. 824.5
10°
799.5
3:1 Slope
Rip Rap
6:1 Slope
6' Wide Berm
3' High
6:1 Slope





RIP RAP DETAIL
(TYPICAL)



RIP RAP DETAIL
(TYPICAL)

6

PROFILE

NOTE BOOK

No.

SURVEYED

PLOTTED

GRADES CHECKED

B. M.'s NOTED

STRUCTURE NOTATIONS CHECKED

BY

DATE

850

845

840

835

830

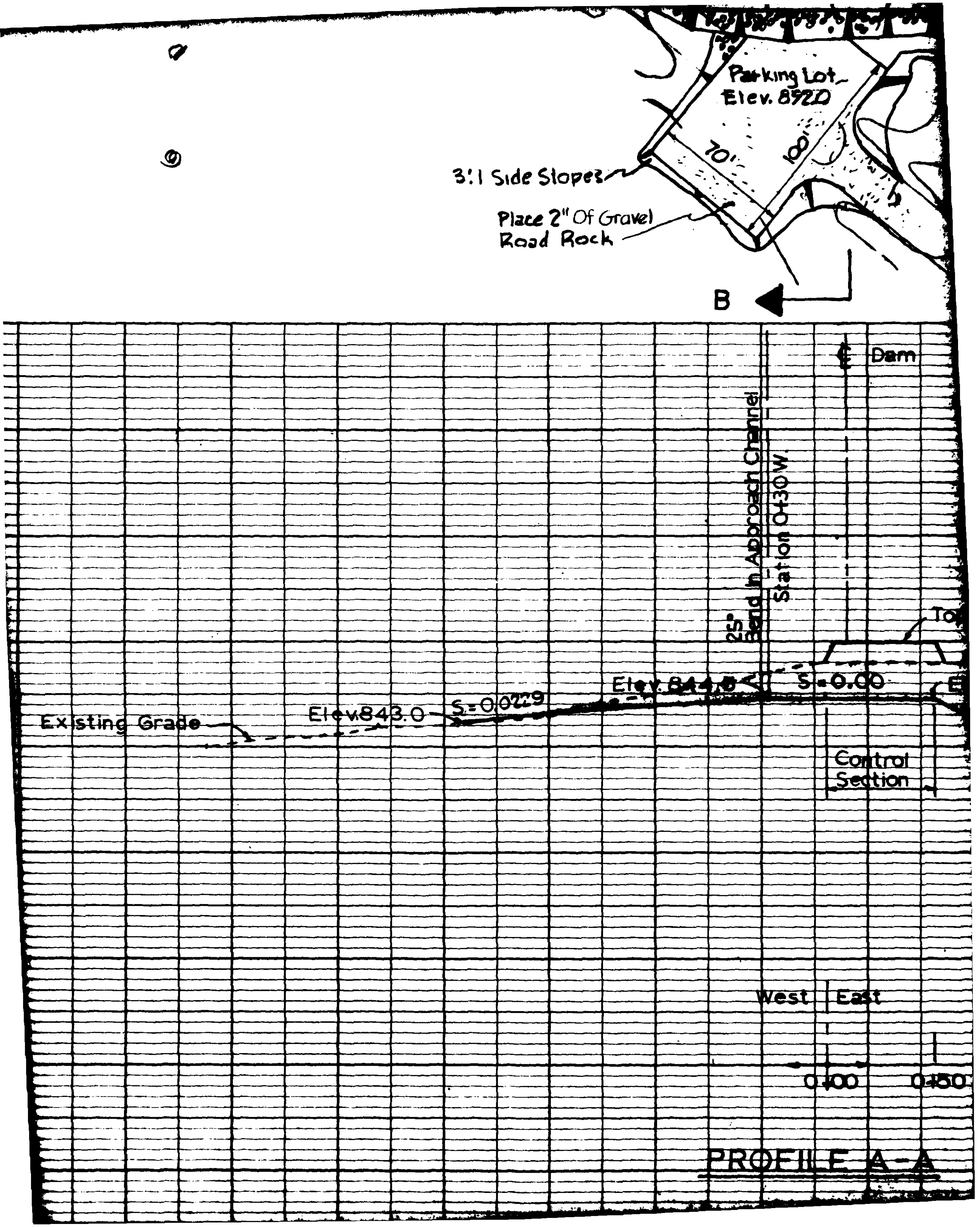
