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WHITE RIVER BASIN

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GRISHAM LAKE DAM

HOWELL COUNTY, MISSOURI

MO 31574

PHASE 1 INSPECTION REPORT NATIONAL DAM SAFETY PROGRAM



**United States Army
Corps of Engineers**

*... Serving the Army
... Serving the Nation*

St. Louis District

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

FOR: STATE OF MISSOURI

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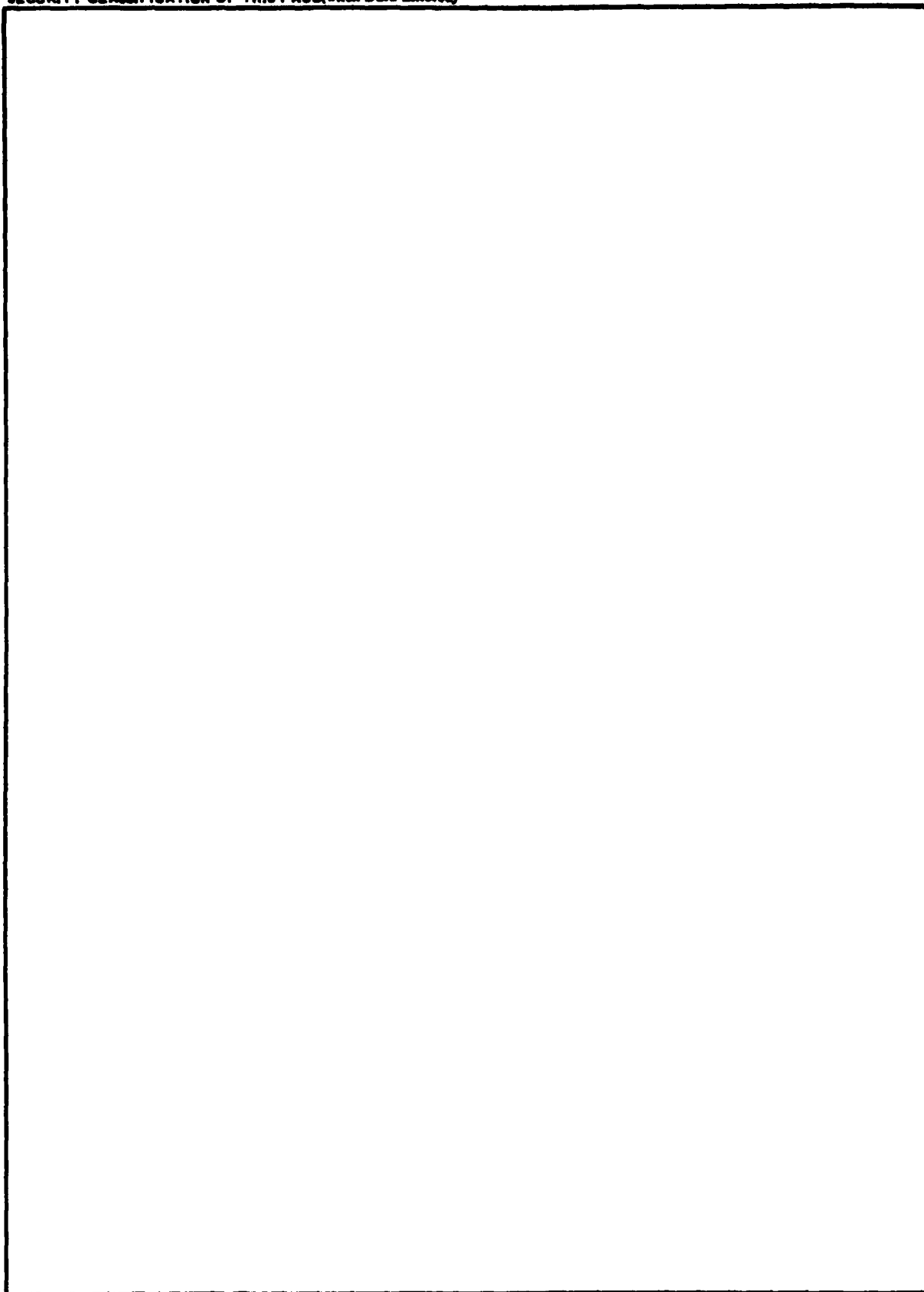
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DEPARTMENT OF THE ARMY
ST. LOUIS DISTRICT CORPS OF ENGINEERS
210 TUCKER BOULEVARD, NORTH
ST. LOUIS, MISSOURI 63101

REPLY TO
ATTENTION OF

SUBJECT: Grisham Lake Dam
Howell County, Missouri
Missouri Inventory No. 31574

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

- a. Dam inspected under this program.
- b. Dam is not performing as a dam as it had in the past.
- c. Should it hold water and function as a dam at some future time, the spillway is seriously inadequate because it will not pass 50 percent of the Probable Maximum Flood without overtopping the dam.

SIGNED

SUBMITTED BY: _____
Chief, Engineering Division

24 DEC 1980

Date

APPROVED BY: _____
Colonel, CE, District Engineer

29 DEC 1980

Date

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WHITE RIVER BASIN

GRISHAM LAKE DAM
HOWELL COUNTY, MISSOURI
MISSOURI INVENTORY NO. 31574

PHASE I INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM

Prepared By

Anderson Engineering, Inc., Springfield, Missouri
Hanson Engineers, Inc., Springfield, Illinois

Under Direction Of

St. Louis District, Corps of Engineers

For

Governor of Missouri

AUGUST, 1980

PHASE I REPORT
NATIONAL DAM SAFETY PROGRAM
SUMMARY

Name of Dam: Grisham Lake Dam
State Located: Missouri
County Located: Howell
Stream: Tributary of Howell Creek
Date of Inspection: June 19, 1980

Grisham Lake Dam was inspected by an interdisciplinary team of engineers from Anderson Engineering, Inc. of Springfield, Missouri and Hanson Engineers, Inc. of Springfield, Illinois. The purpose of this 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 they have been developed with the help of several Federal and State agencies, professional engineering organizations, and private engineers. Based on these guidelines, the St. Louis District, Corps of Engineers has determined that this dam is in the high hazard potential classification, which means that loss of life and appreciable property loss could occur if the dam fails. The estimated damage zone extends approximately three miles downstream of the dam. Located within this zone are 24 [±] dwellings, commerce and residential buildings and Highway 17 (barrier), all in the City of West Plains.

The dam is in the small size classification, since it is less than 40 ft high, and the maximum storage capacity is greater than 50 ac-ft but less than 1,000 ac-ft.

Our inspection and evaluation indicates that the spillway does not meet the criteria set forth in the guidelines for a dam having the above size and hazard potential. The spillway will pass 22 percent of the Probable Maximum Flood without overtopping. 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 guidelines require that a dam of small size with a high downstream hazard potential pass 50 to 100 percent of the PMF. Considering the height of the dam (25 feet), the maximum storage capacity (179 acre-feet) and the small volume of permanent water storage, 50 percent of the PMF has been determined to be the

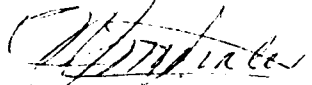
appropriate spillway design flood. The 100-year flood (1 percent probability flood) will not overtop the dam. The 1 percent probability flood is one that has a 1 percent chance of being equaled or exceeded in any given year.

The embankment appears to be in good condition. Deficiencies visually observed by the inspection team were: (1) Few small trees on upstream face of embankment; (2) very dense brush and tree growth on downstream face of embankment; (3) lack of wave protection for upstream face of embankment; (4) minimal elevation difference between the spillway crest and the low point of the dam crest; and (5) lack of a non-erodible spillway control section.

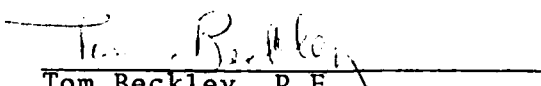
Another deficiency was the lack of seepage and stability analysis records.

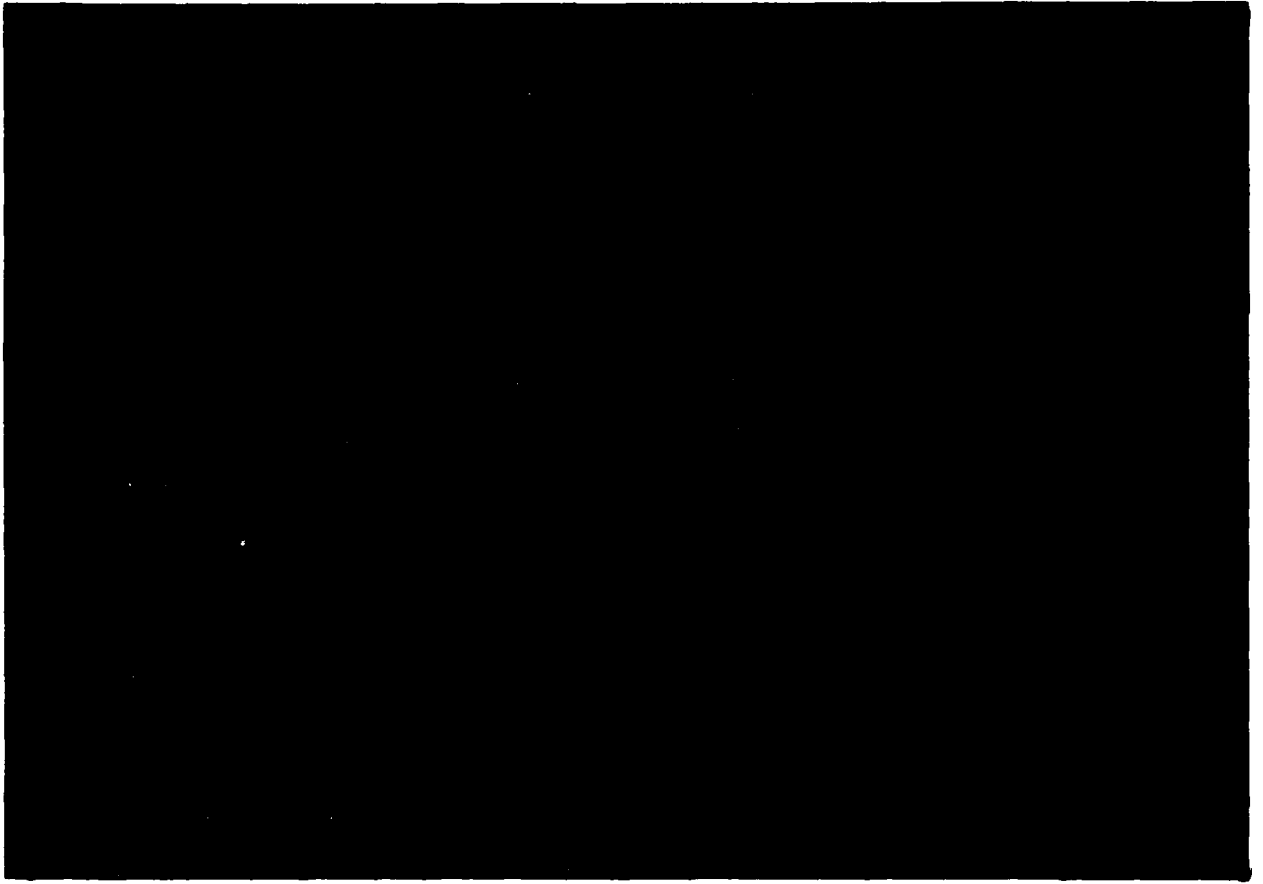
It is recommended that the owners take the necessary action without undue delay to correct the deficiencies reported herein. A detailed discussion of these deficiencies is included in the following report.


Steve Brady, P.E.
Anderson Engineering, Inc.


Nelson Morales, P.E.
Hanson Engineers, Inc.


Dave Daniels, P.E.
Hanson Engineers, Inc.


Tom Beckley, P.E.
Anderson Engineering, Inc.



