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FAILURE MODE ANALYSIS REPORT

# STRUCTURES ASSESSMENT PROGRAM PHASE I

Cave Buttes Dam • Powerline FRS • Vineyard Road FRS • Rittenhouse FRS
Spook Hill FRS • Signal Butte FRS • Apache Junction FRS

*Prepared for*
**Flood Control District of Maricopa County**
January 2002
FCD No. 2001C014/PCN 050.35.31

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*Prepared by*
Kimley-Horn and Associates, Inc.

Kimley-Horn
and Associates, Inc.

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Suite 250
7600 N. 15th Street
Phoenix, Arizona
85020

February 12, 2002

Mr. Tom Renckly, P.E.
Structures Division Manager
Flood Control District of Maricopa County
2801 West Durango Street
Phoenix, Arizona 85009

**Re: Structures Assessment Program, Phase I**
**Failure Mode Analysis Report**
**FCD 2001C014**
**KHA No.: 091131006**

Dear Tom:

Kimley-Horn and Associates, Inc. is pleased to submit the draft Failure Mode Analysis Report for the above referenced project. We have provided one copy each of the report to the following Structures Assessment Technical Committee agencies: Natural Resources Conservation Service, Arizona Department of Water Resources, U.S. Army Corps of Engineers, and the Bureau of Reclamation. We have requested that the reviewers provide comments by Monday, March 11, 2002 to KHA by email. If possible we request the same for District comments.

Tom, it has been a great pleasure to provide the failure modes analyses services to the District. The failure mode sessions have benefited and added great value to the Structures Assessment Program.

If you have any questions, please contact me at my direct line at 906-1182.

Sincerely,

KIMLEY-HORN AND ASSOCIATES, INC.

Robert A. Eichinger

Robert A. Eichinger, P.E.
Project Manager
Email: bob.eichinger@kimley-horn.com

Encl: Draft Failure Mode Analysis Report – 6 copies

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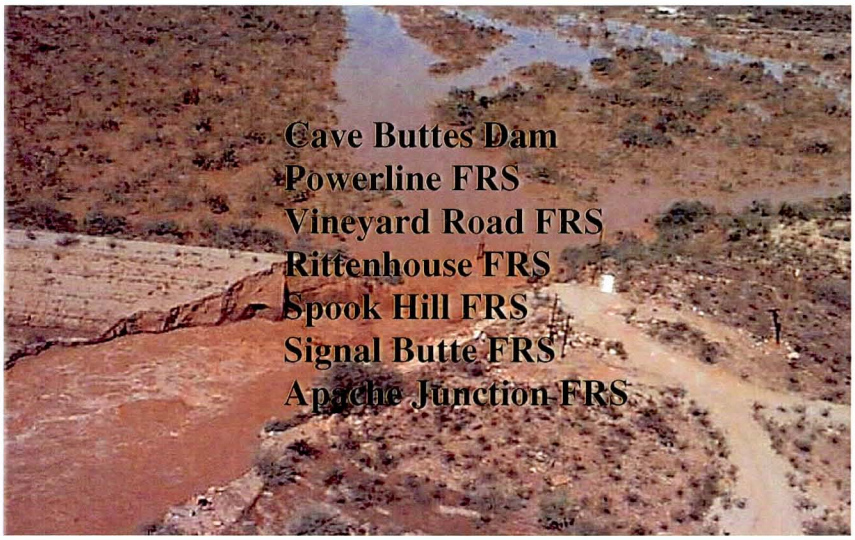
# STRUCTURES ASSESSMENT PROGRAM – PHASE I

## *FAILURE MODE ANALYSIS REPORT - DRAFT*




**FLOOD CONTROL DISTRICT**
**OF MARICOPA COUNTY**
**FCD 2001C014**
**PCN PLAN 050.35.31**

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**JANUARY 2002**

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# FAILURE MODE AND CONSEQUENCE ANALYSIS REPORT

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# FAILURE MODE AND CONSEQUENCE ANALYSIS REPORT

## PREFACE

The "Failure Mode and Consequence Analysis Report" documents the Failure Modes Analysis conducted for the Flood Control District of Maricopa County for seven earth embankment dams and flood retarding structures. These seven structures are Cave Buttes Dam, Powerline Flood Retarding Structure (FRS), Vineyard Road FRS, Rittenhouse FRS, Spook Hill FRS, Signal Butte FRS, and Apache Junction FRS.

**Methodology and Approach**: A Failure Mode and Consequence Analysis (FMCA) is a formal examination of "potential" failure modes for an existing dam by an assembled FMCA team. The analysis is based on a thorough review of existing data and information for the dam under investigation, from first hand input from field/maintenance and operational personnel, site specific inspections and observations, previous completed engineering analyses, identification of potential failure modes, failure causes and failure development, and an understanding of the consequences of failure. The Failure Mode Analysis is intended to provide enhanced understanding and insight on the risk exposure associated with the dam. This is accomplished by including the traditional engineering approach and means but also going beyond and above those means for assessing the safety of a dam and by intentionally seeking input from the diverse team of individuals who have information on the performance and operation of the dam. An FMCA utilizes all of the available data and information from a standard engineering analysis of an existing dam and should be viewed as a supplement to the traditional process in which the safety of a dam is judged upon by it's ability to pass performance and standards-based criteria (for stability, design floods, and other conditions).

As a result of the FMCA's "question asking approach" the failure mode examination process has the potential to:

- Enhance the dam safety inspection process.
- Enhance and focus the visual surveillance and instrumented monitoring program.
- Identify gaps, shortcomings, or oversights in data, information or analyses necessary to further evaluate a potential failure mode.
- Help identify the most effective dam safety risk reduction measures.

The failure mode analysis is carried out by:

1. Collecting all data, studies and information on the investigation, analysis, design, construction, and performance and operational history of the project dam.

2. Establishment of a FMCA core team of at least three technically experienced persons in dam safety (and familiar with dam failure mechanisms) to review all the

background information to gain a general understanding of the dam and with these specific questions in mind during the review:

- How could this dam fail? Leading to the identification of potential failure modes (based on site-specific consideration of conditions).
- What happens if the dam fails? Leading to the consequences of failure.
- Are the identified potential failure modes recognized and being appropriately monitored by visual surveillance or instrumented monitoring?
- What actions (immediate or long term) can be taken to reduce dam failure likelihood or to mitigate failure consequences?

3. Brainstorming the identified potential failure modes and processes with an extended FMCA team (persons most familiar with investigation, design, analysis, performance, operation and maintenance of the dam).

4. Documenting the failure mode analysis including immediately recording the major findings and understandings for the dam under investigation from the brainstorming session (step 3). The major findings and understanding are then reviewed by the extended FMCA team for meaning, intent, and completeness of though.

The FMCA report for the dam documents the results of the FMCA by identifying potential failure modes, discusses the positive factors that exist at the dam that may reduce the potential that the specific failure mode would occur, discusses adverse factors that may be conducive to the potential failure mode, and presents risk reduction measures for that failure mode. The potential failure modes are categorized based on judgement of that failure mode occurring at that dam. The FMCA document is a "living" document. It is intended to be used for assisting in making dam safety decisions (prioritizing actions), used for guidance on future dam safety inspections, and is to be updated as new data or evidence warrants enhancement of the understanding of the performance, operation, and risk of the dam. The benefit of a living document format is that the increased understanding and insight regarding the dam and performance of the dam lives on.

**FMCA Goal**: The goal of the failure mode and consequences analysis is to achieve an understanding of the most significant site-specific failure modes and inundation impacts, and the potential consequences of failure for each structure. The analysis is fundamental to dam safety risk assessment and to any further steps employing quantitative risk analysis. It is important to note that the effort expended in this activity can provide significant benefits through:

- Assisting with decisions on a course of action relative to the safety of the dam being evaluated
- Enhancing the long term dam safety monitoring and dam surveillance program
- Improving the emergency preparedness program
- Prioritization of dam safety modifications for the full inventory of dams (However, this study is limited only to the seven dams)
- Reducing the risk of dam failure and lives at risk both upstream and downstream of the dam.

These potential benefits are possible whether or not further work on risk quantification is accomplished.

**Failure Mode Categories**: The failure mode analysis identified potential failure modes for each structure. The failure modes were placed into one of four categories as follows. Except in special cases, only those potential failure modes in Category I will be considered in the risk (quantitative) analysis phase of the risk assessment exercise.

- Category I – Failure modes of greatest significance.
- Category II – Failure modes of lesser (but not inconsequential) significance.
- Category III – Failure modes for which insufficient information is presently available to make a judgement on the significance of the failure mode. The development of additional data and information is warranted. Additional records research may be justified.
- Category IV – Failure modes which are not physically possible or which are clearly not credible.

The failure mode analysis has identified and developed "potential" failure modes. It is not the purpose of the analysis or the report to provide an opinion of the safety of each structure in regards to operation, stability, or structural soundness. The analysis and report does not offer, nor was it in the scope or purpose, to investigate the safety of each dam or to state that there is any concern regarding the immediate safety of each structure. The analysis and report only identifies potential failure modes or mechanisms that are based on site-specific conditions at each dam, that if these conditions potentially were to develop and/or simultaneously occur, a potential dam failure could result. At the time of conducting the failure mode analysis and preparation of this report, the seven structures were found to be in a satisfactory operating condition.

However, it is prudent to recognize that there exist for all dams specific ways that failure could come about that warrant attention and diligent monitoring. The identification of a condition or process as a "potential failure mode" does not imply that the dam is about to fail or even necessarily that there is a dam safety deficiency at the site. Rather it identifies physically possible conditions or processes (generally with a remote but still credible chance of occurrence) that persons associated with owning, inspecting, analyzing and operating the dam should be aware. Some of the potential failure modes are highlighted (or prioritized) for attention of the dam owners and operators. They are highlighted because the specific conditions at the dam and appurtenant structures are such that these failure modes are physically possible and are considered the most realistic and most credible potential failure modes definable at the site. The site-specific conditions relative to potential failure modes at Cave Buttes Dam, Powerline Flood Retarding Structure (FRS), Vineyard Road FRS, Rittenhouse FRS, Spook Hill FRS, Signal Butte FRS, and Apache Junction FRS are described in this FMCA report.

# SUMMARY OF FAILURE MODES BY CATEGORY FOR EACH DAM OR FLOOD RETARDING STRUCTURE

| Failure Mode Category | DAM OR FLOOD RETARDING STRUCTURE | | | | | | |
|---|---|---|---|---|---|---|---|
| | Cave Buttes Dam | Powerline FRS | Vineyard Road FRS | Rittenhouse FRS | Spook Hill FRS | Signal Butte FRS | Apache Junction FRS |
| **Category I**<br>*Highlighted Failure Modes – Those failure modes of greatest significance considering need for awareness, potential for occurrence, magnitude of consequence and likelihood of adverse response (physical possibility was evident, fundamental flaw or weakness is identified and conditions and events leading to failure seemed reasonable and credible) are highlighted.* | ▪ Failure from piping (seepage erosion) of embankment soil into joints and through joints in the dam foundation - Main dam.<br>▪ Failure from piping Zone II soil into foundation – Training Dike No. 2. | ▪ Failure from overtopping of Powerline FRS.<br>▪ Failure from a potential earth fissure(s) through the dam embankment in association with a significant flooding event.<br>▪ Failure from piping through transverse cracks causing breach of dam in association with a major flooding event. | ▪ Failure from overtopping of dam crest due to very large flooding events approaching the PMF.<br>▪ Failure from potential earth fissure(s) through the dam embankment in association with a significant flooding event.<br>▪ Failure from seepage erosion through transverse cracks causing breach of dam in association with a major flooding event.<br>▪ Failure from seepage erosion resulting from a flow path through foundation and along the dam/foundation contact. | ▪ Failure from overtopping of dam crest due to PMF.<br>▪ Failure from seepage erosion through transverse cracks causing breach of dam in association with a major flooding event.<br>▪ Failure from seepage erosion resulting from a flow path through foundation and along the dam /foundation contact. | ▪ Adverse consequences resulting from emergency spillway discharges during major rainfall events.<br>▪ Failure due to seepage erosion along a transverse crack through embankment in association with a major flooding event. | ▪ Adverse consequences resulting from emergency spillway discharges during major rainfall events. | ▪ Adverse consequences resulting from normal emergency spillway discharges during major rainfall events. |
| **Category II**<br>*Failure Modes Considered but not Highlighted – These are judged to be of lesser significance and likelihood. Each failure mode is described and included with reasons for and against the potential occurrence of the* | ▪ Failure from piping Zone II soil from dam into alluvium at cutoff trench - Main dam.<br>▪ Failure from piping material from the main dam into the foundation through open joints.<br>▪ Failure from piping along outlet works – Main dam. | ▪ Failure from potential piping around/or from 12 inch corrugated metal pipe outlet at station 141+00.<br>▪ Failure from potential erosion of abutment and/or emergency spillway during spillway discharges.<br>▪ Failure from potential embankment slope instability. | ▪ Failure from seepage erosion due to piping through foundation.<br>▪ Failure from potential erosion of abutments during spillway discharges.<br>▪ Failure from potential embankment slope instability. | ▪ Failure from potential erosion of left abutment of the dam during spillway discharges.<br>▪ Failure from seepage erosion through an earth fissure(s) causing a transverse crack through the dam embankment in association with a major flooding event. | ▪ Failure from overtopping of Spook Hill FRS.<br>▪ Failure from seepage erosion or piping around principal spillway outlet (7.5 ft by 7.0 ft reinforced concrete box). | ▪ Failure due to seepage erosion along a transverse crack through embankment which ruptures or encounters a defect in the HDPE liner leading to failure). | ▪ Failure due to seepage erosion through a transverse crack in the embankment during a major flooding event.<br>▪ Failure from seepage erosion due to seepage erosion of foundation material along contact. |

# SUMMARY OF FAILURE MODES BY CATEGORY FOR EACH DAM OR FLOOD RETARDING STRUCTURE

| Failure Mode Category | DAM OR FLOOD RETARDING STRUCTURE | | | | | | |
|---|---|---|---|---|---|---|---|
| | Cave Buttes Dam | Powerline FRS | Vineyard Road FRS | Rittenhouse FRS | Spook Hill FRS | Signal Butte FRS | Apache Junction FRS |
| **Category II (con't)** *failure mode. The reason for the lesser significance is noted and summarized in the documentation report or notes.* | ▪ Failure from cracking due to geometry / settlement / shrinkage – main dam abutments and Dike No. 1 at Station 35+60 and Dike No. 2 At Station 50+16.<br>▪ Failure from emergency spillway erosion.<br>▪ Failure from piping of Zone II material into the foundation – Training Dike No. 1.<br>▪ Failure from internal instability of Zone II materials – Training Dike No. 1.<br>▪ Failure From Potential Fissure From Subsidence – Training Dike No. 2.<br>▪ Failure from potential piping from utility crossings at Cave Creek Road – Training Dike No. 2. | | ▪ Failure from piping around principle spillway (54-inch reinforced concrete pipe) and five abandoned irrigation outlets (three 24-inch and two 18-inch rcps).<br>▪ Failure from seepage erosion due to flow/seepage erosion through animal burrows. | ▪ Failure from seepage erosion and erosion tunnel developing around gated irrigation outlets (two 24-inch gated outlets).<br>▪ Failure from potential embankment slope instability.<br>▪ Failure from seepage erosion or piping due to a flow path developing through the foundation.<br>▪ Failure from seepage erosion due to flow/seepage erosion through animal burrows. | ▪ Failure from slope erosion leading to dam breach in association with major flooding event and rain at the dam.<br>▪ Failure from piping along base of the dam (lower portion between original ground and foundation).<br>▪ Failure from seepage erosion along buried utilities. | | |

# SUMMARY OF FAILURE MODES BY CATEGORY FOR EACH DAM OR FLOOD RETARDING STRUCTURE

| Failure Mode Category | DAM OR FLOOD RETARDING STRUCTURE | | | | | | |
|---|---|---|---|---|---|---|---|
| | Cave Buttes Dam | Powerline FRS | Vineyard Road FRS | Rittenhouse FRS | Spook Hill FRS | Signal Butte FRS | Apache Junction FRS |
| **Category III**<br>*More Information or Analyses are Needed in order to Classify – These potential failure modes, to some degree, lacked information to allow a confident judgement on significance and thus a dam safety investigative action or analyses can be recommended. Because these actions are required before resolution, these may also be highlighted.* | ▪ Potential Cave Creek Dam failure cause a problem at Cave Buttes Dam.<br>▪ Failure From Overtopping Training Dike No. 2 From Station 103+60 To 140+00. | ▪ Failure from seepage erosion along the dam foundation contact or through the foundation. | ▪ No Category III potential failure modes were identified by the FMCA team for Vineyard Road FRS at this time. | ▪ No Category III potential failure modes were identified by the FMCA team for Rittenhouse FRS at this time. | ▪ No Category III potential failure modes were identified by the FMCA team for Spook Hill FRS at this time. | ▪ No Category III potential failure modes were identified by the FMCA team for Rittenhouse FRS at this time. | ▪ No Category III potential failure modes were identified by the FMCA team for Apache Junction FRS at this time. |
| **Category IV**<br>*Failure Mode Ruled Out Potential failure modes may be ruled out because the physical possibility does not exist, information came to light which eliminated the concern that had generated the development of the potential for the failure mode, or the failure mode is clearly non-credible or reasonable to postulate.* | ▪ Training Dike No. 1 failure modes:<br>➢ Zone III does not meet filter criteria for Zone II.<br>➢ Shrinkage Cracking<br>▪ Training Dike No. 2 failure modes:<br>➢ Unauthorized installation of underground utilities.<br>Training Dike No. 3 failure modes: none identified. | ▪ Failure from piping around principle spillway – 36-inch reinforced concrete pipe.<br>▪ Failure from potential seismic activity impacting structure. | ▪ Failure from potential erosion of emergency spillway during spillway discharges.<br>▪ Failure from potential seismic activity impacting structure. | ▪ Failure from seepage erosion around principal spillway (33 inch reinforced concrete pipe).<br>▪ Failure from potential erosion of emergency spillway during spillway discharges.<br>▪ Failure from potential seismic activity impacting structure. | | ▪ Failure from overtopping the dam from extreme flood events. | ▪ Failure from seepage erosion due to a transverse crack adjacent to emergency spillway (at embankment/spillway contact).<br>▪ Failure from potential embankment slope instability. |

# SUMMARY OF FAILURE MODES BY CATEGORY FOR EACH DAM OR FLOOD RETARDING STRUCTURE

| Failure Mode Category | DAM OR FLOOD RETARDING STRUCTURE | | | | | | |
|---|---|---|---|---|---|---|---|
| | Cave Buttes Dam | Powerline FRS | Vineyard Road FRS | Rittenhouse FRS | Spook Hill FRS | Signal Butte FRS | Apache Junction FRS |
| **Other Considerations** | | | | | These issues were discussed but a potential failure mode was not identified for evaluation.<br>1. Emergency Spillway<br>2. Seismic<br>3. Fissures and Differential Settlement<br>4. Animal/Vegetation<br>5. Settlement/Erosion of Holocene Soils<br>6. Slope Stability<br>7. ADOT Freeway cyclic loading<br>8. Dam is sectioned | These issues were discussed, but a potential failure mode was not identified for evaluation.<br>1. Upstream impacts<br>2. Slope stability<br>3. Settlement, subsidence, and fissuring<br>4. Excavation for Holocene soils<br>5. Potential erosion along foundation contact<br>6. Piping around outlet pipes<br>7. Emergency spillway transverse crack<br>8. Seismicity<br>9. Piping Dam Breach<br>10. Pass Mountain Diversion Dike | The following issues were discussed, but a failure mode was not identified.<br>1. Fissure developing below dam embankment.<br>2. Seepage erosion due piping around ATT cable.<br>3. Seepage erosion due to piping at abandoned 4-inch waterline.<br>4. Seismic activity in conjunction with impoundment.<br>5. No failure modes were found to be associated with the principal spillway and emergency spillway. |

# District Actions and Potential Risk Reduction Measures Identified from Failure Modes and Consequences Analysis

## CAVE BUTTES DAM

| District Action | Potential Risk Reduction Measures |
|---|---|
| 1. As-Built Utility Crossings. There is an identified need to put the as-built details for utility crossings (of Dike No. 2 at Cave Creek Road) on the as-built drawings of Dike No. 2. The details should include type of utility, depth of embedment, piping countermeasures, location, size of utility, and any field markers indicating the location of the utility in the field. | 1. Monitoring During Flooding. As a risk reduction measure, the FMCA team identified specific locations to monitor during flood event. These include the abutments of the main dam and Training Dikes No. 1 and No. 2. |
| 2. Impoundment Documentation. Need for good recording of high water both in stage and in elevation, including times and dates of high water in reservoir. The reservoir staff gages should be incremented in 1/10th foot intervals for elevations within five feet of the emergency spillway crest. The reservoir stage (elevation, time and date) should be recorded accurately during storm events and at frequent intervals when the reservoir is rising quickly. | 2. Monitor left abutment seepage in downstream groin during flooding events. |
| 3. Estimate Incipient Damaging Discharge. The threshold of damage flow from emergency spillway discharge floods needs to be estimated. | 3. Monitor right abutment (groin and rock slope) for any seepage escaping through joints. |
| 4. Repair Depression on Dike No. 2. Training Dike No. 2 depression area on upstream slope at Station 64+80 should be repaired. Kimley-Horn and Associates (June 2000) provided an outline of a repair plan in the Individual Structures Assessment Report. | 4. Monitor downstream slopes and toe of Dike No. 2 during flooding events/impoundment. |
| 5. Monitoring During Flooding. The FMCA team identified specific locations to monitor during flood event. These include the abutments of the main dam and Training Dikes No. 1 and No. 2. | 5. Monitor downstream toe during regular inspections and during flood events. |
| | 6. Use Peck observational method, which is to construct downstream weir at end of groin. Use weirs instead of flumes. Flumes pass sediment and weirs do not. |
| | 7. Monitor during inspections and during flooding events at principal outlet headwall. |
| | 8. Pave top of dam at abutments. Cracks of any significant size would likely promulgate through pavement. |
| | 9. Determine the estimated threshold flow that would result in damaging flows from the spillway to include in emergency preparedness plan. |
| | 10. Close observation during flooding events to pick up any seepage. |

## CAVE BUTTES DAM

| District Action | Potential Risk Reduction Measures |
|---|---|
| | 11. Monitor area for ground subsidence. |
| | 12. Evaluate groundwater levels in region. |
| | 13. Put utilities on Cave Buttes Dam as-builts. |
| | 14. Locate and mark utility locations in field. |
| | 15. Note utility contact persons in case of emergency. |
| | 16. Note procedure and method of isolating water line segments under dam. |
| | 17. Monitor Cave Creek Dam during flooding events. |
| | 18. There is additional analysis that could be conducted to provide better insight on the potential failure modes and consequences of failure to Cave Creek Dam. Additional analysis includes a dambreak evaluation of Cave Creek Dam and the resulting flood wave analysis on Cave Buttes Dam. The Bureau of Reclamation has a computer program "LS Wave" that may be suitable for this type of analysis. |
| | 19. Alternative Analysis Report by Kimley-Horn and Associates, Inc. (August 2000) identified potential secondary outlet in Dike No. 2 located near left abutment. |

# District Actions and Potential Risk Reduction Measures Identified from Failure Modes and Consequences Analysis

## POWERLINE FLOOD RETARDING STRUCTURE

| District Action | Potential Risk Reduction Measures |
|---|---|
| 1. Nature of Earth Fissure Risks. The weight of evidence indicates that earth fissure development is continuing in the vicinity of Powerline FRS and there is a plausible scenario in which an earth fissure could cross the alignment of the dam some time in the future. Investigations of earth fissuring in the vicinity of Powerline Dam are ongoing. Conclusions and recommendations from these investigations are forthcoming. For the interim period, it is recommended that known earth fissures in the vicinity of the dam are routinely monitored under the District's Dam Safety Program. | 1. (1) Keep the crest of the dam level and at the design elevation; (2) Monitor the impoundment levels during storm events; (3) Provide training to O&M crews regarding earth fissure development and other key observational parameters such as seepage erosion failure processes; (4) Operation of irrigation outlets – close gates except as needed. |
| 2. Identification of Fissure Development. Train District O&M staff on how fissures develop and to identify fissures in the field. | 2. Confirm crest profile elevation. |
| 3. Need Crack Inspections Records and Mapping And Soil Testing Data Done For Powerline. It is of critical importance to find the records of documentation of cracks by the SCS in the inspection of the central filter trench during construction in 1989. | 3. Verify capacity of Powerline FRS. |
| 4. Lack of Stability Analyses. The FMCA Team recommends that slope stability analyses for the embankment and the central filter be documented. A record of a stability analysis for the original design or the modifications has not been found. | 4. Confirm hydrology of Powerline FRS (100-yr and PMF). |
| 5. Confirm Hydrologic Routings Through the Reservoir. The inflows and outflows for the full PMF, ½ PMF, and 100-year events and the consequences of the outflows downstream should be confirmed. (Note: It is recommended that the analysis include the 250 yr., 500 yr., etc. to define the threshold recurrence interval for overtopping and provide input for incremental damage analysis.) | 5. Recommend a joint agency fissure/subsidence-monitoring program. |
| | 6. Segment the dam into smaller dams. |
| | 7. Eliminate the north 40% of dam. |
| | 8. Inspections for fissure expression after rain events to include not only dam but impoundment area and areas downstream of the dam. |
| | 9. Need crack mapping and soil moisture change information for Powerline. |
| | 10. Conduct soils properties evaluation. |
| | 11. Monitor irrigation outlet and inspect during regular inspections and during flood events. |
| | 12. Keep slide gate on irrigation outlet closed until needed (this would prevent development of precursor seepage paths developing from leakage out of the culvert flowing along the sides of the culvert). |

## POWERLINE FLOOD RETARDING STRUCTURE

| District Action | Potential Risk Reduction Measures |
|---|---|
| | 13. Video inspection of interior of CMP pipe. |
| | 14. Monitor emergency spillway performance during flood events. |
| | 15. Monitor training dikes and left abutment during flood events. |
| | 16. Conduct embankment slope stability analyses. |
| | 17. Need to evaluate existence of Holocene soils in foundation of dam. |
| | 18. Need to prepare crack mapping and find NRCS crack report. |
| | 19. Remedial measure is to add drain to central filter. |
| | 20. Video interior of principal spillway pipe. |
| | 21. Monitor principal spillway outlet and inspect during regular inspections and during flood events. |
| | 22. Examine and inspect dam and outlets and surrounding ground after earthquake (greater than a minimum of 0.05g) and high potential fissure areas. |

# District Actions and Potential Risk Reduction Measures Identified from Failure Modes and Consequences Analysis

## VINEYARD ROAD FLOOD RETARDING STRUCTURE

| District Action | Potential Risk Reduction Measures |
|---|---|
| 1. (1) Ensure that the crest of the dam is kept level at design elevation; (2) Continue monitoring the impoundment levels during storm events; (3) Provide training to O&M crews regarding identification of earth fissure development and other key observational parameters such as evidence of seepage erosion processes; (4) FMCA recommends that slope stability analyses for the embankment and the central filter be documented. A record of a stability analysis for the original design or the modifications has not been found. | 1. Confirm dam crest profile elevation.<br>2. Verify capacity of Vineyard Road FRS.<br>3. Raise and level crest as necessary to return to design elevation.<br>4. Construct the buffer zone (area between downstream toe of dam and CAP) and CAP canal to receive and pass discharges from overtopping of Vineyard over the side bank of the canal if necessary.<br>5. Recommend a joint agency fissure/subsidence-monitoring program.<br>6. Segment dam into smaller dams.<br>7. Training O&M crews to know signs of fissure development.<br>8. Because small floods may initiate fissuring and thus provide advance warning of the potential presence of a fissure under the dam, inspections after rain events should include not only the dam but also the impoundment area and the areas downstream of the dam.<br>9. Adjust easement further out to control development.<br>10. Groundwater recharge instead of pumping.<br>11. Fissures should be monitored if discovered.<br>12. Employ CAP canal as a visual indicator of developing fissure.<br>13. Evaluate the need for finger drains.<br>14. Monitor cracks and develop crack mapping program. |

District Actions and Potential Risk Reduction Measures Identified
from Failure Modes and Consequences Analysis

## VINEYARD ROAD FLOOD RETARDING STRUCTURE

| District Action | Potential Risk Reduction Measures |
|---|---|
| | 15. Train O&M crews on mechanism of transverse and longitudinal crack formation. |
| | 16. Need to evaluate existence of Holocene soils in foundation of dam. |
| | 17. Monitor emergency spillway performance during flood events. |
| | 18. Monitor training dikes and abutments during flood events. |
| | 19. Conduct embankment slope stability analyses. |
| | 20. Watch for slope distortion - integrate monument data. |
| | 21. Confirm design slope by survey. |
| | 22. Video interior of principal spillway pipe. |
| | 23. Monitor principal spillway outlet and inspect during regular inspections and during flood events. |
| | 24. Locate and mark abandoned irrigation outlets on embankment slopes. |
| | 25. Verify plugging of irrigation pipes. |
| | 26. Continue animal abatement and repair program. |
| | 27. Monitor emergency spillway performance during flood events. |
| | 28. Monitor training dikes and left abutment during flood events. |
| | 29. Examine and inspect dam and outlets and surrounding ground after earthquake (greater than a minimum of 0.05g) and high potential fissure areas. |

District Actions and Potential Risk Reduction Measures Identified
from Failure Modes and Consequences Analysis

# RITTENHOUSE FLOOD RETARDING STRUCTURE

## District Action

1. (1) Keep the crest of the dam level at the design elevation ; (2) Monitor the impoundment levels during storm events (this includes both remote and as well as on-site monitoring); (3) Provide training to O&M crews regarding identification of earth fissure development and other key observational parameters such as indications of seepage erosion; (4) Operation of irrigation outlets – close gates except as needed.
2. Confirm Utility Crossing. The details of the telephone utility crossing of dam at Station 141+00 should be confirmed. Document on dam as-built drawings.
3. Lack of Material Testing on Key Parameters. The FMCA team has found that there is relatively few grain size analysis and Atterberg limits test available for the embankment fill and borrow. Thus, the characteristics of the embankment material are uncertain.
4. Lack of Stability Analyses. The FMCA team recommends that slope stability analyses for the embankment and the central filter be documented. A record of a stability analysis for the original design or the modifications has not been found.

## Potential Risk Reduction Measures

1. Confirm dam crest profile elevation.
2. Verify capacity of Rittenhouse FRS.
3. Evaluate the role of the CAP canal and the Queen Creek Wash in the spillway discharge and dam breach scenarios.
4. Monitor cracks and develop crack mapping program.
5. Train O&M crews to on mechanism of transverse and longitudinal crack formation.
6. Need to evaluate existence of Holocene soils in foundation of dam.
7. Evaluate finger drains.
8. Monitor emergency spillway performance during flood events.
9. Monitor training dikes and abutments during flood events.
10. Extend training dike beyond main section of the dam.
11. Recommend a joint agency fissure/subsidence-monitoring program.
12. Segment dam into smaller dams.
13. Training O&M crews to know signs of fissure development.
14. Inspections after rain events to include not only dam but impoundment area and downstream areas of dam.
15. Adjust easement further out to control development.
16. Groundwater recharge instead of pumping.
17. Earth fissure can be monitored if discovered.
18. CAP canal is a good indicator of visualization of existing fissure.

## RITTENHOUSE FLOOD RETARDING STRUCTURE

| District Action | Potential Risk Reduction Measures |
|---|---|
| | 19. Small floods may provide warning for earth fissure expression. |
| | 20. Video interior of irrigation pipes. |
| | 21. Monitor irrigation outlets and inspect during regular inspections and during flood events. |
| | 22. Close the gates under normal operations. Open only when District decides to provide water to downstream. |
| | 23. Conduct slope embankment stability analyses. |
| | 24. Watch for slope distortion - integrate monument data. |
| | 25. Confirm design slope by survey. |
| | 26. Need to evaluate existence of Holocene soils in foundation of dam. |
| | 27. Continue animal abatement and repair program. |
| | 28. Video interior of principal spillway pipe. |
| | 29. Monitor principal spillway outlet and inspect during regular inspections and during flood events. |
| | 30. Upsize diameter of outlet pipe. |
| | 31. Place filter material around pipe. |
| | 32. Run SITES model by NRCS to evaluate spillway performance. |
| | 33. Examine and inspect dam and outlets and surrounding ground after earthquake (greater than a minimum of 0.05g) and high potential fissure areas. |

District Actions and Potential Risk Reduction Measures Identified
from Failure Modes and Consequences Analysis

# SPOOK HILL FLOOD RETARDING STRUCTURE

## District Action

1. Buried Stream Channels under Dam. Relatively deep excavation of the Holocene soils was done at four buried stream channels to reach the strongly cemented deposits. These excavations were about 25 feet at Station 222+00, 15 feet at Station 242+00, 15 feet at Station 266+00 and 30 feet at Station 280+00. The SCS quality control testing from the original construction indicates that the fill was compacted to a mean of 98.7 percent of ASTM D698 maximum dry density and mean moisture content of 1.3 percent below the optimum moisture content. The fill appears to be of high quality and not susceptible to significant settlements and cracking when wetted. However special observation and review of monitoring data should be done in the areas of deep localized sub-excavation to confirm that that there are no significant differential settlements or cracks.

2. Utility Crossings. The details and locations of utility crossings should be added to the as-built plans of the dam.

3. Right-of-Way Permit Process. There is a need to improve right-of-way process to ensure the dam is not inappropriately "invaded". The FMCA team identified a need to improve right-of-way permit process pertaining to as-builts for modifications by others such as utilities.

4. Clear Zones Upstream and Downstream Toes. The FMCA team identified a need for clear zones at upstream and downstream toes for emergency response and maintenance.

## Potential Risk Reduction Measures

1. Prepare emergency action plan according to minimum standards in FEMA 64 and ADWR rules and regulations.
2. Increase principal spillway outlet capacity to Spook Hill floodway.
3. Restore CAP canal bank at emergency spillway on opposite bank to full height.
4. Add other outlets to discharge to CAP canal.
5. Monitor cracks and develop crack mapping program.
6. Train O&M crews to on mechanism of transverse and longitudinal crack formation.
7. Repair any significant cracks upon their development. ← procedure?
8. Evaluate need for central filter.
9. Confirm crest profile elevation. Keep crest level.
10. Verify capacity of Spook Hill FRS.
11. Verify hydrology for PMF.
12. Inspect for seepage outside of principal spillway outlet box area on downstream.
13. Place elevation staff gage at inlet.
14. Set limits for repair of slopes - develop slope repair guidelines.
15. Continue seeding of slopes.
16. Consider rock mulch at groins.