825

820

815

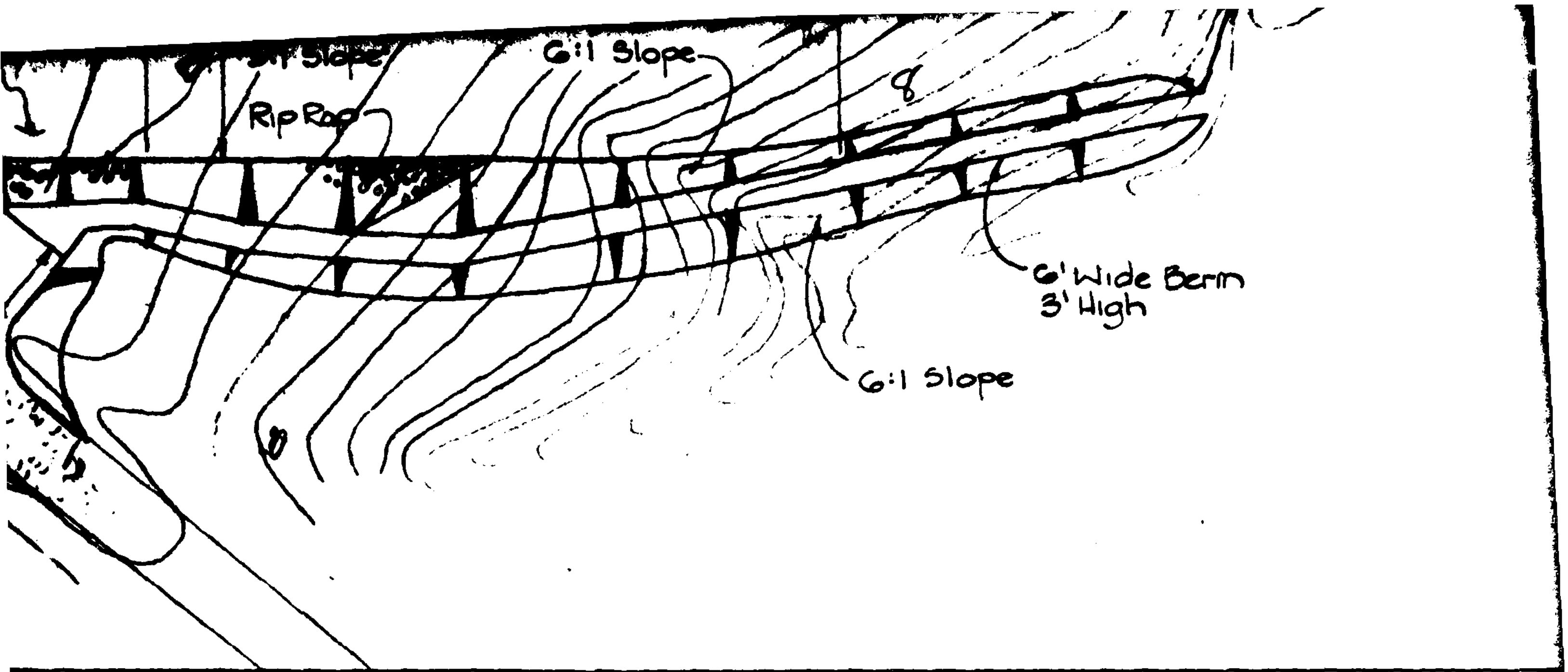
810

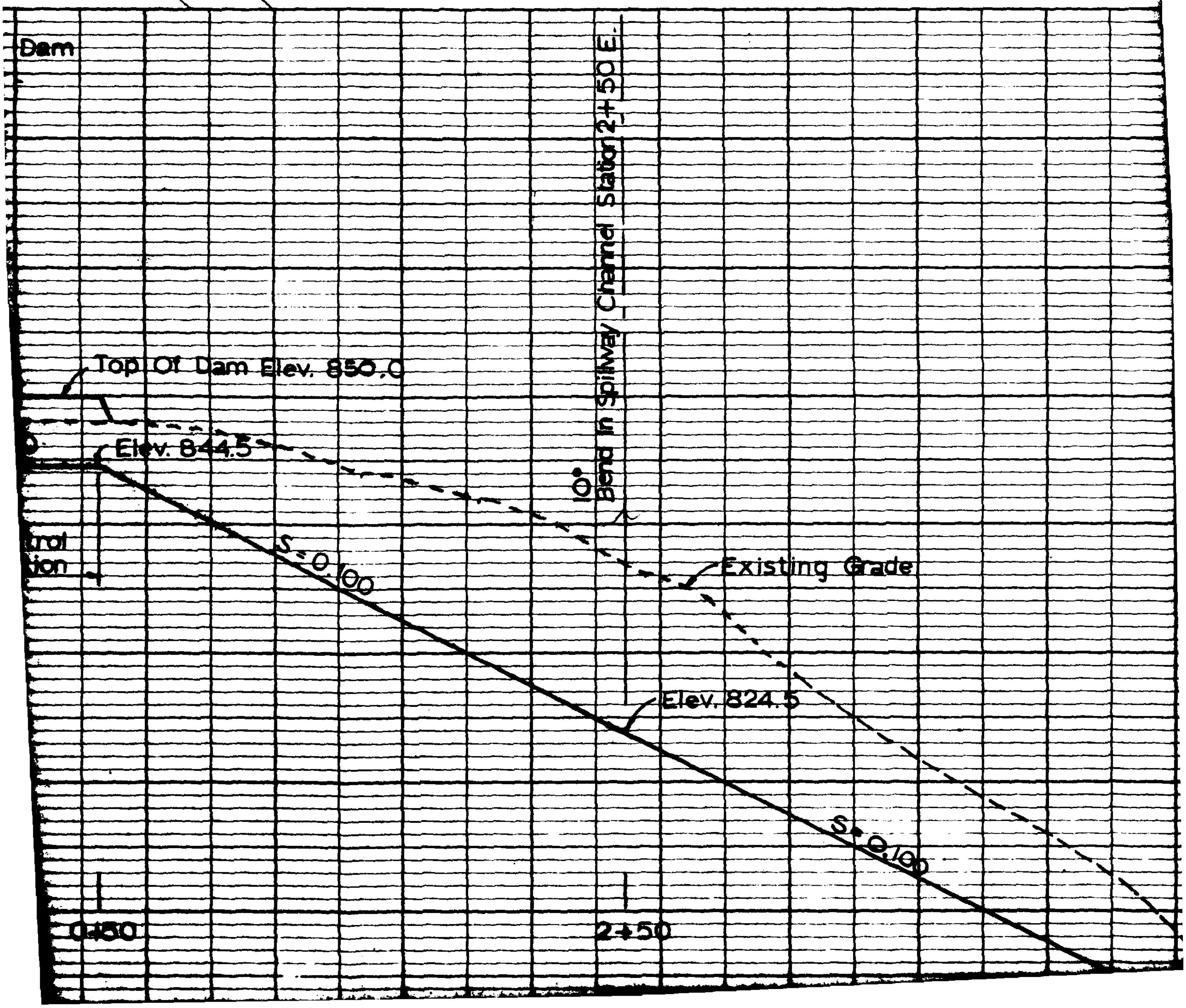
Existing



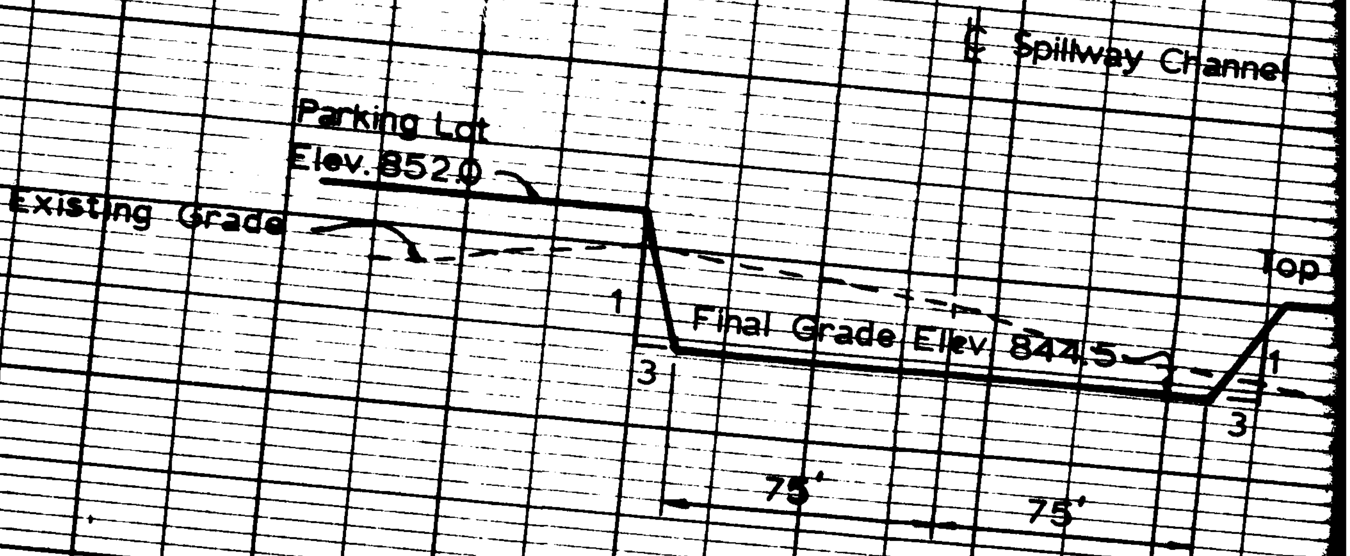
Parking Lot
Elev. 892.0
70'
100'
3:1 Side Slopes
Place 2" Of Gravel Road Rock
B
Dam
25° Bend In Approach Channel
Station 0+30 W.
Elev. 844.5
S=0.00
Elev. 843.0
S=0.0229
Existing Grade
Control Section
West
East
0+00
0+50

PROFILE A-A


3:1 Slope
Rip Rap
6:1 Slope
6' Wide Berm
3' High
6:1 Slope


Dam
Top Of Dam Elev. 850.0
Elev. 844.5
S=0.100
10° Bend In Spillway Channel Station 2+50 E.
Existing Grade
Elev. 824.5
S=0.100
2+50