AERIAL VIEW OF LAKE AND DAM

PHASE 1 INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM
GRISHAM LAKE DAM
MISSOURI INVENTORY NO. 31574

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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 St. Louis District, Corps of Engineers, District Engineer directed that a safety inspection be made of Grisham Lake Dam in Howell County, Missouri.

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 a 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, "Recommended Guidelines for Safety Inspection of Dams, Appendix D." 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:

Grisham Lake Dam is an earth fill structure approximately 25 ft high and 765 ft long at the crest. The appurtenant works consists of an earth cut swale spillway at the north abutment.

Sheet 3 of Appendix A shows a plan, profile, and typical section of the embankment.

B. Location:

The dam is located in the central part of Howell County, Missouri on a tributary of Howell Creek. The dam and lake are within the West Plains, Missouri 15 minute quadrangle sheet (Section 13, T24N, R09W - latitude 36°44.8'; longitude 91°53.8'). Sheet 2 of Appendix A shows the general vicinity.

C. Size Classification:

With an embankment height of 25 ft and a maximum storage capacity of approximately 179 acre-ft, the dam is in the small size category.

D. Hazard Classification:

The St. Louis District, Corps of Engineers has classified this dam as a high hazard dam. The estimated damage zone extends approximately three miles downstream of the dam. Located within this zone are 24 dwellings, commerce and residential buildings and Highway 17 (barrier), all in the City of West Plains. Photograph number 12 of Appendix D shows a portion of the dwellings in the estimated damage zone. The affected features within the damage zone were verified by the inspection team.

E. Ownership:

The dam is owned by Mr. G. E. Grisham. The owner's address is 213 E. Main Street, West Plains, Missouri (Telephone number 417/256-7159).

F. Purpose of Dam:

The dam was constructed primarily for lakeside home development.

G. Design and Construction History:

There are no plans or design calculations available for this dam. The dam was constructed in 1964 by Mr. E. T. Stokes of West Plains, Missouri. Mr. Stokes, an equipment dealer, constructed the dam with his own equipment and personnel. The lake was to have approximately 13 acres of surface area when full.

A core trench 10 feet wide and 6 to 10 feet deep was cut into "good clay". The material for the core was obtained from the lake area. All of the borrow for the embankment was obtained from the lake area. Mr. Stokes indicated that the material removed from the borrow site consisted of "sand and creek gravel" over clay. The core was constructed from the selected clay obtained from the borrow area.

Mr. Stokes stated that after completion of the dam, the reservoir was filling at a normal expected rate. Approximately one year after the dam was completed, a "heavy" rainfall raised the lake level to about 2 feet below the spillway. Three days after the rainfall, Mr. Stokes stated that he heard a loud noise from the vicinity of the dam. When he arrived at the dam site, he stated that he heard the sound of "rushing water". He did not observe any flow of surface water downstream of the dam. Seven hours after the loud noise, Mr. Stokes stated that the reservoir area was dry.

During the period of time that the water was draining, no apparent surface flow was observed downstream of the dam.

An area near the south abutment, believed to be the failure area, was excavated to a depth of 15 to 20 feet. A large crevice

was detected in the bedrock. After cleaning out the crevice area, 40 cubic yards of concrete were placed in and around the crevice. Clays obtained from the lake bed were then placed and compacted in the excavated area.

Mr. Stokes indicated that subsequent to the repair work, the dam has never held water. No additional repairs or modifications have been made to the dam.

H. Normal Operating Procedures:

All flows will be passed by the uncontrolled earth cut swale spillway at the north abutment. Information obtained from Mr. Stokes and Mr. Grisham indicates that the dam has never been overtopped.

1.3 PERTINENT DATA:

Pertinent data about the dam, appurtenant works, and reservoir are presented in the following paragraphs. Sheet 3 of Appendix A presents a plan, profile, and typical section of the embankment.

A. Drainage Area:

The drainage area for this dam, as obtained from the U.S.G.S. quad sheet, is approximately 168 acres.

B. Discharge at Dam Site:

- (1) All discharge at the dam site is through an uncontrolled spillway.
- (2) Estimated Total Spillway Capacity at Maximum Pool (Top of Dam - El. 1070): 5 cfs
- (3) Estimated Capacity of Principal Spillway: 5 cfs
- (4) Estimated Experience Maximum Flood at Dam Site: Spillway section has not been used; maximum reservoir elevation approximately 1068.0 feet, MSL
- (5) Diversion Tunnel Low Pool Outlet at Pool Elevation: Not Applicable
- (6) Diversion Tunnel Outlet at Pool Elevation: Not Applicable
- (7) Gated Spillway Capacity at Pool Elevation: Not Applicable
- (8) Gated Spillway Capacity at Maximum Pool Elevation: Not Applicable

C. Elevations:

All elevations are consistent with an assumed mean sea level elevation of 1074.00 for nail set in west side of 15 inch oak tree

10 feet right of Station 0 + 20, (estimated from quadrangle map).

- (1) Top of Dam: 1070.0 feet, MSL (Low Point)
- (2) Principal Spillway Crest: 1069.8 feet, MSL
- (3) Emergency Spillway Crest: Not Applicable
- (4) Principal Outlet Pipe Invert: Not Applicable
- (5) Streambed at Centerline of Dam: 1048.0 feet, MSL
- (6) Pool on Date of Inspection: 1048.4 feet, MSL
- (7) Apparent High Water Mark: 1057.0 feet, MSL
- (8) Maximum Tailwater: Not Applicable
- (9) Upstream Portal Invert Diversion Tunnel: Not Applicable
- (10) Downstream Portal Invert Diversion Tunnel: Not Applicable

D. Reservoir Lengths:

- (1) At Top of Dam: 1,560 feet
- (2) At Emergency Spillway Crest: Not Applicable
- (3) At Principal Spillway Crest: 1,550 feet

E. Storage Capacities:

- (1) At Top of Dam: 179 acre-feet
- (2) At Emergency Spillway Crest: Not Applicable
- (3) At Principal Spillway Crest: 175 acre-feet

F. Reservoir Surface Areas:

- (1) At Top of Dam: 19.3 acres
- (2) At Emergency Spillway Crest: Not Applicable
- (3) At Principal Spillway Crest: 19 acres

G. Dam:

- (1) Type: Rolled Earth
- (2) Length at Crest: 765 feet
- (3) Height: 25 feet
- (4) Top Width: 11 feet

- (5) Side Slopes: Upstream varies from 1V on 1.7H to 1V on 2.3H;
Downstream varies from 1V on 3.8H to 1V on 5.8H
- (6) Zoning: Apparently Homogeneous
- (7) Impervious Core: None
- (8) Cutoff: Key Trench to Clay
- (9) Grout Curtain: None

H. Diversion and Regulating Tunnel:

- (1) Type: Not Applicable
- (2) Length: Not Applicable
- (3) Closure: Not Applicable
- (4) Access: Not Applicable
- (5) Regulating Facilities: Not Applicable

I. Spillway:

I.1 Principal Spillway:

- (1) Location: North Abutment
- (2) Type: Earth Cut Swale
- (3) Upstream Channel: Earth Cut, grass covered channel
- (4) Downstream Channel: Brush and wood covered earth
channel with moderate side slopes

I.2 Emergency Spillway:

- (1) Location: Not Applicable
- (2) Type: Not Applicable

J. Regulating Outlets:

There are no regulating facilities associated with this
dam.

SECTION 2 - ENGINEERING DATA

2.1 DESIGN:

No engineering data exist for this dam. No documentations of construction records were available. There are no documented maintenance data.

A. Surveys:

No information regarding pre-construction surveys was obtained. Sheet 3 of Appendix A presents a plan, profile, and cross section of the dam from survey data obtained during the site inspection. A nail set in the west side of a 15 inch oak tree at Station 0 + 20, 10 feet right of centerline, was used as a site datum of assumed elevation 1074.0.

B. Geology and Subsurface Materials:

The topography of the site is rolling to hilly. This area is in the Ozarks geologic region of the state. Generally the soils around the dam consist of thick, well drained, residual cherty, silty clays above bedrock. Shallow auger probes in the embankment showed it to consist of cherty silt clays that were yellowish-brown in color. The soil portion appeared to be in the Unified Soil Group of CL. The underlying rock is of the Canadian series of the Ordovician system. The rocks of this series are principally arenaceous, and cherty dolomite and sandstone. The Jefferson City dolomite formation is believed to be the parent bedrock in the lake basin. The thickness of the Jefferson City formation ranges from 125 to 350 feet.

The publication "Caves of Missouri" indicates that two known caves exist in Howell County. Both caves are several miles from the site. The quad sheet for this area shows the presence of sinks in all directions from the lake basin and characterizes the residual soils.

The "Geologic Map of Missouri" indicates the nearest fault zone to be approximately 20 miles northwest of the site. The Missouri Geologic Survey has indicated that the faults in this area are generally considered to be inactive.

C. Foundation and Embankment Design:

No foundation and embankment design information was available. Seepage and stability analyses apparently were not performed as required in the guidelines. Mr. Stokes indicated that a clay core approximately 10 feet wide and 6 to 8 feet deep was incorporated into the dam and keys into a clay material. He also stated that the material for the embankment was obtained from the lake area.

D. Hydrology and Hydraulics:

No hydrologic or hydraulic design computations for this dam were available. Based on a field check of spillway dimensions, embankment elevations, and a check of the drainage area on U.S.G.S. quad sheets, hydrologic analyses using U. S. Army Corps of Engineers guidelines were performed and appear in Appendix C, Sheets 1 to 9.

E. Structures:

There are no structures associated with this dam.

2.2 CONSTRUCTION:

No construction inspection data have been obtained.

2.3 OPERATION:

Normal flows would be passed by an uncontrolled earth cut spillway located in the north abutment. No operating facilities exist.

2.4 EVALUATION:

A. Availability:

No engineering data, seepage or stability analyses, or construction test data were available.

B. Adequacy:

The engineering data available were inadequate to make a detailed assessment of the design, construction, and operation of this structure. Seepage and 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 stability analyses should be performed for appropriate loading conditions and made a matter of record.

C. Validity:

To our knowledge, no valid engineering data on the design or construction of the embankment are available.

SECTION 3 - VISUAL INSPECTON

3.1 FINDINGS:

A. General:

The field inspection was made on June 19, 1980. The inspection team consisted of personnel from Anderson Engineering, Inc. of Springfield, Missouri and Hanson Engineers, Inc. of Springfield, Illinois. The team members were:

Steve Brady - Anderson Engineering, Inc. (Civil Engineer)
Tom Beckley - Anderson Engineering, Inc. (Civil Engineer)
Dave Daniels - Hanson Engineers, Inc. (Geotechnical Engineer)
Nelson Morales - Hanson Engineers, Inc. (Hydraulic Engineer)

Photographs of the dam, appurtenant structures, reservoir, and downstream features are presented in Appendix D.

B. Dam:

The embankment appears to be in good condition. The upstream face had some scattered trees. There was no sloughing or significant erosion. There was no wave protection such as riprap for the upstream face.

The horizontal alignment was good with a gentle curve. The crest of the dam had a sag curve extending the full length of the embankment. The elevation of the crest varied from 1070.0 at the center to 1071.6 at the north end of the embankment. The elevation at the center of the embankment was 0.2 feet higher than the spillway channel.

No surface cracking or unusual movement of the embankment was obvious.

The downstream face of the embankment was densely covered with brush and trees. A thorough inspection of the downstream face was difficult due to the heavy growth. No animal burrows were observed. No sloughing or serious erosion was evident on the embankment or at the embankment-abutment contacts. The embankment and downstream valley were investigated for seepage, but none was found.

Auger probes in the crest of the embankment indicated a red-brown clayey silt with chert fragments (ML-CL).