District Actions and Potential Risk Reduction Measures Identified
from Failure Modes and Consequences Analysis

## SPOOK HILL FLOOD RETARDING STRUCTURE

| District Action | Potential Risk Reduction Measures |
|---|---|
| 5. Animal Burrows. It was concluded that animal burrows at this site is not a dam safety issue due to shallow depths of penetrations and primarily located on talus at toes and landscape soil layer. In addition the District O&M Division conducts an ongoing animal control program and animal borrow repair program. | 17. As long as slope erosion gully does not invade crest performance should be fine.<br>18. Monitor and observe downstream slope and toe during regular inspections and during flood events.<br>19. Identify and locate (including depth) of all buried utilities. Put on as-builts of dam.<br>20. Improve right-of-way process.<br>21. See KHA report "Policy and Program Report" and recommendations regarding utility crossings. |

District Actions and Potential Risk Reduction Measures Identified
from Failure Modes and Consequences Analysis

# SIGNAL BUTTE FLOOD RETARDING STRUCTURE

## District Action

1. No Individual EAP. An individual Emergency Action Plan needs to be developed according to ADWR and FEMA 64 criteria.
2. As-Builts Needed For Cutoff Wall Below Emergency Spillway. Documentation states that during construction a cutoff wall was constructed below the emergency spillway. This documentation is in the ADWR construction inspection reports. However, the cutoff wall is not reflected on the As-built Plans.
3. Need construction Quality Assurance test results on HDPE lining.
4. Crack Monitoring. Monitor for development of longitudinal cracks and be aware of future cracks.
5. Slope Stability. Slope stability analysis needs to be conducted for record conditions.
6. Field Inspections. Locations for special inspection for settlement cracks should be identified. These are the areas of localized deep sub-excavation of the Holocene soils to the contact of the strongly cemented calcareous soils.
7. Staff Gage. It was observed by the FMCA team that the principal outlet does not have a staff gage, but the gated outlet does.

## Potential Risk Reduction Measures

1. Prepare emergency action plan according to minimum standards in FEMA 64 and ADWR rules and regulations.
2. Use lower level principal spillway outlets at Signal Butte during flood event.
3. Increase principal spillway outlet capacity to Spook Hill FRS.
4. Monitor cracks and develop crack mapping program.
5. Train O&M crews to on mechanism of transverse and longitudinal crack formation.
6. Repair cracks at dam.
7. Locate seam welding QC results.
8. Could protect any penetrations into the dam with filter material downstream of the penetration.
9. Lower principal spillway outlet opening.
10. Maintenance activities are good.
11. Evaluate flood routing with sediment pool available.

District Actions and Potential Risk Reduction Measures Identified
from Failure Modes and Consequences Analysis

## APACHE JUNCTION FLOOD RETARDING STRUCTURE

### District Action

1. Repair of transverse cracks on case-by-case basis. Repair of longitudinal crack (repair/seal).
2. Inspection procedures for event related inspections (during and post-event).
3. Extend central filter through the collapsing Holocene soils to the underlying cemented deposits between Stations 44+25 and 52+50.
4. Stability Analyses. It is recommend that further stability analyses be conducted for the record.
5. Utility Crossings. Need proper documentation of utility crossings. No as-built plan drawings were found for the 4-inch abandoned waterline line located at Station 85+24.
6. O&M Plan. The Operations and Maintenance Plan should be updated.

### Potential Risk Reduction Measures

1. Prepare emergency action plan according to minimum standards in FEMA 64 and ADWR rules and regulations.
2. Monitor cracks and develop crack mapping program.
3. Train O&M crews to on mechanism of transverse and longitudinal crack formation.
4. Repair cracks.
5. Train operation and maintenance personnel and inspectors to know where and what to observe to identify seepage erosion.
6. Inspect and monitor during event and post-event.
7. Monitor emergency spillway discharges.
8. Update stability analyses to actual cases.
9. Watch for slope distortion - integrate monument data.
10. Confirm design slope by survey.
11. Survey crest at bulge stations.
12. Install drains to central filter.

# FAILURE MODE AND CONSEQUENCE ANALYSIS
# For
# CAVE BUTTES DAM

## PHOENIX, ARIZONA
## NATDAMID AZ 1007

**June 12, 2001**

## 1.0 INTRODUCTION

### General Description

Cave Buttes Dam is located on Cave Creek Wash in Maricopa County, Arizona. Cave Buttes Dam is located near Cave Creek Road about 17 miles north of downtown Phoenix and less than a mile downstream of the existing, but no longer operational, Cave Creek Dam. The project consists of the main dam structure, a detached emergency spillway, a principal spillway, and three training dikes (see Individual Structures Assessment Report prepared by Kimley-Horn and Associates, Inc., June 2000).

Cave Buttes Dam is classified as a large, high hazard dam. The construction contract was awarded to Washington Construction Company of Missoula, Montana. Construction was completed in March 1980. The U.S. Army Corps of Engineers provided construction supervision. The drainage area for Cave Buttes Dam is 191 square miles.

Seepage was noted in the left groin of Cave Buttes Dam on January 14, 1993 in association with the impoundment of January 1993. Photographs attached to the District memorandum dated June 7, 1999 show the seepage at the east edge of the grouted rip-rap groin, left abutment, approximately 45-feet above the downstream toe of the slope. The water was clear with no traces of sediment.

Aside from this incident, the dam has performed satisfactorily to date.

### Dam Data

Dam type: Zoned earth fill (main dam and dikes no. 1 and 2)
Dam height: 109-ft above streambed (main dam)
Dam length: 2,275-ft (main dam)
Dam crest: width 20-ft; elevation 1679.1-ft
Spillways: Principal - 45-inch RCP in concrete cradle; inlet elevation 1560.7-ft;
Emergency spillway - 540-ft wide bedrock lined spillway; crest elevation 1657.1-ft
Freeboard: 5.0-ft
Reservoir Surface: 1,820-ac at spillway crest
Storage: 46,600-af at spillway crest
Hazard Classification: High

**Purpose and Scope**

In general, the purpose of the Failure Mode and Consequence Analysis (FMCA) exercise was to:

- Identify potential site-specific failure modes for the dam.
- Discuss non-quantitatively the likelihood of the identified failure modes.
- Determine whether or not and how important failure mechanisms are being monitored
- Examine the consequences of failure and the adverse consequences of successful operation (e.g. – large spillway releases)
- Identify possible risk reduction actions that may be taken to reduce the likelihood of failure or to mitigate adverse consequences
- Determine what information, investigations or analyses may be needed to resolve uncertainties relative to potential failure modes.

(Note: In this phase, the FMCA team only examined the general nature of the "consequences" for the failure modes identified and where appropriate estimated how they may be different than previously anticipated. Greater detail on the estimate of the magnitude of the "consequences" for each significant failure mode will be addressed in the quantitative portion or risk analysis part of the risk assessment for the dam. At this time it is anticipated that more detailed discussion on the emergency response plan for each structure will also be possible.)

**Team Members**

Larry Von Thun, Engineering Consultant and FMCA facilitator
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Ken Euge, Geological Consultants, Principal
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Larry Lambert, Flood Control District of Maricopa County, Dam Safety Engineer
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Justin Beeler, Kimley-Horn and Associates, Inc, Session Recorder
Tim Murphy, Flood Control District of Maricopa County, Civil Engineer (consequences)

## 2.0 MAJOR FINDINGS AND UNDERSTANDINGS GAINED

The following is a summary of the major findings and understandings for Cave Buttes Dam as a result of the Failure Mode and Consequence Analysis. The major findings and understandings given below are organized as follows. First the most important design, construction, geologic, and performance differences or unique aspects related to the potential for failure mode development of Cave Buttes Dam are given. Findings related to identification of potential failure modes are given first. Then findings related to consequences are given. Thirdly, general findings and observations made during the analysis are listed. Lastly, identified risk reduction measures and action items for Cave Buttes Dam are provided. The "Embankment Criteria and Performance Report, U.S.

Army Corps of Engineers, March 1981" hereafter referred to as the "Performance Report" and construction photographs were the principal documents available to the FMCA Team. See Cave Buttes Dam - Appendix for cross sections of the Dam and Dikes 1, 2 and 3.

**Key Findings Related To Failure Mode Development – "Static Loading Failures – Seepage Erosion – Foundation"**

- Treatment Of Foundation Joints in the Main Dam. The joints and depressions in the rock foundation beneath Zone I of the main dam were apparently treated (FMCA team assumed per documents treatment was slush grout, filling or covering the joints) but subsequent investigations exposed untreated joints illustrating that not all joints were treated. Documentation of this finding is from the report titled " Embankment Criteria and Performance Report, U.S. Army Corps of Engineers, March 1981". The "Foundations Report" cited in the Performance Report was not available (See below "Additional Reports"). The presence of untreated joints in the foundation allows direct access of water at or near reservoir head to come in contact with the core materials essentially immediately upon filling.
- Seepage In Left Abutment Groin. The seepage water that has appeared on the left abutment groin (in a previous reservoir filling) is almost certainly passing through foundation jointing (because of its rapid appearance in association with the reservoir filling). If the flow is small enough such that it does not erode the downstream shell materials or if the flow can be controlled without eroding the shell materials at the groin and if the flow is not carrying any embankment material it would not constitute a failure mode. To verify that these conditions remain under control this seepage needs to be closely monitored during significant filling episodes for presence of fines (a weir could be placed to capture and monitor this seepage), quantity of flow as well as time and location of onset of seepage determined (Photographs of the groin seepage are provided in the Cave Buttes Dam appendix of this section).
- Zone III Does Not Meet Filter Criteria for Zone II. In the main dam and in training dikes 1 and 2, Zone III material was designed by the Corps to act as a filter for Zone II material based on the data and analysis in the geotechnical criteria report for Cave Buttes Dam. This report is titled "Embankment Criteria and Performance Report, U.S. Army Corps of Engineers, March 1981". The District conducted a filter analysis (see report "Filter Analysis & Evaluation of Properties of Zones I & II" – December 2001 in Appendix of Cave Buttes Dam of this section) for Zone III and has concluded that portions of Zone III does not meet current filter criteria for Zone II.
- Susceptibility to Shrinkage Cracking. All Zone I and some of Zone II materials are considered susceptible to shrinkage cracking and brittle fractures (because of the amount of fines and clay content) in response to differential settlement and possibly hydraulic fracturing processes. It is also likely that these soils have experienced drying shrinkage cracking to depths up to about 20 feet below crest of dam. The rational for this opinion is based on the weight of evidence experienced at Vineyard Road FRS. However nothing has been observed to date at Cave Buttes Dam.
- Short Detention Duration. The detention/retention time for water in the reservoir is relatively short (on the order of an average of 15 days from elevation 1642 feet). This

is an important factor in judging failure mode development likelihood. The team considered that only static/seepage related failure modes that could be significant (highlighted) were those where the reservoir water had an direct path to the potential vulnerable location immediately upon filling (open crack or open joint). Thus the potential failure modes, which require appreciable time (months to years) for flow path or phreatic surface development are all judged to be considered but not highlighted. (Note: the question of the effect of multiple floods in close sequence is of concern and should be studied)

- Seismic Risk. A seismic exposure evaluation completed by AMEC indicates the peak horizontal acceleration (PHA) created by the maximum credible earthquake (MCE) is only about 0.10g (reference AMEC report, December 2001). There is a very low probability of slope instability due to seismic loading. The likelihood of an earthquake simultaneously with a significant impoundment is considered very remote. Seismic loading is considered to be insufficient to threaten the integrity of any of the Cave Buttes structures (main dam and training dikes).
- Subsidence. The potential for subsidence (settlement) related embankment cracking is judged to exist at Dike No. 2 based on as-built construction drawings of the foundation bedrock profile and a discontinuity in the bedrock surface at approximately Station 37+00.
- Potential Emergency Spillway Erosion. Based on a review of the as-built construction drawings, the emergency spillway has a potential for erosion during discharge of floodwater through the spillway (loss of materials on the sides of the channel and the potential for headcutting from the downstream end toward the emergency spillway sill and reservoir during discharge). The potential emergency spillway discharges is approximately the 250-year flood.
- Low Piping Potential for Principal Spillway. The principal spillway outlet works was excavated into rock and encased with concrete but a portion is underlain by up to approximately 12 feet of lean concrete which was backfilled as a positively projecting pipe (pipe extended above grade). There is thus, potential for a seepage erosion failure along the entire principal spillway conduit although a seepage pathway passing along a part of the conduit is considered low. There appears, based on this construction, to be a very low potential for piping failure. This is based on review of as-built drawings and the Cave Buttes Dam construction photographs.
- Foundation Joints Not Treated. The bedrock foundation joints beneath the core were not treated at Dike No. 1 and at Dike No. 2 abutments. (At Dike No. 1 there is the possibility of direct evidence of piping occurring based on an inspection photographs - see Appendix of this report and Individual Structures Assessment Report by Kimley-Horn and Associates for Cave Buttes Dam – dated June 2000). There is a potential for piping at Dike No. 1 and at Dike No. 2 abutments because the foundation grouting was not as complete as is standard practice today.
- Potential Embankment Cracking. A review of the as-built construction photographs, plans and profiles of the main dam show the existence of abrupt changes in rock at main dam right abutment, Dike No. 1 and Dike No. 2 east side. These are judged to be locations of potential differential settlement, hydraulic fracturing and embankment cracking. A seepage erosion failure passing through embankment cracks, loose zones at the foundation interface and defects in Zone III is potentially possible.

## Consequences

- Dam Breach. A realistic dam breach outflow from a seepage erosion type failure needs to be performed for the main dam and the breach parameters need to be reviewed for use in the risk analysis. The reservoir elevation for this breach should be established at the spillway crest because it is not realistic for a failure to develop in the time it would take to release any reservoir head higher than the high capacity emergency spillway. Once verified the incremental damage assessment from an assumed failure needs to be made for this outflow condition.
- Need for Extended Consequence Evaluations. The consequence estimations to date only extend to the CAP facilities, for the risk analysis the total damage estimate needs to be made.
- Dike No. 2 Consequences Different from Main Dam. The failure of Dike 2 has different downstream consequences than does the main dam. These consequences need to be determined in general and for the risk analysis.

## General Findings

- Documentation. It was noted that it is critical from a monitoring and potential failure mode evaluation standpoint to document what to look at and what has changed (e.g., utilities at Cave Creek Road - did not get on as-builts).
- Settlement History and Future Monitoring- From interpretation of record crest surveys, evidence suggests that minor settlement has been measured to date. This is reasonable and to be expected since the dam has not yet undergone saturation. It is important to continue to monitor closely in future especially after flood events. It is important to survey the dam and dikes regularly to monitor for general settlement and any differential settlement.

## Action Items/Risk Reduction

- As-Built Utility Crossings. There is an identified need to put the as-built details for utility crossings (of Dike No. 2 at Cave Creek Road) on the as-built drawings of Dike No. 2. The details should include type of utility, depth of embedment, piping countermeasures, location, size of utility, and any field markers indicating the location of the utility in the field.
- Impoundment Documentation. Need for good recording of high water both in stage and in elevation, including times and dates of high water in reservoir. The reservoir staff gages should be incremented in $1/10^{th}$ foot intervals for elevations within five feet of the emergency spillway crest. The reservoir stage (elevation, time and date) should be recorded accurately during storm events and at frequent intervals when the reservoir is rising quickly
- Estimate Incipient Damaging Discharge. The threshold of damage flow from emergency spillway discharge floods needs to be estimated.
- Repair Depression on Dike No. 2. Training Dike No. 2 depression area on upstream slope at Station 64+80 should be repaired. Kimley-Horn and Associates (June 2000) provided an outline of a repair plan in the Individual Structures Assessment Report.

- Monitoring During Flooding. As a risk reduction measure, the FMCA team identified specific locations to monitor during flood event. These include the abutments of the main dam and Training Dikes No. 1 and No. 2.

## 3.0 POTENTIAL FAILURE MODES

Potential failure modes identified by the FMCA team are presented below. The failure modes were placed into one of four categories as follows. Except in special cases, only those potential failure modes in Category I will be considered in the risk (quantitative) analysis phase of the risk assessment exercise.

- Category I – Failure modes of greatest significance (highlighted for risk analysis.
- Category II – Failure modes of lesser significance (but not inconsequential).
- Category III – Failure modes for which insufficient information is presently available to allow the team to make a judgement on the significance of the failure mode. The development of additional data and information is warranted. Additional records research may be justified.
- Category IV – Failure modes which are not physically possible or which are clearly not credible.

For each of the potential failure modes identified, a problem statement is briefly described and the factors that make the failure mode more likely (adverse factors) or less likely (positive factors) to occur are listed following the problem statement (except Category IV). In addition, any identified potential actions for risk reduction for each potential failure are then provided.

## CATEGORY I - FAILURE MODES OF GREATEST SIGNIFICANCE

### *Failure From Piping (Seepage Erosion) Of Embankment Soil Into Joints And Through Joints In The Dam Foundation - Main Dam (Category I)*

Failure Mode Description: Embankment soils may potentially be piped (removed by seepage erosion) along or near the dam foundation contact. Water at or near reservoir head could be present near the Zone 1 core due to open untreated joints in the abutment. This water could cause seepage erosion at the contact or could enter the core materials due to separation zones in the core created by hydraulic fracturing or other (differential movement) trigger event. Such preferential seepage paths through the core would enhance the seepage erosion rate. This failure mode is consistent with the documented seepage in the left abutment during the January 1993 impoundment at the dam (see groin seepage photographs in appendix of this section), the discovery of untreated joints in the abutment, and observations of the abutment topography.

Adverse Factors:

(1) Apparent and documented evidence of open joints flow on the left abutment (see Cave Buttes Dam appendix of this report).

(2) Rock structures are present in foundation profile of main dam (shear zone/distortion).
(3) The steepness of abutment rating by judgment of the team is 5 to 7 (relatively steep slope for abutment – on a scale 1 to 10 with 10 the most critical).
(4) Change in slope on left abutment rock contact.
(5) Possible erosion of Zone II as well as Zone I materials.
(6) Plasticity index relatively low in some dam testing results but generally greater than 15. (See Cave Buttes appendix in this section for the data on the plasticity index test results in the borrow and in the dam.)

Positive Factors:
(1) No direct evidence of open joints.
(2) Little settlement of dam to date.
(3) Plasticity index relatively high in borrow testing and in dam.
(4) Long downstream Zone II section.
(5) Plugging of any crack or joint could result from Zone II materials (See Cave Buttes appendix in this section for gradation information).

Potential Actions for Risk Reduction:
(1) Monitor left abutment seepage in downstream groin during flooding events.
(2) Monitor right abutment (groin and rock slope) for any seepage escaping through joints.

Note – the team originally classified the following failure mode as Category II (considered but not highlighted) but decided that because the failure flood would pass through a different basin that it should also be included in the risk analysis. Also it did meet the teams criteria of having the potential for water at the critical point upon the onset of reservoir filling)

***Failure From Piping Zone II Soil Into Foundation - Training Dike No. 2 (Category I)***

Failure Mode Description: Zone II materials rest directly on untreated joints on the right abutment foundation of Training Dike No. 2. These Zone II materials may be piped through joints in the foundation. (See Cave Buttes appendix in this section for typical cross section of Dike No. 2).

Adverse Factors:
(1) The plasticity index at the location tested at 4 to 10 (See appendix for data on materials testing for Dike No. 2).
(2) Jointing is present and there is a potential for jointing to be open at near foundation surface.

Positive Factors:
(1) There is not enough time to saturate the Training Dike No. 2 embankment materials to cause a pipe. The time of impoundment is short.

(2) The plasticity index from borrow sources is non-plastic to 25.
(3) There is a wide gradation of Zone II materials - plugging of pipe pathway likely.

Potential Actions for Risk Reduction:
(1) Monitor downstream slopes and toe of Dike No. 2 during flooding events/impoundment.

## CATEGORY II – FAILURE MODES OF LESSER SIGNIFICANCE (BUT NOT INCONSEQUENTIAL).

***Failure From Piping Zone II Soil From Dam Into Alluvium At Cutoff Trench - Main Dam (Category II – Considered but not highlighted).***

Failure Mode Description: Potential for seepage path to develop that would pipe Zone II materials into the alluvium on the sidewalls of the cutoff trench.

Adverse Factors:
(1) The upper 15 feet of alluvium is loose (41% relative density, sand, gravel, gravelly sand and silty sand and gravel.
(2) There are possible old permeable remnant channels under main dam. This may provide for a potential seepage path.

Positive Factors:
(1) There is no direct evidence for open cracks/open work gravel in the cutoff trench walls.
(2) The plasticity index of Zone II is 15 – 25, but some low (<10).
(3) The time to develop a seepage path through the dam and into the alluvium does not appear adequate, as impoundment duration is very short.

Potential Actions for Risk Reduction:
(1) Monitor downstream toe during regular inspections and during flood events.

***Failure From Piping Material From The Main Dam Into The Foundation Through Open Joints (Category II– Considered but not highlighted).***

Failure Mode Description: Potential for Zone II and/or Zone I materials to pipe into foundation through open joints.

Adverse Factors:
(1) The joints observed in core and foundation filled with sand.

Positive Factors:
(1) Time - The time to develop a seepage path through the embankment does not appear to be adequate, as impoundment duration is short.

(2) Plasticity index of embankment materials generally indicates good erosion resistance.
(3) Slush grouting of the foundation was reportedly conducted during construction, which would have reduced the number of open joints, (but not all were treated based on later explorations).

Potential Actions for Risk Reduction:
(1) Use Peck observational method, which is to construct downstream weir at end of groin. Use weirs instead of flumes. Flumes pass sediment and weirs do not.

***Failure From Piping Along Outlet Works - Main Dam (Category II– Considered but not highlighted).***

Failure Mode Description: Potential for seepage erosion along principal spillway conduit.

Adverse Factors:
(1) A review of the construction photographs for the principal spillway pipe indicated part of the pipe was not imbedded in rock. Constructed as positively projecting pipe.

Positive Factors:
(1) The outlet works were constructed in rock through Zone I and Zone II.
(2) No evidence of leakage of the outlet works pipe.
(3) Pipe resting on a concrete cradle.

Potential Actions for Risk Reduction:
(1) Monitor during inspections and during flooding events at outlet headwall.

***Failure From Cracking Due To Geometry/Settlement/Shrinkage – Main Dam Abutments And Dike No. 1 At Station 35+60 And Dike No. 2 At Station 50+16. (Category II – Considered but not highlighted)***

Failure Mode Description: Cracking (due to any of the above causes) has the potential to open a direct seepage path that would expose the more vulnerable embankment materials to seepage erosion potential immediately upon reservoir filling.

Adverse Factors:
(1) The dam foundation and abutment contact geometry could allow cracking (note the conditions are not as bad as appears on as-builts due to different vertical and horizontal scales.) However, page 7 of the Embankment Report discusses a 1:1 slope in left abutment and break in slope on right abutment - U.S. Army Corps of Engineers Embankment Performance Criteria Report).
(2) Geometry not continuous throughout the dam.
(3) The dam has not been tested yet - dry for long periods of time.

(4) The foundation materials have not been tested for dispersive soils.
(5) The tensile regime is there already.

Positive Factors:
(1) Time – consideration is will saturation take place to produce settlement? (doubtful saturation will take place).
(2) No significant differential settlement to date (0.2 feet over 125 feet since construction in 1980).
(3) P.I of Zone I and Zone II would resist seepage erosion of material.
(4) Gradation of Zone II would work to plug any crack under erosion process.
(5) Dispersive soils in area/vicinity are rare.

Potential Actions for Risk Reduction:
(1) Pave top of dam at abutments. Cracks of any significant size would likely promulgate through pavement.

***Failure From Emergency Spillway Erosion (Category II – Considered but not highlighted).***

Failure Mode Description: Large discharges in the emergency spillway with high erosive velocities could erode and potentially cause a headcut to form towards the reservoir.

Adverse Factors:
(1) The emergency spillway was excavated in jointed fractured rock.
(2) Discharge flow velocities are high.
(3) Low spots/areas of the spillway were constructed in fill or backfilled.
(4) Drop at the end of the emergency spillway could initiate head-cutting process.

Positive Factors:
(1) Limited spillway service time of seven (7) days during PMF.
(2) Large floods that would involve significant use of spillway is an infrequent storm event (estimated at greater than 250 year return period for any use of spillway).

Potential Actions for Risk Reduction:
(1) Determine the estimated threshold flow that would result in damaging flows from the spillway to include in emergency preparedness plan.

Consequences: The consequences of this failure mode are that the spillway discharges are anticipated to carry material eroded from the spillway (and this could increase downstream damages). The process most likely will not be a rapid development. There is no major anticipated change in flow (discharge), and the reservoir would not be lost if the emergency spillway were to erode as described.

***Failure From Piping Of Zone II Material Into The Foundation - Training Dike No. 1 (Category II – Considered but not highlighted).***

Failure Mode Description: The joints in Training Dike No. 1 foundation were not treated as was done at the Main Dam. Thus it is possible that the Zone II material could pipe through open joints in the rock

Adverse Factors:

(1) The Principal Outlet could clog and result in greater impoundment time allowing saturation of embankment and foundation to develop.
(2) Open joints are more likely and open more widely near the bedrock surface.
(3) Cyclic flooding events (series of back to back storms) could exacerbate the problem by allowing the flow path to begin to develop and then push on with the series of events.
(4) Fill from main dam indicated that the material has a P.I. < 10.
(5) Depressed area at upstream crest shows slump at Station 38+61 (not confirmed by survey).

Positive Factors:

(1) Short impoundment duration - design estimate approximately 15 days at average elevation of 1642 feet.
(2) Injection grouting on 40 foot spacing at 25 feet depth.
(3) Dam loaded in 1993 to elevation of 1635 feet - seepage path needs to develop more rapidly - outflow not very significant.

Potential Actions for Risk Reduction:

(1) Close observation during flooding events to pick up any seepage.

Consequence: If a failure were to occur during the time period, it is reasonable to say the water elevation at Dike No. 1 would not be greater than 1642 feet. Consequently, the discharge for a breach height of 10 feet and a breach width of 50 feet is approximately 4,700 cfs.

***Failure From Internal Instability Of Zone II Materials - Training Dike No. 1 (Category II – Considered but not highlighted).***

Failure Mode Description: Zone II is a widely graded material. If steady seepage were passing through this dam it is possible that the finer fraction would pipe through the coarser fraction.

Adverse Factors:

(1) Gradation of Zone II materials does not (appear to) meet filter criteria for internal instability. (See appendix for filter criteria data).

Positive Factors:

(1) There is not enough time for development of seepage path and gradual removal of material – this would likely take years to remove enough material to produce a collapse of the crest and the impoundment time is short. See Corps of Engineers Design Memorandum No. 3 GM Phase II July 1967.

(2) The cohesiveness of the material may prevent removal of fines even if gradation indicated internal instability.

Potential Actions for Risk Reduction:

(1) None action identified unless dam becomes a storage dam.

***Failure From Potential Fissure From Subsidence - Training Dike No. 2. (Category II – Considered but not highlighted).***

Failure Mode Description: Dike No. 2 rests on alluvial fill and may be subject to local subsidence (although none is experienced at this time). An earth fissure could cause a structural failure of the embankment by opening a crack through the structure. Alternatively the fissure could potentially undermine the dam embankment by abruptly causing separation of structure at the foundation with the dam bridging the fissure. If the fissure occurred in association with a large flood the structural failure from the fissure would provide a path for a seepage erosion breach. Potentially there will not be a high level of water impounded during fissure expression. In such a case there would be a minimal loss of water. This failure mode was considered by the FMCA team to be a very low potential threat to Dike No. 2.

Adverse Factors:

(1) Of all the Cave Buttes embankments, Training Dike No. 2 is the most susceptible.

(2) Earth fissure could develop in response to initial major flow into the basin of this dike (which has not yet occurred).

(3) Homogeneous Zone II material in Dike No. 2 embankment.

(4) Materials in dike are dry, and perhaps brittle enough to facilitate and support a crack formed by fissuring.

Positive Factors:

(1) No evidence as yet over this region of north Phoenix regarding the existence of fissures.

(2) Long time to develop earth fissure - conditions may or may not exist.

(3) There has been little settlement of dikes and dam to date.

(4) No sustained water in impoundment area.

(5) Zone II is well graded to 3/4 inch but maybe irrelevant.

(6) Zone II materials are present in Dike No. 2 west of Cave Creek Road.

(7) Less subsidence susceptibility west of Cave Creek Road. Bedrock is shallower.

(8) Not time dependent west of Cave Creek Road.

Potential Actions for Risk Reduction:
(1) Monitor area for ground subsidence
(2) Evaluate groundwater levels in region.

***Failure From Potential Piping From Utility Crossings At Cave Creek Road - Training Dike No. 2. (Category II – Considered but not highlighted).***

Failure Mode Description: The pipe conduits at the Cave Creek Road crossing include three utility crossings comprised of a 6-foot, 5-foot, and a 4-foot diameter pipes. There appears to be no evidence of caving; grout has been placed around water and sewer pipe; pipes were installed by jacking; and steel casing around water pipe with grout in annular space.

Adverse Factors:
(1) Conduits are located below dam from upstream to downstream.
(2) May not be built like shown on construction plans.

Positive Factors:
(1) There appears to be 10 feet of cover over the utilities in reservoir and downstream.
(2) No sustained reservoir head - little time to develop seepage path.
(3) Utilities are below cutoff of dike trench.

Potential Actions for Risk Reductions:
(1) Put utilities on Cave Buttes Dam as-builts.
(2) Locate and mark utility locations in field.
(3) Note utility contact persons in case of emergency.
(4) Note procedure and method of isolating water line segments under dam.

## CATEGORY III– FAILURE MODES FOR WHICH INSUFFICIENT INFORMATION IS PRESENTLY AVAILABLE FOR MAKING ENGINEERING JUDGEMENTS.

***Potential Cave Creek Dam Failure Cause A Problem At Cave Buttes Dam (Category III).***

Failure Mode Description: Cave Creek Dam is an existing concrete arch buttress dam that is non-operational and is located within the reservoir impoundment area for Cave Buttes Dam. Cave Creek Dam is within the same drainage area and on the same ephemeral wash as Cave Buttes Dam (Cave Creek Wash). Cave Creek Dam is located approximately 3,000 feet upstream of Cave Buttes Dam. Cave Buttes Dam was designed to "replace" Cave Creek Dam. The problem is the potential for differential impoundment behind Cave Creek Dam which could potentially fail Cave Creek Dam. The resulting flood wave from Cave Creek Dam could but is considered unlikely to cause a problem at Cave Buttes Dam.

Adverse Factors: None identified.

Positive Factors:
(1) Cave Creek Dam spillway elevation at 1637 feet with 12,000 acre-feet of storage.
(2) Limited failure breach likely with 2 to 4 buttresses of Cave Creek Dam.
(3) Cave Creek Dam is 3,000 feet upstream of Cave Buttes Dam - floodwave may experience attenuation before reaching Cave Buttes Dam.

Potential Actions for Risk Reduction:
(1) Monitor Cave Creek Dam during flooding events.
(2) There is additional analysis that could be conducted to provide better insight on the potential failure modes and consequences of failure to Cave Creek Dam. Additional analysis includes a dambreak evaluation of Cave Creek Dam and the resulting flood wave analysis on Cave Buttes Dam. The Bureau of Reclamation has a computer program "LS Wave" that may be suitable for this type of analysis.

Consequence: The consequence is that debris from the failure of Cave Creek Dam may block the inlet of Cave Buttes Dam.

***Failure From Overtopping Training Dike No. 2 From Station 103+60 To 140+00 (Category III).***

Failure Mode Description: There is a potential for overtopping Dike No. 2 from Station 103+00 to Station 140+00. The crest of the dike is at elevation 1679.1-feet at Station 103+60 to elevation 1715.0-feet at Station 104+00 (at crest slope of 1 percent). The Dike No. 2 crest elevation west of Cave Creek Road is 1679.1-feet. (PMF elevation is 1674.1-feet).

Adverse Factors:
(1) Potential for subsidence is greater on the east end of Training Dike No. 2.
(2) Culvert under Cave Creek Road on upstream side potential may not have adequate capacity to carry inflow from the east side to the west side, thus it may impound flows and create overtopping of the dike.
(3) Need to know if training dike can handle inflow from upstream area (need to better define upstream drainage area for dike).

Positive Factors:
(1) Short duration of impoundment time.
(2) The training dike is used for infrequent events.

Potential Actions for Risk Reduction:

(1) Alternative Analysis Report by Kimley-Horn and Associates, Inc. (August 2000) identified potential secondary outlet in Dike No. 2 located near left abutment.

**CATEGORY IV – FAILURE MODES WHICH ARE NOT PHYSICALLY POSSIBLE OR WHICH ARE NOT CLEARLY CREDIBLE.**

***Training Dike No. 1 Failure Modes Considered But Ruled Out Are (Category IV):***

1. **Piping of Zone II into Zone III**. Zone III does not meet current filter criteria for Zone II (see Cave Buttes Dam Appendix of this section), however, because of short impoundment/detention time there may be not enough time for transport.
2. **Shrinkage cracking**. Not considered a problem based on coarser materials used for this Dike. Plasticity Index < 10. No.200 is 20 - 30%. (For it to be a concern for drying shrinkage cracking the threshold PI > 15 and No. 200 > 40%).

***Training Dike No. 2 Failure Modes Considered But Ruled Out Are (Category IV):***

The unauthorized installation of underground utilities through or under Training Dike No. 2 is not likely. Any earthwork activity near, adjacent, or involving Training Dike No. 2 will require a permit and review process involving the District, ADWR, and possibly the U.S. Army Corps of Engineers.

***Training Dike No. 3 Failure Mode Considered But Ruled Out Are (Category IV):***

No failure modes were considered for Training Dike No. 3. Training Dike No. 3 does not sustain water impoundment for a long enough time to produce a seepage and piping related concern for a homogeneous embankment. Also the events that would put water against the Dike are very infrequent (near or at PMF level).

## 4.0 SUMMARY AND CONCLUSIONS

Construction of Cave Buttes Dam appears to have experienced embankment quality problems (See "Filter Analysis & Evaluation of Zones I & II" in Appendix). Except for a minor seepage incident during the January 1993 storm event (see groin seepage photographs in Appendix), the dam has performed normally for 21 years. It is very well maintained and monitored.