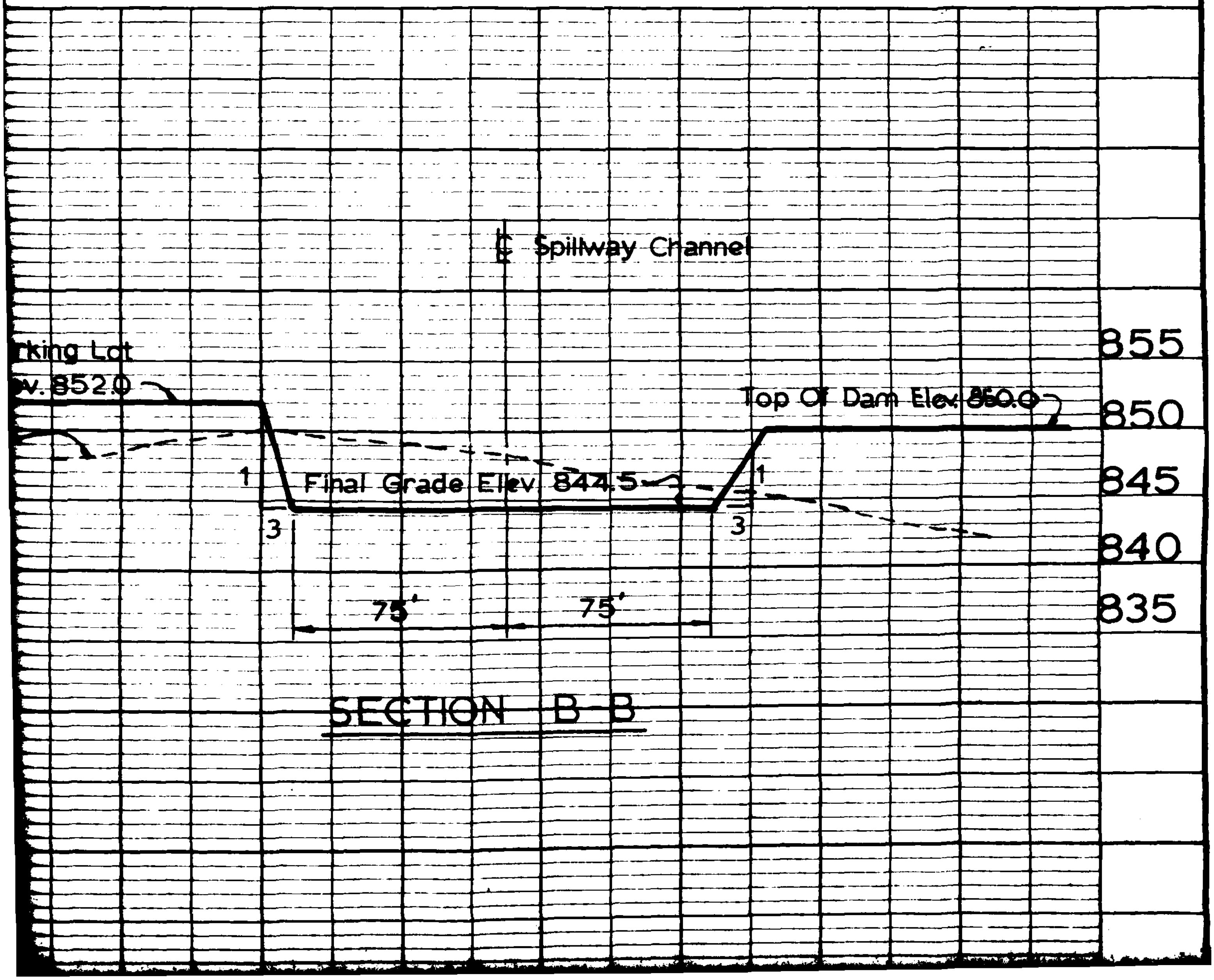
9 (TYPICAL)