C. Appurtenant Structures:

C.1 Principal Spillway:

The spillway is an earth cut swale in the north abutment. No significant erosion was observed in the channel. The approach

area had a scattering of brush and small trees. There is no permanent control section at the crest of the spillway. The spillway has apparently never been used. The elevation of the channel is 0.2 foot lower than the low point of the embankment.

C.2 Emergency Spillway:

There is no emergency spillway associated with this dam.

D. Reservoir:

The watershed is wooded with some pastureland. The slopes into the reservoir area are moderate. The lake was only about 0.5 acre in plan area the day of inspection. According to the builder the lake has maintained this level for all but the first year after construction. The crevice that was repaired, or other apparent crevices does not permit permanent water storage.

E. Downstream Channel:

The downstream channel is wooded for several hundred feet starting about 150 feet downstream of the toe. Some brush cover was observed at the inlet to the channel. The downstream channel slopes were light to moderate.

3.2 EVALUATION:

Trees and brush on the dam constitute a potential seepage hazard and encourage animal burrowing. There is no wave protection provided for the upstream face of the embankment. A non-erodible control section is not provided for the spillway; therefore, progressive erosion could lower the elevation of the spillway, and thus lower the normal pool elevation of the reservoir should the dam ever attain normal pool elevation. The tree and brush growth in the spillway channel could restrict flood flows.

The difference of 0.2 feet between the spillway and the low point of the crest of dam and the small capacity of the spillway indicates that overtopping is eminent should the pool level reach the spillway elevation.

As the dam has not provided permanent water storage for 15 of the 16 years of its existence, the above deficiencies are not as critical from an urgency standpoint. However, the correction of these deficiencies represents standard practice for the construction and maintenance of dams. The occurrence of a large flood could result in rapid fill up of the lake and substantially increase the risk associated with the above deficiencies.

SECTION 4 - OPERATIONAL PROCEDURES

4.1 PROCEDURES:

There are no operating facilities associated with this dam. The pool level is normally controlled by rainfall, runoff, evaporation and apparent leakage from the reservoir. Should the lake ever fill up, an additional pool level control would be the capacity of the uncontrolled spillway.

4.2 MAINTENANCE OF DAM:

The presence of tree and brush growth on the embankment indicates that minimal maintenance is done.

4.3 MAINTENANCE OF OPERATING FACILITIES:

There are no operating facilities for this dam.

4.4 DESCRIPTION OF ANY WARNING SYSTEM IN EFFECT:

The inspection team is unaware of any existing warning system for this dam.

4.5 EVALUATION:

The vegetation on the dam, lack of riprap, and a non-erodible spillway control section and the minute elevation difference between the low point of the crest of the dam and the spillway crest are deficiencies which could become serious should the dam provide permanent water storage.

SECTION 5 - HYDRAULIC/HYDROLOGIC

5.1 EVALUATION OF FEATURES:

A. Design Data:

No hydrologic or hydraulic design computations for this dam were available.

B. Experience Data:

No recorded rainfall, runoff, discharge or reservoir stage data were available for this lake and watershed. The builder and owner indicated that the spillway has never been used. Information from the builder indicates that the highest water level in the lake was approximately 2 feet below the top of dam elevation 1068. The apparent high water mark observed by the inspection team was elevation 1056.4. Our hydrologic and hydraulic analyses using U. S. Army Corps of Engineers guidelines appear in Appendix C.

C. Visual Observations:

The approach area to the spillway has a few scattered trees and some brush. There is no non-erodible spillway control section. The spillway outlet channel is diverted away from the embankment, and spillway releases would not be expected to endanger the dam. The small difference in elevation (0.2 feet) of the spillway crest and the low point of the crest of the dam creates a potential overtopping should the dam ever provide permanent water storage.

D. Overtopping Potential:

The hydraulic and hydrologic analyses (using the U. S. Army Corps of Engineers guidelines and the HEC-1 computer program) were based on: (1) a field survey of spillway dimensions and embankment elevations; and (2) an estimate of the reservoir storage and the pool and drainage areas from the West Plains, Missouri 15 Minute U.S.G.S. quad sheet.

Based on the hydrologic and hydraulic analysis presented in Appendix C, the spillway will pass 22 percent of the Probable Maximum Flood. 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 recommended guidelines from the Department of the Army, Office of the Chief of Engineers, require that this structure (small size with high downstream hazard potential) pass 50 percent to 100 percent of the PMF, without overtopping. Considering the height of dam (25 feet), the maximum

storage capacity (179 acre-feet) and the small volume of permanent water storage, 50 percent of the PMF has been determined to be the appropriate spillway design flood. The 1 percent probability flood will be storage in the lake without spillway spilling or overtopping of the dam.

Application of the probable maximum precipitation (PMP), minus losses, resulted in flood hydrograph peak inflow of 4,020 cfs. For 50 percent of a PMF, the peak inflow was 2,010 cfs.

The routing of the PMF through the spillway and dam indicates that the dam will be overtopped by 1.9 feet at elevation 1071.9. The duration of the overtopping will be 18.5 hours, and the maximum outflow will be 3,405 cfs. The maximum discharge capacity of the spillway is 5 cfs. The routing of 50 percent of the PMF indicates that the dam will be overtopped by 1.3 feet at elevation 1071.3. The maximum outflow will be 1,584 cfs, and the duration of overtopping will be 12.0 hours. Overtopping of an earthen embankment could cause serious erosion and could possibly lead to failure of the structure.

SECTION 6 - STRUCTURAL STABILITY

6.1 EVALUATION OF STRUCTURAL STABILITY:

A. Visual Observations:

Observed features which could adversely affect the structural stability of this dam are discussed in Sections 3.1B and 3.2.

B. Design and Construction Data:

No design and construction data for the foundations and embankment were available.

Seepage and stability analyses comparable to the requirements of the guidelines were not available, which constitutes a deficiency which should be rectified.

C. Operating Records:

No operating records have been obtained.

D. Post-Construction Changes:

The only post-construction changes that have been done to the dam are the reported repair to the lake bed by excavating and concreting the apparent failed area.

E. Seismic Stability:

The structure is located in seismic zone 1. An earthquake of this magnitude would not generally be expected to cause severe structural damage to a well constructed earth dam of this size.

SECTION 7 - ASSESSMENT/REMEDIAL MEASURES

7.1 DAM ASSESSMENT:

This Phase I inspection and evaluation should not be considered as being comprehensive since the scope of work contracted for is far less detailed than would be required for an in-depth evaluation of dams. Latent deficiencies, which might be detected by a totally comprehensive investigation, could exist.

A. Safety:

The embankment is generally in good condition. Several items were noted during the visual inspection which should be investigated further, corrected or controlled. These items are: (1) few small trees on the upstream face of embankment; (2) very dense brush and trees on downstream face of embankment; (3) lack of wave protection for upstream face of embankment; and (4) minimal elevation difference between the spillway crest and the low point of the dam crest.

Another deficiency was the lack of seepage and stability analyses records.

The dam will be overtopped by flows in excess of 22 percent of the Probable Maximum Flood. Overtopping of an earthen embankment could cause serious erosion and could possibly lead to failure of the structure.

B. Adequacy of Information:

The conclusions in this report were based on the performance history as related by others, and visual observation of external conditions. The inspection team considers that these data are sufficient to support the conclusions herein. Seepage and stability analyses comparable to the "Recommended Guidelines for Safety Inspection of Dams" were not available, which is considered a deficiency.

C. Urgency:

The remedial measures recommended in paragraph 7.2 should be accomplished in the near future. If the deficiencies listed in paragraph A are not corrected, and if good maintenance is not provided, the embankment condition will continue to deteriorate and possibly could become serious in the future. The items recommended in paragraph 7.2A should be pursued without undue delay.

D. Necessity for Additional Inspection:

Based on the result of the Phase I inspection, no additional inspection is recommended.

E. Seismic Stability:

The structure is located in seismic zone 1. An earthquake of this magnitude would not generally be expected to cause severe structural damage to a well constructed earth dam of this size.

7.2 REMEDIAL MEASURES:

The following remedial measures and maintenance procedures are recommended. All remedial measures should be performed under the guidance of a professional engineer experienced in the design and construction of dams.

A. Alternatives:

- (1) Spillway size and/or height of dam should be increased to pass 50 percent of the PMF. In either case, the spillway should be protected to prevent erosion.

B. O & M Procedures:

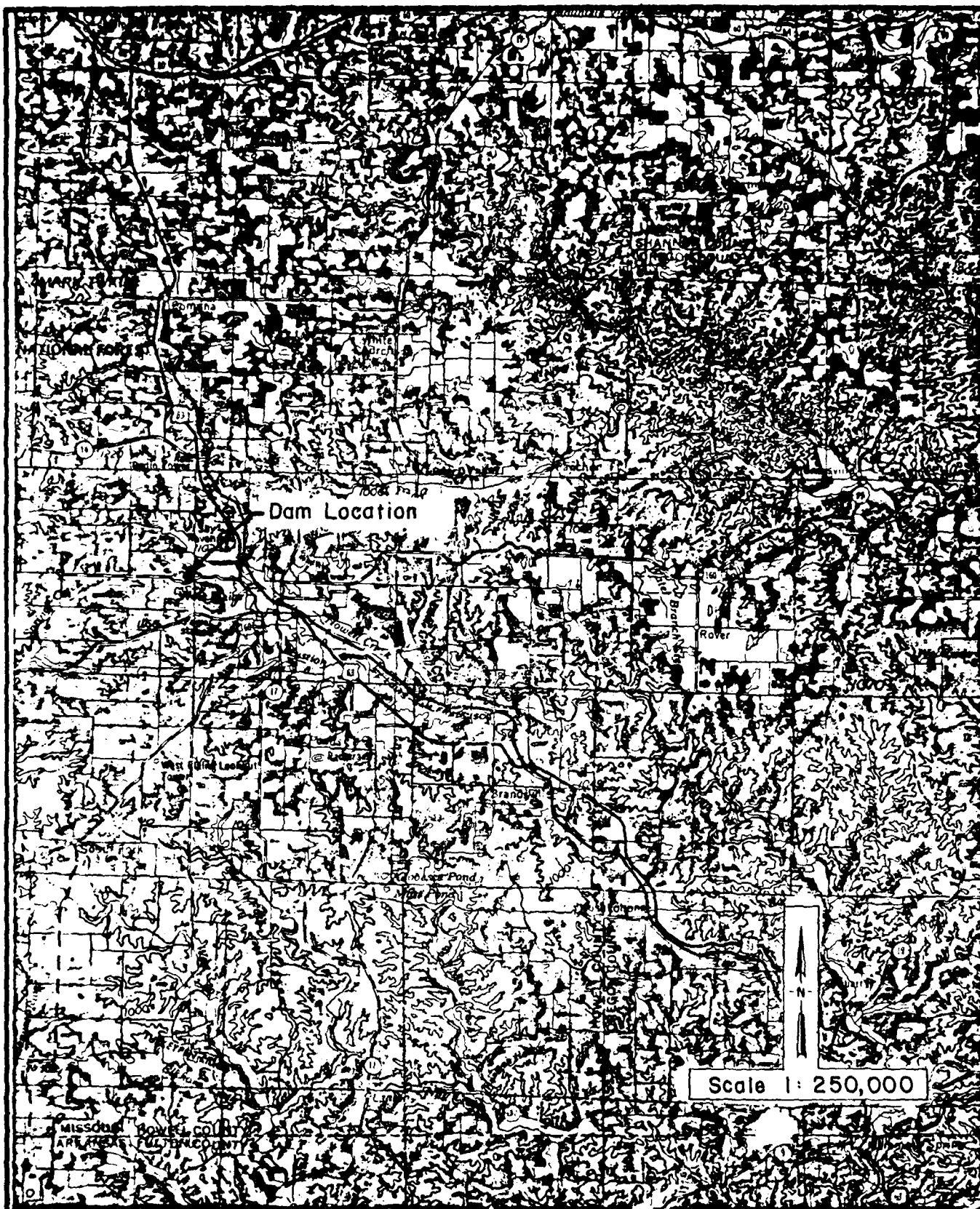
- (1) Seepage and stability analyses comparable to the requirements of the recommended guidelines should be performed by an engineer experienced in the construction of dams.
- (2) A non-erodible spillway control section should be provided so that progressive erosion of the spillway would not lower the normal pool of the reservoir.
- (3) The vertical alignment of the crest of the dam should be improved to provide a greater elevation difference between the spillway and dam crest to limit the overtopping potential.
- (4) Wave protection should be provided for the upstream face of the dam.
- (5) Brush and tree growth should be removed from the embankment and the spillway channel. This should be done under the guidance of a professional engineer experienced in the design and construction of dams. Indiscriminate clearing methods could jeopardize the safety of the dam.
- (6) A detailed inspection of the dam should be made periodically by an engineer experienced in the design and construction of dams.