However, it is prudent to recognize that there exist for all dams specific ways that failure could come about that warrant attention and diligent monitoring. The identification of a condition or process as a "potential failure mode" does not imply that the dam is about to fail or even necessarily that there is a dam safety deficiency at the site. Rather it identifies physically possible conditions or processes (generally with a remote but still credible chance of occurrence) that persons associated with owning, inspecting, analyzing and operating the dam should be aware. Some of the potential failure modes are

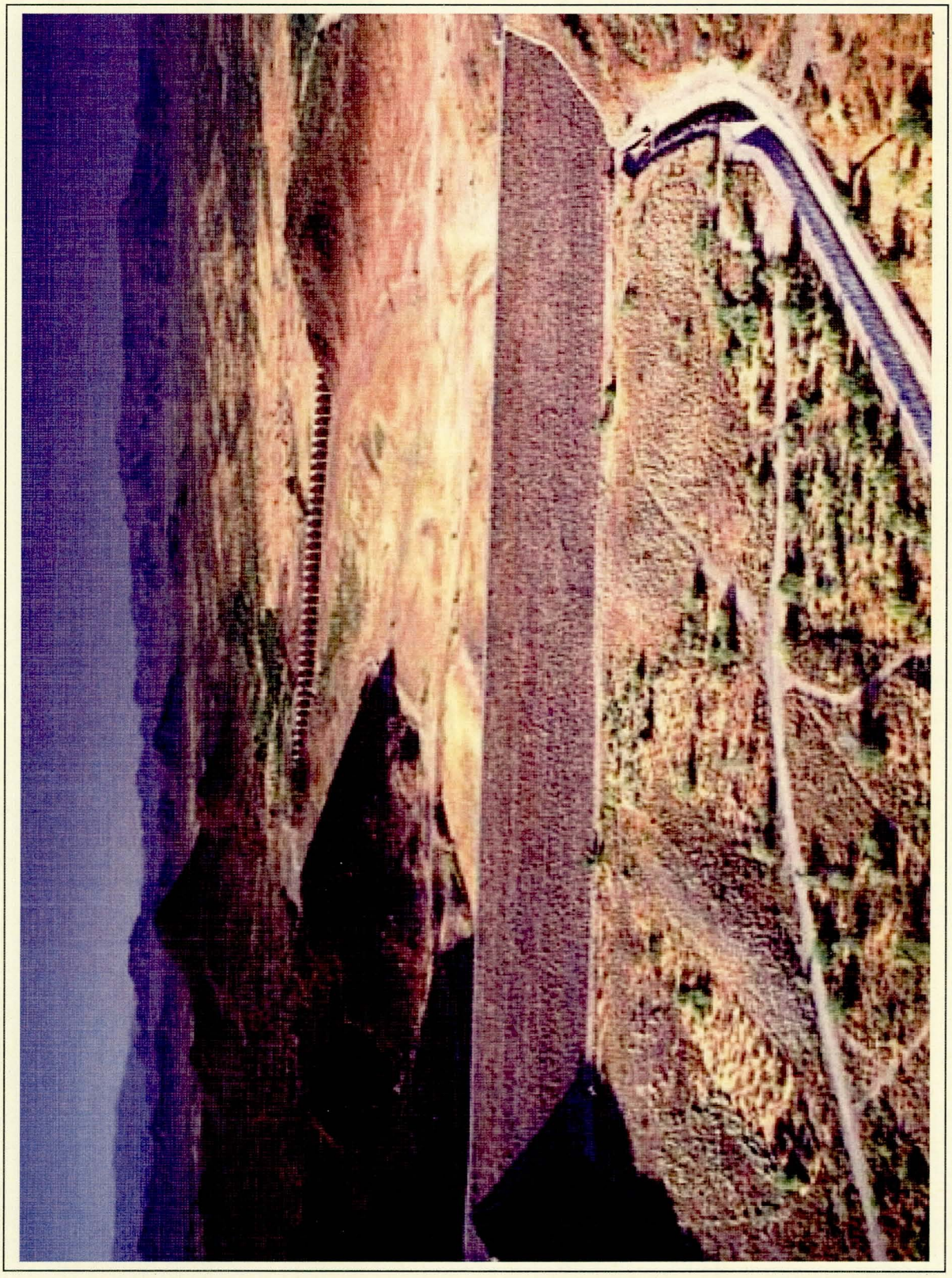
highlighted (or prioritized) for attention of the dam owners and operators. They are highlighted because the specific conditions at the dam and appurtenant structures are such that these failure modes are physically possible and are considered the most realistic and most credible potential failure modes definable at the site.

Two Category I potential failure modes, one for the main dam and one for Dike No. 2, were identified to be included in the quantitative risk analysis. These two static loading potential failure modes are physically possible, but neither seems particularly likely at this time. However, awareness, monitoring, and training (classroom training) of the possibility of these and other Category II potential failure modes identified and during inspections/flood events are key risk reduction actions that are warranted.

There are a large number of people and structures at risk in the flood path in the event of a spillway discharge or dam failure by breach. It appears likely that casualties could occur due to a dambreak. A very significant consequence of dam failure would be the adverse impact on the Central Arizona Project canal. It is highly recommended that an individual emergency action plan be developed for Cave Buttes Dam.

Note: The briefing paper "Filter Analysis & Evaluation of Zones I & II" was subsequently prepared (after the FMCA) by the Flood Control District and is included in the Appendix of this section. The briefing paper outlines a statistical evaluation of Zone I and II embankment materials and documents evidence of a shut down work order for construction at the main dam due to non-compliance of project specifications.

**Cave Buttes Dam Appendix - FMCA Reference Materials (following).**



Cave Buttes Dam looking upstream toward Cave Creek Dam.
Photo date: October 2001 Photograph Copyright Kimley-Horn and Associates, Inc.

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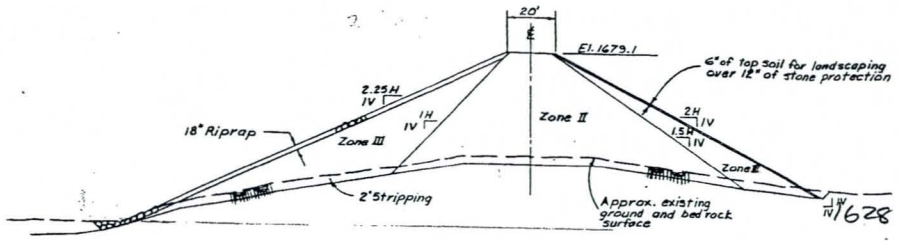
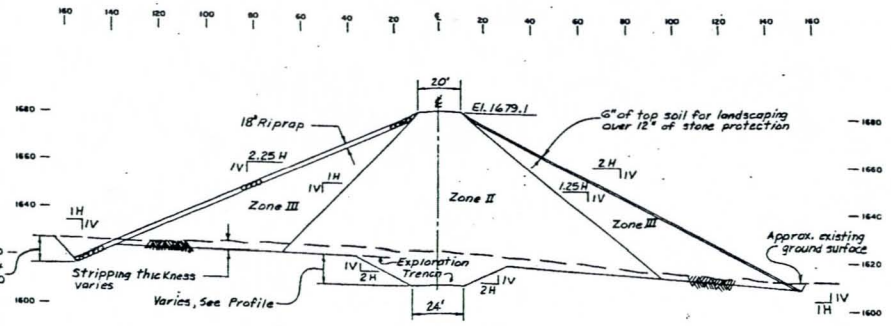


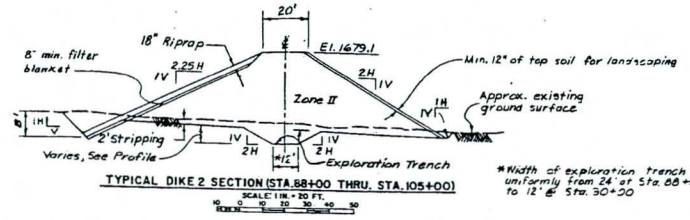

TYPICAL SECTION DIKE 1 (STA. 38+00)

SCALE: 1 IN. = 20 FT.




TYPICAL DIKE 2 SECTION (STA. 50+15 THRU. STA. 88+00)

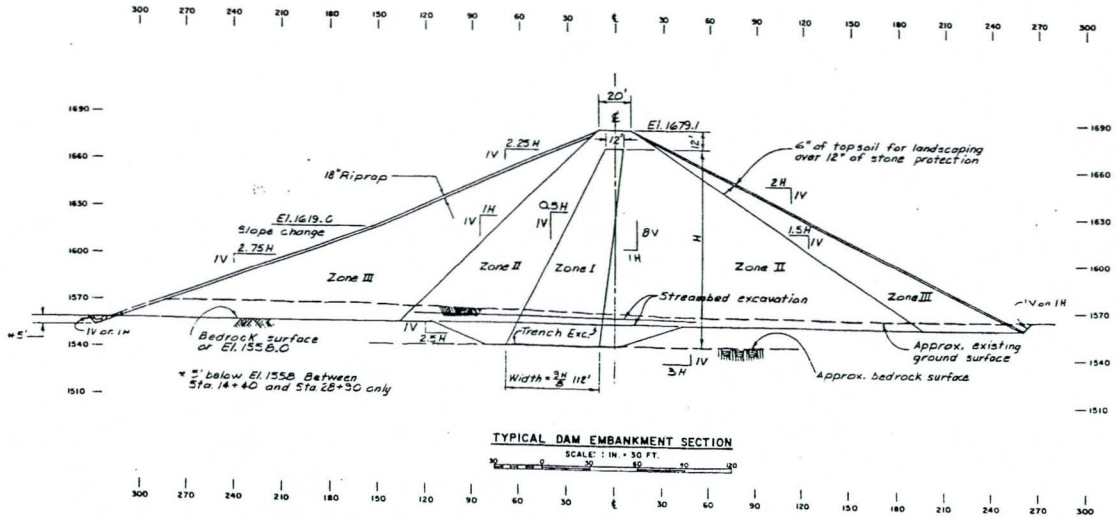
SCALE: 1 IN. = 20 FT.




TYPICAL DIKE 2 SECTION (STA. 88+00 THRU. STA. 105+00)

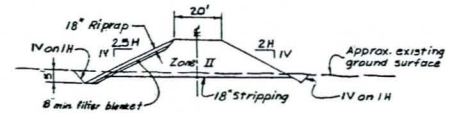
SCALE: 1 IN. = 20 FT.

*Width of exploration trench varies uniformly from 24' at Sta. 88+00 to 12' @ Sta. 30+00




TYPICAL DAM EMBANKMENT SECTION

SCALE: 1 IN. = 30 FT.




TYPICAL SECTION DIKE 3 (STA. 27+50)

SCALE: 1 IN. = 20 FT.

DATUM IS MEAN SEA LEVEL

| SYMBOL | DESCRIPTIONS | DATE | APPROVAL |
|---|---|---|---|
| | | | |

REVISIONS

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

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GILA RIVER BASIN
NEW RIVER AND PHOENIX CITY STREAMS, ARIZONA

CAVE BUTTES DAM

DAM, DIKES NO. 1, NO. 2 & NO. 3

TYPICAL EMBANKMENT SECTIONS

SUBMITTED BY: CHIEF, PLANNING & REPORTS BRANCH

APPROVAL RECOMMENDED: CHIEF, ENGINEERING DIVISION

APPROVED: COLONEL, C.E. DISTRICT ENGINEER

SPEC. NO. DACW09-____-B-____

DISTRICT FILE NO. 242/200

SHEET 1 OF 1 SHEETS

PLATE 3

# Filter Analysis & Evaluation of Properties of Zones I & II
# Failure Mode Analysis-Cave Buttes Dam
# Structures Assessment Dam Safety Program, Phase I,
# Flood Control District of Maricopa County

*George H. Beckwith, P. E.-Geotechnical Engineer*

*27 December 2001*

## 1.0 INTRODUCTION

This report presents the results of filter analysis and evaluation of the materials properties of Zone I (core) and Zone II (transition zone) made as a part of the Failure Mode Analysis for the Cave Buttes Dam. The analysis was performed pursuant to discussions on June 12, 2001 in the Risk Assessment phase of the Failure Mode and Consequence Analysis (FMCA). This analysis is being performed as a part of Phase I of the DISTRICT's Structures Assessment and Dam Safety Programs. Details of the design and construction of Cave Buttes Dam presented in the *Embankment Criteria and Performance Report* (USACOE, 1981) provide the basis for the analysis.

The capacity of Zone III (the pervious zone) to seal cracks in Zone II (the transition zone) was analyzed. The analysis focused on seepage-induced erosion processes involving a continuous pathway through the dam and it's foundation as opposed to piping that is initiated at the downstream face and progresses upstream (Von Thun, 1996; Milligan, 1996). Among the factors considered was the possibility of portions of the pathway being created by hydraulic fracturing (Sherard, 1973, 1985; Lo and Kaniru, 1990). Also considered were possible of pathways along the rock foundation-embankment interface, through open fractures in the rock foundation, along the primary spillway conduit and through layers in either the foundation soils or Zone III with pores so large that the Zone I and II soils could wash through (Milligan, 1996; Skempton and Brogan, 1994; Kenney and Lau,1984).

The filter analysis is presented in Section 2.0. Evaluation of the Verification Testing Program undertaken by the USACOE in the early part of embankment construction and other quality control testing is presented in Section 3.0. This information was assessed in relation to the properties of Zones I and II and the potential for seepage erosion failure modes passing through these zones.

A list of documents that the DISTRICT obtained from the USACOE since the Failure Mode Analysis session is presented in Section 4.0. These reports should be reviewed in the course of the final Risk Analysis process.

## 2.0 FILTER ANALYSIS

The results of the analysis are shown on the attached calculation sheets (5 pages).

## 2.1 Approach

Methodology in Chapter 26 of the SCS (now the Natural Resources Conservation Service or NRCS) National Engineering Handbook, *Gradation Design of Sand and Gravel Filters* (SCS, 1994) was used in the analysis. These procedures are the product of comprehensive research by the SCS (Sherard et al, 1984a, 1984b; Sherard and Dunnigan, 1985, 1989) and are essentially the same as those of the Bureau of Reclamation (BOR, 1987) and USACOE (USACOE, 2000). They have been endorsed by various organizations involved with the design, construction and operation of dams including the International Commission on Large Dams (ICOLD, 1994) and Canadian Dam Safety Association (Milligan, 1996).

Filtration properties and criteria to prevent gap-graded filters and minimize segregation were evaluated with the methods in SCS Chapter 26. More recent work by Foster and Fell (2001) was also considered. A procedure presented by Kenney and Lau (1984) and Skempton and Brogan (1994) was employed to analyze the internal stability of Zone III. This procedure evaluates whether the pore size of a granular soil is sufficient to allow the fines to be washed out through the granular skeleton of coarser particles under normal hydraulic gradients.

## 2.2 Notation

The notation used in the calculations is as follows:

D15F= The particle size of the filter material (Zone III) for which 15 percent by weight is finer.

A= The percent by weight of the coarse band of the filter material passing the no. 200 sieve.

D85B= The particle size of the specified fine band of the base material (Zone II) for which 85 percent by weight is finer.

## 2.3 Base Soil Category (BSC) of Zone II.

Zone II is composed of sandy silts, silty sands, clayey sands and sandy clays. The soils generally have up to about 30 percent gravel and classify as SM, ML, CL/ML, SC and SM/SC in the Unified Soil Classification System. As shown on Calculation Sheet 5, the specifications called for between 20 and 60 percent by weight passing the no. 200 sieve. The percent passing the no. 200 sieve determined in the field control testing for the main dam ranged from 6 to 86 percent. Median values determined for the main dam in the verification testing were 50 percent for the left abutment, 38 percent for the main embankment and 31 percent for the right abutment.

SCS Chapter 26 (SCS, 1994) subdivides base materials into four categories for filtration analysis. Zone II falls into both BSC 2 (silty and clayey sands and gravel with 15 to 39 percent passing the no. 200 sieve) and BSC 3 (sands, silts, clays and silty & clayey sands with 40 to 85 percent passing the no. 200 sieve).

**2. 4 Characteristics of Zone III**

The specified gradation limits of Zone III is shown on Calculation Sheet 5. According to the USACOE (1981), the Zone III material was excavated from borrow areas upstream and downstream of the dam in the Cave Creek stream channel. The material consists of sandy gravel and gravelly sand with varying amounts of silt and cobbles. The project specifications called for Zone III to have no more than 12 percent by weight passing the no. 200 sieve, while construction testing showed a mean value of 7 percent minus no. 200 material and a standard deviation of 3.6 percent. Lenses of coarse granular soil with as little as one percent passing the no. 200 sieve were reported in the stream bed alluvium in the dam foundation.

**2.5 Filtration Analysis.**

Filtration requirements are defined in terms of D15F. For the coarse band of Zone III, D15F= 5.0 mm while D15F= 0.15 mm for the fine band. The analysis was done for the specified fine band of Zone II. Because a considerable amount of Zone II is finer than the specified limits, the analysis for BSC 2 overestimates the filtration capacity of Zone III.

The results of the filtration analysis are as follows:

- A D15F equal to or less than 0.7 mm is required for BSC 2. Thus the coarse band of Zone III fails.
- For BSC 3, a D15F equal to or less than 6.0 mm is required as compared to the actual value of 5.0 mm. Thus, Zone III meets filtration criteria for the portion of Zone II that falls into BSC 3.

The analysis shows that that Zone III does not meet filtration criteria for the finer portion of Zone II. This material constitutes a large part of the embankment.

**2.6 Filter Band Limits to Prevent Gap Grading.**

For the particle sizes of the filter, for which 60 percent by weight or less are finer, the ratio of the course band size to the fine band size should be 5 or less. As shown on Sheet 2, the actual ratios are 8.2 to 50. Secondly, the coefficient of uniformity (CU) should be equal to or less than 6. The actual CU-values are 41 for the coarse band and 125 for the fine band. Thus, Zone III does not meet SCS Chapter 26 criteria for prevention of gap grading.

Analysis of the internal stability of Zone III by the Kenney and Lau (1984) procedure is presented on Sheets 3 and 4 and is illustrated on Figure 3. The results suggest Zone III is internally unstable.

### 2.7 Criterion for Segregation

SCS Chapter 26 includes a criterion for prevention of segregation. This criterion stipulates maximum D90F sizes for various ranges of D10F.

- For the coarse band of Zone III, D90F should be equal to or less than 35 mm. The actual value is 230 mm.
- For the fine band, a D90F-value equal to or less than 20 mm is required. The actual value is 53 mm.

The analysis indicates that the Zone III materials do not conform to the criterion for susceptibility to the segregation.

### 2.8 Discussion of Results

The analysis described above indicates that there is a significant possibility of defects in Zone III that could potentially form a part of continuous seepage pathways from the reservoir through the dam or through the dam and it's foundation. Much of Zone III apparently contains substantial cobbles. When placement in the main embankment began in September, 1977, more plus 6-inch material was present than in the test fill. The material was so coarse that conventional laboratory and field density tests could not be performed. Because Zone III is a pit-run, rather that a processed material, the likelihood of relatively open graded lenses extending through the zone seems greater. Zone III is relatively thin near the top of the dam.

## 3.0 EVALUATION OF PROPERTIES OF ZONE I & ZONE II

Further considerations relevant to analysis of seepage-induced erosion failure modes are discussed below.

### 3.1 Verification Testing Program

According to USACOE (1981), excavation of the cutoff trench and foundation treatment for the main dam was done between November 1977 and July 1978. The main dam embankment was constructed between August 1978 and August 1979. Modifications to the contract were made during construction because of flooding. Modification No. 2 authorized emergency flood work including an emergency by-pass channel and revised diversion ditch. Modification No. 8 authorized accelerated construction of the main embankment to elevation 1600.

The South Pacific Division (SPD) of the USACOE instructed the Arizona Resident Office to carry out a verification testing program in teletype SPDEG-G dated 18

September 1978. Further instructions for the details of the program were given by SPC personnel in a meeting at the Arizona Resident Office on 3 October 1978. It is stated in the Verification Testing Report of the USACOE Los Angeles District (Appendix B, USACOE, 1981) that ***"The verification testing was requested by SPD because of their perception of non-compliance with the project specifications observed during project visits."*** Modification No. 10 to the construction contract provided for verification testing of the embankment materials and abutment treatment. The Verification Testing was carried out by the USACOE with support from Fugro, Inc. who submitted a separate report. The Fugro, Inc. is presented in Appendix B1 (USACOE, 1981).

Earthwork operations for the main dam were apparently shut down from late September to sometime in December 1978 while the verification testing program was being performed and evaluated. The construction surface was at about elevation 1572, or somewhat above original natural grade in the stream channel, at the time of the verification testing.

It was ultimately concluded in the USACOE report that ***"the dam has been determined to be safe"*** and it was recommended that ***"construction proceed in accordance with project specifications"***. However, it is considered relevant to risk analysis that the foundation treatment and critical parts of the embankment were constructed at a time when SPD personnel believed that the work did not conform to project specifications.

### 3.2 Statistical Analysis of Field Compaction Testing

The statistical analysis of the field compaction testing of Zones I and II by the USACOE presented in the "Embankment Criteria and Performance Report" (USACOE, 1991) was reviewed. The use of statistical methods for the evaluation of earthwork is based on the concept that quality control is possible only through the use of probability theory and statistical principles (Hilf, 1991). Hilf (1991) describes pioneering work by the Bureau of Reclamation (BOR) in the application of statistical analysis to the quality control of the fine-grained zones in dams. Based on studies made in the 1950's and 1960's, the BOR recommended that the maximum allowable standard deviation be 3.0 for variations in percent compaction (ratio of field dry density to maximum dry density) and 1.5 for variations of the field moisture content from the optimum moisture content. The standard deviations for the Cave Buttes Dams exceed the recommended values.

To place the quality of Zones I and II of the main dam and Dike No. 1 into perspective, the statistical analysis was compared with data for the fine-grained zones of 80 BOR dams presented by Hilf (1991). This assessment indicates that Zones I and II are of relatively poor quality.

- For percent compaction of the main dam, S=3.8 for Zone I and 4.5 for Zone II. In contrast, only 11 percent of the BOR dams had S-values equal or greater that 3.8 and only 6 percent had S-values equal or greater than 4.5.

- For the variation of the field moisture content from the optimum moisture content for the main dam, S= 1.8 for Zone I and 1.6 for Zone II. For the BOR dams, 24 percent had S-values equal of greater than 1.6 and 14 percent had S-values equal or greater than 1.8.

- For variation or the moisture content from optimum, S=2.1 for Zone II of Dike No. 1. For percent compaction, S=4.6 for Zone II of Dike No. 1.

The statistical analysis indicates that seepage into the lower part of embankment might create localized differential settlements in low-density zones sufficient to produce cracking or reduce major principal stresses producing the susceptibility to hydraulic fracturing. Data presented by Lawton et al (1989; 1991; 1992), Brandon et al (1990) and Booth (1976) suggest that differential settlements of Zones I and II in response to wetting might be relatively large.

### 3.3 Relative Susceptibility of Zone I & II Soils to Seepage Erosion

Twenty borings were drilled, under the direction of Fugro, Inc., for the verification testing. In the initial borings near the left abutment, the rotary wash method was used. This was discontinued at the direction of the USACOE when large water losses were noted and hydraulic fracturing was suspected. The soils believed to be susceptible to hydraulic fracturing were apparently excavated and replaced.

Three hollow-stem auger borings, ranging from 11 to 48 feet in depth, and one 54-inch bucket-auger boring, 47 feet in depth, were drilled to investigate Zone II. Grain-size analysis were performed on 52 samples and Atterberg limits tests on 10 samples. The following soil classifications were made for the 52 samples:

- 63 percent were classified as silty sands and sandy silts (Unified Soil Classification: SM & ML) with plasticity indices of 3 or less.
- 37 percent were classified as clayey sands, sandy clays and silty clays (Unified Soil Classification: CL/ML, SM/SC & SC) with plasticity indices of 4 to 17.

The Atterberg limits tests for Zone II are plotted on a Unified Soil Classification System plasticity chart on Figure 2 which includes the Resistance Categories for internal erosion develop by Sherard (1953). The ML and SM soils fall into Sherard's Resistance Category 3 and have a particularly high susceptibility to erosion. According to Van Zyl and Harr (1981) and Meyer et al (1994), the global seepage-gradients required to induce internal erosion of these soils may be as low as 0.12.

In contrast to Zone II, most of the Zone I soils were classified as sandy clays (Unified Soil Classification: CL) having plasticity indices between 12 and 29. The Atterberg limits tests for Zone I are plotted on Figure 3. Most of the Zone I soils fall into Sherard's Resistance Category 1: overall, they have a relatively high resistance to seepage-induced erosion. Threshold gradients may be 0.5 or more.

## 4.0 ADDITIONAL DOCUMENTS

Subsequent to the Failure Mode Analysis meeting, the DISTRICT obtained the following documents from the USACOE:

- Report Titled: "Cave Butte Dam Design Memorandum No. 3, General Design Memorandum –Phase II Project Design Part 1, July, 1976"
- Report Titled: "Cave Buttes Dam Test Fill, Zones I, II and III, January 1979"
- Document Titled: "Specifications for Cave Buttes Dam, November 1979"
- Report Titled: "Cave Buttes Dam Foundation Report, August 1983"

It is recommended that the FMCA Team review these documents as a part of the final Risk Analysis for Cave Buttes Dam.

## 5.0 REFERENCES

Booth, A. R., 1975, *The Factors Influencing Collapse Settlements of Collapsing Soils,* Proceedings of the Sixth Regional Conference for Africa on Soil Mechanics and Foundation Engineering, Vol. 2, South African Institution of Civil Engineers

Brandon, T. L., Duncan, J. M., and Gardner, W. S., 1990, *Hydrocompaction Settlement of Deep Fills,* Journal of Geotechnical Engineering, ASCE, Vol. 116, No. 10, October.

Bureau of Reclamation (BOR), 1987, *Chapter 5 Filters,* Design Standards 13, Embankment Dams, U. S. Department of Interior, Denver.

Foster, M., and Fell, R., 2001, *Assessing Embankment Dam Filters That Do Not Satisfy Design Criteria,* Journal of Geotechnical and Environmental Engineering, ASCE, Vol. 127, No. 5, May.

Hilf, J. W., 1991, *Compacted Fill,* Foundation Engineering Handbook, Hsai-Yang Fang, Ed., Von Nostrand Reinhold, New York.

ICOLD, 1994, *Embankment Dam Filters and Drains,* Bulletin 95, Paris

Kenny, T. C., and Lau, D., 1984, *Internal Stability of Granular Filters,* Canadian Geotechnical Journal, Vol. 22.

Lawton, E. C., Fragaszy, R. J., and Hetherington, M. D., 1992, *Review of Wetting-Induced Collapse in Compacted Soil,* Journal of Geotechnical Engineering, ASCE, Vol. 118, No. 9, September

Lawton, E. C., Fragaszy, R. J. and Hardcastle, J. H., 1991, *Stress Ratio Effects on Collapse of Compacted Clayey Sand,* Journal of Geotechnical Engineering, ASCE, Vol. 117, No. 5, May

Lawton, E. C., Fragaszy, R. J., and Hardcastle, J. H., 1989, *Collapse of Compacted Clayey Sand,* Journal of Geotechnical Engineering, ASCE, Vol. 115, No. 9, September

Leonards, G. A., and Narain, J., 1963, *Flexibility of Clay and the Cracking of Earth Dams,* Journal of Soils and Foundations Division, ASCE, Vol. 89, No. SM2, March.

Lo, K. Y., and Kaniaru, K., 1990, *Hydraulic Fracture of Earth and Rock-fill Dams,* Canadian Geotechnical Journal, Vol. 27.

Meyer, W, Schuster, R. L., and Sabol, M. A., 1994, *Potential for Seepage Erosion of a Landslide Dam,* Journal of Geotechnical Engineering, ASCE, Vol. 120, No. 7, July

Milligan, V., 1996, *Thoughts on the Development of Filter Design Concepts and Their Current Application,* Joint 1996 CDSA/CANCOLD Embankment Design Workshop, Niagra Falls, Ontario.

SCS, 1994, *Chapter 26, Gradation Design of Sand and Gravel Filters,* Part 633, National Engineering Handbook, May.

Sherard, J. L., and Dunnigan, L. P., 1989, *Critical Filters for Impervious Soils,* Journal of Geotechnical Engineering, ASCE, Vol. 115, No. 7, July

Sherard, J. L., and Dunnigan, L. P., 1985, *Filters and Leakage Control of Embankment Dams,* Seepage and Leakage from Embankment Dams and Impoundments, R. L. Volpe & W. E. Kelly, Eds., American Society of Civil Engineers, New York.

Sherard, J. L., Dunnigan, L. P., and Talbot, J. R., 1984a, *Filters for Clays and Silts,* Journal of the Geotechnical Engineering Division, ASCE, June.

Sherard, J. L., Dunnigan, L. P. and Talbot, J. R., 1984b, *Basic Properties of Sand and Gravel Filters,* Journal of the Geotechnical Engineering Division, ASCE, June.

Sherard, J. L., Woodward, R. J., Gizienski, S. F., and Clevenger, W. A.,1963, *Earth and Earth-Rock Dams, Engineering Problems of Design and Construction,* John Wiley & Sons, Inc., New York.

Leonards, G. A., and Narain, J., 1963, *Flexibility of Clay and the Cracking of Earth Dams,* Journal of Soils and Foundations Division, ASCE, Vol. 89, No. SM2, March..

Skempton, A. W., and Brogan, J. M., 1994, *Experiments With Piping of Sandy Gravels,* Geotechnique, Vol. 44, No. 3

Stearns, C. E., Smith, R. J., and Stevenson, J. C., 1978, *Cracking of Dams in Arizona,* Report of the Soil Conservation Service Crack Study Team, April

U. S. Army, Corps of Engineers (USACOE), 2000, *Appendix D, Filter Design,* EM 1110-2-1913, U. S. Department of the Army.

USACOE, 1981, *Embankment Criteria and Performance Report, Cave Buttes Dam,* Gila River Basin, New River and Phoenix City Streams, Los Angeles District, March.

Von Thun, J. L., 1996, *Understanding Seepage and Piping Failures- the No. 1 Dam Safety Problem of the West,* ADSO Western Regional Conference, Lake Tahoe, Nevada, April.

Von Thun, J. L.,1985, *Application of Statistical Data from Dam Failures and Accidents to Risk-Based Decision Analysis on Existing Dams,* BOR, Engineering and Research Center, Denver Colorado, October.

Van Zyl, D., and Harr, M. E., *Seepage Erosion Analysis of Structures,* Tenth International Conference , Soil Mechanics and Foundation Engineering, Vol. 1, A. A. Balkema/Rotterdam.

## FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

PROJECT CAVE BUTTES DAM PAGE 1 OF 5
DETAIL FILTER ANALYSIS - COMPUTED GHB DATE 6-22-01
ZONES II to III CHECKED BY _____ DATE _____

NOTES: 1. Geotechnical data from "Embankment Criteria and Performance Report" (USACOE, 1981)
2 Filter analysis performed in accordance with: Chapt. 26 "Gradation Design of Sand and Gravel Filters" (SCS, 1994).

BASE SOIL (Zone II) CATAGORY (BSC)

Minus no. 200 - Zone II (% passing by dry wt.)