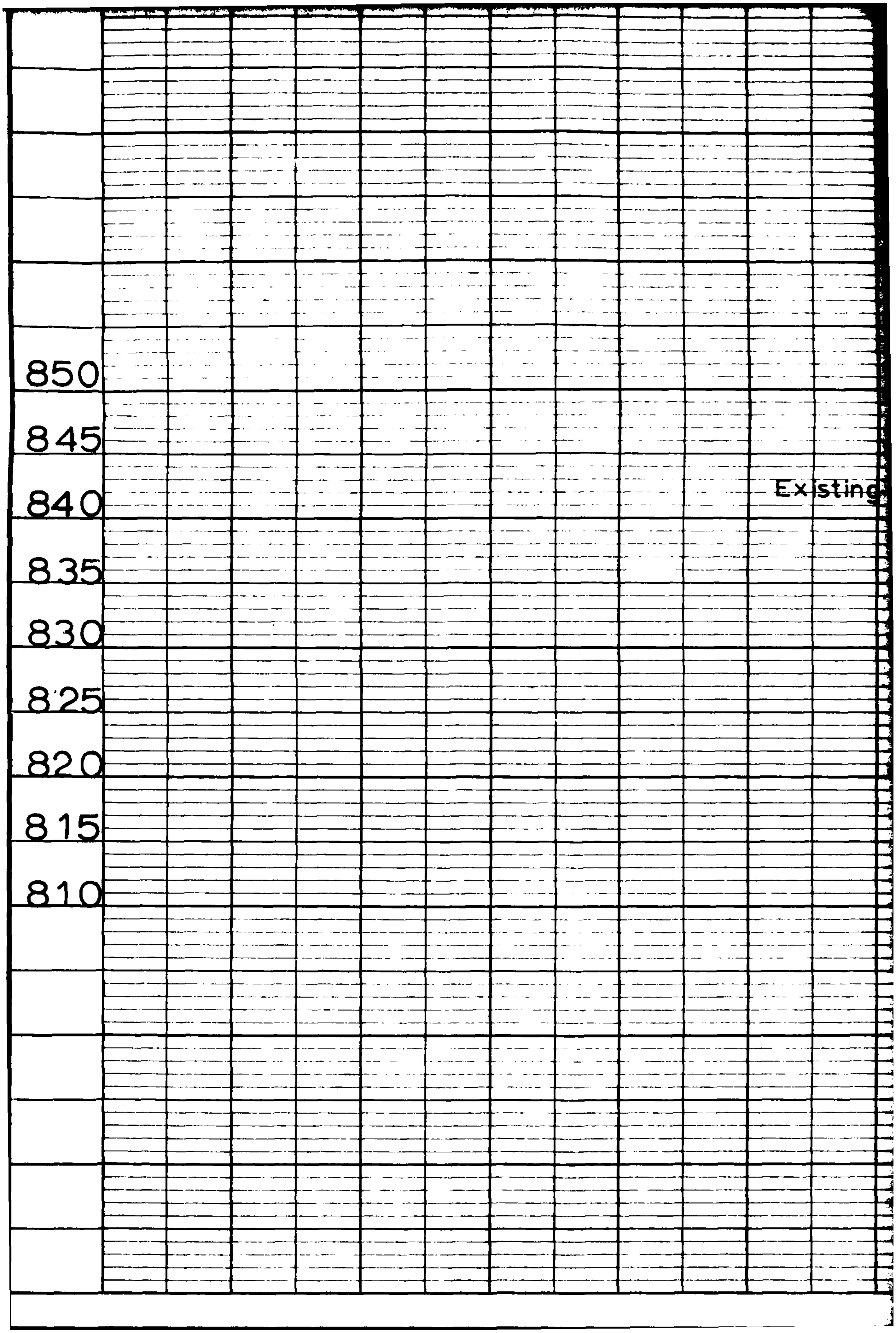
SECTION B B

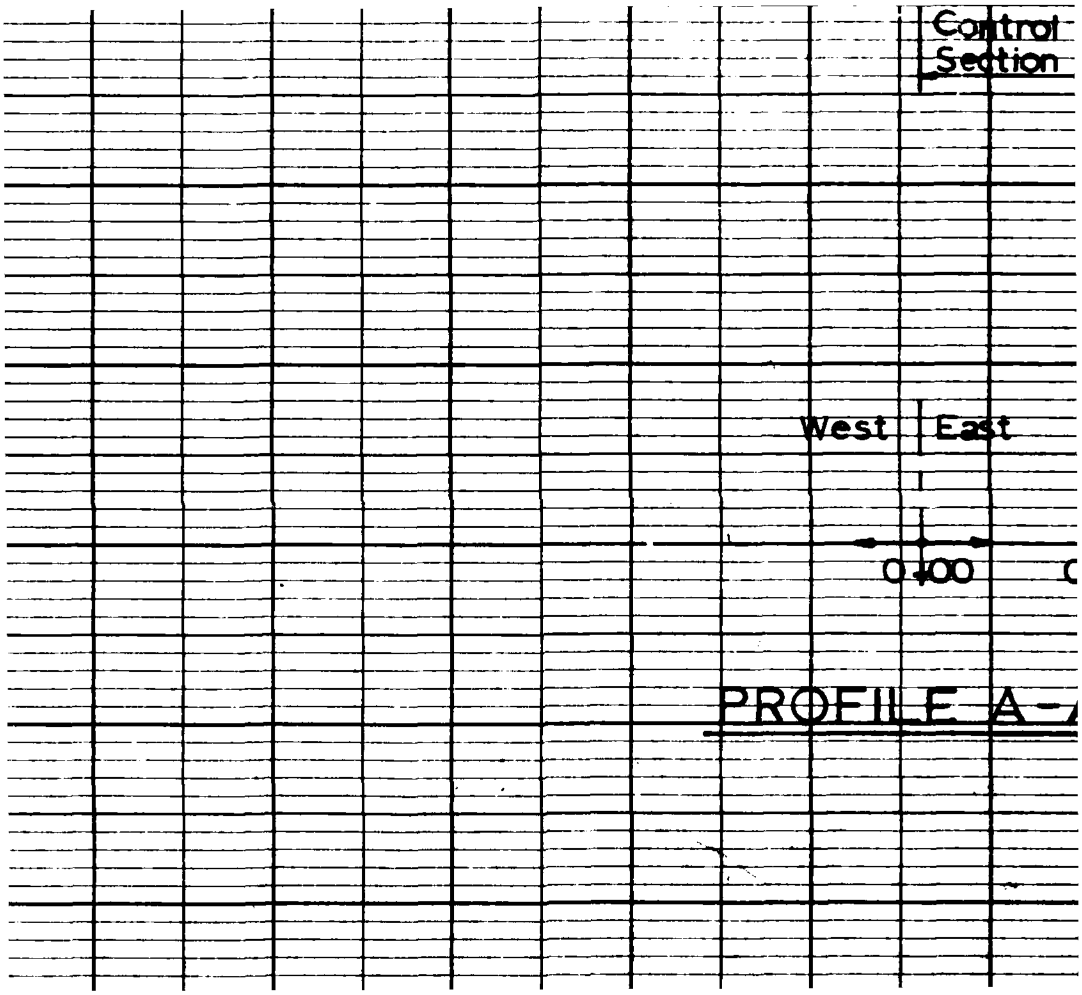
RIP RAP DETAIL
(TYPICAL)