APPENDIX A

Dam Location and Plans



LOCATION MAP



VICINITY MAP

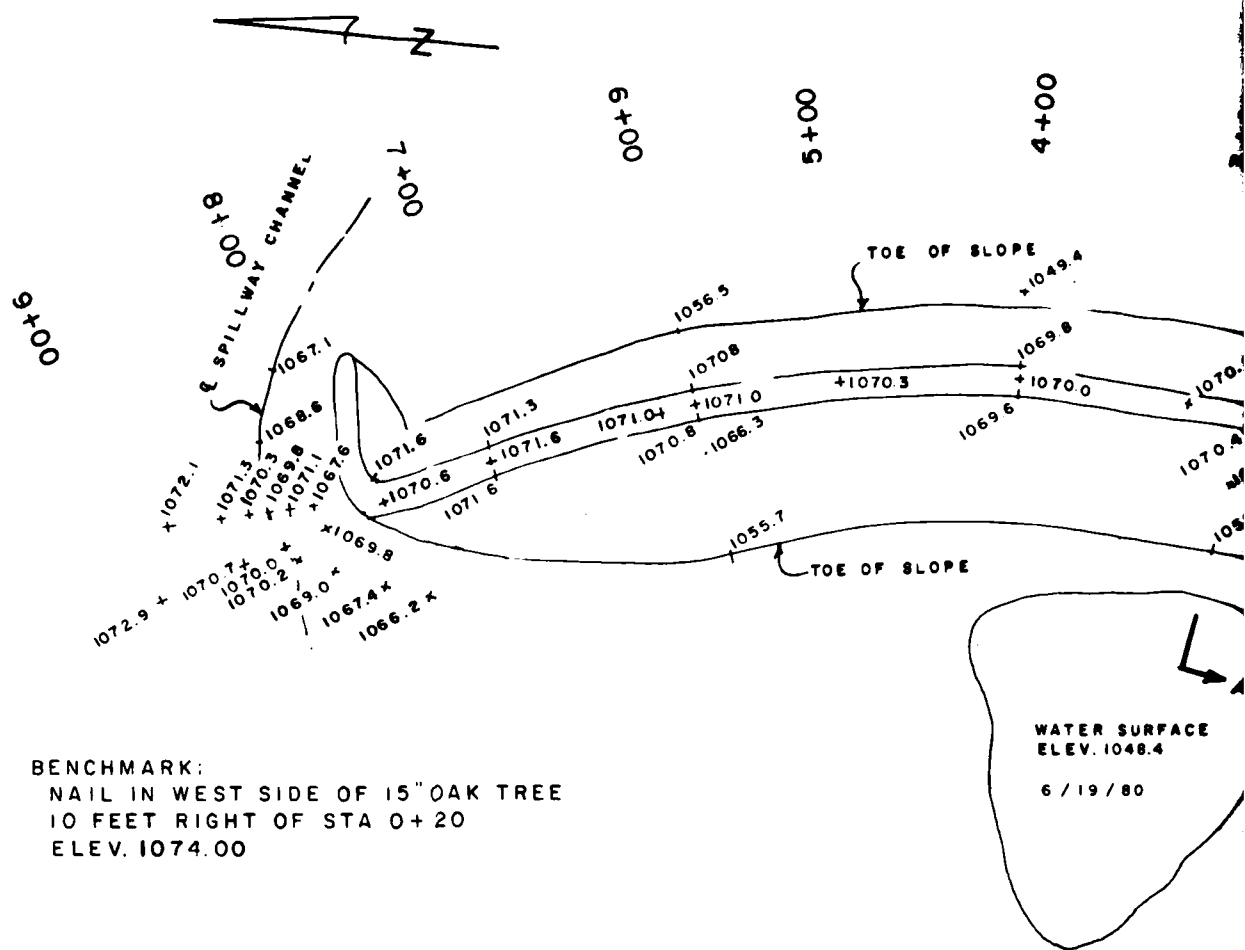


HANSON
ENGINEERS
INCORPORATED

SPRINGFIELD, IL • PEORIA, IL • ROCKFORD, IL

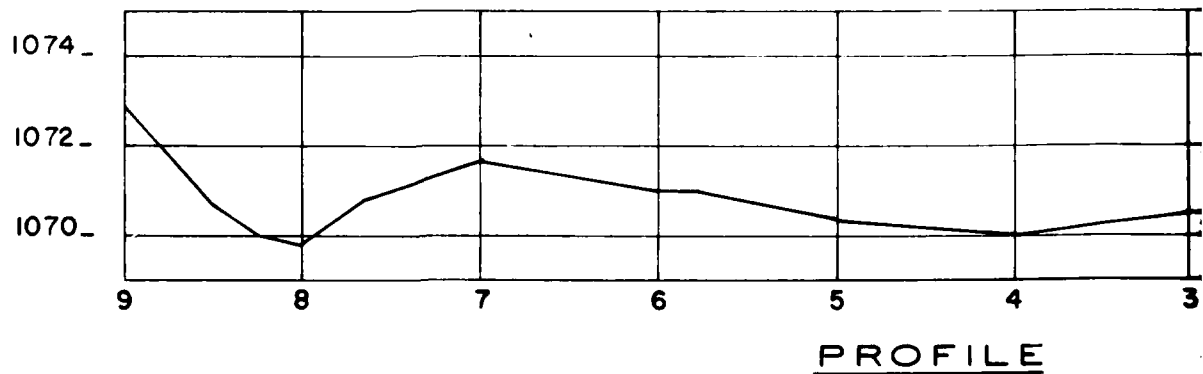
Grisham Lake Dam
Howell County, Missouri
Mo. I.D. No. 31574

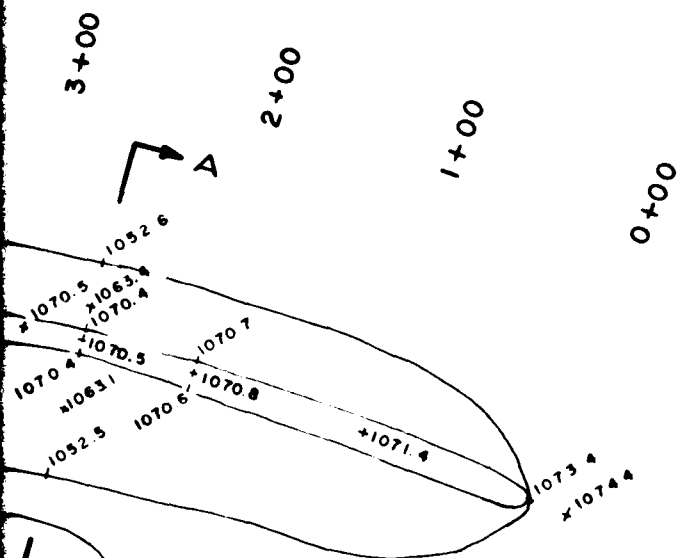
Sheet 2, Appendix A



BENCHMARK:
NAIL IN WEST SIDE OF 15" OAK TREE
10 FEET RIGHT OF STA 0+20
ELEV. 1074.00

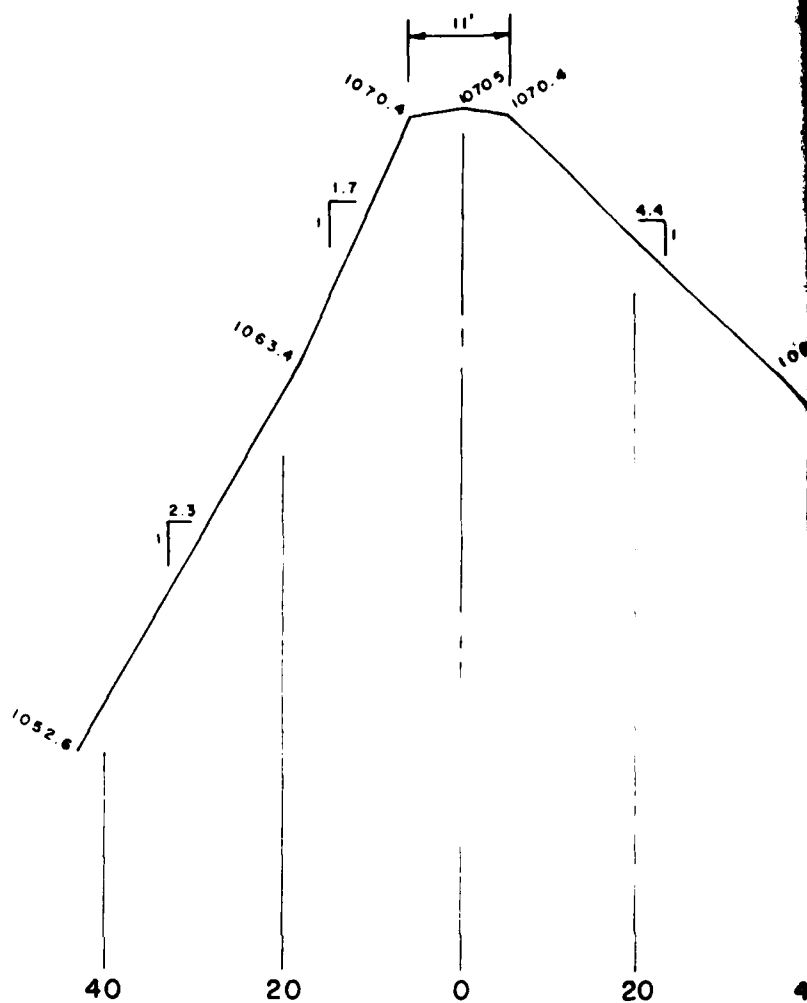
PLAN VIEW
SCALE 1" = 100'