20 - 60% specified
6 - 86% Constr. Testing results - Main Dam
Mean - no. 200 for Main Dam

33.3 % Table 2 (USACOE, 1981)

Median - no. 200 from Verification Testing (APPA, USACOE, 1981)

50% Lt Abmt
31% Rt Abmt
38% Main Dam

Thus, most of Zone II is BSC 3 (15-39% - no. 200), but a significant fraction is BSC 2 (40-85% - no 200) as defined by Tab 26-1 (SCS, 1994)

ACTUAL D15F (dia. at which 15% by weight is finer)

For Zone I
D15F = 5.0 mm coarse band of specs
D15F = 0.15 mm fine band of specs

REQUIRED D15F (Zone I)

For BSC 2 $\leq$ 0.7 mm ← $\boxed{\text{Coarse band fails}}$

For BSC 3 $\leq \left(\frac{40-A}{40-15}\right)\left[(4 \times D85B) - 0.7\text{mm}\right] + 0.7\text{mm}$

For coarse band of filter, - no. 200, A = 0, D85B = 1.1 mm (fine band Zone II)

Thus:

$$D15F \leq \left(\frac{40-0}{40-15}\right)\left[(4 \times 1.1) - 0.7\right] + 0.7$$

$$\leq (1.6)(4.4-0.7) + 0.7 = 6.0\text{mm} \leftarrow \boxed{\text{OK}}$$

## FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

PROJECT CAVE BUTTES DAM PAGE 2 OF 5
DETAIL FILTER ANALYSIS COMPUTED GHB DATE 6-22-00
ZONE III CHECKED BY _____ DATE _____

FILTER BAND LIMITS TO PREVENT GAP GRADING (TAB 26-4, SCS, 1994)

For % passing ≤ 60%, the max dia/min. dia ratio is ≤ 5

| % Passing | Diameter (mm) Coarse | Diameter (mm) Fine | $D_c/D_f$ | Remarks |
|---|---|---|---|---|
| 60 | 82 | 10.0 | 8.2 | |
| 50 | 62 | 5.0 | 12.4 | |
| 40 | 40 | 2.0 | 20.0 | FAILS |
| 30 | 25 | 0.6 | 41.7 | |
| 20 | 10 | 0.23 | 50.0 | |
| 10 | 2 | 0.08 | 25.0 | |

Coefficient of uniformity, $CU = \frac{D60F}{D10F} \leq 6$

For coarse band

$CU = \frac{82}{2} = 41$

For fine band

$CU = \frac{10}{0.08} = 125$

Fails

SEGREGATION CRITERIA (TAB 26-7, SCS, 1994)

D90F = 230 mm (coarse band)
D90F = 53 mm (fine band)

For coarse band: D10F = 2.0; thus D90F ≤ 35 ← fails

For fine band: D10F = 0.08; thus D90F ≤ 20 ← fails

## FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

PROJECT CAVE BUTTES DAM PAGE 3 OF 5
DETAIL INTERNAL STABILITY COMPUTED GHB DATE 6.22.01
OF FILTER: ZONE III CHECKED BY _____ DATE _____

ANALYSIS OF INTERNAL STABILITY (Kenney & Lau, 1985)

| BAND | F % Passing | Df (mm) | 4Df (mm) | 4Df % passing | H |
|---|---|---|---|---|---|
| Fine | 10 | 0.08 | 0.32 | 28 | 18 |
| | 20 | 0.23 | 0.92 | 33 | 13 |
| | 30 | 0.6 | 2.4 | 43 | 13 |
| | 40 | 2.0 | 8.0 | 56 | 16 |
| | 50 | 5.0 | 20.0 | 68 | 18 |
| | 60 | 10.0 | 40.0 | 82 | 22 |
| | 70 | 22.0 | 88.0 | — | — |
| Coarse | 5 | 0.6 | 2.4 | 11 | 6 |
| | 10 | 2.0 | 8.0 | 18 | 8 |
| | 20 | 10.0 | 40.0 | 37 | 17 |
| | 30 | 25.0 | 100.0 | 70 | 40 |
| | 40 | 40.0 | 160.0 | 81 | 41 |
| | 50 | 62.0 | 248.0 | 92 | 42 |
| | 60 | 82.0 | 328.0 | — | |
| | 70 | 100.0 | 400.0 | — | |

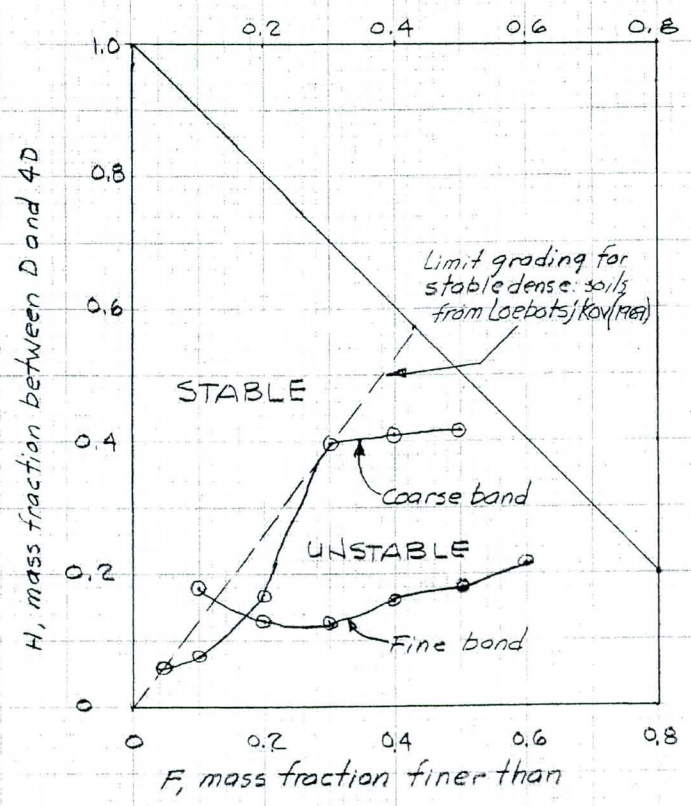
## FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

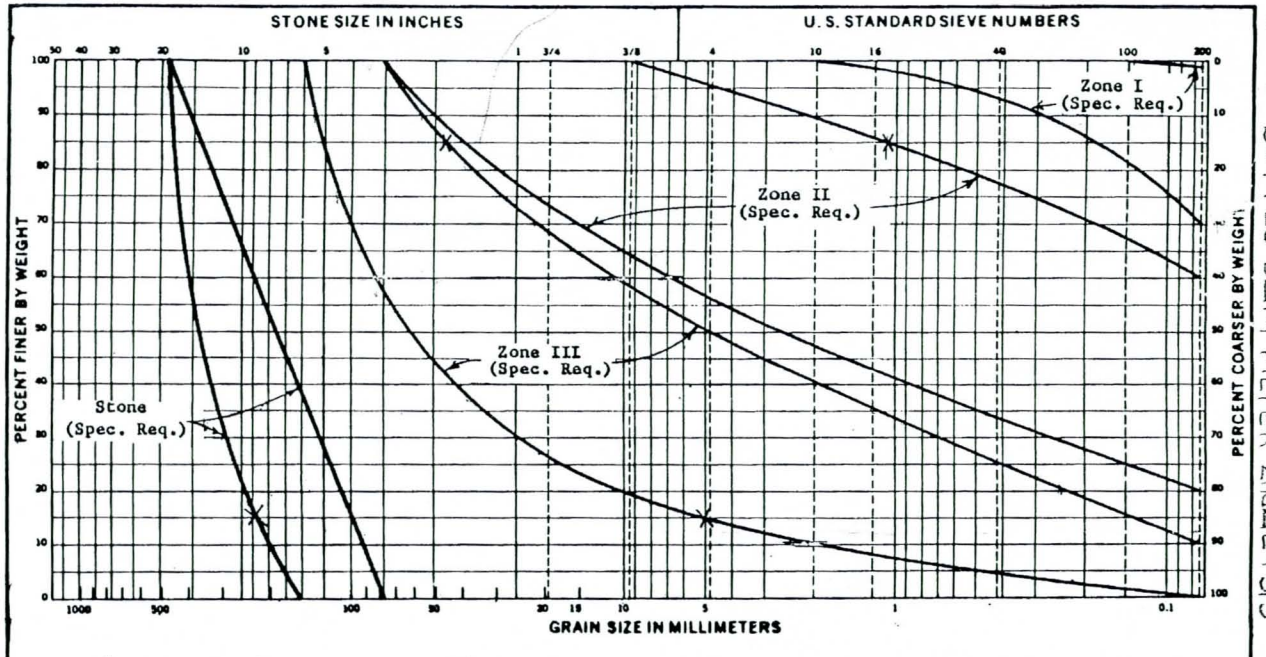
PROJECT CAVE BUTTES DAM PAGE 4 OF 5

DETAIL FILTER ANALYSIS - COMPUTED GHB DATE 6.22.01

INTERNAL STABILITY CHECKED BY _____ DATE _____

AFTER: Kenney, T.C., and Lau, D, 1985, "Internal Stability of Granular Filters", Canadian Geotechnical Journal, Vol. 22, No. 2






$$\frac{D_{15}\text{ (coarse fraction of Zone II)}}{D_{85}\text{ (fine fraction of Zone I)}} = \frac{0.04}{0.0085} = 4.7$$

$$\frac{D_{15}\text{ (coarse fraction of Zone III)}}{D_{85}\text{ (fine fraction of Zone II)}} = \frac{5.0}{1.1} = 4.5$$

$$\frac{D_{15}\text{ (coarse fraction of Stone)}}{D_{85}\text{ (fine fraction of Zone III)}} = \frac{220}{46} = 4.8$$

GILA RIVER BASIN
New River and Phoenix City Streams
Arizona

CAVE BUTTES DAM

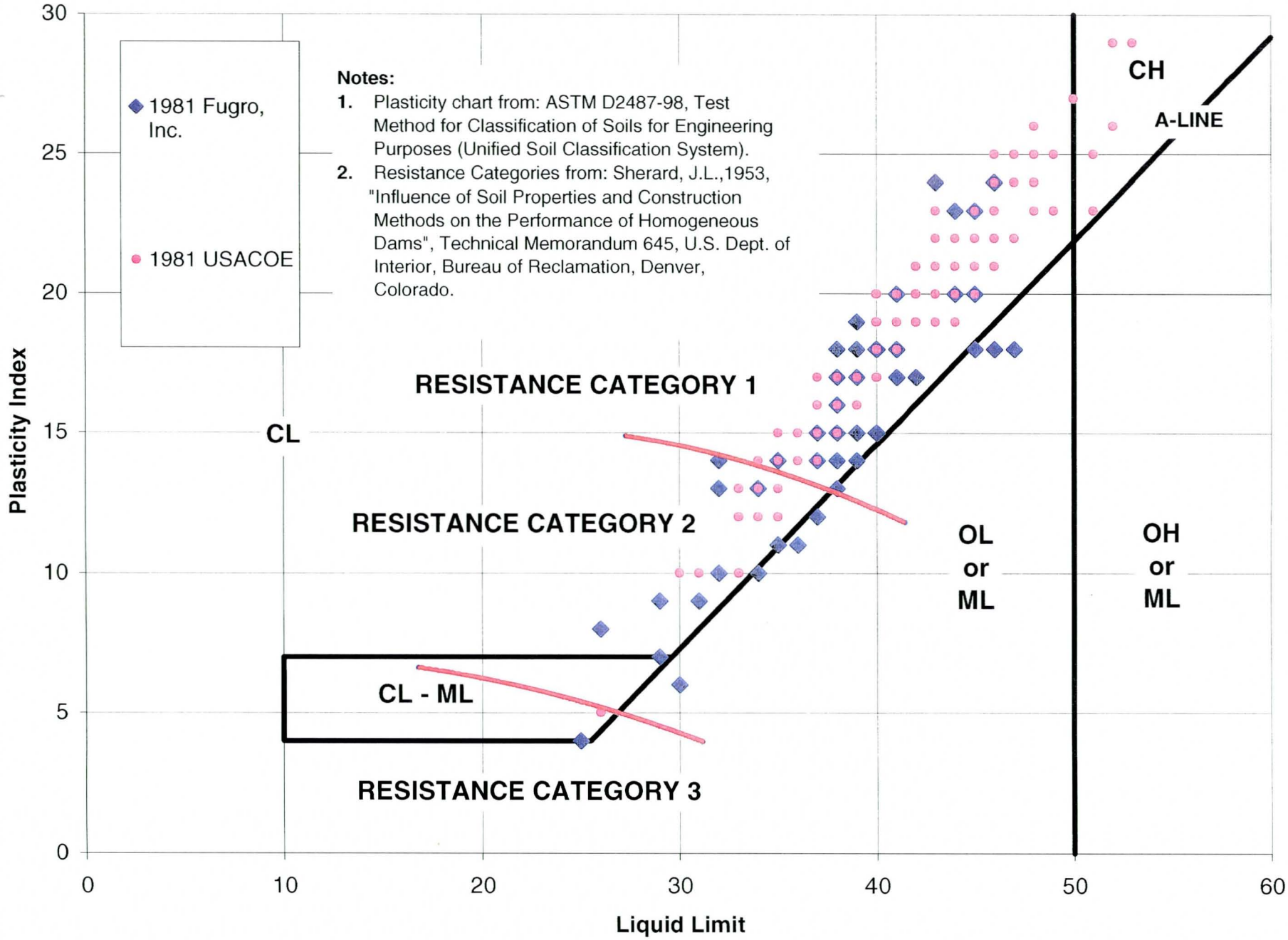
Embankment Filter Requirements

APPENDIX IA Figure 1

CAVE BUTTES DAM FILTER ANALYSIS
PG 5 of 5
GHB 6.22.01

**FIGURE 1**

Cave Buttes Dam Embankment Material Properties - ZONE 1
Verification Testing Program



# FIGURE 2

Cave Buttes Dam Embankment Material Properties - ZONE 2
Verification Testing Program

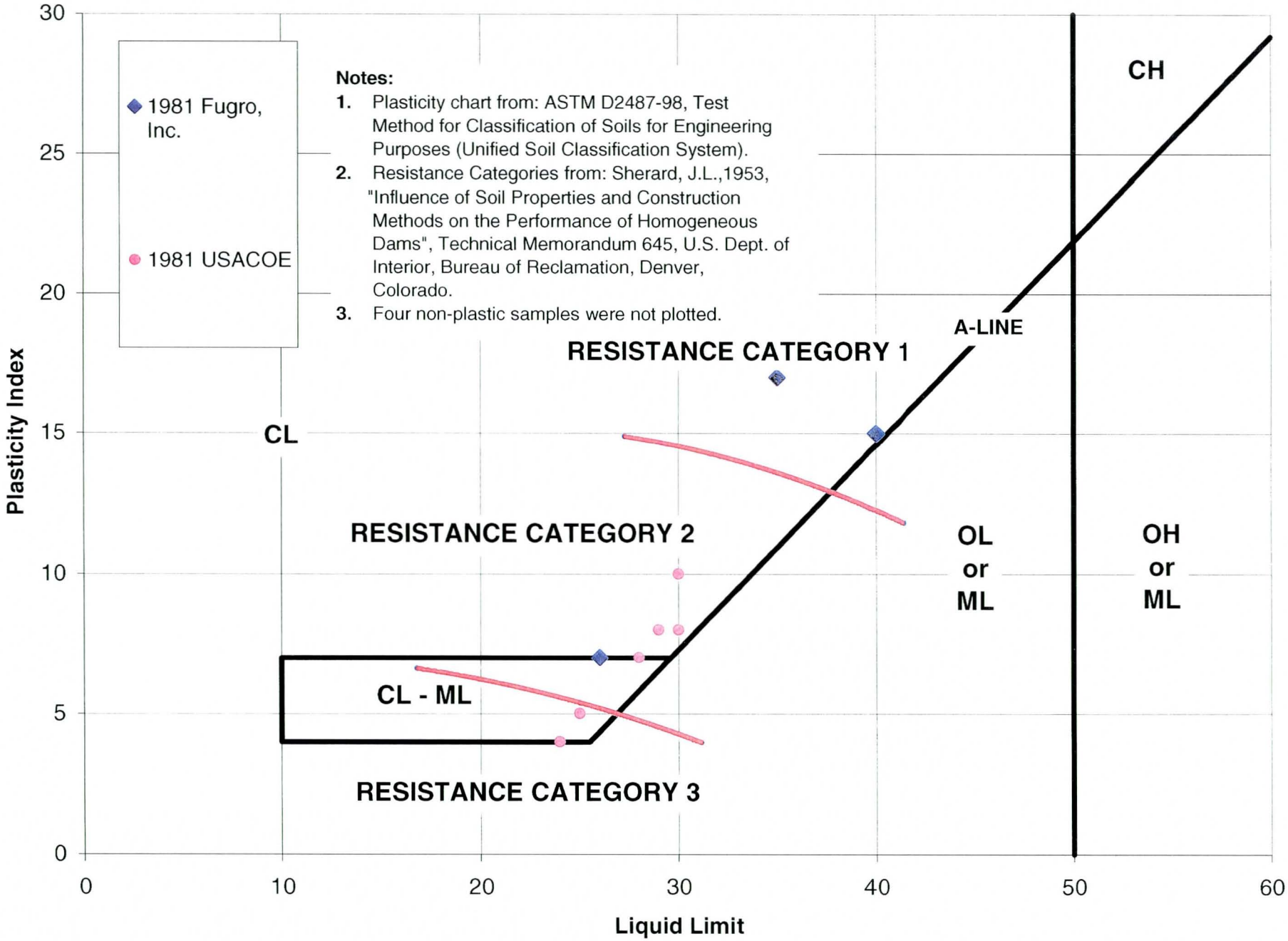
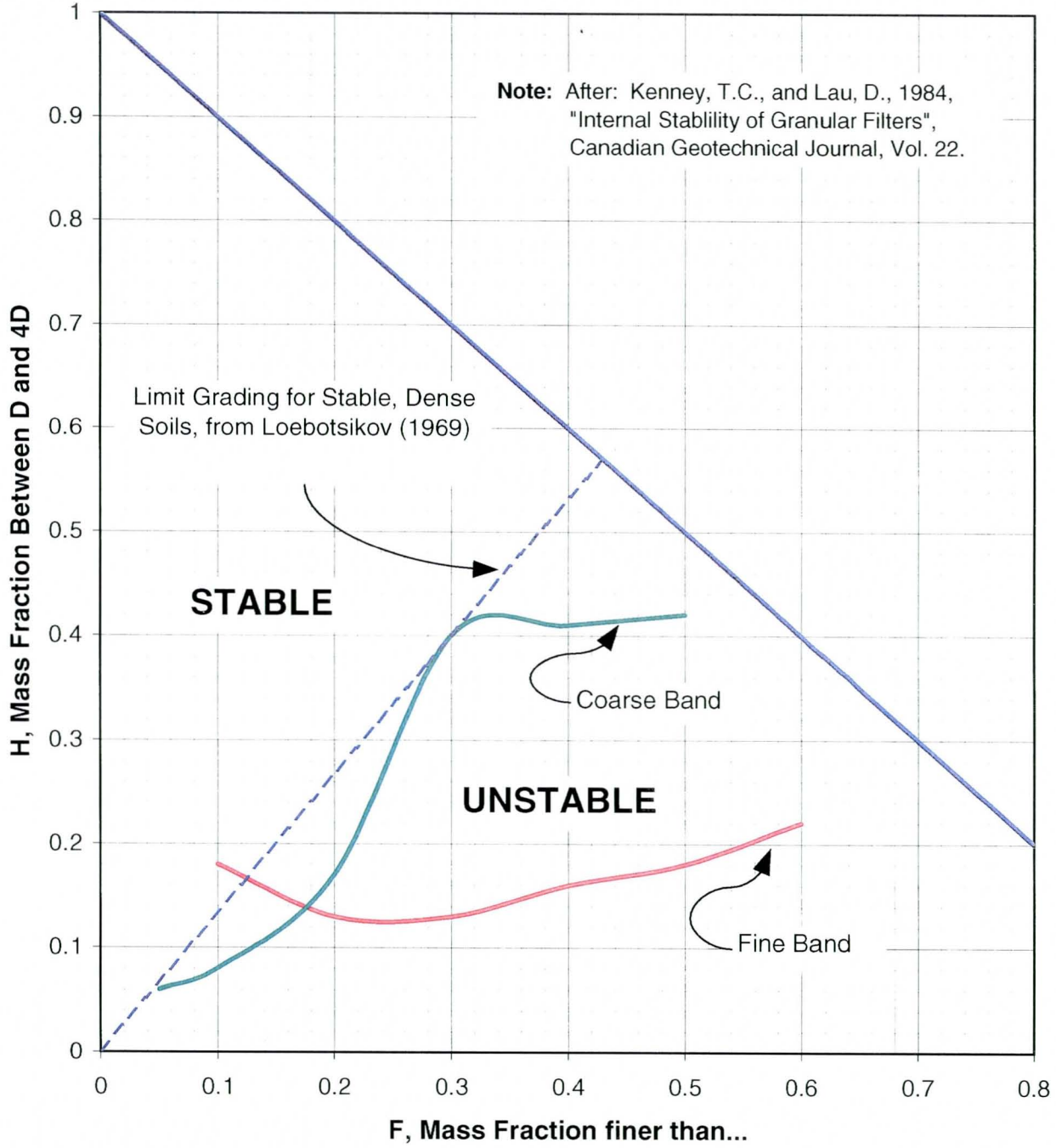


**FIGURE 3**

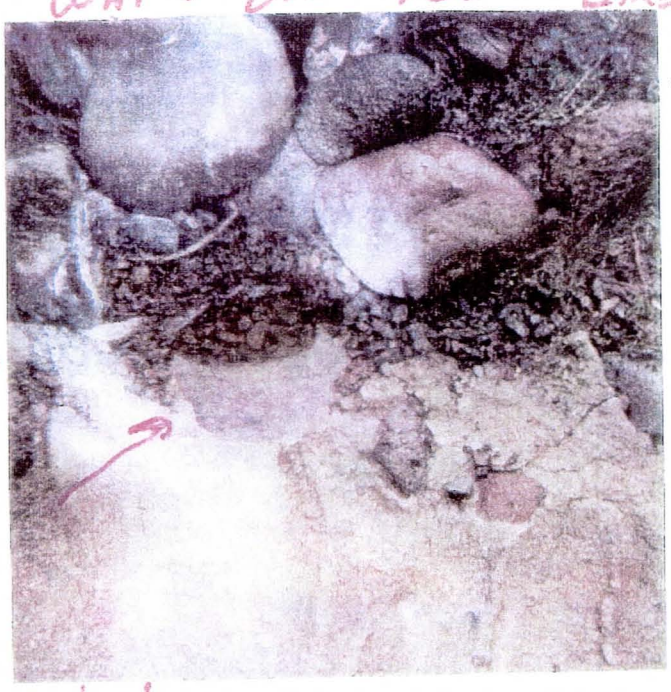
Internal Stability Analysis,Cave Buttes Dam Zone III Filter



WATER ELV. 1630.0 1400 HRS.

1/14/93 CAVE BUTTES DAM
EAST SIDE OF EAST GROINS
NEAR TOE AREA (#1)

WATER ELV. 1630.0 1400 HRS.




1/14/93 CAVE BUTTES
EAST SIDE OF EAST
GROIN SLIGHTLY ABOVE
TOE AREA

Cave Buttes Dam #1 1/14/93

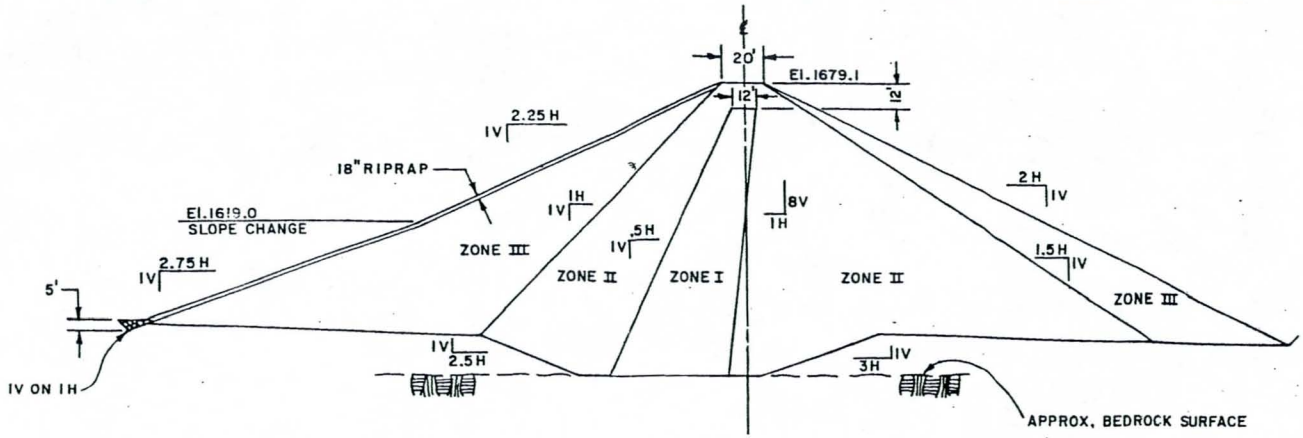
Seepage at the east edge of grouted rip-rap groin, left abutment, approx 45' above downstream toe of slope. Water was clear with no traces of sediments. Seepage occurred during a large storm event and it rained for 3 days and grounds were very saturated. The staff gauge reading at the P.O. indicated a 75' impoundment.

WATER ELV. 1630.0 1400 HRS

1/14/93 CAVE BUTTES DAM
2ND SEEPAGE AREA BELOW
OTHER PHOTO'S ~~EAST~~ SIDE
EAST SIDE GROIN

Cave Buttes Dam #2 1/14/93

Second seepage area approx. 10 to 13 feet below The other with all the same conditions existing as above #1




## TYPICAL DAM EMBANKMENT SECTION

## SPECIFICATION REQUIREMENTS FOR EMBANKMENT MATERIALS

| MATERIAL | ZONE I | ZONE II | ZONE III |
|---|---|---|---|
| GRADATION REQUIREMENTS | -NO.200 >70% | -NO.200 > 20% | -NO.200 <12 % |
| MOIST. CONTENT (BELOW OR ABOVE OPTIMUM) | -2% TO +2% | -2% TO +2% | — |
| LAYER THICKNESS | 12" | 12" | 24" |
| COMPACTIVE EFFORT * | 6 PASSES 50-TON ROLLER | 10 PASSES 50-TON ROLLER | 8 PASSES 50-TON ROLLER |

* ACTUAL NUMBER OF PASSES TO BE SPECIFIED WILL BE DETERMINED BY A TEST FILL.

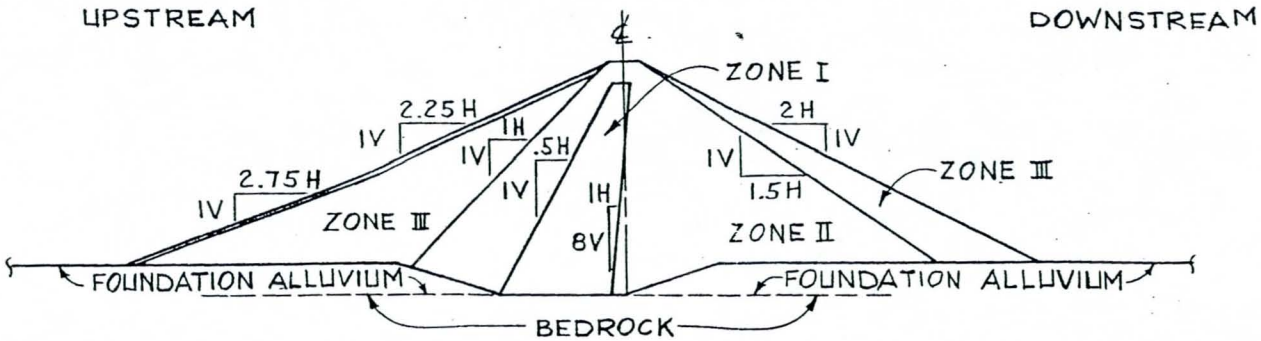
GILA RIVER BASIN
NEW RIVER & PHOENIX CITY STREAM, ARIZONA

CAVE BUTTES DAM

SPECIFICATION REQUIREMENTS

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS
TO ACCOMPANY REPORT DATED:

APPENDIX 1A FIGURE 18




# EMBANKMENT DESIGN VALUES

| MATERIALS | UNIT WEIGHT* | | | SHEAR STRENGTH (Φ IN DEGREES & C IN T/SF) | | | | | | PERM. COEFF. (K) FT./DAY | SPEC. GRAV. (G) |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | DRY P.C.F. | WET P.C.F. | SAT P.C.F. | "S" | | "R" | | "Q" | | | |
| | | | | Φ | C | Φ | C | Φ | C | | |
| ZONE I | 95 | 115 | 122 | 30 | – | 14 | 0.5 | 13 | 0.8 | 0.001 | 2.68 |
| ZONE II | 118 | 128 | 137 | 34 | – | 17 | 0.4 | – | – | 0.10 | 2.61 |
| ZONE III | 129 | 139 | 145 | 39 | – | 30 | – | – | – | 10.0 | 2.77 |
| DAM FOUNDATION ALLUVIUM | 129 | 139 | 145 | 39 | – | 30 | – | – | – | 10.0 | 2.77 |
| DIKE 2 FOUNDATION (0'-8') | 100 | 104 | 125 | – | – | 18 | 0.5 | – | – | 50.0 | 2.68 |
| DIKE 2 FOUNDATION (8'-15') | 110 | 112 | 130 | – | – | 18 | 1.5 | – | – | 3.0 | 2.68 |

* Unit Weights Shown Are Based On 95 Percent Of Maximum Dry Density.

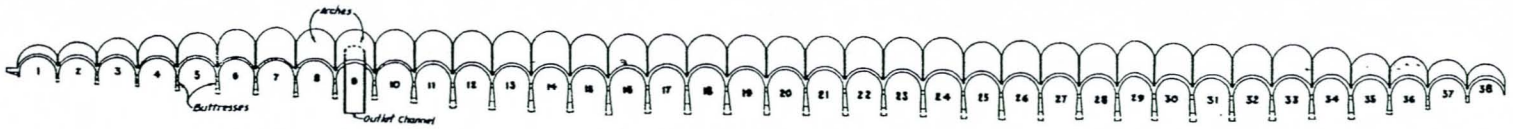
GILA RIVER BASIN
NEW RIVER & PHOENIX CITY STREAMS, ARIZONA
CAVE BUTTES DAM

SUMMARY OF DESIGN VALVES

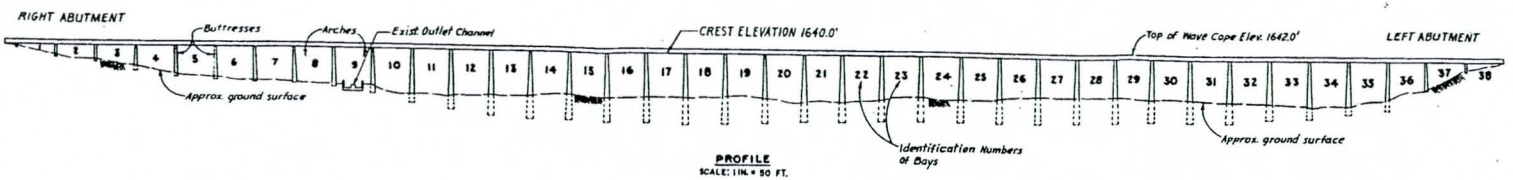
U. S. ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS
TO ACCOMPANY REPORT DATED:

APPENDIX IA FIGURE 10

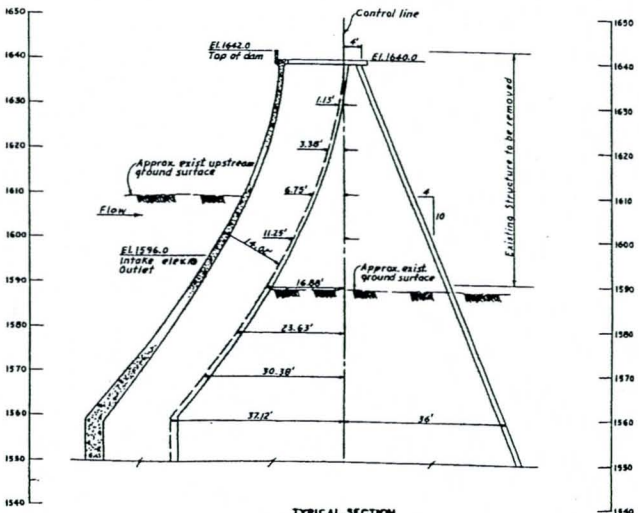
CORPS OF ENGINEERS



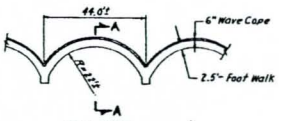

PLAN
SCALE: 1 IN. = 50 FT.



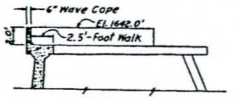

PROFILE
SCALE: 1 IN. = 50 FT.




TYPICAL SECTION
SCALE: 1 IN. = 10 FT.

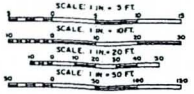



TOP VIEW
SCALE: 1 IN.=20 FT.




SECTION A-A
SCALE: 1 IN.= 5 FT.

Note:
Reinforcing Steel in the existing concrete structure is not shown.




DATUM IS MEAN SEA LEVEL

| SYMBOL | DESCRIPTIONS | DATE | APPROVAL |
|---|---|---|---|
| | REVISIONS | | |

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

GILA RIVER BASIN
NEW RIVER AND PHOENIX CITY STREAMS, ARIZ.

EXISTING CAVE CREEK DAM

DESIGNED BY: J.H.H.
DRAWN BY: J.K.K.
CHECKED BY:
SUBMITTED BY:
APPROVAL RECOMMENDED:
APPROVED:
SPEC. NO. DACW09-
DISTRICT FILE NO. 242/145
SHEET 1 OF 1 SHEETS

APPENDIX I

PLATE 4

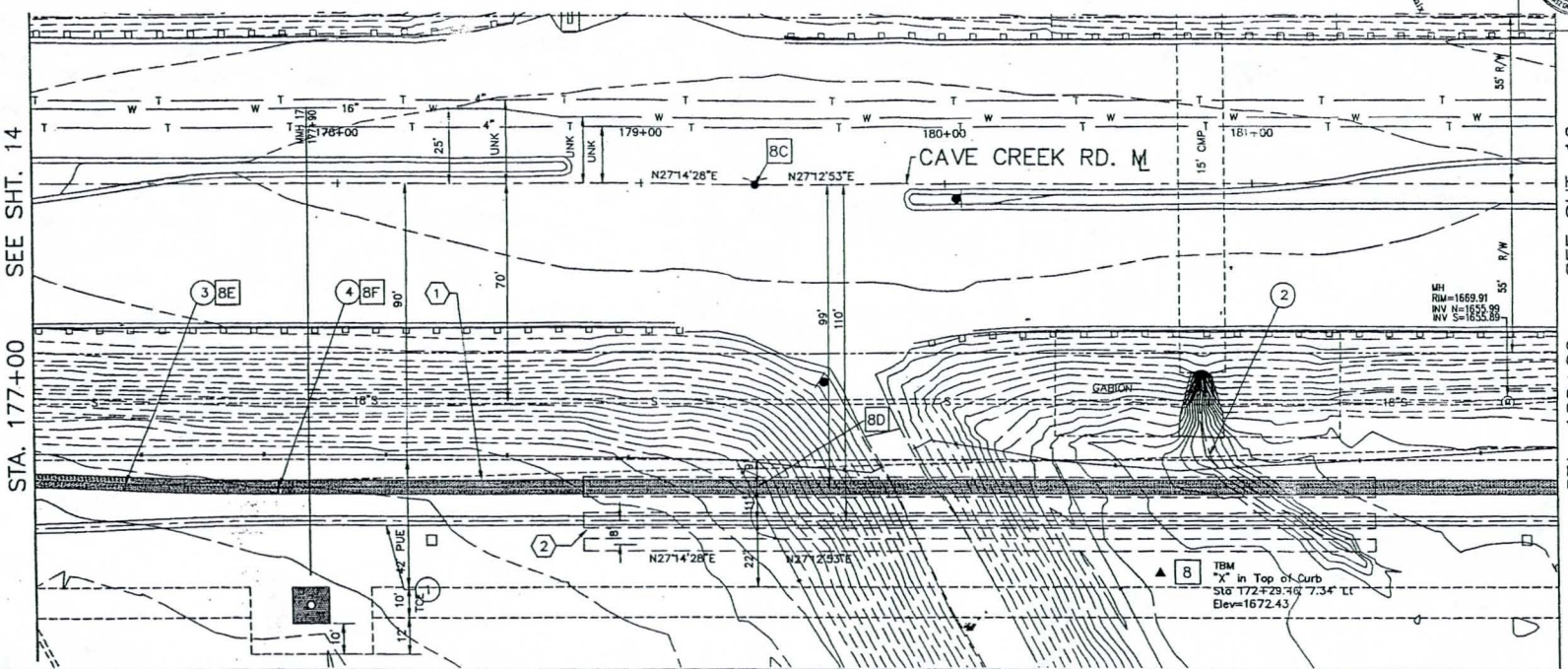
PHOENIX STREETS-MARICOPA CO.