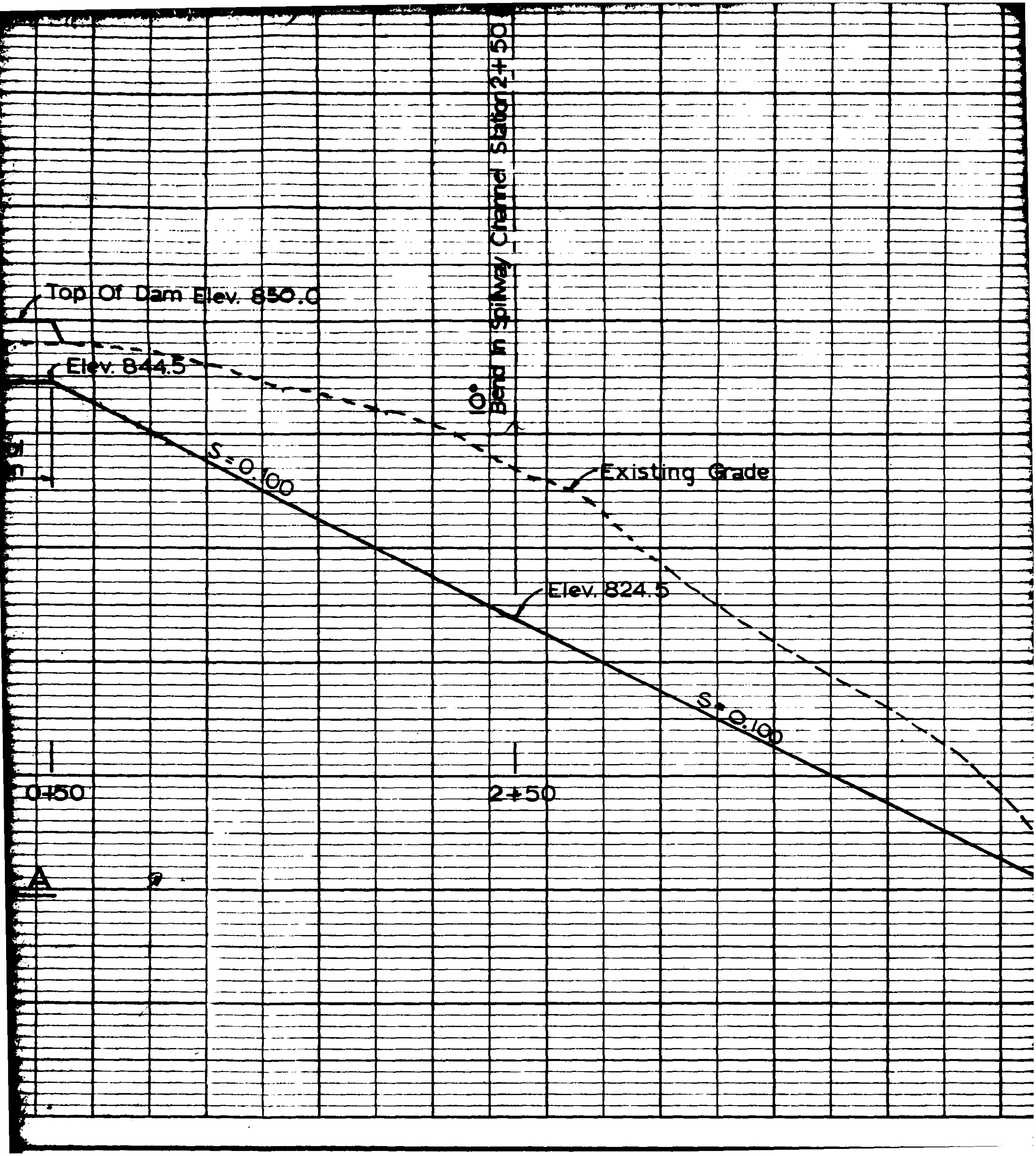
9

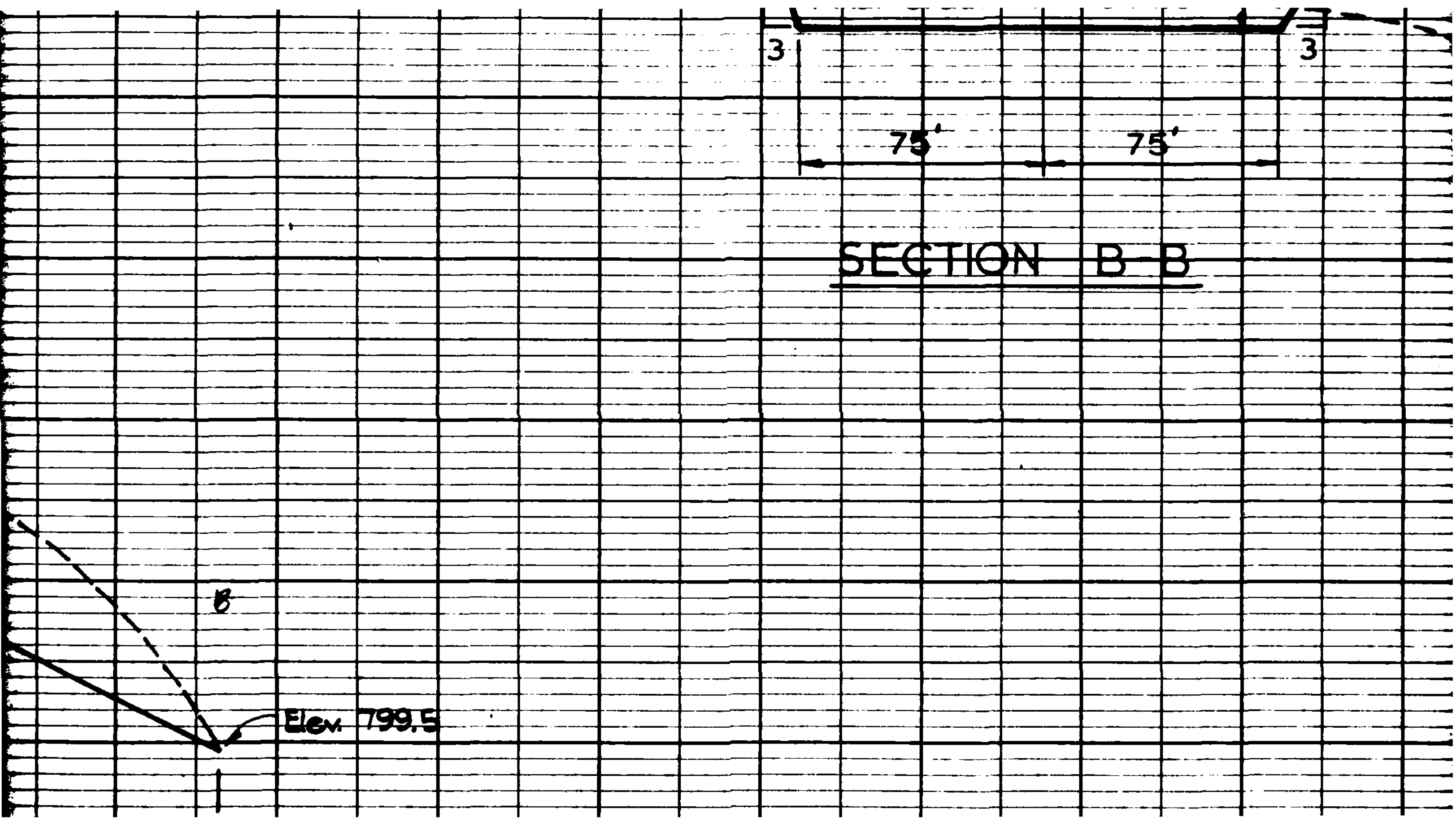


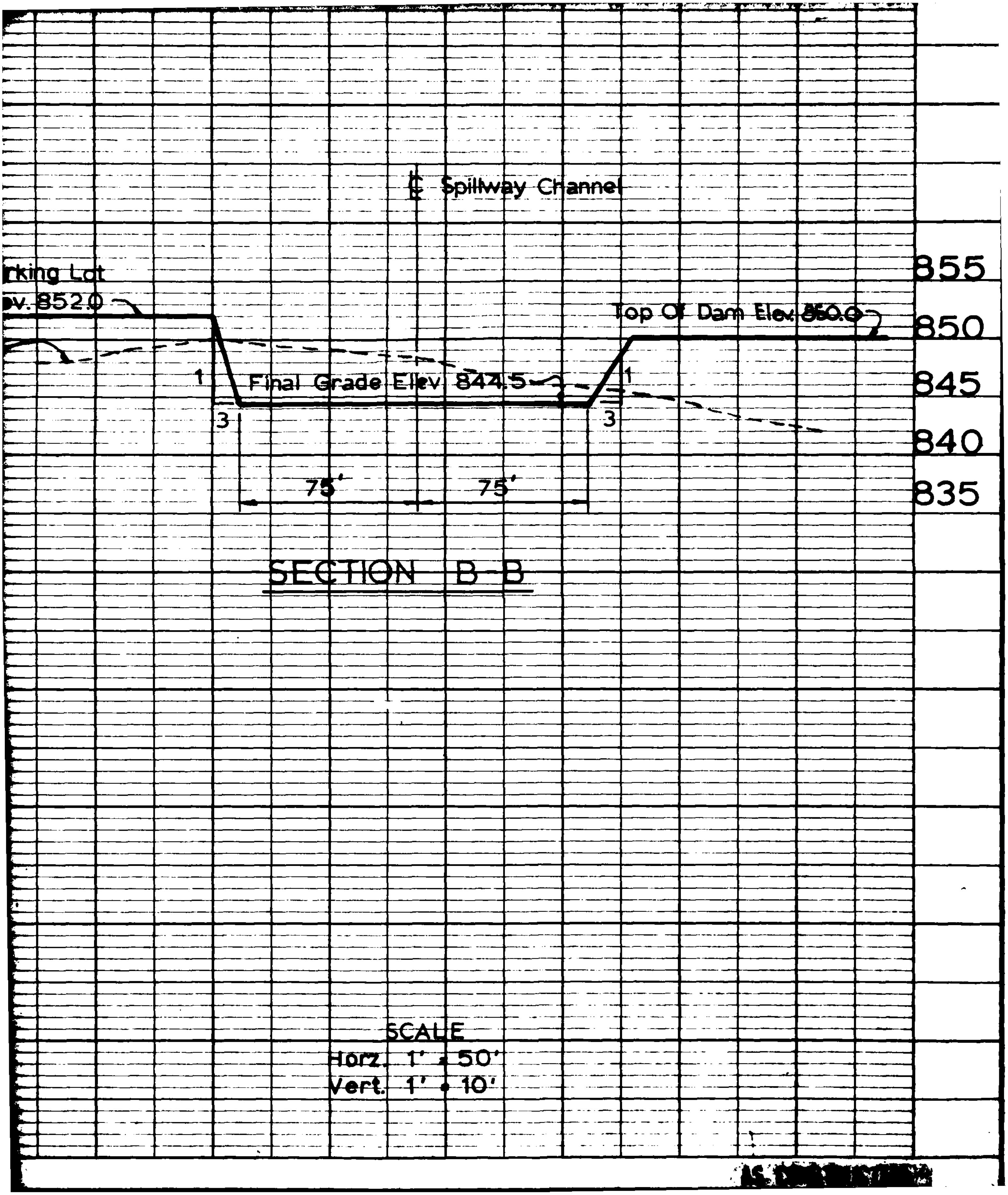
SECTION B-B


850
845
840
835
830
825
820
815
810
Existing

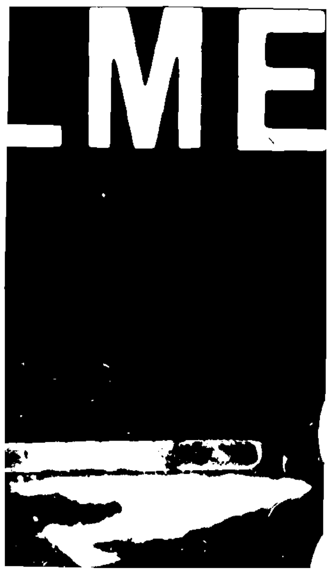

Control
Section
West
East
0+00
PROFILE A-


Top Of Dam Elev. 850.0
Elev. 844.5
10° Bend In Spillway Channel Station 2+50
S=0.100
Existing Grade
Elev. 824.5
S=0.100
0+50
2+50
A


3
3
75'
75'
SECTION B-B
B
Elev. 799.5



Spillway Channel
rking Lot
ev. 852.0
Top Of Dam Elev. 850.0
Final Grade Elev. 844.5
1
3
1
3
75'
75'
855
850
845
840
835
SECTION B-B
SCALE
Horz. 1' = 50'
Vert. 1' = 10'


LME