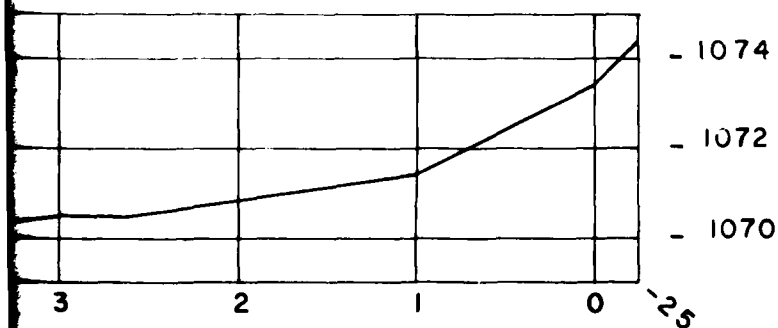


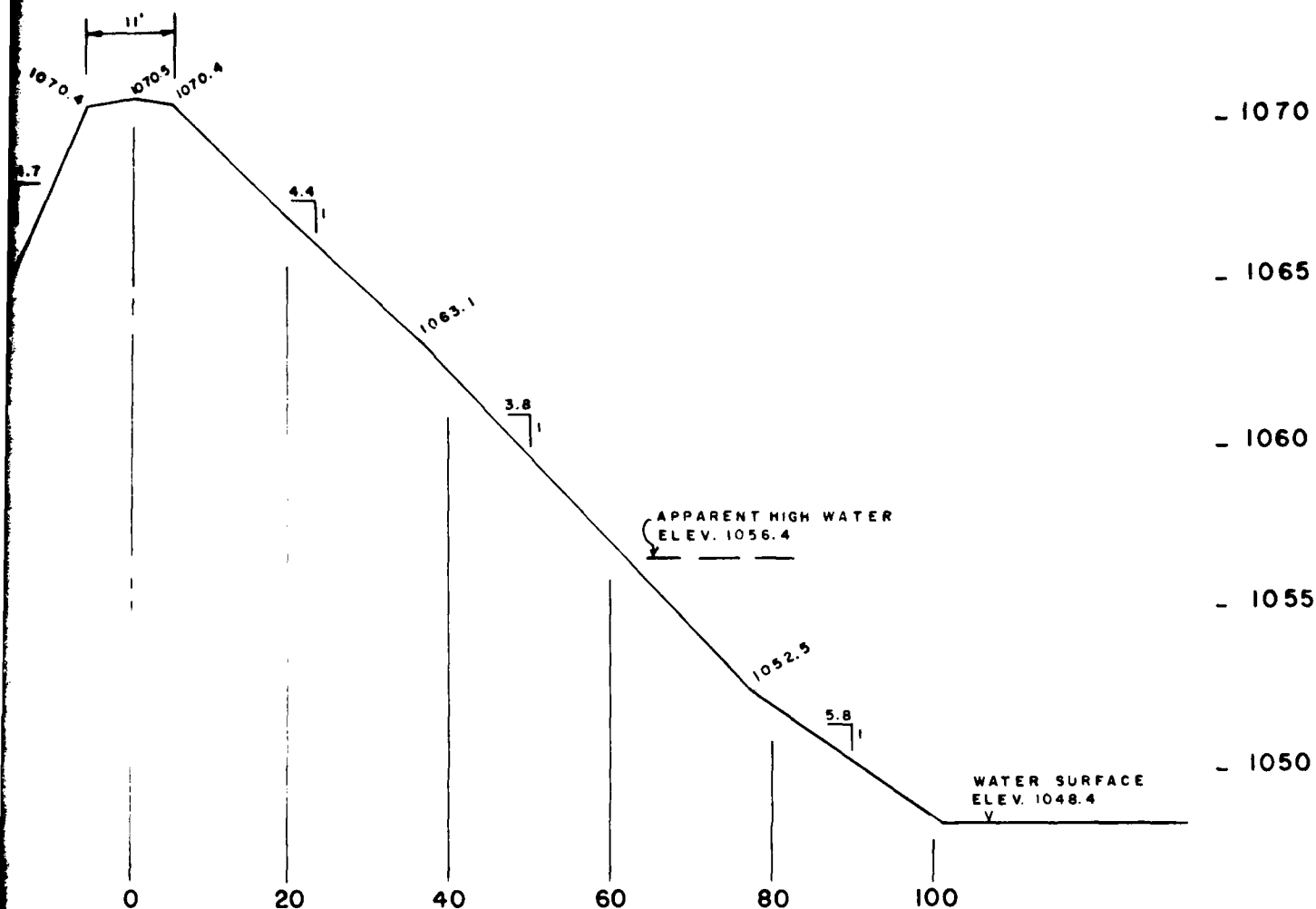
ACE

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SECTION A-A ST





SECTION A-A STA 2+65

SHEET 3 APPENDIX A

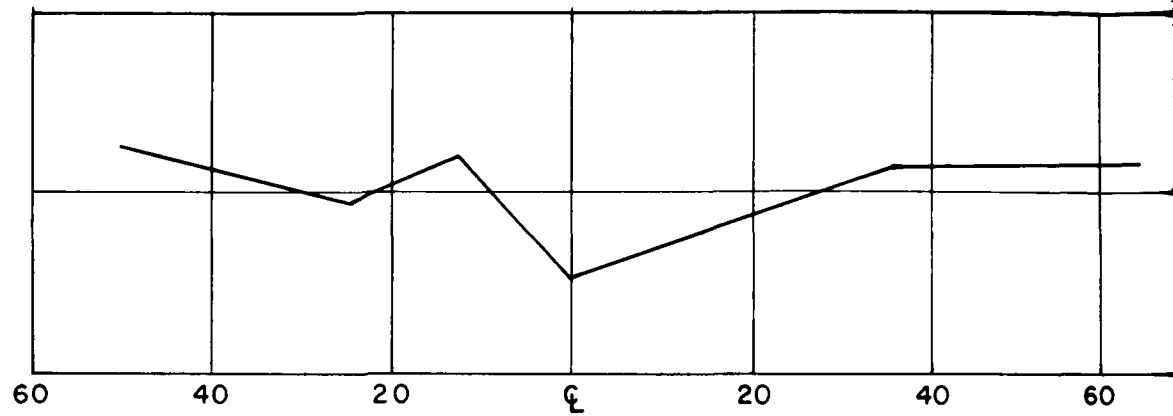
ANDERSON ENGINEERING, INC.
730 NORTH BENTON AVENUE
SPRINGFIELD, MISSOURI 65802

GRISHAM LAKE DAM

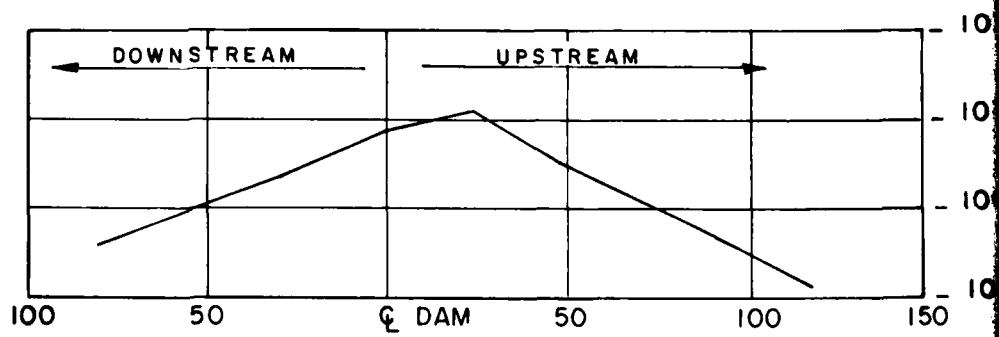
MO. No. 31574

PLAN & PROFILE

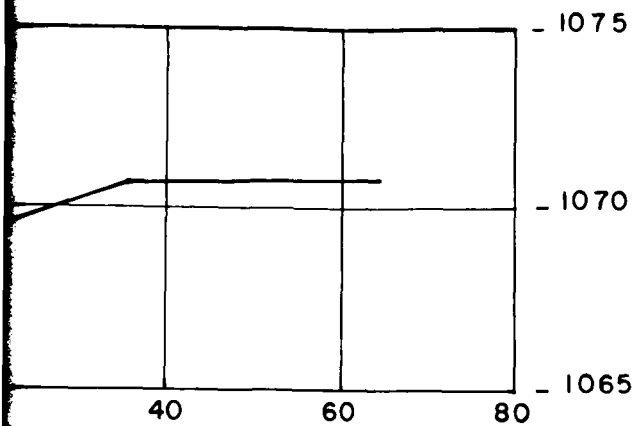
HOWELL COUNTY, MO.



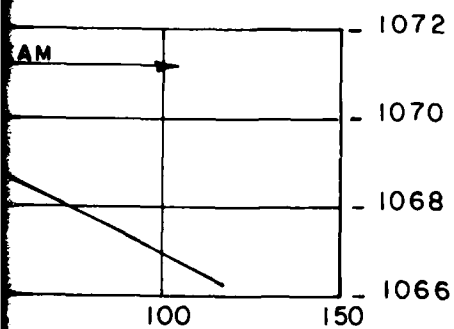
SPILLWAY SECTION
17 FT. DOWNSTREAM ϕ DAM



SPILLWAY PROFILE



SECTION
E DAM



FILE

SHEET 3A APPENDIX A

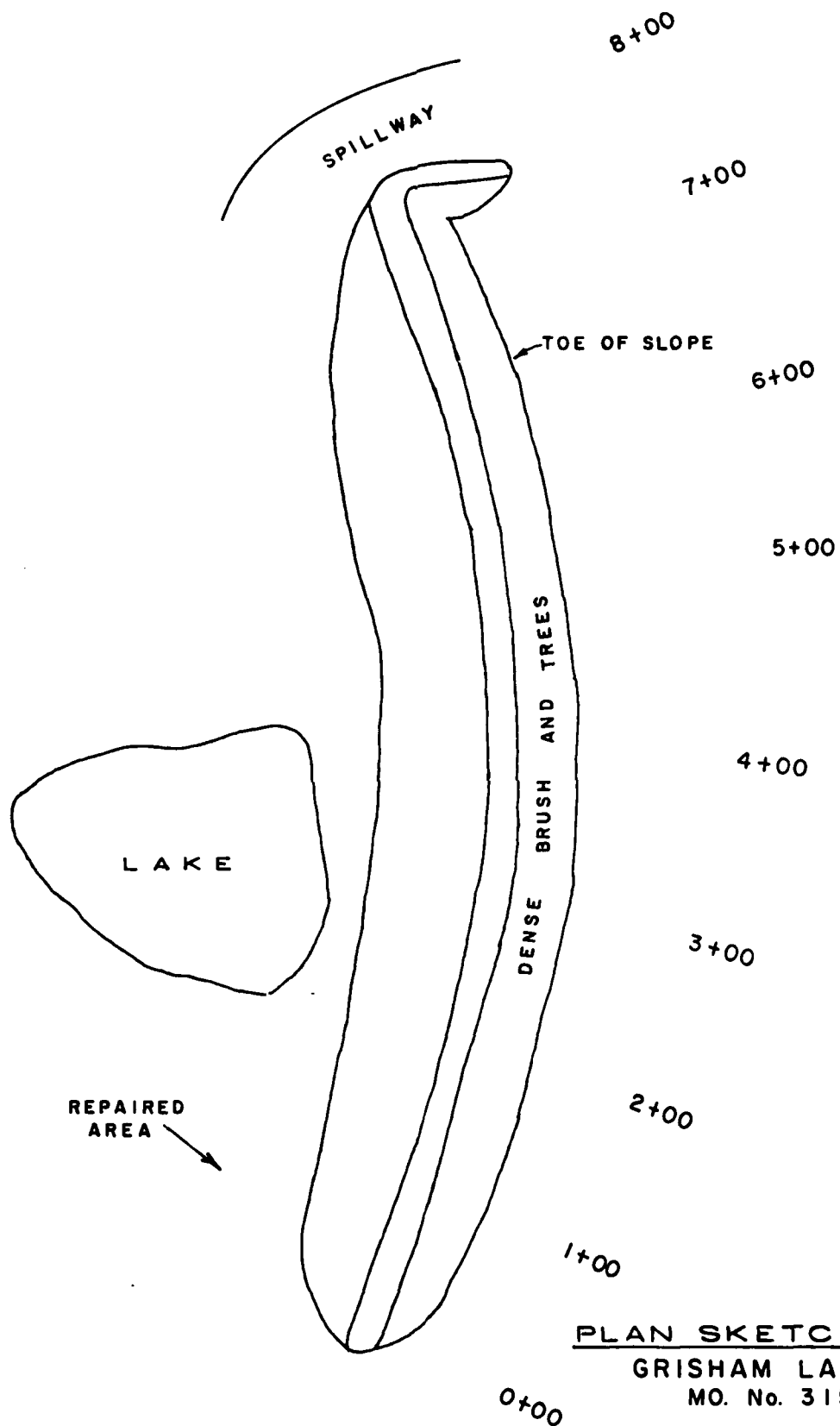
ANDERSON ENGINEERING, INC.
730 NORTH BENTON AVENUE
SPRINGFIELD, MISSOURI 65802

GRISHAM LAKE DAM

MO. No. 31574

SPILLWAY
SECTION E PROFILE

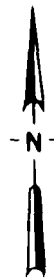
HOWELL COUNTY, MO.