CAVE BUTTE DAM SPUR

| F.H.W.A. REGION | STATE | PROJ. NO. | NO. | TOTAL | AS BUILT |
|---|---|---|---|---|---|
| 9 | ARIZ | W-964817 W-964791 | 15 | 50 | |

CONSULTING ENGINEER

DES J.B. DR: H.C. CK: L.M. DATE: 3/98

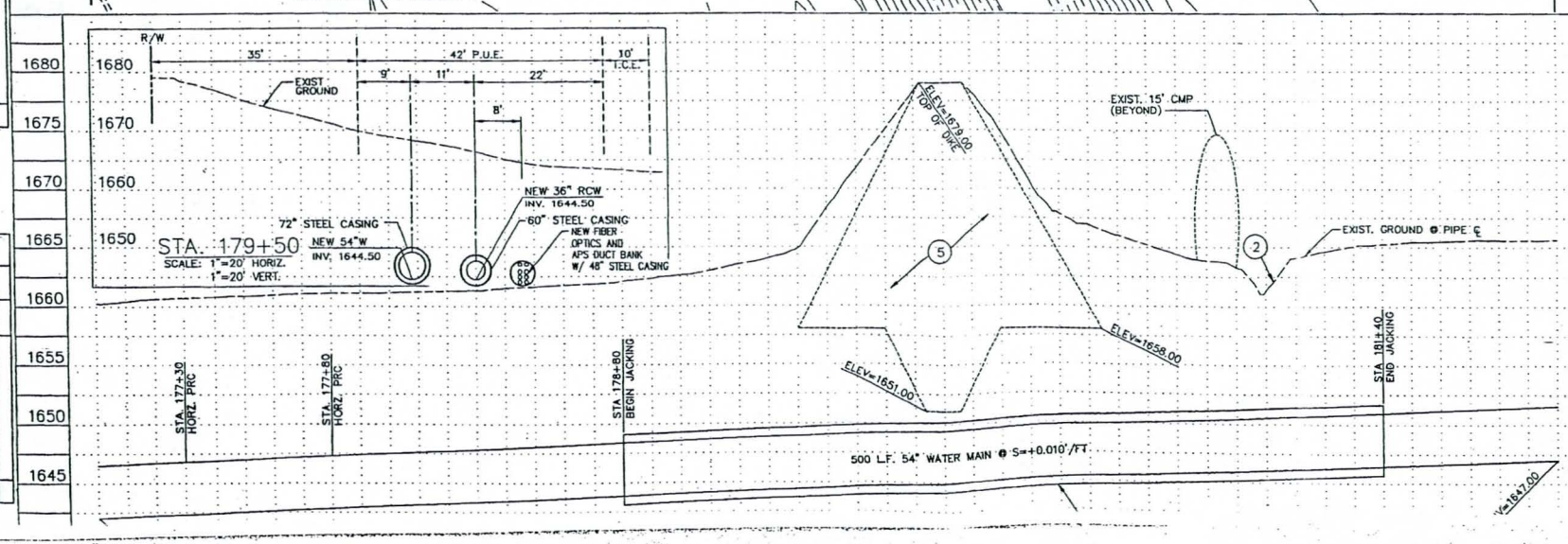



## CONSTRUCTION NOTES:

### NEW WATER LINE

| NO. | STATION to STATION | SIZE | L.F. |
|---|---|---|---|
| 1 | Sta 177+00 to 182+00 | 54" | 500 |
| 2 | Sta 178+80 to 181+40 Bore and Jacking See MAG Specifications | 72" | 260 |

1. 36" Reclaimed Water Line See Sheet 35
2. Maintain Uninterrupted Flows in Wash
3. Sta 177+30 PRC R=1000
4. Sta 177+80 PT R=1000
5. Cave Butte Dam Dike No.2 Core Material Type II See Detail 2, Sht. 49




### COORDINATE TABLE

| NO. | DESCRIPTION | NORTHING | EASTING |
|---|---|---|---|
| 8 | "X" in Top of Curb | 989,420.05 | 670,822.63 |
| 8C | B/C Flush Sta 179+37.27 | 990,045.99 | 671,153.14 |
| 8D | Angle Pt ℄ Pipe | 990,000.70 | 671,241.17 |
| 8E | PRC ℄ Pipe | 989,816.97 | 671,145.17 |
| 8F | PT ℄ Pipe | 989,860.85 | 671,169.17 |

CALL FOR THE BLUE STAKES 263-1100 Blue Stake Center CALL COLLECT


PROJECT ENGINEERING CONSULTANTS, LTD.
2320 W. PEORIA AVE. C-122 PHOENIX, AZ 85029
PHONE (602) 906-1901


CITY OF PHOENIX, ARIZONA
STREET TRANSPORTATION DEPARTMENT

# CAVE BUTTES DAM

GILA RIVER BASIN

NEW RIVER AND PHOENIX CITY STREAMS, ARIZONA

EMBANKMENT CRITERIA

AND

PERFORMANCE REPORT

Los Angeles District

March 1981

## PERTINENT DATA

CONTRACT NO. DACW-09-77-B-0031
Awarded on: 28 September 1977
Completed on: 11 March 1980
CONTRACTOR: Washington Construction Company, Missoula, Montana
CAVE BUTTES DAM

| | | |
|---|---|---|
| Drainage area | sq mi | 191 |
| Dam (rolled earthfill) | | |
| Crest elevation | ft msl | 1,679.1 |
| Maximum height above streambed | ft | 109 |
| Freeboard | ft | 2,260 |
| Spillway (detached) | | 5 |
| Crest elevation | ft msl | 1,657.1 |
| Crest length | ft | 510 |
| Elevation of maximum water surface | ft msl | 1,674.1 |
| Outlet works (ungated conduit) | | |
| Diameter of conduit | ft | 3.75 |
| Length | ft | 528.75 |
| Intake elevation | ft msl | 1,560.7 |
| Saddle dike No. 1 | | |
| Crest length | ft | 930 |
| Maximum height above existing ground | ft | 39 |
| Saddle dike No. 2 or east dike | | |
| Crest length | ft | 9,035 |
| Maximum height above existing ground | ft | 55 |
| Saddle dike No. 3 or west dike | | |
| Crest length | ft | 3,245 |
| Maximum height above exisiting ground | ft | 10 |
| Reservoir area at spillway crest | acre | 1,820 |
| Capacity (gross) at spillway crest | acre-ft | 46,600 |
| Storage allocation below spillway crest | | |
| Flood control (net) | acre-ft | 40,900 |
| Sedimention | acre-ft | 5,700 |
| Standard project flood | | |
| Total volume | acre-ft | 42,200 |
| Peak inflow | cfs | 54,000 |
| Peak outflow | cfs | 486 |
| Drawdown time | day | 48 |
| Maximum probable flood | | |
| Total volume | acre-ft | 122,000 |
| Peak inflow | cfs | 172,000 |
| Peak outflow | cfs | 100,600 |
| Drawdown time | hr | 61 |

CAVE BUTTES DAM
GILA RIVER BASIN
NEW RIVER AND PHOENIX CITY STREAMS, ARIZONA
EMBANKMENT CRITERIA AND PERFORMANCE REPORT

## GENERAL

1. Authority. Authority for the preparation of the Embankment Criteria and Performance Report for Cave Buttes Dam is contained in ER 1110-2-1901, dated 1 August 1972.

2. Project Purpose. The Cave Buttes Dam project is part of the New River and Phoenix City streams flood control project, and was authorized by the Flood Control Act of 1965 (Public Law 89-298, 89th Congress). The project provides flood protection to the City of Phoenix and vicinity.

3. Project Location. The Cave Buttes Dam project is located in Maricopa County, Arizona, approximately 19 miles north of downtown Phoenix. Cave Buttes Dam is located on Cave Creek approximately 0.7 mile downstream (south) of the existing Cave Creek Dam. The drainage basin above this project has an area of 191 square miles, and extends in a northeasterly direction from the damsite to the county line of Maricopa and Yavapai counties. The location of the project is shown on plate 1.

4. Project Description. A detailed description of the Cave Buttes Dam is given in Design Memorandum No. 3, General Design Memorandum, Phase II Appendix 1A, dated July 1976. Important features of the dam include the main embankment, three dikes, outlet works, outlet channel, unlined spillway, and access roads. The main embankment is a compacted, multi-zoned, earthfill structure consisting of an impervious core, transition zones on either side of the core, outer zones of pervious material, and cobble stone slope protection. The core is founded on bedrock within a cutoff trench. Top of the embankment is at elevation 1,679.1 feet, a maximum of 109 feet above the existing streambed. The length of the embankment is 2,260 feet, and the top width of the embankment is 20 feet. Dike No. 1 is located about 90 feet east of the main dam and is 930 feet in length with a crest elevation of 1,679.1 feet. The height of Dike No. 1 is about 39 feet above existing ground. The dike cross section is similar to the dam embankment without the impervious core and the cutoff trench. Dike No. 2 is located approximately 6,000 feet northeast of the dam embankment. The total length of the dike is 9,035 feet. The western 5,345 feet is a saddle dike with a crest elevation of 1,679.1 feet, a maximum height of 55 feet above existing ground. The eastern 3,690 feet of the dike has a variable height ranging from 10 to 6 feet above the existing ground. The dike embankment consists of two designs: the first consists of a transition material core with outer zones of pervious material on either side of the core, the second consists of homogeneous transition material throughout. Dike No. 3 is located approximately 2.5 miles northwest of the main dam. The dike is 3,245 feet in length with a crest elevation 1,679.1 feet, a maximum height of 10 feet above the existing ground. The dike consists of transition material obtained from a designated borrow area with 1.5 feet riprap upstream slope protection. The outlet works are founded on

bedrock at the left abutment of the main dam. The outlet works consist of an approach channel, intake tower, conduit, energy dissipator, and outlet channel. The spillway is located in a natural saddle approximately 1,600 feet northwest of the main embankment. The crest length is 510 feet at elevation 1,657.1 feet. A general plan of the project is shown on plate 2. Typical sections are shown on plate 3.

5. Construction History. The contract for construction of Cave Buttes Dam was awarded to Washington Corporations of Missoula, Montana on 28 September 1977 under Contract No. DACW-09-77-C-0058. Construction began in October 1977 and was completed in March 1980. Major storms occurring in March 1978, December 1978, and January 1979, and verification testing of the main embankment, directed by the South Pacific Division Office (SPD) during October 1978, created delays in construction. Construction of major features of the project and their approximate dates of construction are listed below:

| Construction Feature | Approximate Dates of Construction |
|---|---|
| Dam - Foundation and cutoff trench | November 1977 - July 1978 |
| Embankment | August 1978 - August 1979 |
| Dike No.1 - Foundation | November 1977 - July 1978 |
| Embankment | August 1978 - August 1979 |
| Dike No.2 - Foundation and cutoff | October 1977 - August 1979 |
| Embankment | October 1977 - August 1979 |
| Closure | August 1979 |
| Dike No.3 - Embankment | November 1978 |
| Spillway - Excavation | January 1978 - August 1978 |
| Concrete | April 1979 |
| Outlet Construction | November 1977 - September 1978 |
| By-pass Channel | July 1979 - August 1979 |

## SITE GEOLOGY

6. General. A detailed description of the Cave Buttes Dam site geology is provided in Design Memorandum No. 3, General Design Memorandum, Phase II, Appendix 1A, dated July 1976. The dam site at streambed elevation (1,570 feet) spans Cave Creek between two rock ridges that parallel the valley. Bedrock forming the ridges and underlying the stream bottom is a metaigneous rock, consisting of moderately hard green schist, greenstone, and granite. A thin veneer of older alluvium (talus material) covers the upper slopes of the ridges and becomes much thicker on the east side of the valley near the base of the slope. The materials in the valley are recent and older alluvium, which consist of various combinations of sand, gravel, cobbles, and some boulders existing to a depth of approximately 35 feet. There is evidence of ancient folding and some faulting in the rock formations in the vicinity of the Cave Buttes Dam.

7. Seismicity. Earthquake records indicate that Arizona is outside the circum-Pacific belt that includes the Pacific Coast and adjacent parts of the Western Mountain Region of the United States. The largest known earthquake in the State's history was one of intensity VIII, Modified Mercalli Scale (M.M.), recorded in 1910 approximately 170 miles from the project area. Thirty-nine earthquakes of maximum intensity IV to VI have occurred within a 150-mile radius of the project area from 1852 through 1974. Evaluation of the geologic and seismic conditions within a 150-mile radius of the project site indicates that the embankments are located in an area of low seismicity. The maximum intensity of shaking to be expected at the site, based on a 100-year record, would be on the order of M.M. IV.

## SITE INVESTIGATIONS

8. Investigations of the Proposed Site. In February 1970, a preliminary subsurface exploration was started for the plan recommended in the interim report (Interim Report on Survey for Flood Control, dated 15 January 1964). The data from this exploration, consisting of core borings, power auger holes, dozer trenches, and a seismic refraction survey, indicated that the top 30 feet consisted of recent alluvium having several layers of gravel and large nested boulders. Underlying the alluvium was a tuffaceous agglomerate formation which extended to crystalline bedrock at depths greater than 200 feet. Due to unfavorable foundation and abutment conditions disclosed at the site of the dam embankment, an alternative site about 7,000 feet upstream of the interim report site, where the overall foundation conditions appeared to be more favorable, was investigated. Data obtained from similar explorations at this site, indicated hard bedrock at the abutments and in the streambed below the recent alluvium, at a maximum depth of approximately 35 feet. Because of these more favorable foundation conditions, the upstream site was selected for further studies. Subsurface investigations were made at the relocated Cave Buttes Dam site in the foundation of the dam, dikes, outlet works, spillway, and the potential borrow areas. The subsurface investigations began in March 1970. The field explorations consisted of site reconnaissance, deep and shallow seismic refraction surveys, diamond core drilling, bucket-type power auger drilling, excavating trenches with a dozer and Gradall, and conducting in-place density tests and mass gradation tests.

9. Dam Foundation. The subsurface exploration in the streambed consisted of drilling 6 diamond core holes, varying in depth from 60 to 100.6 feet, and excavating 4 test trenches (TT 70-20 through -22 and -36). The trenches were excavated with a D-8 dozer to depths ranging from 16 to 32 feet to examine the foundation materials and to obtain disturbed samples for classification testing and moisture content determination. In TT 70-35, in addition to sampling for classification and moisture content, in-place density tests were conducted to evaluate the density of the foundation materials. Two trenches (TT 70-37 and -39) were excavated at the toe of the abutments to determine the depth of overburden. The locations of the core holes and test trenches are shown on plates 4 and 5.

10. Dam Abutments. The investigation of the right abutment consisted of drilling 2 diamond core holes to depths of 75 and 101.3 feet. The investigation of the left abutment consisted of drilling 2 diamond core holes to depths of 30 and 75 feet. All holes were drilled through the overburden into the bedrock. The locations of the core holes are shown on plate 4.

11. Dike No. 1 Foundation. The investigation for this dike consisted of drilling 2 diamond core holes to depths of 25 feet and excavating one test trench (TT 70-38). The trench was excavated to a depth of 4 feet to examine the overburden materials, to obtain disturbed samples, and to determine the depth to rock. The locations of the core holes and test trench are shown on plates 4 and 6.

12. Dike No. 2 Foundation. The investigation for this dike consisted of drilling 2 diamond core holes and 16 power-auger holes (TH 70-38 through -50 and 70-57 through -59). One diamond core hole was drilled to a depth of 51.6 feet in the right abutment. The second core hole was drilled on the axis of the dike, approximately 350 feet from the toe of the abutment, to a depth of 60 feet. The auger holes were drilled to depths ranging from 12 to 34 feet along the axis of the embankment. In each test hole standard penetration tests were conducted and disturbed samples obtained for laboratory classification, moisture content determination, and compaction tests. Two undisturbed cubic-foot samples were taken at depths of approximately 5 and 10 feet for detailed laboratory testing of shear strength, consolidation, and permeability. In-place densities were taken within the top 2.5 feet of the foundation materials and at the locations of undisturbed sampling. The locations of the core holes and test holes are shown on plate 7.

13. Dike No. 3 Foundation. The investigation for this dike consisted of drilling 3 power-auger holes (TH 70-89,-90 and 70-92) to depths ranging from 9 to 10.5 feet along the dike axis to obtain disturbed samples for laboratory classification, moisture content determination, and compaction tests. The standard penetration test was conducted in each test hole and in-place densities were taken in the top 1.5 feet of the foundation materials. The locations of the test holes are shown on plate 6.

14. Outlet Works. The foundation investigation for the outlet works consisted of drilling 5 diamond core holes varying in depth from 40 to 100.3 feet. In addition, 3 test trenches (TT 75-1,-2 and -19) were excavated to bedrock along the centerline of the outlet conduit and at the location of the intake structure to examine the bedrock surface and the excavation materials. The locations of the diamond core holes and test trenches are shown on plates 4 and 5.

15. Spillway. The investigation for the spillway, located approximately 1/2 mile northwest from the dam site, consisted of drilling 6 diamond core holes ranging in depth from 25.2 to 60.3 feet and excavating 1 test trench (TT 70-26) to a depth of 25 feet. A Caterpillar D-8 dozer, with hydraulic blade and rippers, was used to excavate the trench and to determine the ripability, breakdown properties of the rock, and possible size of the excavated material. The locations of the diamond core holes and test trench are shown on plates 4 and 6.

## BORROW AREA INVESTIGATIONS

16. Impervious Borrow (Zone I). The basin of the existing Cave Creek Dam was investigated to determine the gradation and properties of the materials for construction of Zone I, the impervious core of the dam. The investigations of the area, located about 3/4 mile upstream from the dam site, consisted of drilling 22 power-auger holes (TH 70-24 through -33, 70-51 through -53 and 70-80 through -88) through the sediments in the basin and sampling one existing trench (TT 70-40). The test holes were drilled to depths ranging from 3 to 19 feet to obtain disturbed samples for laboratory classification, moisture content determination, compaction tests, and detailed testing for shear strength, consolidation, and permeability. To determine balance factors, the standard penetration test was conducted in each test hole and in-place density tests were taken at depths ranging from 0 to 8 feet. The locations of the test holes and test trench are shown on plate 8.

17. Transition Borrow (Zone II). An area approximately 1 mile northeast from the dam site was investigated to determine the gradation and properties of the materials for construction of the transition zones of the main embankment and random zone for Dike No. 1. The investigation of the area consisted of drilling 46 power-auger holes (TH 74-10 through -55) to depths ranging from 4.5 to 15 feet. The materials encountered were visually classified and disturbed samples of representative materials were obtained for laboratory classification, moisture content determination, compaction tests, and detailed testing for shear strength, consolidation, and permeability. To determine balance factors, the standard penetration test was conducted in each test hole. The locations of the tests holes are shown on plate 9.

18. Pervious Borrow (Zone III). The Cave Creek streambed, upstream and downstream from the damsite, was investigated to determine the gradation and properties of materials for construction of Zone III, the pervious shells of the embankments. Detailed laboratory testing was conducted on representative material from these areas to determine shear strength and permeability.

a. Upstream Borrow. The investigation of this area, located between the existing Cave Creek Dam and the site of the subject embankment, consisted of excavating 21 test trenches (TT 70-23, -26, -27, -29, -30, and TT 75-4 through -18) in the streambed. The trenches were excavated with a D-8 dozer and a Gradall to depths ranging from 11 to 30 feet to examine the sides of the cuts and to sample the excavated materials for laboratory classification. The locations of the test trenches are shown on plate 10.

b. Downstream Borrow. The investigation of this area, located approximately 1 mile downstream from the dam axis, consisted of excavating 18 test trenches (TT 70-8 through -12, -15 and -24, TT-1, TT 71-10 through -12, -5, -8, -14 and TT 74-1 through -4) in the streambed. The trenches were excavated with a D-8 dozer to depths ranging from 12 to 18 feet to examine the sides of the cuts and to sample the excavated materials for laboratory classification. The locations of the test trenches are shown on plate 10.

19. Dike No. 1 Borrow. The random material to construct Dike No. 1 was obtained from the same source as the transition borrow for the main embankment.

20. Dike No. 2 Borrow. The area immediately upstream from Dike No. 2 was investigated to determine the suitability of the material for construction of the dike embankment. The investigation consisted of drilling 17 test holes (TH 70-34 through -37, TH 70-76 through -79 and TH 74-1 through -9) to depths ranging from 7.5 to 20 feet. Disturbed samples were obtained for laboratory classification, moisture content determination, and compaction tests. The standard penetration test was conducted in each test hole and in-place density tests were taken at depths ranging from 1 to 2 feet. The locations of the test holes are shown on plate 7.

21. Dike No. 3 Borrow. The area immediately upstream and downstream from Dike No. 3 was investigated to determine the suitability of the material for construction of the dike embankment. The investigation consisted of drilling 4 test holes (TH 70-89 through -92) to a maximum depth of 10 feet. Disturbed samples were obtained for laboratory classification, moisture content determination, and compaction tests. The standard penetration test was conducted in each test hole and in-place density tests were taken at a depth of 6 inches. The locations of the test holes are shown on plate 6.

## RIPRAP INVESTIGATIONS

22. An area located approximately 1 mile southeast from the dam site was investigated for a source of facing stone. The investigation consisted of drilling 6 diamond core holes (D-28 through D-33) to depths ranging from 37.5 to 101.1 feet. The locations and logs of the core holes are shown on plate 12. Other sources for stone were (a) existing stockpiles of cobbles and boulders near 7th Street and Beardsley Road located approximately 4 miles southwest from the dam site and (b) cobbles and boulders processed from the dam foundation and pervious borrow excavations.

## PRE-CONSTRUCTION FOUNDATION CONDITIONS

23. Dam Foundation. A generalized profile of the foundation along the centerline of the embankment, showing the thickness of streambed recent and older alluvium is shown on plate 13. The foundation conditions along the centerline of the embankment had changed considerably since the investigations were started in 1970. After completion of the foundation investigation large quantities of the foundation materials had been removed to a maximum depth of approximately 15 feet by commercial producers of sand and gravel. Several small dikes had been dozed into place to control surface runoff and to prevent flooding of the pits during their excavation. Waste material, stripped from approximately the top 1 foot of the streambed, had been randomly piled in the vicinity of the proposed embankment and the designated upstream pervious borrow area. The test data indicated that approximately the top 15 feet of the foundation materials were predominantly loose to medium dense. They classified mostly as sandy gravels, gravelly sands and borderline gravelly

sands-silty gravelly sands. At depths ranging from approximately 15 feet to the bedrock surface, the materials were predominantly medium dense to dense, and classified mostly as clayey sandy gravels and borderline sandy gravels-clayey sandy gravels. It was estimated that the total depth of foundation alluvium contained approximately 15 percent cobbles and boulders. Data from test trenches excavated in the foundation indicated that after stripping, the predominant portion of foundation materials had percentages of fines, material passing the No. 200 sieve, ranging from 1 to 9 percent. An occasional thin, discontinuous layer had a percentage of fines greater than 9 percent. In-place density tests taken in the foundation material indicated an average dry unit weight, for the top 15 feet, of approximately 118 pcf corresponding to a relative density of approximately 41 percent. At depths greater than 15 feet, a limited amount of tests indicated an average dry unit weight of approximately 129 pcf corresponding to a relative density of approximately 80 percent. The top 3 to 5 feet of the bedrock was fractured and rippable (see log TT 70-36). The foundation rock was moderately hard to hard, fine grained and foliated. The water losses from pressure tests were very small and core recovery recorded, in diamond core holes, was generally high (see geologic core logs, on plates 11 and 12).

24. Dam Abutments. The profile along the centerline of the dam, see plate 14, shows the location of the rock at the abutments and the depth of overburden, near the toe of the abutment. The rock of both abutments was schist, weathered near the surface, moderately hard to hard, platy and micaceous. The lower portion of the right abutment, from streambed to about El. 1,620 feet, had an approximate slope of 1 Vertical on 2 Horizontal, then flattened to 1V on 4H. The lower part of the left abutment was covered with talus material and older alluvium to a maximum depth of approximately 30 feet. The slope of the abutment was nearly 1V to 1H.

25. Dike Foundations.

a. Dike No. 1. The foundation material of Dike No. 1 consisted of moderately hard to hard schist bedrock. Along the centerline of the dike the overburden materials varied in thickness from 0 to 2 feet. In accordance with the Unified Soil Classification System the overburden materials classified as clayey sand with gravel.

b. Dike No. 2. Most of the right abutment of Dike No. 2, station 50 to about 53+50, was covered with a thin layer of overburden. Rock outcrops were present at a few scattered places. The entire right abutment was schist, and the embankment foundation and left abutment, station 53+50 to 115 consisted of older alluvium of the valley bottom. A generalized profile along the centerline of the dike, showing bedrock at the right abutment and the thickness of alluvium in the foundation and the left abutment, is shown on plate 13. The alluvial materials in the foundation east of station 54 were predominantly silty sand and clayey sand. The soils were finer east of Cave Creek Road. The materials to a depth of approximately 5 feet had an average dry density of 102 pcf and 81 percent relative compaction. Below this depth, the materials became progressively harder due to caliche cementation, and cobbles became more numerous. At a depth of approximately 10 feet, the average dry density was 100 pcf and 87 percent relative compaction. Undisturbed samples tested

for compression characteristics at depths of 5 and 10 feet indicated potential foundation deformation of approximately 4 and 2 percent, respectively, under embankment loading in an unsaturated condition. Upon saturation of the foundation materials, approximately 5 percent additional settlement in the foundation would be anticipated. The permeability of the near surface material was approximately 50 feet per day decreasing with depth to approximately 3 feet per day at a depth of 10 feet. At station 97, the alinement of Dike No. 2 crossed the centerline of existing Cave Creek Road and required ramping up the existing road to the elevation of the top of the dike.

c. Dike No. 3. The near surface foundation materials of Dike No. 3 was sandy clay or clay underlain by sandy silt and clayey sand highly cemented and containing a small percentage of gravel size material. The in-place density of the top 12 inches of material was on the order of 80 percent of maximum density. The material became harder from 1 to 3 feet. At depths greater than 4 feet the materials had SPT blow counts greater than 50 and, therefore, were very stiff.

26. Outlet Works. The outlet works is located near the toe of the left abutment. The 3.75-foot diameter reinforced concrete conduit was founded entirely on greenstone/schist bedrock. The foundation for the ungated intake structure was founded on similar rock. A profile of the foundation near the centerline of the outlet works is shown on plate 13.

27. Spillway. The spillway was excavated in rock. The rock was predominantly a moderate to highly fractured and foliated schist with occasional moderately fractured greenstone intervals, weathered to approximately 8 feet with foliation dipping 60 to 90 degrees to the northwest. Rock in the hill west of the spillway saddle was granite. Below the layer of weathered rock, the schist became harder. Due to the infrequency of spillage, the remoteness of the site, and the nature of the materials, this site was considered satisfactory for an unlined spillway with a concrete sill along its full width and sides to control scour during a remote spillway flow. The alinement for the sill was located near the highest point in the saddle.

## FOUNDATION AND ABUTMENT TREATMENT

28. Dam. A detailed description of the foundation and abutment treatment is provided in the Foundation Report. The foundation and abutment treament of the main dam consisted of excavating to suitable bedrock within the bottom of the foundation trench and the entire abutment area. All blocks, slabs, and boulders were removed from the bottom of the trench to provide a uniform bedrock surface for placement of fill. Pressure cement grouting was generally conducted at the mid-point of Zone I, along a single line, at 40-foot centers to a depth of 25 to 75 feet except from station 13+07 to 29+63 where the spacing was reduced to approximately 20-foot centers. When grout take was high or seepage occurred at the surface, split-spacing and offset of grout holes was conducted. After grouting was performed, air blowing of the Zone I contact area and a zone 12 feet upstream and downstream of the Zone I limits in the Zone II contact area was accomplished. A small amount of dental

concrete was placed in fill areas which were inaccessible for proper compaction of the embankment fill within the air blown areas. Mortaring with neat cement was done along seams and joints within the air blown areas to assure all openings would be completely filled prior to placing core material. The proposed contact grouting plan is shown on plate 14 and the typical foundation treatment is shown on plate 15. The foundation treatment outside the limits of the cutoff trench consisted of excavating to elevation 1558 to remove loose sands and gravels and to strip unsatisfactory material. The exposed foundation materials were then compacted with 8 passes of a 50-ton rubber tire roller prior to placement of embankment materials.

29. Dike No. 1. The foundation and abutment treatment for Dike No. 1 consisted of excavating 2 feet over the entire abutment and foundation areas to suitable rock. Air cleaning of the rock surface which was to receive Zone II material, was accomplished within the central 24 feet of the dike embankment. Grouting was accomplished along the centerline of the dike at 40-foot centers and to depths of 25 feet (see plate 14). Removing loose rock, clearing, grubbing, wetting, and proofrolling was performed in areas under the Zone III materials.

30. Dike No. 2. The foundation treatment for Dike No. 2 consisted of excavating a foundation trench with a base width of 24 feet and to a depth of about 10 feet between station 103+00 and the right abutment (station 53+50). The remainder of the streambed foundation and abutment was excavated to a depth of at least 2 feet. The bottom of the foundation trench and streambed foundation was compacted with 8 passes of a 50-ton rubber tire roller. The bedrock at the right abutment was air cleaned in the central 24-foot width prior to placing embankment materials (see plates 16 through 18). Grouting was accomplished along the abutment centerline of the dike at 40-foot centers and to depths of 25 feet.

31. Dike No. 3. The foundation treatment for Dike No. 3 consisted of about 1.5 feet of stripping and compaction with 8 passes of a 50-ton rubber tire roller prior to placing embankment materials.

31a. Outlet Works. The outlet structure was constructed on fractured greenstone bedrock, except for that part of the excavation from station 50+85 to 48+10, where some alluvium and highly weathered and sheared greenstone was encountered. Plate 24 shows the plan and profile (stations 60+00 to 46+80) of the outlet works. The alluvium and the highly fractured and sheared greenstone were overexcavated to remove compressible material from the base of the outlet works structure. The area over excavated was approximately 200 feet long by 10 feet wide by a maximum of 15-feet deep. The area was cleaned and backfilled with concrete to form an incompressible and impervious foundation for the outlet works conduit.

## EMBANKMENT DESIGN

32. Design Values. The selection of design values was based on the results of detailed laboratory test conducted on samples from the designated borrow areas and the foundation materials. A detailed description of the adopted

values is provided in Design Memorandum No. 3, General Design Memorandum (GDM), Phase II, Appendix 1A, dated July 1976. A summary of the GDM design values is presented in table 1. Test fills were conducted on the Zone I, II, and III materials used in the construction of the dam embankment, to verify if the specified compaction procedures produced the necessary results. A separate report on this testing is presented in Appendix A. In the test fill report an evaluation was made of the effects of incremental rolling by a 50-ton pneumatic-tired roller and the effects of placement and compaction of overlying lifts on the in-situ density.

Table 1. GDM DESIGN VALUES

EMBANKMENT MATERIALS

Zone I Material

| | | |
|---|---|---|
| Moist unit weight | | 115 pcf |
| Saturated unit weight | | 122 pcf |
| "S" strength | Ø = 30 deg. | C = 0 tsf |
| "R" strength | Ø = 14 deg. | C = 0.5 tsf |
| "Q" strength | Ø = 13 deg. | C = 0.8 tsf |
| Coefficient of permeability (K) | | 0.001 fpd |

Zone II Material

| | | |
|---|---|---|
| Moist unit weight | | 128 pcf |
| Saturated unit weight | | 138 pcf |
| "S" strength | Ø = 34 deg. | C = 0 tsf |
| "R" strength | Ø = 17 deg. | C = 0.4 tsf |
| Coefficient of permeability (K) | | 0.10 fpd |

Zone III Material

| | | |
|---|---|---|
| Moist unit weight | | 139 pcf |
| Saturated unit weight | | 145 pcf |
| "R" strength | Ø = 39 deg. | C = 0 tsf |
| "S" strenght | Ø = 30 deg. | C = 0 tsf |
| Coefficient of permeability (K) | | 10 fpd |

33. Main Embankment Section. The embankment has a maximum height of about 109 feet above the existing streambed. The crest width is 20 feet. The upstream slope is 1V on 2.25H from the top of the dam to elevation 1,619. Below elevation 1,619, the slope is 1V on 2.75H. The downstream slope is 1V on 2H. The embankment is a zoned rolled-earth fill. The general plan, and details of the embankment are shown on plates 2 and 3. Since the amount of available sources of Zone III material were limited, the outer zones of the embankment are sized to fully utilize the available quantities of Zone III materials.

a. Zoning of Embankment. The cross section of the embankment was designed to make optimum usage of the materials available from nearby sources and from required excavations. The zoning selected would provide effective

Table 2. SUMMARY OF EMBANKMENT CONTROL TESTS

| Item<br>Control Test | Zone I (impervious)<br>Dam | Zone II (random)<br>Dam | Dike 1 | Dike 2 | Composite | (Zone II)<br>Dike 3 | Zone III (pervious)<br>Composite |
|---|---|---|---|---|---|---|---|
| a. Percent Fines: | | | | | | | |
| No. of Tests | 371 | 565 | 32 | 132 | 729 | 21 | 89 |
| Mean (-No. 200) | 82.2 | 33.3 | 28.4 | 29.5 | 32.4 | 73.5 | 7.0 |
| Standard Deviation | 11.5 | 12.1 | 10.0 | 8.3 | 11.6 | 10.5 | 3.6 |
| b. Field Density: | | | | | | | |
| No. of Tests | 374 | 576 | 32 | 147 | 755 | 21 | 55(1) |
| Mean (pcf) | 102.9 | 120.2 | 120.2 | 119.3 | 120.1 | 104.8 | 137.9 (1) |
| Standard Deviation | 5.2 | 7.2 | 8.1 | 6.0 | 7.0 | 5.3 | 6.9(1) |
| c. Field Moisture: | | | | | | | |
| No. of Tests | 374 | 576 | 32 | 147 | 755 | 21 | - |
| Mean (% dry wt.) | 20.9 | 11.3 | 10.6 | 10.5 | 11.1 | 14.0 | - |
| Standard Deviation | 2.6 | 2.7 | 2.4 | 2.1 | 2.6 | 2.5 | - |
| d. Percent Compaction: | | | | | | | |
| No of Tests | 374 | 576 | 32 | 147 | 755 | 21 | 14(2) |
| Mean (% max.) | 99.9 | 100.1 | 100.4 | 98.4 | 99.8 | 97.9 | 100.3 |
| Standard Deviation | 3.8 | 4.5 | 4.9 | 3.6 | 4.4 | 4.4 | 6.3(2) |
| e. Moisture Variation: | | | | | | | |
| No. of Tests | 374 | 576 | 32 | 147 | 755 | 21 | - |
| Mean (% from opt) | 0.7 | -0.7 | -1.5 | -1.0 | -0.8 | -3.8 | - |
| Standard Deviation | 1.8 | 1.6 | 2.1 | 1.4 | 1.6 | 3.6 | - |
| f. Relative Density | | | | | | | |
| No. of Tests | - | - | - | - | - | - | 68(3) |
| Mean (% Rel. dens.) | - | - | - | - | - | - | 94.9(3) |
| Standard Deviation | - | - | - | - | - | - | 7.9(3) |

Notes:

(1) Densities were conducted by large scale water displacement method. For Sand-Cone densities: No. of test = 67, Mean density = 129.3 pcf, and Standard deviation = 4.3%

(2) Relative compactions conducted by DWR method.

(3) Relative densities conducted by ASTM Standard D 2049. For Sand Cone densities: No. of tests = 67, Mean density = 129.3 pcf, and Standard deviation = 4.3%

f. Summary of Field Compaction Control Data. As required by ER 1110-2-1925, "Field Control Data for Earth and Rockfill Dams," test results were summarized by the Resident Engineer staff and transmitted, through the Foundation and Materials Branch, to the South Pacific Division Offices every month during active construction periods. At the completion of earthfill operations, 13 monthly reports had been forwarded with the following results.

(1) Zone I and Zone II Materials. Specifications required the placement moisture contents to be within the range of 2 percent below to 2 percent above the optimum moisture content with a desired relative compaction of 97 and 95 percent of the maximum density for Zone I and Zone II materials, respectively. The following list summarizes the results.

Summary of Compaction Control Data for
Zone I and II Materials

| | Dam Zone I (impervious) | Dam Zone II (random) | Dike 1 Zone II (random) | Dike 2 Zone II (random) | Dike 3 Zone II (random) |
|---|---|---|---|---|---|
| Total No. of Tests | 446 | 872 | 40 | 175 | 23 |
| No. Outside Limits: | | | | | |
| Total | 151 | 395 | 17 | 73 | 21 |
| Moisture Content | 85 | 227 | 11 | 41 | 15 |
| Density | 35 | 84 | 2 | 19 | 0 |
| Moisture & Density | 31 | 84 | 4 | 13 | 6 |
| No. Reworked | 83 | 288 | 8 | 35 | 2 |
| No. Reworked & Retested | 63 | 173 | 6 | 17 | 2 |

The results of the field control tests are presented graphically in figures 1 through 14.

(2) Zone I Materials, Main Embankment. Results of field control testing indicated Zone I materials were finer grained than anticipated in design. The percent passing the No. 200 sieve ranged from 33 to 99 percent and the majority of the materials had No. 200 values greater than the 70 percent design value. These finer grained materials had corresponding lower field densities. Field densities ranged from 84 to 120 pcf and most of the tests were lower than the design 115 pcf. The field moisture contents varied from 14.2 to 28.2 percent and the mean field moisture content of 20.9 compared approximately with the design value of 21 percent. The percent relative compaction ranged from 85 to 109 percent. The mean percent compaction of all the materials tested was 99.9 percent which is above the 97 percent design requirement. The percent variation from optimum moisture content ranged from -3.6 to 8.2 percent. The design requirements of $\pm$ 2.0 percent from optimum moisture content were within one standard deviation from the mean.

(3) Zone II Material, Main Embankment. Results of field control testing indicated Zone II materials were finer grained than anticipated in design. The percent passing the No. 200 sieve ranged from 6 to 86 percent and the majority of the materials had percent passing the No. 200 sieve values greater than the 20 percent design value. These finer grained materials had

corresponding lower field densities. Field densities ranged from 90 to 140 pcf and most of the tests were lower than the design 128 pcf. The mean field moisture content of 11.1 was above the design value of 8.5 percent. The percent relative compaction ranged from 70 to 116 percent. The mean percent compaction of all the materials tested was 99.8 percent, well above the 95 percent design requirement. The percent variation from optimum moisture content ranged from -8.3 to 4.8 percent. The design requirements of ± 2.0 percent from optimum moisture content are within one standard deviation from the mean.

(4) Zone II Material, Dike No. 3. Results of field control testing indicated Dike No. 3, Zone II materials were finer grained than anticipated in design. The percent passing the No. 200 sieve ranged from 45 to 85 percent and 50 percent of all samples had percent passing the No. 200 sieve values less than the 70 percent upper limit design value. Field densities ranged from 96 to 118 pcf and all of the tests were lower than the design 128 pcf. The field moisture contents varied from 8.5 to 18.4 percent and the mean field moisture content of 14.0 percent was much higher than the design value of 8.5 percent. The percent relative compaction ranged from 85 to 105 percent. The mean percent compaction of all the materials tested was 97.9 percent, above the 95 percent design requirement.

(5) Zone III Materials. Moisture control for placing Zone III material was not required. The desired relative density of the zone was 80 percent and the desired relative compaction in accordance with the DWR test results was 95 percent of the maximum density. The following list summarizes the results.

Summary of Compaction Control
Data for Zone III Material

| | ASTM 2049 | DWR Max-Vibrated |
|---|---|---|
| Total No. of Tests | 68 | 14 |
| No. Outside Density Limits: | 10 | 4 |
| No. Reworked | 6 | 0 |
| No. Reworked & Retested | 4 | 0 |

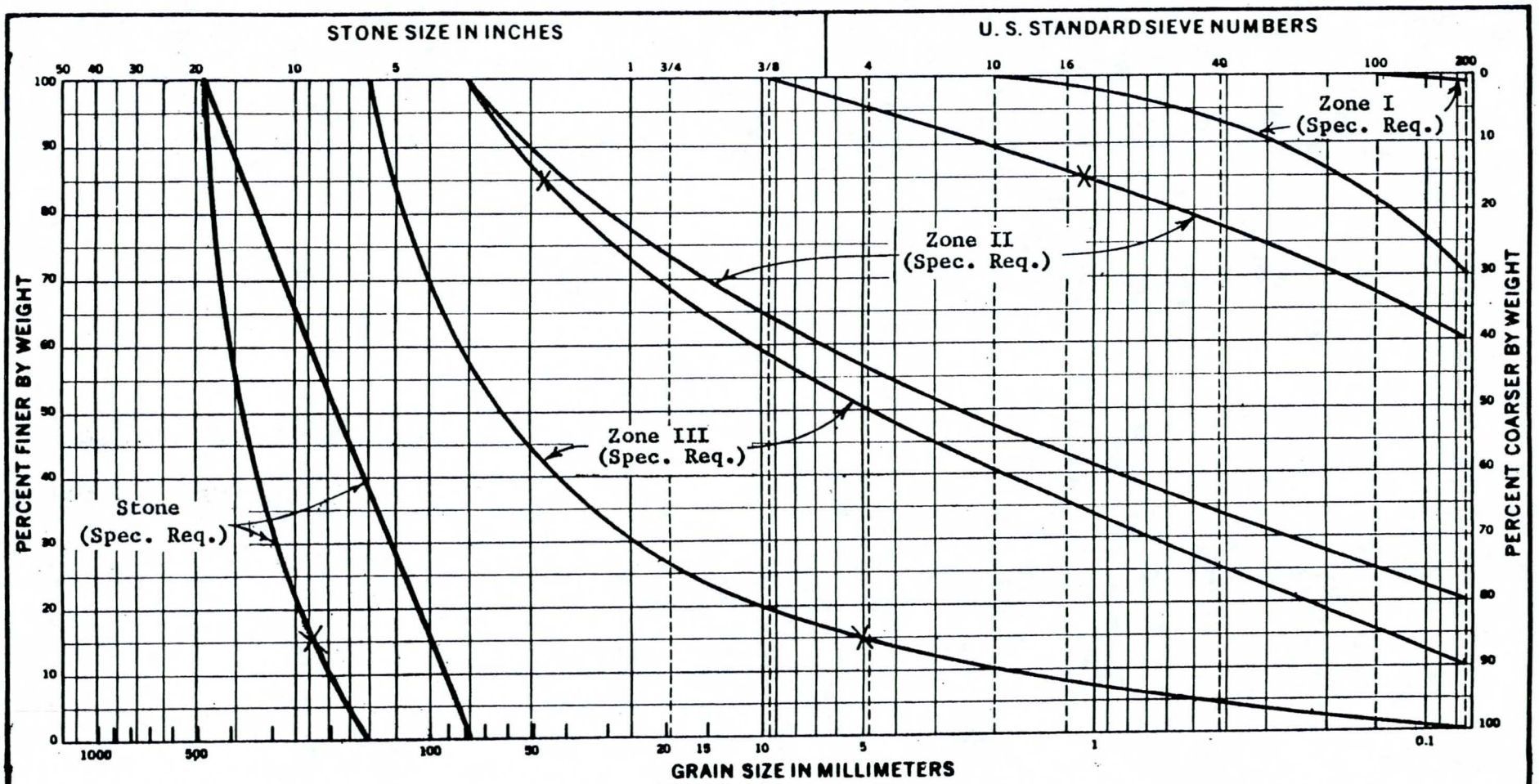
41. Record Tests. In order to verify that design assumptions were met, record samples were periodically obtained by Foundations and Materials Branch personnel. These samples were shipped to the SPD Soils Laboratory for record testing. Record tests were performed to evaluate the material type, density, permeability, shear strength, consolidation, and other soil parameters. Undisturbed cubic foot samples were obtained in Zone I and Zone II materials at locations upstream and downstream of the centerline of the embankment as planned in the "Cave Buttes Dam Engineering Considerations and Instructions for Field Personnel." Three field density determinations were also required at all samples locations.

a. Permeability. Permeabilities of record samples were measured in both the horizontal and vertical directions in accordance with the procedure described in Engineer Manual EM 110-2-1906 "Laboratory Soil Testing," 30 November 1970.

(1) Zone I Materials, Main Embankment. The vertical and horizontal permeabilities of undisturbed record samples obtained from Zone I of the main embankment are summarized in table 3. The vertical permeabilties averaged [illegible]x$10^{-4}$ fpd and the horizontal permeabilities averaged 3.56x$10^{-4}$ fpd. The horizontal permeabilities averaged higher than the vertical permeabilities, and both horizontal and vertical permeabilities were considerably lower than the design permeability of 1.0x$10^{-3}$ fpd.

(2) Zone II Materials, Main Embankment. The vertical and horizontal permeabilities of undisturbed record samples obtained from Zone II of the main embankment are summarized in table 3. The vertical permeabilities averaged 6.50x$10^{-2}$ fpd and the horizontal permeabilities averaged 1.75x$10^{-1}$ fpd. The horizontal permeabilities averaged higher than the vertical permeabilities. The horizontal permeabilities were slightly higher and the vertical permeabilities were slightly lower than the design permeability of 1.0x$10^{-1}$ fpd, however, both the horizontal and vertical permeabilites were in the same order of magnitude of the design value.

(3) Zone II Materials, Dike No. 2. The vertical and horizontal permabilities of undisturbed record samples obtained from Zone II of Dike No. 2 are summarized in table 3. The vertical permeabilities averaged 4.28x$10^{-2}$ fpd and the horizontal permeabilities averaged 8.6x$10^{-2}$ fpd. The horizontal permeabilities averaged slightly higher than vertical permeabilities, and both horizontal and vertical permeabilities were considerably lower than the design permeability of 1.0x$10^{-1}$ fpd.




$$\frac{D_{15}\text{ (coarse fraction of Zone II)}}{D_{85}\text{ (fine fraction of Zone I)}} = \frac{0.04}{0.0085} = 4.7$$

$$\frac{D_{15}\text{ (coarse fraction of Zone III)}}{D_{85}\text{ (fine fraction of Zone II)}} = \frac{5.0}{1.1} = 4.5$$

$$\frac{D_{15}\text{ (coarse fraction of Stone)}}{D_{85}\text{ (fine fraction of Zone III)}} = \frac{220}{46} = 4.8$$

GILA RIVER BASIN
New River and Phoenix City Streams
Arizona

CAVE BUTTES DAM

Embankment Filter Requirements

APPENDIX IA Figure

# TABLE 2
## VERIFICATION TESTING
## SUMMARY OF ZONE I RESULTS

| SPECIFICATION OR DESIGN VALUES | | LEFT ABUTMENT | | | MAIN EMBANKMENT | | | RIGHT ABUTMENT | | |
|---|---|---|---|---|---|---|---|---|---|---|
| PROPERTY TESTED | SPECIFICATION OR DESIGN VALUE | LOWER DECILE | MEDIAN | UPPER DECILE | LOWER DECILE | MEDIAN | UPPER DECILE | LOWER DECILE | MEDIAN | UPPER DECILE |
| GRADATION (% PASSING NO. 200) | >70 | 67 | 84 | 93 | 72 | 87 | 95 | 90 | 96 | 98 |
| DRY DENSITY (P C F) | 95 | 96.1 | 104.0 | 111.1 | 97.4 | 103.0 | 108.9 | 87.0 | 95.9 | 100.4 |
| MOISTURE CONTENT (% OF DRY WT.) | 21 | 17.4 | 20.8 | 25.4 | 18.5 | 21.7 | 24.8 | 19.3 | 23.7 | 25.0 |
| PERMEABILITY (FT./DAY) | 0.001 | 0.000086 | | | 0.000085 | | | | | |

# TABLE 3
## VERIFICATION TESTING
## SUMMARY OF ZONE II RESULTS

| SPECIFICATION OR DESIGN VALUES | | LEFT ABUTMENT | | | MAIN EMBANKMENT | | | RIGHT ABUTMENT | | |
|---|---|---|---|---|---|---|---|---|---|---|
| PROPERTY TESTED | SPECIFICATION OR DESIGN VALUE | LOWER DECILE | MEDIAN | UPPER DECILE | LOWER DECILE | MEDIAN | UPPER DECILE | LOWER DECILE | MEDIAN | UPPER DECILE |
| GRADATION (% PASSING NO. 200) | >20 <60 | 20 | 50 | 77 | 27 | 38 | 56 | 25 | 31 | 68 |
| DRY DENSITY (PCF) | 118 | 100.1 | 116.2 | 121.5 | 104.9 | 115.4 | 123.7 | 115.9 | 119.0 | 124.5 |
| MOISTURE CONTENT (% OF DRY WT.) | 8.5 | 5.8 | 12.8 | 20.2 | 7.2 | 11.6 | 16.8 | 9.5 | 11.0 | 15.7 |
| PERMEABILITY (FT./DAY) | 0.10 | | | | 0.000642 | | | | | |

A350.602
28

GILA RIVER BASIN
NEW RIVER AND PHOENIX CITY STREAMS, ARIZONA

DESIGN MEMORANDUM NO. 3
GENERAL DESIGN MEMORANDUM — PHASE II

PROJECT DESIGN
PART 1

HYDRO LIBRARY

# CAVE BUTTES DAM

(INCLUDING CAVE CREEK TO PEORIA AVENUE)

U.S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

JULY 1976

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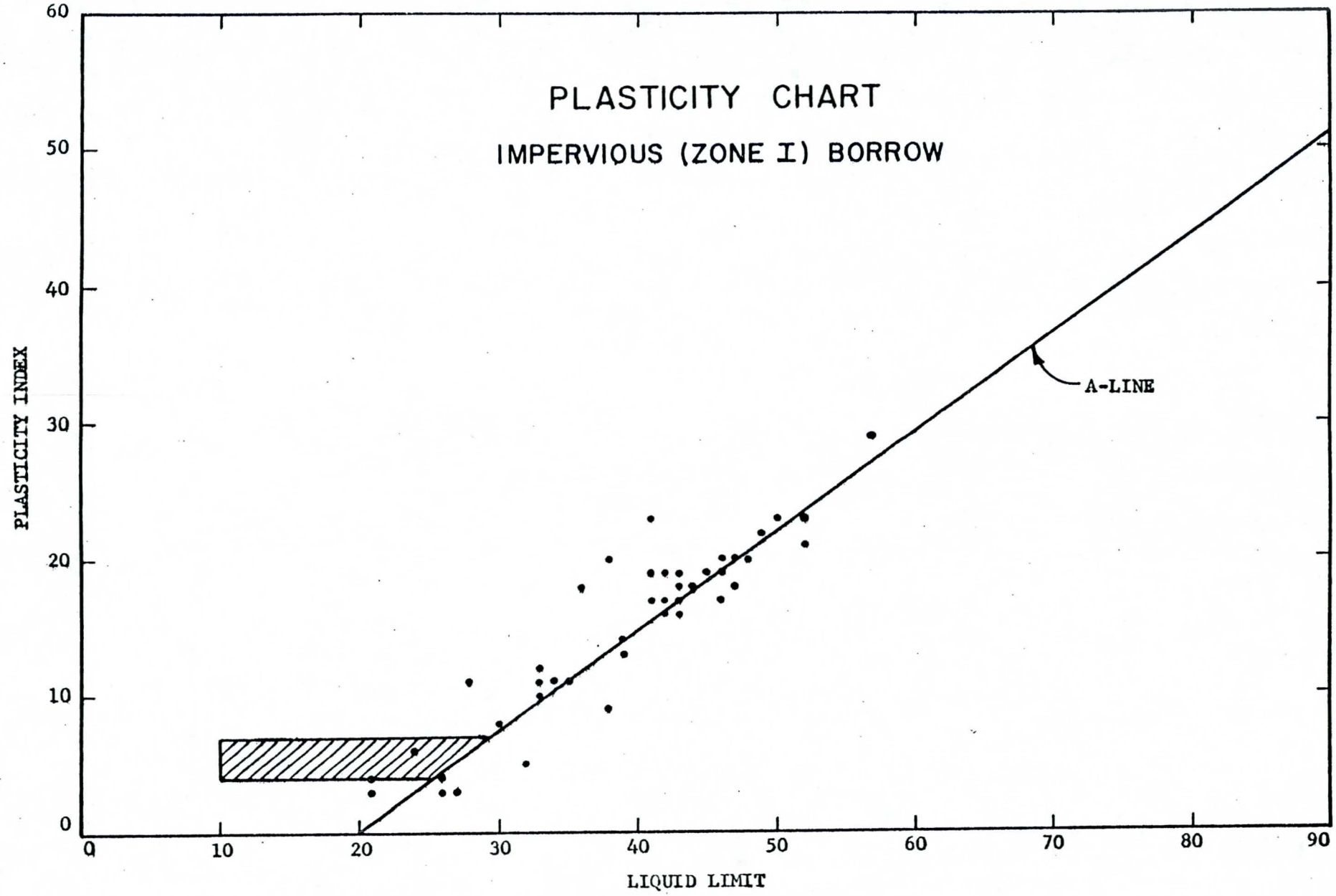
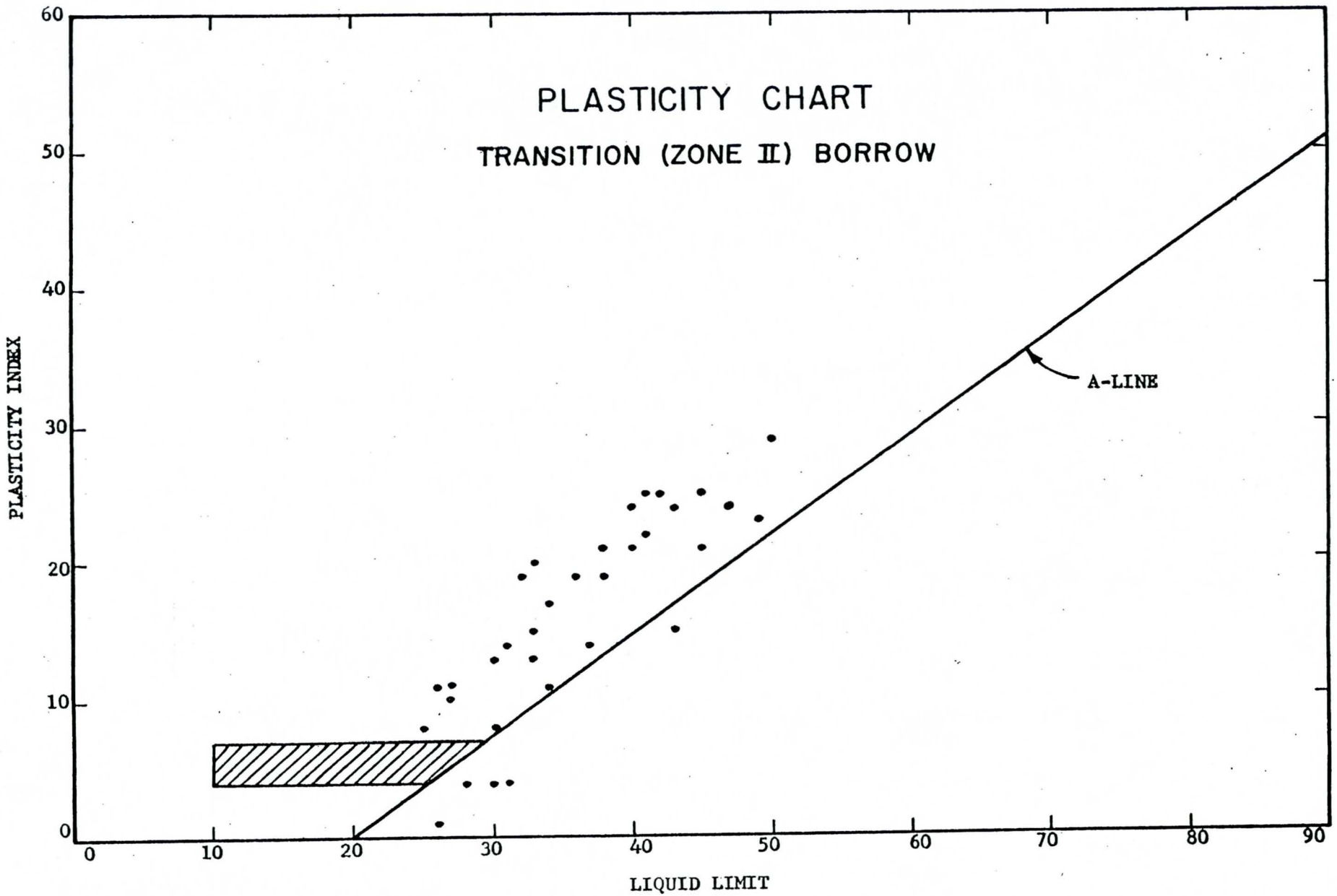
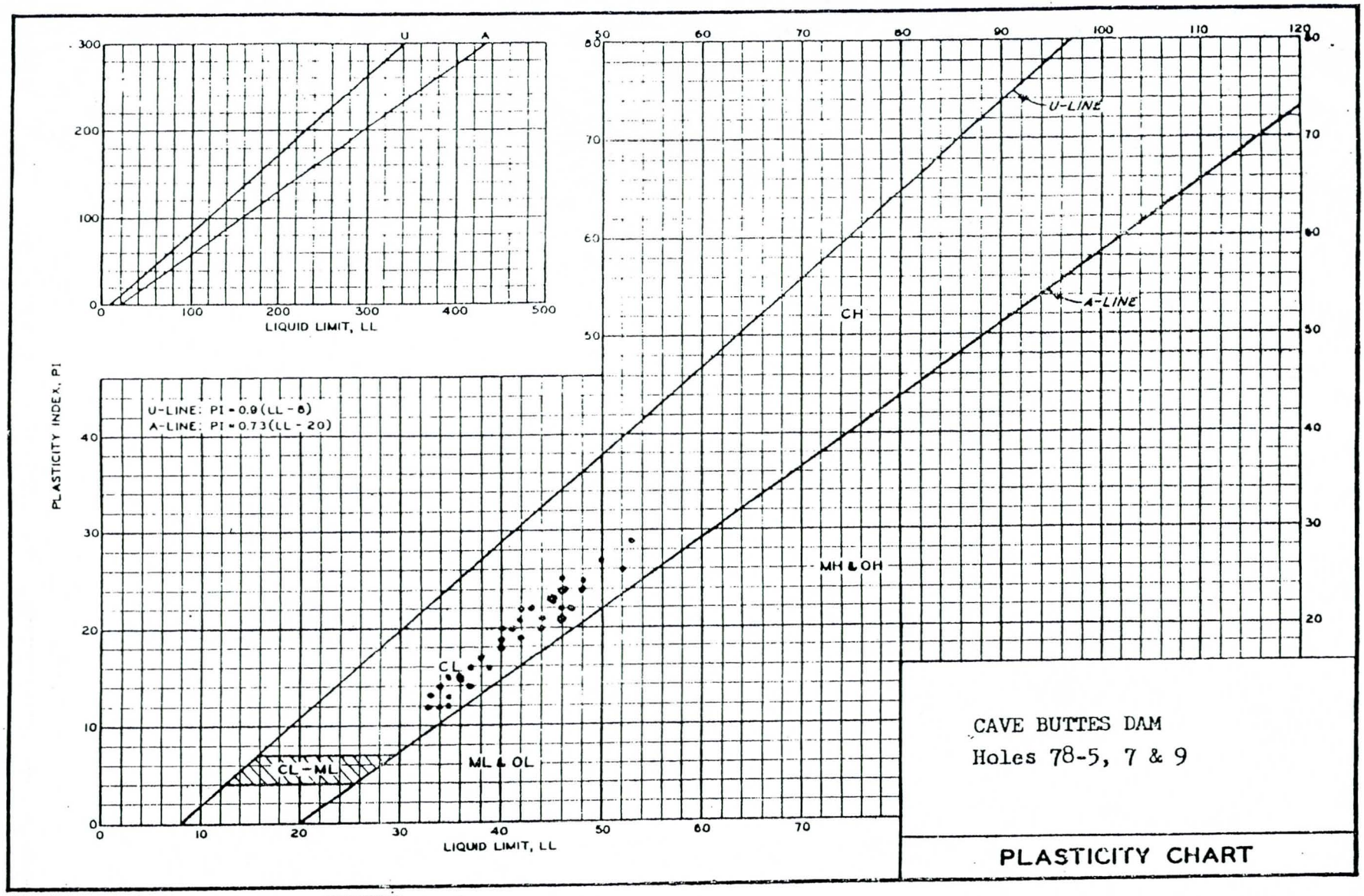
DM-31