PLAN SKETCH OF DAM
GRISHAM LAKE DAM
MO. No. 31574

APPENDIX B

Geology and Soils



LEGEND

GLACIATED PLAINS



WESTERN PLAINS



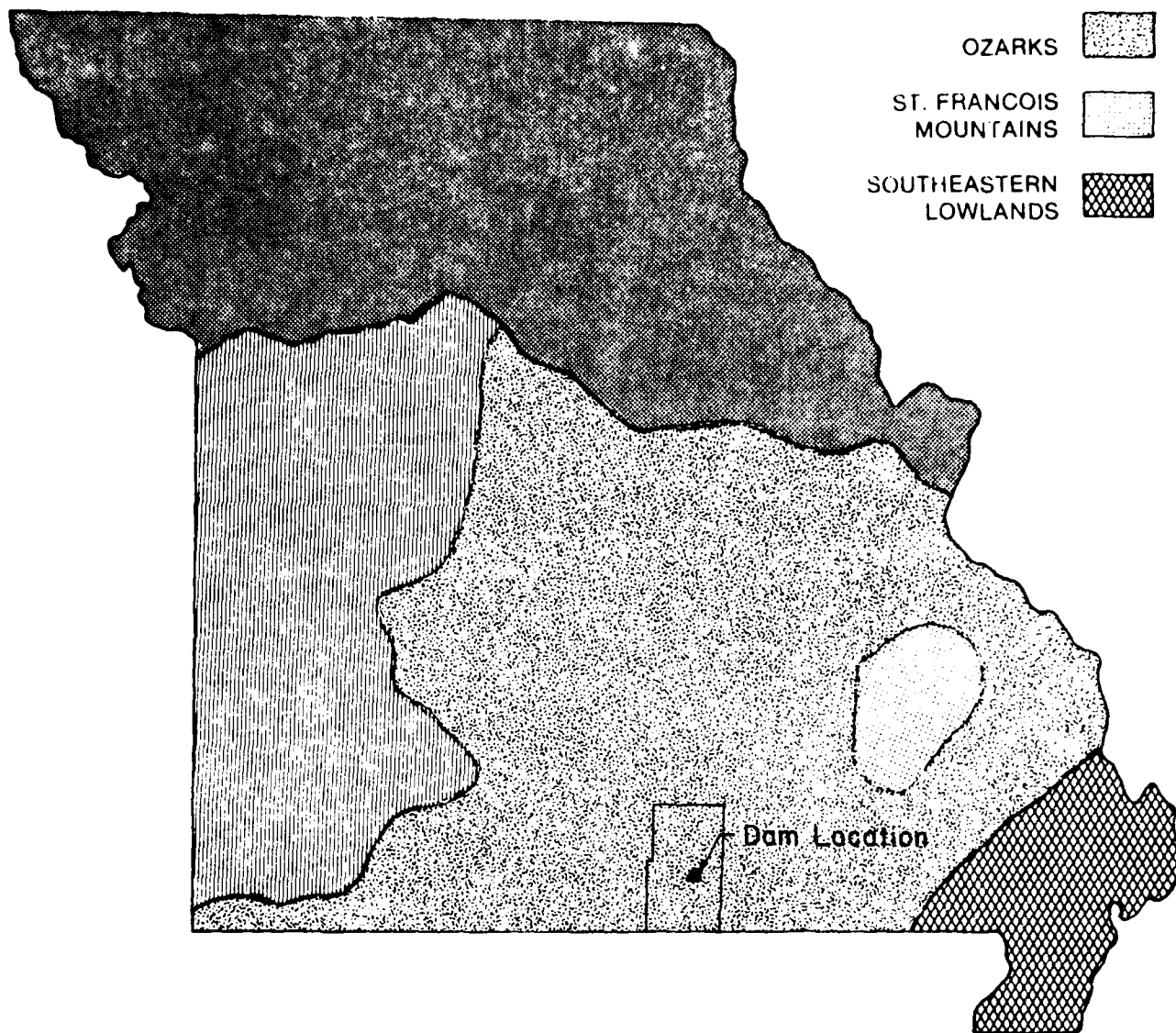
OZARKS



ST. FRANCOIS MOUNTAINS



SOUTHEASTERN LOWLANDS



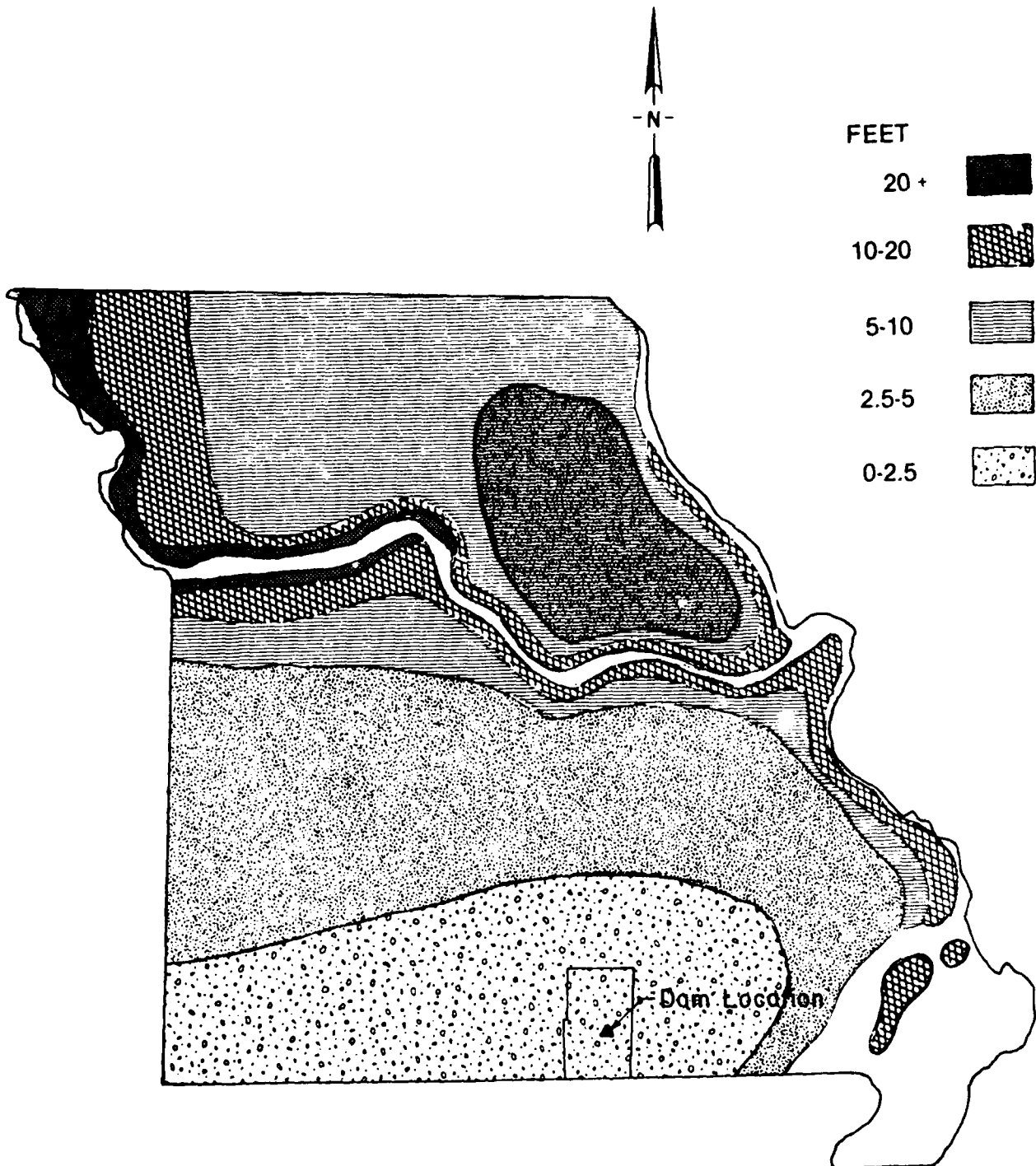
MAJOR GEOLOGIC REGIONS OF MISSOURI



SPRINGFIELD, IL • PEORIA, IL • ROCKFORD, IL

Grisham Lake Dam
Howell County, Missouri
Mo. I.D. No. 31574

SHEET 1, APPENDIX B



THICKNESS OF LOESSIAL DEPOSITS



SPRINGFIELD, IL • PEORIA, IL • ROCKFORD, IL

Grisham Lake Dam
Howell County, Missouri
Mo. I.D. No. 31574

SHEET 2, APPENDIX B

APPENDIX C

Overtopping Analysis



LAKE AND WATERSHED MAP



SPRINGFIELD, IL • PEORIA, IL • ROCKFORD, IL

Grisham Lake Dam
Howell County, Missouri
Mo. I.D. No. 31574

Sheet I, Appendix C

APPENDIX C

HYDROLOGIC AND HYDRAULIC ANALYSIS

To determine the overtopping potential, flood routings were performed by applying the Probable Maximum Precipitation (PMP) to a synthetic unit hydrograph to develop the inflow hydrograph. The inflow hydrograph was then routed through the reservoir and spillway. The overtopping analysis was accomplished using the systemized computer program HEC-1 (Dam Safety Version), July 1978, prepared by the Hydrologic Engineering Center, U.S. Army Corps of Engineers, Davis, California.

The PMP was determined from regional charts prepared by the National Weather Service in "Hydrometeorological Report No. 33." Reduction factors were not applied. The rainfall distribution for the 24-hour PMP storm duration was assumed according to the procedures outlined in EM 1110-2-1411 (SPD Determination). Also, the 1 percent chance probability flood was routed through the reservoir and spillway. Doniphan rainfall distribution (5 min. interval - 24 hours duration), as provided by the St. Louis District, Corps of Engineers, was used in this case.

The synthetic unit hydrograph for the watershed was developed by the computer program using the SCS method. The parameters for the unit hydrograph are shown in Table 1 (Sheet 4, Appendix C).

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 (Sheet 5, Appendix C).

The reservoir routing was accomplished by using the Modified Puls Method. The hydraulic capacity of the spillway was used as an outlet control in the routing. The hydraulic capacity of the spillway and the storage capacity of the reservoir were defined by the elevation-surface area--storage-discharge relationships shown in Table 3 (Sheet 5, Appendix C.)

The reservoir of this dam has never been filled. To consider the effect of the reservoir storage in the routing, an antecedent storm of 25 percent and 50 percent of the PMF was considered assuming the reservoir at elevation 1,057.0 (mean annual high water) to determine the starting reservoir elevation for the routing of 50 percent and 100 percent of the PMF respectively. The 25 percent PMF reached elevation 1,066.8. The 50 percent PMF overtopped the dam. These antecedent storms were assumed to occur four days prior to the corresponding storm. There are no outlet works or dewatering structures in this dam. Thus, the final routing analysis was accomplished starting at the spillway crest elevation (1,069.8) for the PMF, and at elevation 1,066.8 for the 50 percent PMF storm.

The percentage of the PMF that will reach the top of the dam was determined to be 22 percent. An antecedent storm of half this magnitude (11 percent) was considered. In this case, the starting reservoir elevation was 1,062.13.

For the routing of the 1 percent probability flood, the antecedent storm was assumed to be equivalent to 1 in. of runoff. The corresponding reservoir level (1,059.0) was used as the starting reservoir elevation.

The rating curve for the spillway (see Table 4, Sheet 6, Appendix C) was determined assuming critical flow conditions at the control section.

The flow over the crest of the dam during overtopping was determined using the non-level dam option (\$L and \$V cards) of the HEC-1 program. The program assumes critical flow over a broad-crested weir.

A summary of the routing analysis for different ratios of the PMF is shown in Table 5 (Sheet 7, Appendix C).