APPENDIX IA Figure 6




APPENDIX IA Figure



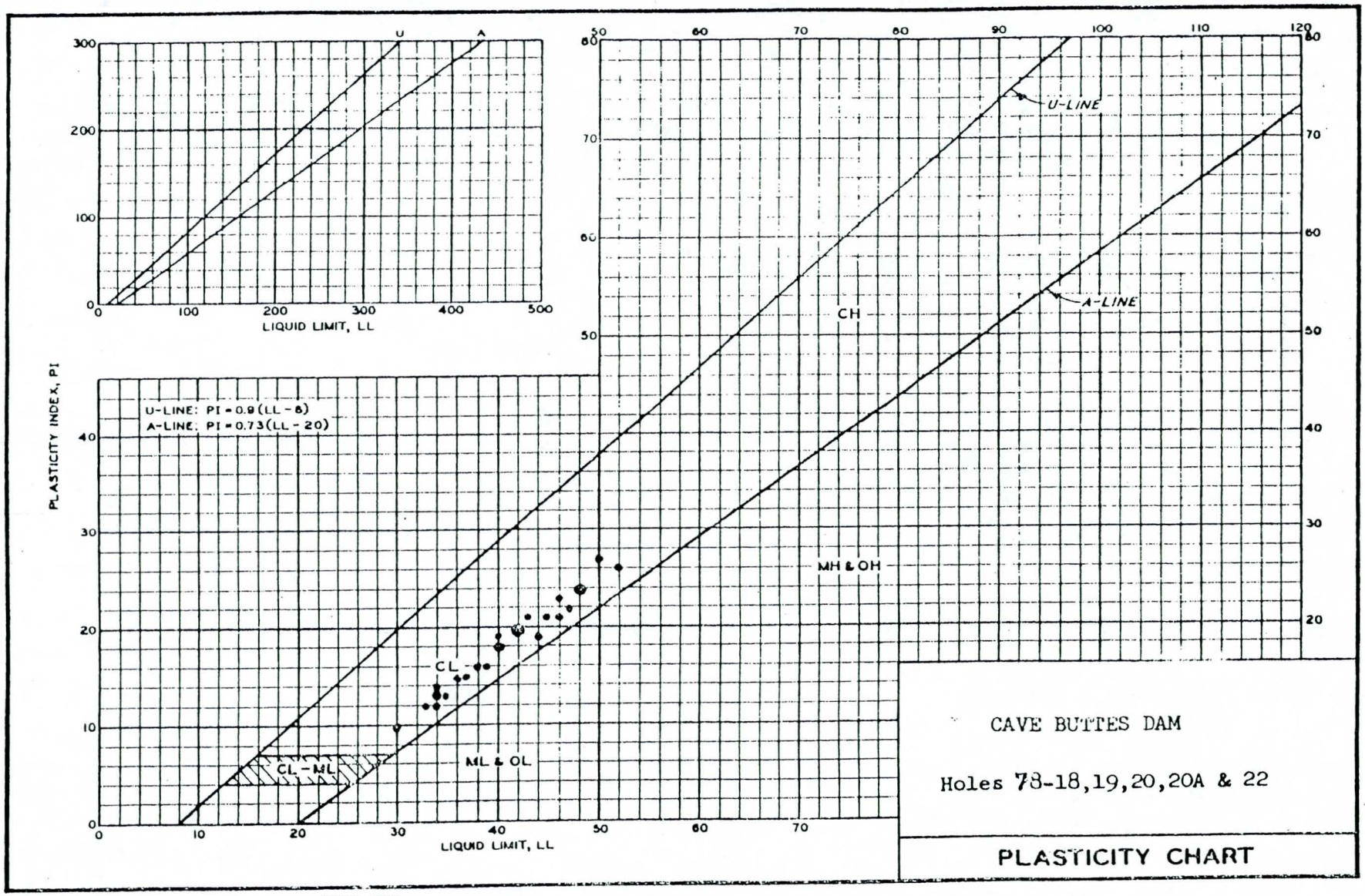

CAVE BUTTES DAM
Holes 78-5, 7 & 9

PLASTICITY CHART

ENG FORM JUNE 1970 4334

TRANSLUCENT

Plate 12




CAVE BUTTES DAM

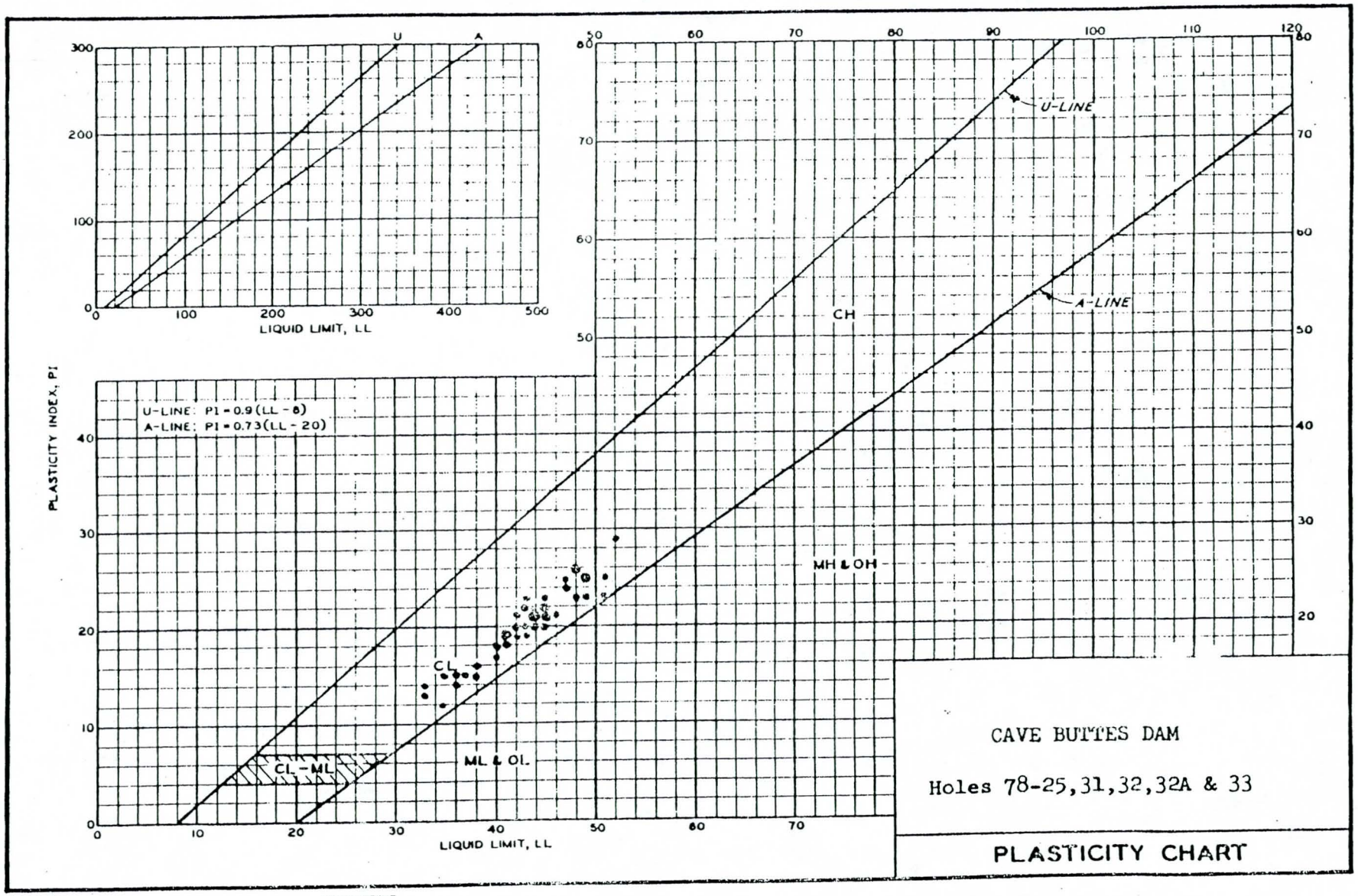
Holes 78-18,19,20,20A & 22

PLASTICITY CHART

ENG FORM JUNE 1970 4334

TRANSLUCENT

Plate 13




CAVE BUTTES DAM

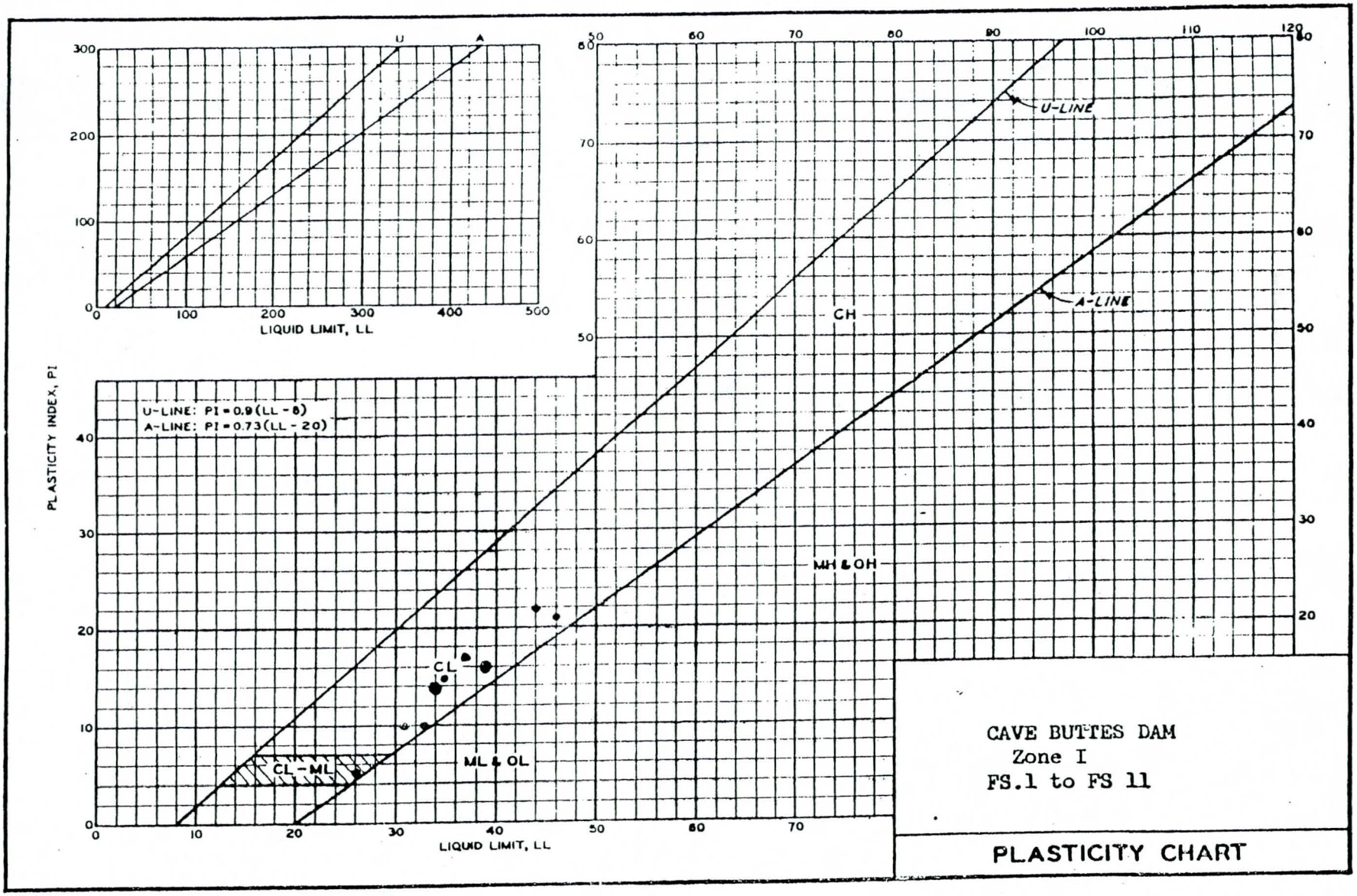
Holes 78-25,31,32,32A & 33

PLASTICITY CHART

ENG FORM 4334
JUNE 1970

TRANSLUCENT

Plate 14



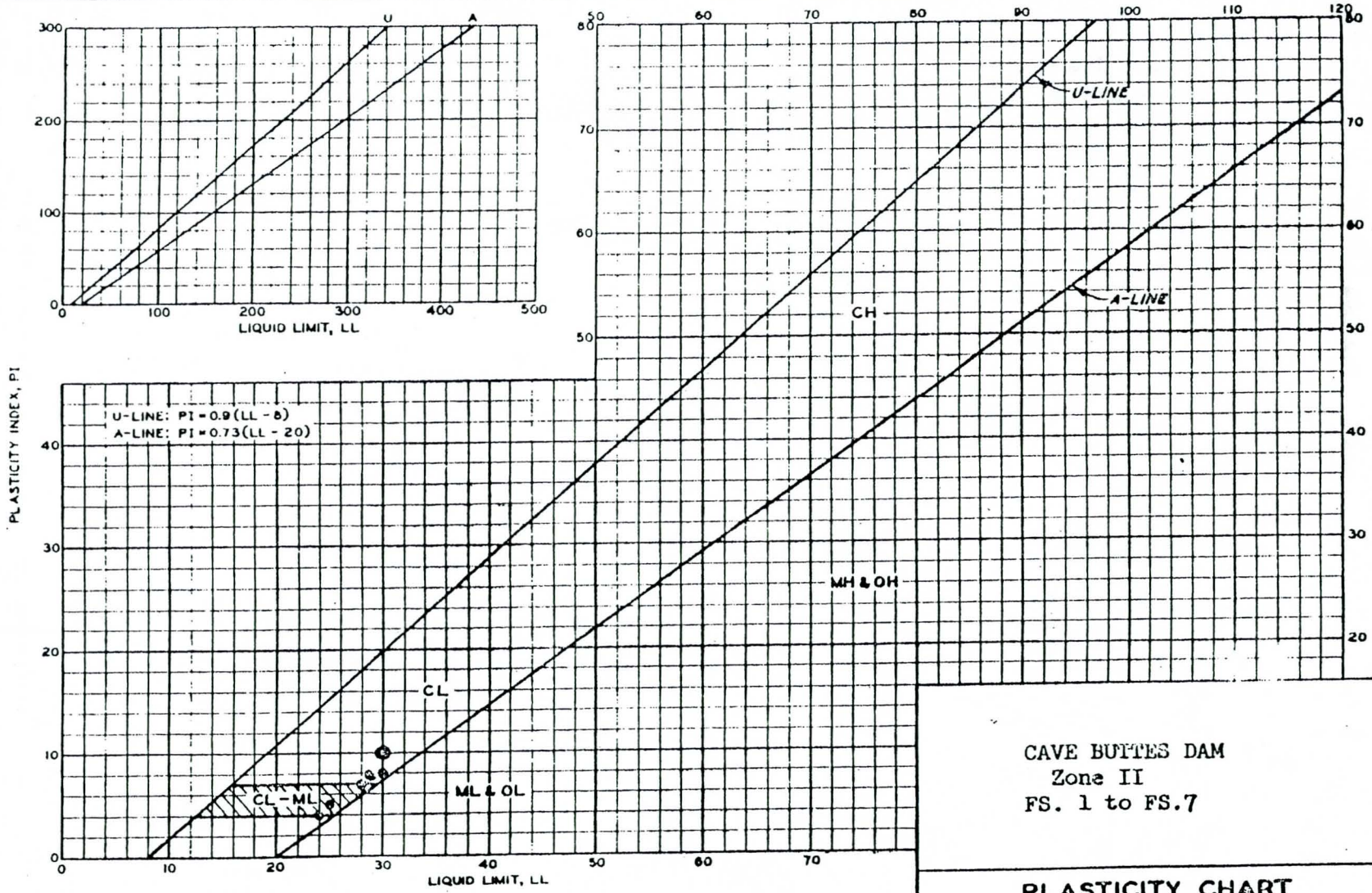

CAVE BUTTES DAM
Zone I
FS.1 to FS 11

PLASTICITY CHART

ENG FORM JUNE 1970 4334

TRANSLUCENT

Plate 15




CAVE BUTTES DAM
Zone II
FS. 1 to FS.7

PLASTICITY CHART

ENG FORM JUNE 1970 4334

TRANSLUCENT

Plate 16

# FAILURE MODE AND CONSEQUENCE ANALYSIS
# For
# POWERLINE FLOOD RETARDING STRUCTURE

## PINAL COUNTY, ARIZONA
## NATDAMID AZ 00082

**July 16 2001**

## 1.0 INTRODUCTION

### General Description

Powerline FRS is located about 35 miles east of downtown Phoenix and five miles southeast of the City of Apache Junction. The project consists of the FRS embankment structure and an emergency spillway. The project is part of the Apache Junction-Gilbert Watershed Protection and Flood Prevention Project.

Powerline FRS is classified as a medium sized, significant hazard dam. The reservoir behind the FRS is 610 acres with a capacity of 4,194 acre-feet. Construction of the Powerline FRS was accomplished under contract to the Ashton Company of Tucson, Arizona. Construction of the FRS and appurtenant structures was completed in 1967.

The dam has performed satisfactorily to date. However, because of significant cracking experienced during and after their first significant impoundment by the adjacent, companion FRS structures, Vineyard and Rittenhouse, Powerline was also provided with the same post construction central filter remediation that each of those structures were given.

### Dam Data

Dam type: Rolled earthfill
Dam height: 21.0-ft
Dam length: 13,398-ft
Dam crest: width 14-ft; elevation 1589.1-ft
Spillways: Principal - 36-inch RCP; inlet elevation 1563.0-ft; Emergency spillway - 600-ft wide earthlined spillway; crest elevation 1583.3-ft
Freeboard: 4.8-ft
Hazard Classification: Significant

### Purpose and Scope

In general, the purpose of the Failure Mode and Consequence Analysis (FMCA) exercise was to:

- Identify potential site specific failure modes for the dam.
- Discuss non-quantitatively the likelihood of the identified failure modes.
- Determine whether or not and how important failure mechanisms are being monitored

- Examine the consequences of failure and the adverse consequences of successful operation (e.g. – large spillway releases)
- Identify possible risk reduction actions that may be taken to reduce the likelihood of failure or to mitigate adverse consequences
- Determine what information, investigations or analyses may be needed to resolve uncertainties relative to potential failure modes.

(Note: For this exercise the FMCA team only examined the general nature of the "consequences" for the failure modes identified and where appropriate estimated how they may be different than previously anticipated . Greater detail on the estimate of the magnitude of the "consequences" for each significant failure mode will be addressed in the quantitative portion or risk analysis part of the risk assessment for the dam. At this time hopefully more detailed discussion on the emergency response plan for each structure will also be possible.)

**Team Members**

Larry Von Thun, Engineering Consultant and FMCA facilitator
Bob Eichinger, Kimley-Horn and Associates, Inc., Project Manager
Jim Scott, URS Corporation, Principal
Ken Euge, Geological Consultants, Principal
Tom Renckly, Flood Control District of Maricopa County, Project Manager,
Larry Lambert, Flood Control District of Maricopa County, Dam Safety Engineer
George Beckwith, Flood Control District of Maricopa County, Geotechnical Engineer
Justin Beeler, Kimley-Horn and Associates, Inc, Session Recorder
Tim Murphy, Flood Control District of Maricopa County, Civil Engineer (consequences)

## 2.0 MAJOR FINDINGS AND UNDERSTANDINGS GAINED

The following is a summary of the major findings and understandings for Powerline Flood Retarding Structure as a result of the Failure Mode and Consequence Analysis (FMCA). Powerline FRS is one of three adjoining flood retention dams (Vineyard Road and Rittenhouse FRS being the other two dams). Because of concerns about drying shrinkage and desiccation cracking each dam was modified after construction but in slightly different ways. In carrying out the FMCA for these dams Vineyard Road FRS was done first followed by Rittenhouse and then Powerline FRS. The potential failure modes considered for the dams were of course very similar and it was natural to compare the likelihood of development of these failure modes among the three dams. Thus in the major findings and understandings given below some of the differences are noted as key findings and conditions of general similarity related to potential failure modes are also noted.

The major findings and understandings given below are organized as follows. First the most important design, construction, geologic, and performance differences or unique aspects related to the potential for failure mode development of Powerline FRS are given. Findings related to failure modes or adverse consequences other than overtopping and spillway discharge are given first. Findings related to consequences are given next

followed by action items (risk reduction and investigations). Finally, general findings that are informational and / or generally similar for all three dams are listed. In making comparisons of the dams and the amount of cracking experienced at the time of the repairs it is instructive to note the year of the original construction of each and the year those repairs were made. The difference between these two dates [shown in brackets] is the number of years drying had taken place before the central trench was excavated. Each trench was mapped for cracking depth, width and extent. They were Vineyard Road FRS – Constructed 1967, repaired 1983, [16 years], Rittenhouse – Constructed 1969, repaired 1979, [10 years], Powerline – Constructed 1968, repaired 1991 [23 years].

**Key Findings / Differences Related To Failure Mode Development – "Static Loading Failures – Seepage Erosion – Fissuring – Foundation Erosion –Etc."**

- New Fissure found Just West of Powerline FRS. AMEC's ongoing Phase II investigations have included review of the geologic profile, history of ground water decline, measured subsidence and reported earth fissures in the vicinity of the Powerline, Vineyard Road and Rittenhouse FRS's. In examination of aerial photographs, AMEC determined that a previously identified liniment west of the dam is an earth fissure. The fissure appears to terminate about one-quarter mile west of Station 110+00 on the east side of Ironwood and just west of the CAP Canal. There is a high probability that the fissure will ultimately migrate through the dam. Observation of the concrete lining of the CAP Canal will be a useful indicator in monitoring growth of this fissure.
- High Potential for Subsidence and Earth Fissuring Owing to the groundwater overdrafts, large differences in bedrock depth over the site and presence of a thick sequence of more compressible lakebed clays within the alluvium there is a relatively high potential for differential subsidence and earth fissuring at Powerline FRS.
- Potential For Differential Settlement From North End Of Structure To South End Of Dam. The dam is underlain by a variable deposit of alluvial sediments. These deposits could be impacted by the ongoing subsidence occurring within the region. The FMCA team noted that on the basis of the thickness of the materials susceptible to subsidence that the structure and/or reservoir could experience tilting in the future. The subsidence amount would be expected to be high on the north end of the dam and reservoir basin and lower on the south end of the dam and reservoir. Since the spillway is on the south side of the reservoir the tilting effect, if it occurred as expected would increase reservoir storage (See settlement profiles – Powerline FRS Appendix). Three factors were identified that could potentially create embankment and foundation cracking and conditions for potential internal seepage-erosion modes of dam failure.

  a) Collapsing Holocene soils extend to as much as 10 feet beneath the cutoff trench and central filter along approximately 64 percent of the embankment. These deposits are susceptible to large "collapse" settlements upon wetting and have the potential to create transverse or diagonal cracks through the embankment and upper part of the foundation. Most are highly erodible and susceptible to seepage erosion.

  b) Ground water declines have lead to land subsidence and earth fissuring at the site and in the vicinity. This process has produced substantial differential settlements of

the dam and reservoir of the Powerline FRS and the adjacent Vineyard FRS and a relatively high risk of earth fissuring beneath the dams. Differential subsidence may have increased reservoir storage capacity.

c) Transverse & Diagonal Cracking. Within a few years after completion of the dam in 1967, extensive transverse and diagonal cracking of the embankment was noted. It was concluded in post-construction geotechnical investigations that the primarily cause is drying shrinkage. A central filter was installed in 1989 as a defense against seepage erosion through cracks. The FMCA Team has not located records of the embankment cracking made by inspection of the central filter trench during the 1989 construction. Thus, the depth, width and intensity of cracking are not entirely known. The limited available soil classification tests suggest that the embankment has less drying shrinkage potential than the Vineyard FRS, but the data is inconclusive. The central filter was extended below the bottom of the cutoff trench over the full length of the dam suggesting that the cracking may have been less severe or as severe as that of the Vineyard FRS. Cracks at the Vineyard FRS extended to more than 21 feet. A considerable number extended below the embankment into the native soils.

- Collapsing Holocene Soils along Foundation of Dam. The FMCA team reviewed foundation excavation records for Powerline FRS. These records indicate that great detail and care was taken in determining the amount of excavation needed to remove Holocene soils on the upstream portion of the section (more care than at Vineyard and Rittenhouse). However, Holocene materials were likely left in on the downstream section. This material is susceptible to seepage erosion. There was an awareness during construction that gravel deposits upstream could allow inflow along the base of the dam and thus this flow needed to be cut off. These inferences were made on the basis of the ADWR inspection reports.
- Central Filter Extends to Foundation. The Powerline central filter repair extends to the full depth to foundation (and somewhat below) and for the entire length of the dam.
- Understanding of Filter Characteristics and Role of Filter. The central filter for Powerline FRS was installed by SCS in 1989 and was intended to be a "crack stopper". The defensive mechanism provided is filtering the movement of fine particles and the building of a filter cake as seepage through an upstream transverse crack or interconnected system of cracks begins. This process prevents seepage-induced erosion of the walls of cracks upstream of the filter, minimizes flow into the filter and thereby prevents seepage erosion of downstream transverse cracks. The specified filter meets generally accepted criteria for filtration. The factors that could potentially create defects that would compromise the function of the filter are segregation during construction and cementation that would allow propagation of cracks through the filter. An internal SCS report (Leckband, 1984; "A Barrier to Cracks in Dry Dams") states that excavations through the central filter of the Vineyard FRS stood vertically for the full height of the dam. This suggests that the filter may be cemented and thereby susceptible to cracking due to settlement, embankment shrinkage, or earth fissures (See Powerline FRS Appendix for SCS note of July 1984). This fact is very significant for Powerline regarding potential cracks from fissuring and subsidence (or differential settlement.).

- No Finger Drains. No drainage of the central filter was provided in the design in the event it is filled with water through rainfall, foundation seepage or other means. This means that high hydrostatic heads could develop in the filter, which would seek an outlet through an opening for drainage such as a downstream crack. However, if no defects are present in the filter, rapid development of a filter cake would be expected to reduce flow into the filter to an amount where finger drains are not needed.
- Potential Loss of Central Filter Material through Crack. The potential exists that there may be a loss of central filter material through a downstream crack should such a crack promulgate to the filter from the downstream face and that there is water present in the central filter to drive the material through the crack. (The relevance of this statement is based on the experience with cracks at Vineyard and Rittenhouse. If the cracks are not more than 0.6 inches wide as was measured for the Rittenhouse FRS, this would not be a problem. If the cracks are 2 or 3 inches wide as for Vineyard, then it is a serious problem.)
- Foundation was Not Pre-Wetted. The foundation soils were not pre-wetted as was done at Rittenhouse FRS to minimize collapse potential of the Holocene soils and associated cracking of the dam (especially longitudinal cracking) upon wetting from a significant flood. This process is believed to have been experienced at Vineyard FRS based upon the observations of Holocene soils left in the foundation and the cracking conditions observed.
- Cutoff Trench Construction. In contrast to Vineyard FRS, there are clearly very large areas at Powerline FRS foundation where an interval of Holocene soils up to 10 feet in thickness was left below the cutoff trench and central filter (Note: Unlike Vineyard Road FRS, it does not appear that extending the cutoff trench through the Holocene soils was intended. This may be because the embankment cracks were shallower and narrower than Vineyard and did not penetrate into the foundation. If this is so, seepage erosion at the embankment-foundation interface was probably not deemed to be a credible scenario. This underscores the importance of locating the SCS crack survey records.)
- Reservoir Drawdown Duration. The original hydrology for the Powerline FRS was analyzed by the NRCS (SCS) in 1964 to 1965. The NRCS analyzed two reservoir drawdown durations based on the proposed initial flood routing conditions- a 10-day drawdown and a 30-day drawdown. A 10-day drawdown duration was based on an initial flood routing beginning at the top of the sediment pool (crest of principal spillway). The 10-day drawdown was analyzed assuming the reservoir would be entirely drained at the end of 10-days. A 30-day drawdown was based on an initial flood pool elevation of 1580.8-feet. The premise of the 30-day drawdown was the reservoir for Powerline would be impounded to an elevation o 1580.8 feet (see Powerline FRS Appendix – excerpts from NRCS Powerline Dam – Hydrology & Hydraulics). In other words, the starting conditions for routing the 100-year flood through the reservoir assumed a pool elevation of 1580.8 feet. It would take 30-days to drawdown the reservoir if this conditioned occurred. It appears that the NRCS documentation (see Appendix) has designed for a 30-day drawdown. The supporting evidence was in the selection of the principal spillway pipe diameter, which was based on the required 30-day discharge rating. However, it is most likely that the reservoir for Powerline FRS will be empty at the initiation of a 100-year event and

that the 10-day drawdown duration would be valid. The FMCA team noted that the available time to develop flow paths and initiate seepage erosion is very short (even in the case of a 30-day drawdown time). The storage volume and routings need to be verified and confirmed.

- Segregation Potential in Central Filter Material. Analysis of the gradations of the central filter had a degree of susceptibility to segregation. There were several instances of the material being dumped from considerable height rather than being tremied into place.
- Reduced Potential for Central Filter Cementation at Powerline. The potential for filter cementation was reduced in the 1991 repair project by the addition of approximately one-foot of material on the dam crest. This cap of material reduces the surface infiltration and thus reduces the/ filter cementation potential.
- Low Potential for Failure of Corrugated Metal Irrigation Outlet Pipe. The FMCA team agreed that the failure mode of piping or seepage erosion along the 12-inch CMP irrigation outlet is not a significant risk. One of the primary reasons is because it is at rather high elevation and thus has limited chance of having a significant head for any sustained period of time to allow development of a seepage path along the pipe boundary.
- Piping at Principal Outlet Not a Failure Mode. The principle spillway conduit is a large diameter ungated (36-inch) reinforced concrete pipe (RCP) resting on concrete cradle and has 5 anti-seep collars. Seepage erosion along the conduit is not considered to be a significant failure mode. Less potential for plugging, less time for piping to develop, tested under head several times with satisfactory performance, and RCP has history of satisfactory performance. (NOTE: Powerline has a 10-day time of release (the same as Vineyard), and thus is subject to high global gradients forcing seepage erosion for a lessor time than Rittenhouse with it's 30-day time of release.)
- Seismic Loading of the Dam was not considered as a Major Issue. A Seismic Exposure Evaluation by AMEC indicates that the peak horizontal acceleration (PHA) at the Powerline FRS due to the maximum credible earthquake (MCE) is about 0.12g. These small dynamic forces would not cause appreciable deformations of the stiff, unsaturated embankment, so earthquakes do not constitute a significant risk. There were no potential failure modes identified due to seismic loading and no "incipient" static loading potential failure mode that would be triggered by seismic load could be identified.
- Dispersivity Tests. Pinhole and SAR test results on materials indicate the embankment soils are non-dispersive.
- High Quality of Embankment. The District's analysis of field compaction testing done by the SCS during the original construction in 1966 and 1967 indicates a standard deviation for variation from the optimum moisture content of 1.0 percent and a standard deviation for percent compaction of 1.6 percent. These values compare with minimum recommended standard deviations of 1.5 percent for moisture content variation and 3.0 percent for percent compaction based on comprehensive studies of the quality control records of dams by the U. S. Bureau of Reclamation. The mean percent compaction for the embankment was 98.2 percent of ASTM D698 maximum dry density and the mean compaction moisture content was 0.3 percent below the

optimum moisture content. This analysis indicates excellent quality control testing procedures and a high quality embankment.

**Key Findings / Differences Related To Failure Mode Development – "Flooding, Overtopping, Spillway Discharges."**

- Overtopping of Powerline FRS. The FMCA team observed that a previous hydrologic and dambreak study for Powerline FRS (Flood Control District – Addendum-A Hydrologic Analysis of Powerline FRS, June 1989) concluded that the dam would be overtopped during a full PMP/PMF event. The height of overtopping estimated is approximately 1.33-ft with a duration of overtopping of approximately 2.8-hours. The amount of overtopping is greater than that estimated at Vineyard Road FRS. However, as noted below, the crest of the dam was raised and the survey following the raising shows the dam to be relatively level, thus the likelihood of failure due to overtopping may actually be less at Powerline than at either Vineyard Road or Rittenhouse.
- Crest Raised (1991). The dam crest was raised to meet the original design crest elevation of 1589.1 feet during the 1991 dam repair and construction of the central filter. This reduces the overtopping potential and most likely creates more storage than accounted for in the hydrologic analysis. The dam crest was raised approximately one-foot.
- Settlement Survey Data. The survey data completed in the past for the dam crest monuments and downstream toe monuments needs to be comprehensively reviewed and the stability of the benchmarks used for the surveys must be verified. Surveyed elevations of the monuments located near the crest do not reflect top of dam elevations.. True elevations of dam features (crest of dam, emergency spillway crest, principal spillway, etc.) need to be established. Need elevation data for sections of dam next to emergency spillway and need to mark elevations on trash rack of principal spillway. Survey data following the 1991 crest raise indicates that the dam crest is higher and more level but the survey data still needs to be verified.
- Emergency Spillway Erosion. There is a potential for erosion of earth-lined emergency spillway (base and sidewalls, due to high flow velocities (range of the estimated velocities 10 to 12 feet per second) at larger discharges. Although the 1989 dam breach analysis showed large spillway erosion, amplification of the inflow hydrograph of the PMF and consequent effects on downstream inundation, there appears to be minimal potential for erosion effecting spillway discharge. There are strongly cemented soils at or just below spillway grade along parts of the alignment. Further study is needed to properly define this issue.

**Consequence Evaluation**

- Central Arizona Project Canal. The CAP canal should be evaluated relative to the consequence of dam failure. The capacity of the CAP Canal embankment to divert outflow from dam breaches into the canal thereby reducing consequences was noted by the FMCA team. Such diversions may help reduce the impacts of dam failure by allowing ponding to occur on the upstream embankment of the canal. The

team also noted that the CAP canal may potentially cause an increase in flooding should the canal fail as the result of a dam breach. The CAP canal has a smaller, ( as compared to CAP at vineyard) but significant potential impact on consequences of dam breach (especially at lower flows). Distortion or cracking of the concrete lining and embankments of the CAP canal may provide an early warning of fissure development. The ability of the CAP canal to handle/store flows resulting from failure at Powerline seems to be considerably less than at Vineyard and Rittenhouse. This is especially true due to the canal flow restrictions at the upstream canal overchute area and at the CAP bridge just downstream. However both of these restrictions could be countered to take advantage of the canal in attenuating/lessening dam failure peak outflow flooding..

- Dambreach Characteristics. Because of the stiff, clayey, unsaturated embankment soils, it is anticipated that the rate of development of a breach at Powerline FRS will be relatively slow. The weight of evidence indicates that the embankment material possesses a relatively high erosion resistance. It is expected that breaches involving both overtopping and seepage erosion failure modes would be narrow and take a long time to form. Relevant case histories of dam breaches involving similar embankment materials include the Church Rock tailings dam near Gallup, New Mexico and the Clear Creek dam near Winslow, Arizona (physical breach models). The estimated breach development time for these dams was estimated to be less than 12 hours each and the sidewalls were near vertical. The development of a breach at Powerline FRS is anticipated to occur relatively slowly.
- Importance of Good Consequence Scenarios. During the process of risk analysis, it is important to develop an understanding of the importance and influence of the CAP canal and the buffer zone between the dam and the CAP canal (and flows into the wasteways) on the area of inundation and to include these effects in estimating the consequences of Dam failure from potential hydrologic and piping dam failures. The canal may help buffer any emergency spillway or dam breach discharges.
- Importance of Incremental Damage Analysis. In the process of Risk Analysis, it will be important to route different flood frequencies, peaks, and volumes to determine recurrence interval of incipient overtopping. The studies should include comparison of the inundation limits from spillway flows and combined spillway and overtopping flows. Differences may be small. Results of study will provide the basis for determination of the need to raise the dam.
- Benefits of Existing District Easements. (for Powerline FRS, Vineyard Road FRS and Rittenhouse FRS)
  a) Existing easements reduce the potential for impacts to the safe operation and proper function of the dams from activities by others.
  b) Existing easements reduce potential impacts to others due to reservoir impoundment or emergency spillway discharges.
  c) Existing easements are useful for identification and monitoring of subsidence fissures and implementation of mitigation measures for fissuring if determined to be required.
  d) Existing easements provide a measure of control of adjacent groundwater pumping

e) Existing easements will be utilized as needed in implementing economical dam replacement and/or dam rehabilitation flood control projects.

**Action Items – Risk Reduction or Investigations**

- Nature of Earth Fissure Risks. The weight of evidence indicates that earth fissure development is continuing in the vicinity of Powerline FRS and there is a plausible scenario in which an earth fissure could cross the alignment of the dam some time in the future. An earth fissure crossing the dam could create a continuous open crack, about ¾-inch in width, through the embankment and foundation. If an undetected earth fissure were present at the time of a major flood, there would be a high risk of a rapid seepage-erosion failure passing through the highly erodible Holocene soils just below the embankment. If the fissure lined up with a serious defect in the central filter, there would be a moderate risk of a seepage erosion failure passing through the embankment. Investigations of earth fissuring in the vicinity of Powerline Dam are ongoing. Conclusions and recommendations from these investigations are forthcoming. For the interim period, it is recommended that known earth fissures in the vicinity of the dam are routinely monitored under the District's Dam Safety Program.
- Risk Reduction Measures. The FMCA team identified several measures that may be taken by the District at the present time to potentially reduce risk. These measures include: (1) Keep the crest of the dam level and at the design elevation; (2) Monitor the impoundment levels during storm events (this includes both remote and as well as on-site monitoring); (3) Provide training to O&M crews regarding earth fissure development and other key observational parameters such as seepage erosion failure processes; (4) Operation of irrigation outlets – close gates except as needed.
- Identification of Fissure Development. The FMCA team recognized the importance of training District O&M staff on how fissures develop and how to identify fissures in the field. (Action item).
- Need The Crack Inspections Records and Mapping And Soil Testing Data Done For Powerline. . It is of critical importance to find the records of documentation of cracks made by the SCS in the inspection of the central filter trench during construction in 1989. (Note: The District subsequently found and evaluated the original compaction tests for the 1967 construction).
- Lack of Stability Analyses. Although slope failure is not believed to be a significant risk, the FMCA Team recommends that slope stability analyses for the embankment and the central filter be documented. A record of a stability analysis for the original design or the modifications has not been found.
- Confirm Hydrologic Routings Through the Reservoir . The inflows and outflows for the full PMF, ½ PMF, and 100-year events and the consequences of the outflows downstream should be confirmed. (Note: It is recommended that the analysis include the 250 yr., 500 yr., etc. to define the threshold recurrence interval for overtopping and provide input for incremental damage analysis.)

**General Findings That Are Informational And / Or Generally Similar For All Three Dams (Powerline, Vineyard Road, and Rittenhouse FRS)**

- Broad Based Value of Risk Assessment. The FMCA team observed the broad-based value of the risk assessment process in risk reduction to all aspects of the dam safety program.
- Impact of Vineyard Road Test Cells. It took a long time to fill the Vineyard Road FRS test cells with water. The data is statistically not adequate to determine the effectiveness of the central drain to work as a crack stopper. (Note: The slow raising of the water level in the Vineyard ponding tests as compared to the rapid rise in stage in the first part of a major flood and consequent surge of seepage into open cracks is an important point. However, ponding tests at Powerline may not be warranted. The SCS ran a considerable number of tests in trenches on the crest of the Powerline FRS. Many showed high infiltration and rapid discharge of seepage from the lower part of the slope.)
- Groundwater Supply Wells. The FMCA team recommends that new groundwater supply wells be located away from the dam, because groundwater withdrawal can contribute to fissure development. The minimum distance from the dam for new groundwater supply wells should be determined through a groundwater study.. AMEC reported that some groundwater wells near the dam and in the adjacent vicinity have a radius of influence of approximately one-mile. There is a large number of groundwater supply wells in the vicinity (high production wells). Groundwater supply wells are potentially a significant factor on localized subsidence for Powerline FRS as there are two within 1 to 1.5 miles northwest and one about two miles to the northeast. There are none within a several miles to the east, south and southwest.
- Small Floods may Assist in the Detection of the presence of Fissures. The FMCA team noted that sheet flow is usually required to erode the earth fissures in the Holocene surface soils and to an extent that there are readily apparent surface manifestations. Thus, inspection of the areas of significant fissure risk after small flooding offers a practical way of detection of fissures. This, along with the other elements of the inspection and monitoring program, provides an effective means of risk management. When earth fissures are detected, damage can be readily repaired with an upstream geomembrane hydraulic barrier penetrating well into the cemented erosion-resistant Late Pleistocene soils. With this inspection and repair procedure, the probability of an undetected earth fissure being present at the time of a major flood will be low. The FMCA team realized that water in the form of stormwater as overland flow or flooding is often associated with development of earth fissures or revealing the existence of already developed fissures. These floodwaters cause the fissure to manifest itself as a surface expressions. Earth fissure development as the result of surface water flow is not tied to a storm event frequency. Small floods, which are much more likely to occur than major floods, which could threaten loss of the dam, may thus provide an early warning for the detection and subsequent repair of an earth fissure.

- Ability to Understand Fissure Development Potential Rational and the Potential to Detect and Monitor Development of Fissures. The FMCA team discussed the mechanism for potential earth fissure development and concluded that the conditions exist at the dam site and in the immediate vicinity for earth fissures to develop. The team recognized there are physical early warning indicators that can be observed and or monitored in the field and that may be linked to the potential formation of an earth fissure. There is a high fissure possibility at Powerline FRS due to high strain potentials. However, upstream and adjacent development may mask the presence of fissures. But, given the extensive investigations made by the U. S. Bureau of Reclamation for the CAP design, the FMCA team believes that the engineering geology and hydrology of the site and the mechanisms of subsidence, accumulation of tensile strains and earth fissuring are well understood. This provides the basis to identify areas of significant earth fissure risks and develop an effective program of inspection and monitoring to manage those risks.
  a) It was concluded that periodic surveys of the dam crest and monitoring of ground water levels will allow identification of the areas of significant tensile strain development and risk of earth fissuring with a good degree of assurance.
  b) The team recognizes that the present desert environment and undeveloped state of the site allows effective observation of earth fissures by periodic air photo interpretation and field inspections: this is a critical part of the monitoring program for early warning of earth fissure migration toward the dam. For this reason, it is recommended that no development be allowed within the parts of the District's easements relevant to earth fissure monitoring.
  c) It is recommended that ground water withdrawal within the easements be regulated to minimize earth fissure risks.

## 3.0 POTENTIAL FAILURE MODES

Potential failure modes identified by the FMCA team are presented below. The failure modes were placed into one of four categories listed below. Except in special cases, only those potential failure modes in Category I will be considered in the risk (quantitative) analysis phase of the risk assessment exercise.

- Category I – Failure modes of greatest significance.
- Category II – Failure modes of lesser significance (but not inconsequential).
- Category III – Failure modes for which insufficient information is presently available to allow the team to make a judgement on the significance of the failure mode. The development of additional data and information is warranted. Additional records research may be justified.
- Category IV – Failure modes which are not physically possible or which are clearly not credible.

For each of the potential failure modes identified, a problem statement is briefly described and the factors that make the failure mode more likely (adverse factors) or less likely (positive factors) to occur are listed following the problem statement (except Category IV). In addition, any identified potential actions for risk reduction for each potential failure are then provided.

## CATEGORY I - FAILURE MODES OF GREATEST SIGNIFICANCE

***Failure From Overtopping Of Powerline FRS (Category I).***

(Note: This failure mode for Powerline was highlighted in order to ensure action relative to verifying crest, spillway and outlet elevations and making reservoir capacity determinations but this potential failure mode will not be carried into the quantitative analysis because it is considered to be remote).

Failure Mode Description: Hydrologic studies conducted on behalf of the Flood Control District for Powerline, Vineyard Road, and Rittenhouse FRS (James Montgomery, June 1985) and internally by the District indicated that there is a potential for Powerline FRS to be overtopped by the full PMF storm event. The overtopping may be on the order of a maximum of from 0.9 to 1.3 feet for a total duration of 2.8 hours. The studies also indicate that there may be a discharge in the emergency spillway for the 100-year event. Overtopping of Powerline FRS would occur at the low point of the dam crest (Survey data currently available do not indicate a major low point on the crest. – see Powerline FRS Appendix for the plot of the crest elevations – especially Station 90+00 low point). Flow would overtop the dam at that point and as flows increased, erosion of the crest and downstream slope would occur and potentially lead to a breach of the dam.

Adverse Factors:

(1) Hydrologic studies for Powerline FRS indicate overtopping to occur for the full PMF.
(2) A higher PMF is possible for this structure (by using HMR 49).
(3) Weekes Wash discharges up to the 100-year flood into Powerline Reservoir (based on District study).