The computer input data and a summary of the output data for the antecedent storms, the PMF and the PMF ratios, are presented on Sheets 8 through 18 of Appendix C. A plot of the inflow-outflow hydrograph for the PMF is shown on Sheet 12, Appendix C.

TABLE 1
SYNTHETIC UNIT HYDROGRAPH

Parameters:

Drainage Area (A)	0.263 sq. miles
Length of Watercourse (L)	0.44 miles
Difference in elevation (H)	95 feet
Time of concentration (Tc)	0.17 hours
Lag Time (Lg)	0.10 hours
Time to peak (Tp)	0.14 hours
Peak Discharge (Qp)	910 cfs
Duration (D)	5 min.

<u>Time (Min.) (*)</u>	<u>Discharge (cfs) (*)</u>
0	0
5	576
10	852
15	374
20	145
25	55
30	22
35	9
40	3
45	0

(*) From the computer output

FORMULA USED:

$$T_c = \left(\frac{11.9 L^3}{H} \right)^{0.385} \quad \text{From California Culverts Practice, California Highways and Public Works, September, 1942.}$$

$$L_g = 0.6 T_c$$

$$T_p = \frac{D}{2} + L_g$$

$$Q_p = \frac{484 A \cdot Q}{T_p} \quad Q = \text{Excess Runoff} = 1 \text{ inch}$$

TABLE 2

RAINFALL-RUNOFF VALUES

<u>Selected Storm Event</u>	<u>Storm Duration (Hours)</u>	<u>Rainfall (Inches)</u>	<u>Runoff (Inches)</u>	<u>Loss (Inches)</u>
PMP	24	35.88	33.12	2.76
1% Prob. Flood	24	7.55	3.59	3.96

Additional Data:

- 1) Soil Conservation Service Soil Group B
- 2) Soil Conservation Service Runoff Curve CN = 78 (AMC III) for the PMF
- 3) Soil Conservation Service Runoff Curve CN = 60 (AMC II) for the 1 percent probability flood
- 4) Percentage of Drainage Basin Impervious 13 percent

TABLE 3

ELEVATION, SURFACE AREA, STORAGE AND DISCHARGE RELATIONSHIPS

<u>Elevation (feet-MSL)</u>	<u>Lake Surface Area (acres)</u>	<u>Lake Storage (acre-ft)</u>	<u>Spillway Discharge (cfs)</u>
1048.0	0	0	-
1057.0	5.6	25	-
1060.0	7.5	45	-
*1069.8	19.0	175	0
**1070.0	19.3	179	5
1075.0	25.2	290	-
1080.0	31.1	431	-

*Principal spillway crest elevation

**Top of dam elevation

The above relationships were developed using information from the West Plains, MO 15 minute quadrangle map and the field measurements.

TABLE 4

SPILLWAYS RATING CURVE

<u>Reservoir Elevation</u> (MSL)	<u>Principal Spillway</u> (cfs)
1069.8	0
*1070.0	5
1070.5	60
1071.0	210
1071.5	450
1072.0	790
1072.5	1,310
1072.6	1,440
1073.3	2,350

*Top of dam elevation

METHOD USED: Assuming critical flow condition at the control section.

FORMULA:

$$\frac{Q^2}{g} = \frac{A^3}{T}$$

Q = Discharge in cubic feet per second

A = Cross sectional area in square feet

T = Water surface width in feet

g = Acceleration of gravity in ft/sec²

TABLE 5

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
-	0	*1069.8	175	0	-
0.22	884	**1070.0	179	5	0
0.50	2010	1071.3	209	1584	1.3
1.00	4020	1071.9	221	3405	1.9

The percentage of the PMF that will reach the top of the dam is 22 percent.

*Principal spillway crest elevation

**Top of dam elevation

A OVERTOPPING ANALYSIS FOR GRISHAM LAKE DAM (# 11)
 A STATE ID NO. 31574 COUNTY NAME : HOWELL
 A HANSON ENGINEERS INC. DAM SAFETY INSPECTION JOB # 80S3001
 B 300 5
 B1 5
 J 1 3 1
 J1 .25 .50 1.0
 K 0 1 5 1
 K1 INFLOW HYDROGRAPH COMPUTATION **
 M 1 2 0.263 0.263 1
 P 0 27.6 102 120 130
 T -1 -60 0.13
 U2 0.17 0.10
 X 0 -.1 2
 K 1 2 0 4 1
 K1 RESERVOIR ROUTING BY MODIFIED PULS AT DAM SITE **
 Y 1 1
 Y1 1 25
 Y41069.8 1070.0 1070.5 1071.0 1071.5 1072.0 1072.5 1072.6 1073.3
 Y5 0 5 60 210 450 790 1310 1440 2350
 \$5 0 25 45 175 179 290 431
 \$E1048.0 1057.0 1060.0 1069.8 1070.0 1075.0 1080.0
 \$1069.8
 \$D1070.0
 \$L 0 160 260 360 440 550 615 660
 \$V1070.0 1070.3 1070.5 1070.8 1071.0 1071.3 1071.6 1073.4
 K 99

25 and 50 PERCENT PMF
 (ANTECEDENT STORMS)
 INPUT DATA

Sheet 8, Appendix C

PEAK FLOW AND STORAGE (END OF PERIOD) SUMMARY FOR MULTIPLE PLAN-RATIO ECONOMIC COMPUTATIONS
 FLOWS IN CUBIC FEET PER SECOND (CUBIC METERS PER SECOND)
 AREA IN SQUARE MILES (SQUARE KILOMETERS)

RATIOS APPLIED TO FLOWS

OPERATION	STATION	AREA	PLAN	RATIO 1	RATIO 2	RATIO 3
				0.25	0.50	1.00
HYDROGRAPH AT	1	0.26	1	964.	1928.	3856.
	(	0.68)	(	27.30)	(54.60)	(109.19)
ROUTED TO	2	0.26	1	0.	267.	3237.
	(	0.68)	(	0.00)	(7.56)	(91.66)

SUMMARY OF DAM SAFETY ANALYSIS

PLAN 1							
ELEVATION		INITIAL VALUE		SPILLWAY CREST		TOP OF DAM	
STORAGE		1057.00		1069.80		1070.00	
OUTFLOW		25.		175.		179.	
		0.		0.		5.	
RATIO OF PMF	MAXIMUM RESERVOIR U.S.ELEV	MAXIMUM DEPTH OVER DAM	MAXIMUM STORAGE AC-FT	MAXIMUM OUTFLOW CFS	DURATION OVER TOP HOURS	TIME OF	
						MAX OUTFLOW HOURS	FAILURE HOURS
0.25	1066.80	0.00	135.	0.	0.00	0.00	0.00
0.50	1070.60	0.60	192.	267.	8.75	17.25	0.00
1.00	1071.86	1.86	220.	3237.	10.00	15.75	0.00

[illegible]

INFLOW HYDROGRAPH COMPUTATION **

0.263

130

2

Ind

1

:

50

450 170

0000

0.0

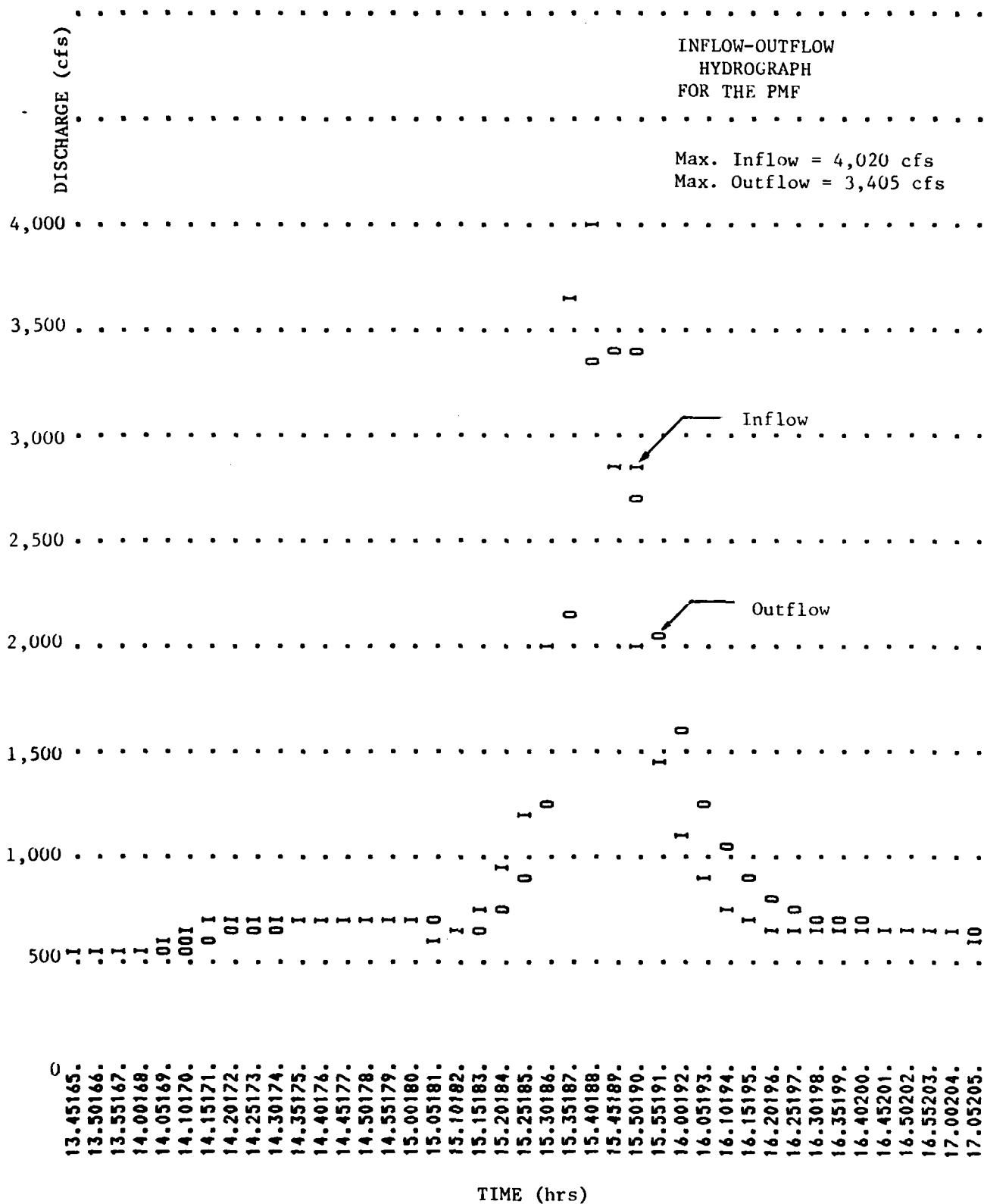
440

1.0

3

Sheet 10, Appendix C

兴行书局



Sheet 13, Appendix C

PEAK FLOW AND STORAGE (END OF PERIOD) SUMMARY FOR MULTIPLE PLAN-RATIO ECONOMIC COMPUTATIONS
 FLOWS IN CUBIC FEET PER SECOND (CUBIC METERS PER SECOND)
 AREA IN SQUARE MILES (SQUARE KILOMETERS)

RATIOS APPLIED TO FLOWS

OPERATION	STATION	AREA	PLAN RATIO	1
				0.50

HYDROGRAPH AT	1	0.26	1	2010.
	(	0.68)	(	56.92)

ROUTED TO	2	0.26	1	1584.
	(	0.68)	(	44.87)

SUMMARY OF DAM SAFETY ANALYSIS

PLAN 1

INITIAL VALUE	SPILLWAY CREST	TOP OF DAM
1066.80	1069.80	1070.00
135.	175.	179.
0.	0.	5.

ELEVATION
STORAGE
OUTFLOW

RATIO OF PMF	MAXIMUM RESERVOIR W.S.ELEV	MAXIMUM DEPTH OVER DAM	MAXIMUM STORAGE AC-FT	MAXIMUM OUTFLOW CFS	DURATION OVER TOP HOURS	TIME OF MAX OUTFLOW HOURS	TIME OF FAILURE HOURS
0.50	1071.34	1.34	209.	1584.	12.00	15.75	0.00

50 PERCENT PMF
 OUTPUT DATA

A OVERTOPPING ANALYSIS FOR GRISHAM LAKE DAM (# 11) (PMF RATIOS)
 A STATE ID NO. 31574 COUNTY NAME : HOWELL
 A HANSON ENGINEERS INC. DAM SAFETY INSPECTION JOB # 80S3001
 B 300 5
 B1 5
 J 1 3 1
 J1 .10 .11 .12
 K 0 1
 K1 INFLOW HYDROGRAPH COMPUTATION ** 5 1
 M 1 2 0.263 0.263 1
 P 0 27.6 102 120 130
 T -1 -60 0.13
 W2 0.17 0.10
 X 0 -.1 2
 K 1 2
 K1 RESERVOIR ROUTING BY MODIFIED PULS AT DAM SITE **
 Y 1 1
 Y1 1
 Y41069.8 1070.0 1070.5 1071.0 1071.5 1072.0 1072.5 1072.6 1073.3
 Y5 0 5 60 210 450 790 1310 1440 2350
 \$S 0 25 45 175 179 290 431
 \$E1048.0 1057.0 1060.0 1069.8 1070.0 1075.0 1080.0
 \$1069.8
 \$D1070.0
 \$L 0 160 260 360 440 550 615 660
 \$V1070.0 1070.3 1070.5 1070.8 1071.0 1071.3 1071.6 1073.4
 K 99

PMF RATIOS
 (ANTECEDENT STORMS)
 INPUT DATA

PEAK FLOW AND STORAGE (END OF PERIOD) SUMMARY FOR MULTIPLE PLAN-RATIO ECONOMIC COMPUTATIONS
FLOWS IN CUBIC FEET PER SECOND (CUBIC METERS PER SECOND)
AREA IN SQUARE MILES (SQUARE KILOMETERS)

OPERATION	STATION	AREA	RATIOS APPLIED TO FLOWS				
			PLAN	RATIO 1	RATIO 2	RATIO 3	
HYDROGRAPH AT	1	0.26					
	(	0.68)	(	10.92)	(	12.01)	(
ROUTED TO	2	0.26					
	(	0.68)	(	0.00)	(	0.00)	(

SUMMARY OF DAM SAFETY ANALYSIS

PLAN 1	ELEVATION	INITIAL VALUE	SPILLWAY CREST	TOP OF DAM
	STORAGE	1057.00	1069.80	1070.00
	OUTFLOW	25.	175.	179.
		0.	0.	5.

RATIO OF PMF	MAXIMUM RESERVOIR U.S.ELEV	MAXIMUM DEPTH OVER DAM	MAXIMUM STORAGE AC-FT	MAXIMUM OUTFLOW CFS	DURATION OVER TOP HOURS	TIME OF MAX OUTFLOW HOURS	TIME OF FAILURE HOURS
0.10	1061.79	0.00	69.	0.	0.00	0.00	0.00
0.11	1062.13	0.00	73.	0.	0.00	0.00	0.00
0.12	1062.47	0.00	78.	0.	0.00	0.00	0.00

PMF RATIOS
(ANTECEDENT STORMS)
OUTPUT DATA

STATE ID NO. 31574 COUNTY NAME : HOWELL
HANSON ENGINEERS INC. DAM SAFETY INSPECTION JOB # 80S3001

PMF RATIOS
INPUT DATA

PEAK FLOW AND STORAGE (END OF PERIOD) SUMMARY FOR MULTIPLE PLAN-RATIO ECONOMIC COMPUTATIONS
 FLOWS IN CUBIC FEET PER SECOND (CUBIC METERS PER SECOND)
 AREA IN SQUARE MILES (SQUARE KILOMETERS)

OPERATION	STATION	AREA	PLAN	RATIOS APPLIED TO FLOWS				
				RATIO 1	RATIO 2	RATIO 3	RATIO 4	RATIO 5
HYDROGRAPH AT	1	0.26	1	804.	844.	884.	925.	965.
	(	0.68)	(	22.77)	(23.90)	(25.04)	(26.18)	(27.32)
ROUTED TO	2	0.26	1	0.	0.	4.	10.	26.
	(	0.68)	(	0.00)	(0.00)	(0.12)	(0.29)	(0.75)

SUMMARY OF DAM SAFETY ANALYSIS

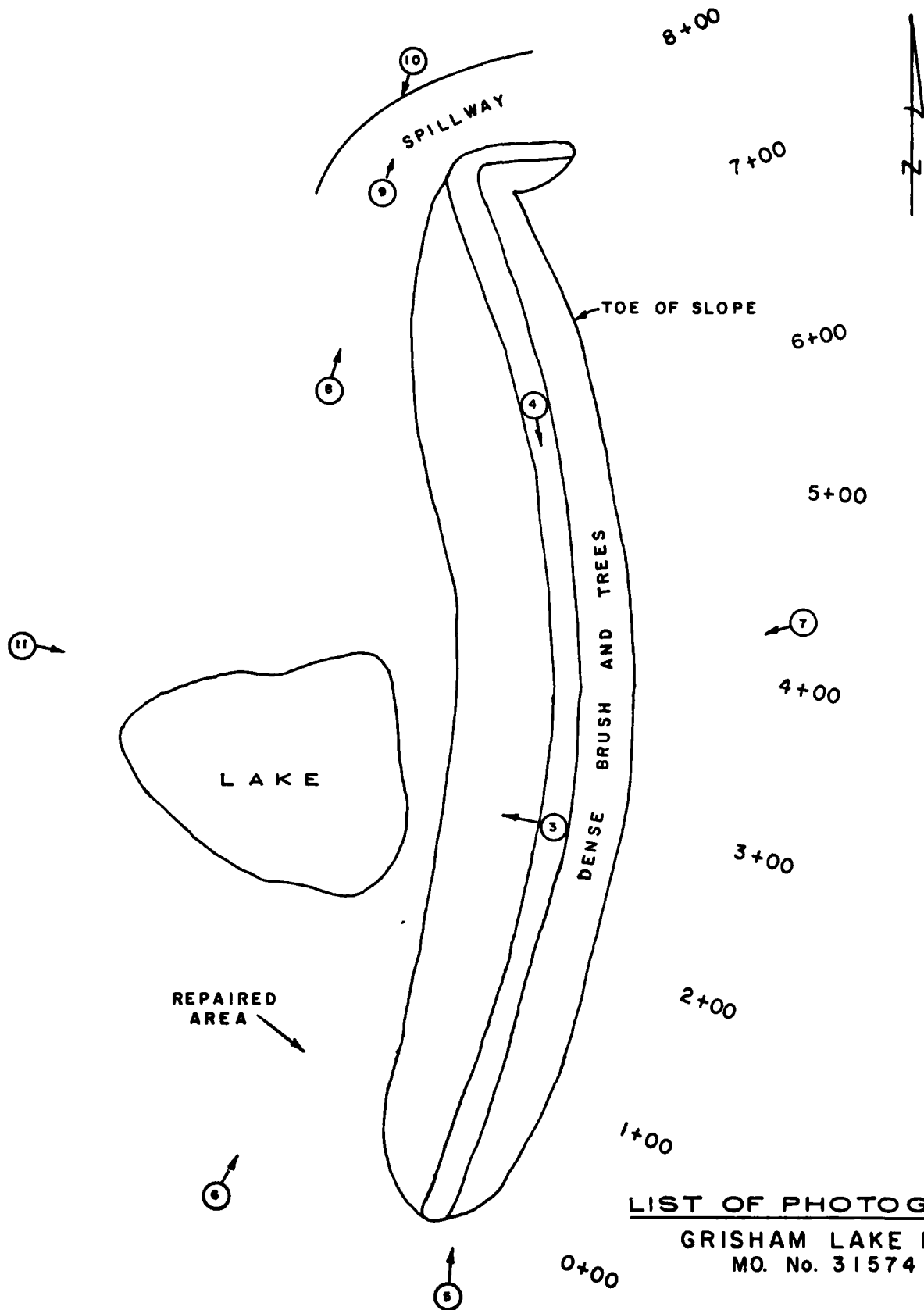
PLAN 1		INITIAL VALUE	SPILLWAY CREST	TOP OF DAM
ELEVATION		1062.13	1069.80	1070.00
STORAGE		73.	175.	179.
OUTFLOW		0.	0.	5.

RATIO OF PMF	MAXIMUM RESERVOIR U.S.ELEV	MAXIMUM DEPTH OVER DAM	MAXIMUM STORAGE AC-FT	MAXIMUM OUTFLOW CFS	DURATION OVER TOP HOURS	TIME OF MAX OUTFLOW HOURS	TIME OF FAILURE HOURS
0.20	1069.42	0.00	170.	0.	0.00	0.00	0.00
0.21	1069.74	0.00	174.	0.	0.00	0.00	0.00
0.22	1069.97	0.00	178.	4.	0.00	24.83	0.00
0.23	1070.05	0.05	180.	10.	5.67	20.58	0.00
0.24	1070.15	0.15	182.	26.	6.75	19.58	0.00

PMF RATIOS
 OUTPUT DATA

APPENDIX D

Photographs

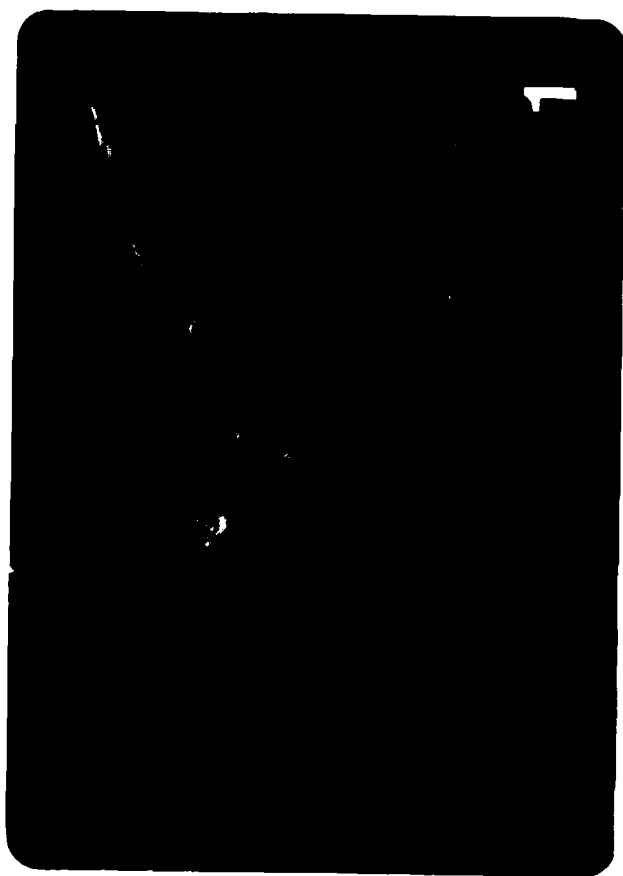


LIST OF PHOTOGRAPHS

GRISHAM LAKE DAM
MO. No. 31574

PHOTOGRAPH INDEX

<u>Photo No.</u>	<u>Description</u>
1	Aerial View of Lake and Dam
2	Aerial View of Lake and Dam
3	View of Lake and Watershed (Looking West)
4	Crest of Dam (Looking South)
5	Crest of Dam (Looking North)
6	Upstream Face of Dam (Looking Northeast)
7	Downstream Face of Dam (Looking West)
8	Spillway Inlet Channel (Looking North)
9	Spillway Outlet Channel (Looking Northeast)
10	Spillway Inlet Channel (Looking South)
11	Lake and Upstream Face of Dam (Looking East)
12	Downstream Hazard Zone (Looking West from Highway 17)







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