Positive Factors:

(1) The dam crest was raised approximately 1 foot during the construction activities for the central filter.
(2) The 1990 crest survey shows a fairly level crest.
(3) May have additional storage in Powerline of approximately 1 foot and increased spillway discharge as well because of the crest being relatively higher by one foot than the spillway.
(4) Regional settlement may provide additional storage in Powerline FRS impoundment.
(5) Hydrologic analysis indicates a short duration of overtopping.

Potential Actions for Risk Reduction:

(1) Confirm crest profile elevation.
(2) Verify capacity of Powerline FRS.
(3) Confirm hydrology of Powerline FRS (100-yr and PMF).

Consequences:

(1) There is less buffer zone between dam and CAP canal than for Vineyard and Rittenhouse but a buffer still exists over most of the dam.
(2) No greater downstream population.
(3) There potentially is a spill in the emergency spillway during 100-year event (unless increased capacity from tilting has occurred that would negate this.
(4) 24-hour and/or multiple storm events may be more critical.
(5) District has extensive easement upstream and downstream of dam - control of development.
(6) Upstream development is expected to occur more rapidly than at Vineyard Road or Rittenhouse FRS.

***Failure From A Potential Earth Fissure(s) Through The Dam Embankment In Association With A Significant Flooding Event (Category I).***

Failure Mode Description: The presence of existing earth fissures in the vicinity and adjacent to Powerline FRS demonstrates the potential for a new earth fissure to manifest itself at the dam embankment. An earth fissure could cause a structural failure of the embankment by opening a crack through the structure. Alternatively, the fissure could potentially undermine the dam embankment by abruptly causing separation of structure at the foundation with the dam bridging the fissure. If the fissure occurred in association with a large flood the structural failure from the fissure would provide a path for a seepage erosion breach. Potentially there will not be a high level of water impounded during fissure expression. In such a case there would be a minimal loss of water. This Failure mode was considered by the FMCA team to be the most significant threat to Powerline FRS.

Adverse Factors:

(1) High tensile strain indicating the possibility of fissure development.
(2) There is recent (1984) and very recent (2001) evidence of earth fissures in vicinity of the dam. One has developed just west of site approximately 1,000 feet away.
(3) Settlement is continuing at the structure.
(4) Good documentation of fissures and subsidence.
(5) Localized groundwater pumping (deep high capacity wells).
(6) Urbanization and development may mask presence of fissures.
(7) Likely existence of Holocene soils - 14 of 26 test pits ended in siltstone, 12 test pits in Holocene soils.
(8) Northern 40% of dam has a very high likelihood of experiencing fissuring. Southern 60% of dam has a moderate likelihood of experiencing fissuring

Positive Factors:

(1) Awareness of existing fissures and fissure development. Can be monitored. District has large easements around structure.

(2) Requires flood to initiate potential breach sequence due to fissure. Small floods may cause fissure to express as surface manifestation without failure.
(3) Central Arizona Project canal provides a distinct, observable marker to help identify fissure development on the downstream side of the dam. Can be used as a "warning" flag.
(4) Probability of fissure developing / manifesting itself for the first time concurrent with a large impoundment is low.

Potential Actions for Risk Reduction:
(1) Recommend a joint agency fissure/subsidence-monitoring program.
(2) Segment the dam into smaller dams.
(3) Eliminate the north 40% of dam.
(4) Inspections for fissure expression after rain events to include not only dam but impoundment area and areas downstream of the dam.

***Failure From Piping Through Transverse Cracks Causing Breach Of Dam In Association With A Major Flooding Event (Category I).***

(Note: This potential failure mode was highlighted as a Category I mode, but is moved by the FMCA team into Category III for need of additional information)
Failure Mode Description: Potential for impounded water to infiltrate and flow into a transverse crack(s) existing in embankment prior to the major flooding event or a transverse crack that develops in association with the flooding. The transverse crack allows the entry of a great enough flow of water to initiate seepage erosion and breaching failure. Two modes were identified for failure: Mode A - Failure through a large (wide) transverse crack that extends across the central filter, and Mode B - failure through entry of flow through multiple upstream transverse cracks, water flows to/through a flaw in the central filter and begins seepage erosion (and loss of central filter material) through a downstream transverse crack.

Adverse Factors:
(1) Evidence of filter placement that could produce segregation.
(2) Potential for Holocene soils across dam section but not direct evidence.
(3) There is a potential for cementation of the filter and sustaining a crack.
(4) Thirty-day drawdown time (versus 10 days for Vineyard FRS).
(5) Did not pre-wet foundation soils on downstream half as was done for Rittenhouse.
(6) No drain for filter.

Positive Factors:
(1) Examined foundation soils during construction to define cut-off trench depth.
(2) The central filter was constructed to base of foundation over entire length of structure.

(3) Low P.I.'s indicate less potential for shrinkage cracks. (There was no initial trench investigation of cracking for Powerline as was done for Rittenhouse and Vineyard because the 1972 impoundment apparently did not result in significant cracking at Powerline)
(4) Soil cap placed on crest during construction activities for central filter may help reduce infiltration/cementation.
(5) Mode B appears significantly less likely due to no indication of intensive cracking.

Potential Actions for Risk Reduction:
(1) Need crack mapping and soil moisture change information for Powerline.
(2) Conduct soils properties evaluation.

## CATEGORY II – FAILURE MODES OF LESSER SIGNIFICANCE (BUT NOT INCONSEQUENTIAL).

***Failure From Potential Piping Around/Or From 12 Inch Corrugated Metal Pipe Outlet At Station 141+00 (Category II – Considered but not highlighted).***

Failure Mode Description: Infiltration of water into embankment material around pipe and soil is carried away due to seepage erosion along the culvert or piping leading to development of an erosional tunnel, caving and breaching.

Adverse Factors:
(1) Corrugated metal pipe - difficult to compact soils around, does not meet regulation for pipe material type, and potential corrosion.
(2) The central filter was a repair/rehabilitation project.
(3) Frequent use increases corrosion potential.
(4) Difficult compaction around CMP pipe due to seepage collars.

Positive Factors:
(1) Central filter constructed around CMP pipe during filter construction activities.
(2) Embankment less likely to undergo shrinkage cracking away from pipe.
(3) Seepage collars help arrest shrinkage crack.
(4) Very low hydraulic gradient at the outlet.
(5) Time for development is long for piping process to occur compared to the time water is in the reservoir.
(6) The outlet is located higher on the right abutment (the invert elevation of the 12-inch CMP is 1574.0 and the emergency spillway elevation is 1583.3-ft (relates to item 4 - low hydraulic head) - requires extreme storm.
(7) Low consequences if failure were to occur.

Potential Actions for Risk Reduction:
(1) Monitor outlet and inspect during regular inspections and during flood events.

(2) Keep slide gate closed until needed (this would prevent development of precursor seepage paths developing from leakage out of the culvert flowing along the sides of the culvert).
(3) Video inspection of interior of CMP pipe.

***Failure From Potential Erosion Of Abutment And/Or Emergency Spillway During Spillway Discharges (Category II – Considered but not highlighted).***

Failure Mode Description: The emergency spillway is a limited service earth-lined spillway. Spillway discharges from extreme events could potentially damage the spillway floor and left abutment due to erosion.

Adverse Factors:
(1) The discharge velocity from the PMF event is estimated at 12.1 feet per second.
(2) No control sill to interrupt potential head cut.
(3) Damage to spillway basin and training walls would be expected at higher flows.
(4) Sixteen-inch waterline constructed at right training dike of spillway.
(5) No erosion protection is provided on the inside wall of the approach channel to the spillway, which is also the far end of left abutment of the dam.

Positive Factors:
(1) No drop at end of spillway to initiate erosion (the principal spillway would be flowing full).
(2) Founded on strongly cemented clayey sand at level approach section of spillway.
(3) Discharges in spillway are directed away from dam.
(4) Powerline and Vineyard Road principal outlets tailwater at downstream end of emergency spillway - creates energy dissipation.
(5) Relatively short duration of flows in spillway.
(6) Low incremental consequences – the main potential adverse consequence is relative to reduced flow capacity may increase overtopping potential (this seems extremely unlikely to be significant factor).

Potential Actions for Risk Reduction:
(1) Monitor emergency spillway performance during flood events.
(2) Monitor training dikes and left abutment during flood events.

***Failure From Potential Embankment Slope Instability (Category II – Considered but not highlighted).***

Failure Mode Description: Potential embankment slope failure due to internal pressure in the central filter.

Adverse Factors:

(1) No drain outlet for central filter. Potentially could cause long duration head and development of a phreatic surface (pore pressure) within the downstream slope.

(2) No stability analysis was located from previous studies.

Positive Factors:

(1) Powerline has experienced less longitudinal and transverse cracking than Vineyard or Rittenhouse.

(2) No evidence of past slope instability.

(3) Powerline has an upstream cutoff trench. This should help prevent inflow of water from pervious deposits upstream into the central drain.

(4) The depth of impoundment provides a low driving head.

(5) The depth of impoundment provides low internal pore pressure.

(6) Trench for central filter stayed open without problems as demonstrated during construction activities for the central filter (35 to 40 foot trench).

Potential Actions for Risk Reduction:

(1) Conduct stability analyses.

## CATEGORY III– FAILURE MODES FOR WHICH INSUFFICIENT INFORMATION IS PRESENTLY AVAILABLE FOR MAKING ENGINEERING JUDGEMENTS.

### *Failure From Seepage Erosion Along The Dam Foundation Contact Or Through The Foundation (Category III).*

(Note: This potential failure mode was rated as a Category III mode. With additional information as stated under "Actions" this potential failure mode would be expected to move into Category II).

Failure Mode Description: This potential failure mode is initiated by longitudinal and transverse cracks carrying water into the foundation where there may be potentially erodible Holocene soils. A seepage erosion flow path could develop with the exit at or near the downstream toe at the dam foundation contact. As foundation materials are washed away an erosional tunnel develops, followed by either loss of the reservoir through the tunnel or caving of the dam into the erosional tunnel and formation of a breach. The Powerline FRS appendix provides a plot of the location of potential Holocene materials, which was developed after the FMCA exercise for Powerline FRS.

Adverse Factors:

(1) Powerline test pits show 12 of 26 holes did encounter Holocene Soils below the cutoff trench.

(2) The foundation was not pre-wetted as was done for Rittenhouse FRS.

Positive Factors:

(1) May not be Holocene soils based on soil logs and based on excavation in channel (gravel channels were noted in cutoff trench at depth).
(2) There was a special effort during construction of foundation by geologist to determine extent of foundation excavation.
(3) Less intense cracking than Vineyard and Rittenhouse - less cracking to supply water to drive the process.
(4) Central drain interrupts and collects flow.
(5) The time of development is longer than available water supply.

Potential Actions for Risk Reduction:

(1) Need to evaluate existence of Holocene soils in foundation of dam.
(2) Need to prepare crack mapping and find NRCS crack report (if one exists).
(3) Remedial measure is to add drain to filter.
(4) Failure mode mainly impacts the lower half of the downstream toe (downstream portion only).

## CATEGORY IV – FAILURE MODES WHICH ARE NOT PHYSICALLY POSSIBLE OR WHICH ARE NOT CLEARLY CREDIBLE.

***Failure From Piping Around Principle Spillway – 36-Inch Reinforced Concrete Pipe (Category IV).***

Failure Mode Description: Infiltration of water into embankment material around pipe and is carried away due to seepage erosion along the pipe or piping leading to formation of an erosional tunnel, caving and formation of a breach.

Adverse Factors:

(1) Cutoff collars make it difficult to compact embankment materials around RCP pipe.
(2) Possibly low plasticity index materials at Powerline - may mean that the material around the RCP is more likely to erode than that at Vineyard and Rittenhouse. However, material properties have not yet been located to verify/quantify this possibility.

Positive Factors:

(1) The pipe is located on concrete cradle.
(2) Five cutoff collars on 20-foot spacing.
(3) Collars interrupt potential shrinkage cracks.
(4) The outlet is ungated – thus there is less time for a seepage erosion or piping path to develop.
(5) It would take a long time to develop a seepage erosion flow path or piping flow path with relatively small head.
(6) The central filter was constructed around cradle and pipe.

(7) The outlet was tested several times (from stormwater impoundments) with good performance.
(8) Reinforced concrete pipe material - good performance with RCP, meets regulations.
(9) Large diameter pipe - easier to inspect.

Potential Actions for Risk Reduction:
(1) Video interior of pipe.
(2) Monitor outlet and inspect during regular inspections and during flood events.

***Failure From Potential Seismic Activity Impacting Structure (Category IV).***

Failure Mode Description: A seismic event occurs that impacts structure by opening or expanding existing transverse cracks.(this then leads to the earlier described transverse crack failure mode).

Adverse Factors:
(1) Natural variations in geology (rock profile, soil depth) amplify ground motions to cause localized higher acceleration values.
(2) Brittle nature of dam embankment.

Positive Factors:
(1) Theoretical analysis for small strain- showed no sign of adverse effects from seismic shaking.
(2) Dam is already segmented and broken up.
(3) Flood event with earthquake is very rare event for consideration.
(4) Opportunity to view in advance.
(5) No shallow groundwater – no liquefaction impact.

Potential Actions for Risk Reduction:
(1) Examine and inspect dam and outlets and surrounding ground after earthquake (greater than a minimum of 0.05g) and high potential fissure areas.

## 4.0 SUMMARY AND CONCLUSIONS

Powerline FRS was constructed pursuant to relatively modern dam design. Construction appears to have been successfully accomplished without any major problems or residual concerns or issues. Potential segregation of the central filter material was identified as a concern by the ADWR inspectors during construction of the central filter remediation. The dam has performed normally and satisfactorily for 34 years. The structure is satisfactorily maintained and monitored.

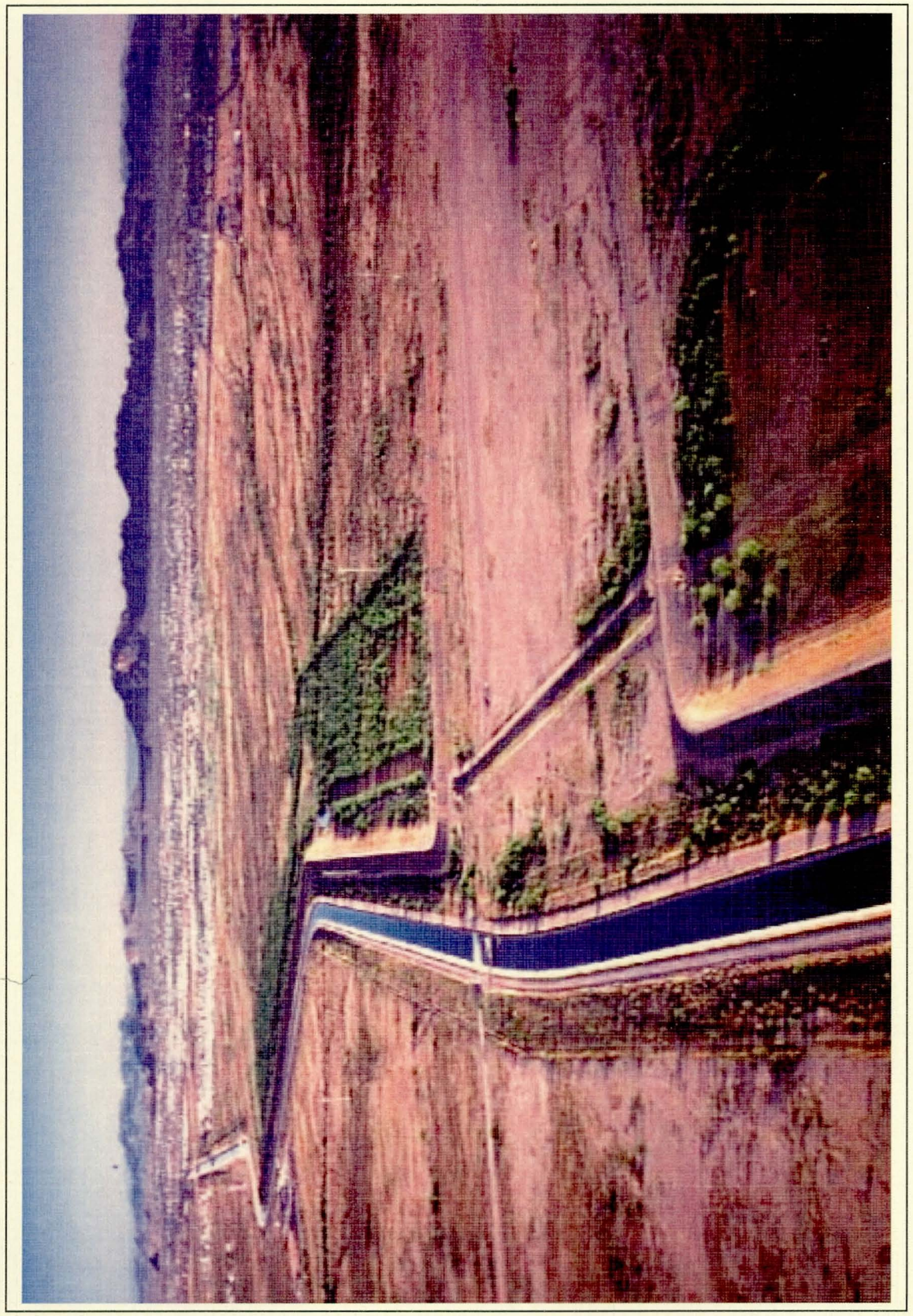
However, it is prudent to recognize that there exist for all dams specific ways that failure could come about that warrant attention and diligent monitoring. The identification of a

condition or process as a "potential failure mode" does not imply that the dam is about to fail or even necessarily that there is a dam safety deficiency at the site. Rather it identifies physically possible conditions or processes (generally with a remote but still credible chance of occurrence) that persons associated with owning, inspecting, analyzing and operating the dam should be aware. Some of the potential failure modes are highlighted (or prioritized) for attention of the dam owners and operators. They are highlighted because the specific conditions at the dam and appurtenant structures are such that these failure modes are physically possible and are considered the most realistic and most credible potential failure modes definable at the site.

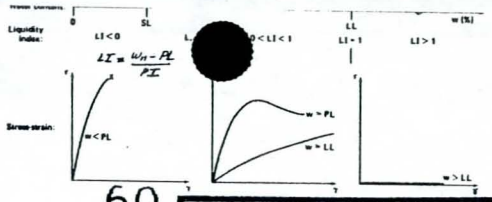
Three potential failure modes were highlighted by the FMCA team (Category I). Two of the Category I failure modes are potential overtopping due to a flood event approaching the PMF and a breach resulting from seepage erosion through a transverse crack during a major flooding event. These will not be carried into the quantitative analysis portion of the risk assessments at this time. The most significant Category I potential failure mode identified was failure due to an earth fissure. The likelihood of this failure mode, however, is dependent on the development of an earth fissure directly beneath or propagated through the dam. Awareness, monitoring, understanding earth fissure development, and training (classroom training) of field personnel to identify the possibility of this potential failure mode before and during inspections/flood events are key risk reduction actions that appear warranted. Other risk reduction measures include a joint agency fissure/subsidence-monitoring program, evaluation of segmenting the dam into smaller dams, and/or evaluation of elimination of the north 40% of dam.

There are not a large number of people and structures at risk in the flood path in the event of a spillway discharge or dam failure by breach. There is, however, the potential that casualties could occur due to a dambreak. A very significant consequence of dam failure would be the adverse impact on the Central Arizona Project canal. Conversely, depending on the magnitude of the dam failure outflow, the canal could mitigate the adverse consequences of dam failure. This appears to be particularly true for any seepage erosion or piping type failure. It is highly recommended that an individual emergency action plan be developed for Powerline FRS.

## POWERLINE FRS APPENDIX - FMCA REFERENCE MATERIALS



Powerline FRS (south end) and emergency spillway (and north end of Vineyard Road FRS).
Photo date: October 2001 Photograph Copyright Kimley-Horn and Associates, Inc.



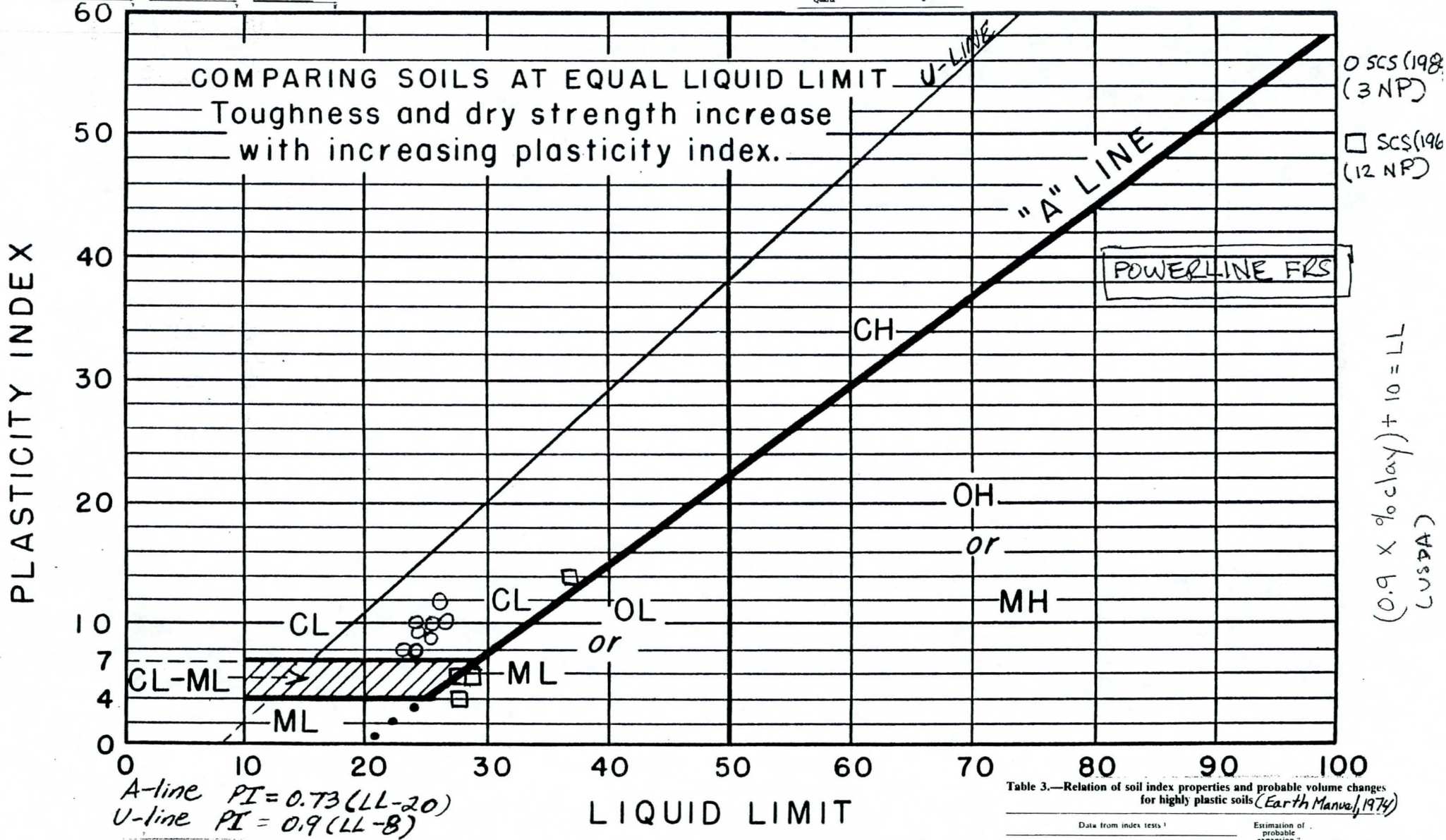

| Characteristic | Soils at Equal LL with Increasing PI | Soils at Equal PI with Increasing LL |
|---|---|---|
| Dry strength | Increases | Decreases |
| Toughness near PL | Increases | Decreases |
| Permeability | Decreases | Increases |
| Compressibility | About the same | Increases |
| Rate of volume change | Decreases | — |

| Mineral | Activity |
|---|---|
| Na-montmorillonite | 4–7 |
| Ca-montmorillonite | 1.5 |
| Illite | 0.5–1.3 |
| Kaolinite | 0.3–0.5 |
| Halloysite (dehydrated) | 0.5 |
| Halloysite (hydrated) | 0.1 |
| Attapulgite | 0.5–1.2 |
| Allophane | 0.5–1.2 |
| Mica (muscovite) | 0.2 |
| Calcite | 0.2 |
| Quartz | 0 |

% < 2 μm

Table 4-1. WES Classification of Potential

| Classification of potential swell | Potential swell $S_p$ percent | Liquid limit LL percent | [illegible] percent | Natural soil suction $\tau^{\circ}_{nat}$ tsf |
|---|---|---|---|---|
| Low | <0.5 | <50 | <25 | <1.5 |
| Marginal | 0.5–1.5 | 50–60 | 25–35 | 1.5–4.0 |
| High | >1.5 | >60 | >35 | >4.0 |




# PLASTICITY CHART

Table 4–. *Data for making estimates of probable volume changes for expansive soils* (Chen 1975)

| Laboratory and field data | | | Probable expansion, percent total volume change | Swelling pressure, ksf | Degree of expansion |
|---|---|---|---|---|---|
| Percentage passing No. 200 sieve | Liquid limit, percent | Standard penetration resistance, blows/ft | | | |
| >35 | >60 | >30 | >10 | >20 | Very high |
| 60-95 | 40-60 | 20-30 | 3-10 | 5-20 | High |

TABLE 3:3 / VOLUME CHANGE POTENTIAL
(Adapted from Holtz and Gibbs[3.11]) (1956)

| Volume Change | Shrinkage Limit | Plasticity Index |
|---|---|---|
| Probably low | 12 or more | 0–15 |

Table 3.—Relation of soil index properties and probable volume changes for highly plastic soils (Earth Manual, 1974)

| Data from index tests[1] | | | Estimation of probable expansion,[2] percent total volume change (dry to saturated condition) | Degree of expansion |
|---|---|---|---|---|
| Colloid content (percent minus 0.001 mm.) | Plasticity index | Shrinkage limit, percent | | |
| >28 | >35 | <11 | >30 | Very high. |
| 20–31 | 25–41 | 7–12 | 20–30 | High. |
| 13–23 | 15–28 | 10–16 | 10–20 | Medium. |
| <15 | <18 | >15 | <10 | Low. |

POWERLINE FRS • ESTIMATED DEPTH OF COLLAPSING HOLOCENE SOILS

9.7.01

| BORING | STATION | DEPTH | USC | TEST PIT | STATION | DEPTH | USC |
|---|---|---|---|---|---|---|---|
| 1 | 134+00 | 12.0 | SM, ML | 32 | 98+00 | 5.0 | SM, SP |
| 2 | 124+00 | 12.0 | SM, SP | 33 | 102+00 | 5.0 | SM, SP |
| 3 | 114+00 | 10.5 | SM, SP | 34 | 106+00 | 11.0+ | SM, SP |
| 4 | 104+00 | 7.5 | SM, SP | 35 | 110+00 | 12.0+ | SM, SP, GP |
| 5 | 94+00 | 12.0 | SM, GP | 36 | 114+00 | 10.0+ | SM, SP |
| 6 | 84+00 | 7.5 | SM | 37 | 118+00 | 10.0 | SM, SP |
| 7 | 74+00 | 6.5 | SM, SC | 38 | 122+00 | 8.0 | SM, SP |
| 8 | 64+00 | 9.0 | SM, SP | 39 | 126+00 | 9.0 | SM, SP |
| 9 | 54+00 | 7.5 | SM, SP | 40 | 130+00 | 10.0+ | SM, SP |
| 10 | 44+00 | 16.5 | SM | 41 | 134+00 | 11.0 | SM |
| 11 | 34+00 | 12.0 | SM, SP | 42 | 138+00 | 6.0 | SM |
| TEST PITS | | | | | | | |
| 15 | 54+00 | 8.5 | SM, SP | | | | |
| 16 | 48+00 | 5.0 | SM | | | | |
| 17 | 44+00 | 10.0 | SM, SP | | | | |
| 18 | 40+00 | 7.0 | SM, SP | | | | |
| 19 | 36+00 | 5.0 | SM, SP | | | | |
| 20 | 32+00 | 10.0 | SM, SP | | | | |
| 21 | 28+00 | 6.5 | SM | | | | |
| 22 | 58+00 | 8.0 | SM | | | | |
| 23 | 62+00 | 8.5 | SM, SP | | | | |
| 24 | 66+00 | 7.0 | SM | | | | |
| 25 | 70+00 | 3.0 | SM, SP | | | | |
| 26 | 74+00 | 5.5 | SM | | | | |
| 27 | 78+00 | 6.0 | SM, ML | | | | |
| 28 | 82+00 | 6.0 | SM | | | | |
| 29 | 86+00 | 6.0 | SM | | | | |
| 30 | 90+00 | 8.0 | SM, SP | | | | |
| 31 | 94+00 | 5.0 | SM | | | | |

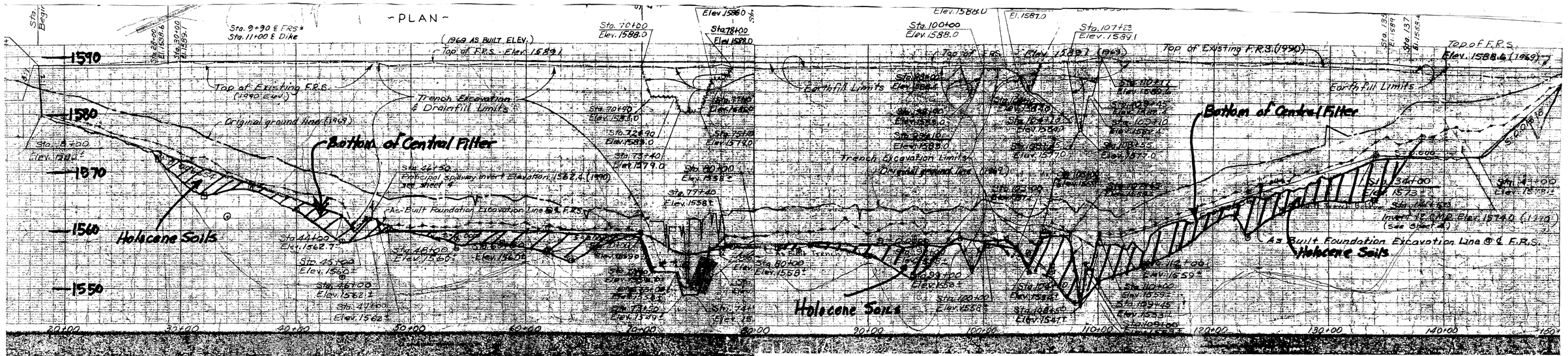
70 − 28 = 42

96 − 80 = 16

137 − 104 = 33

9100' = 1.63 mi

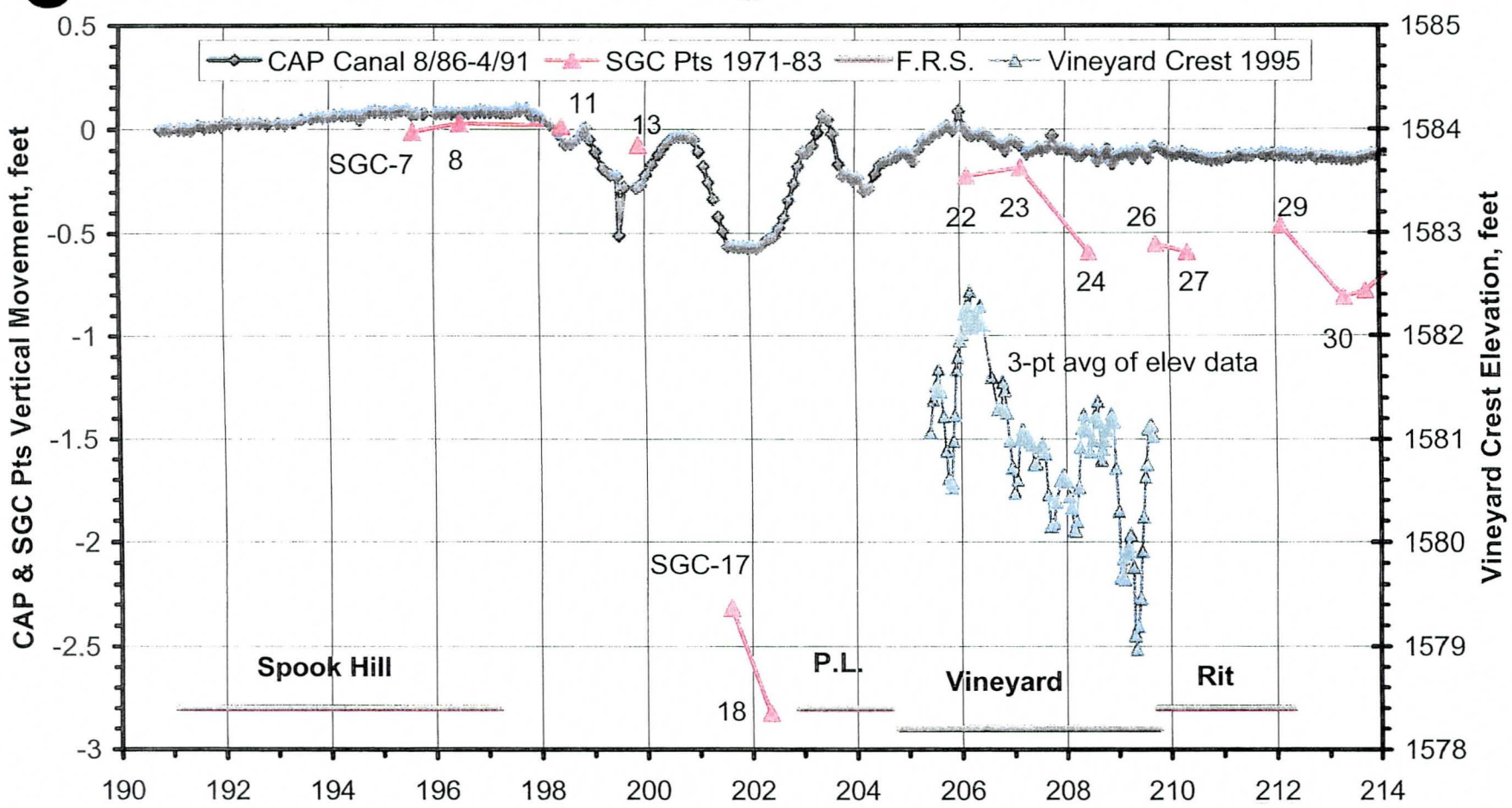
$\frac{1.63}{2.54}$ = 64% of dam has Holocene Soils below Central Filter



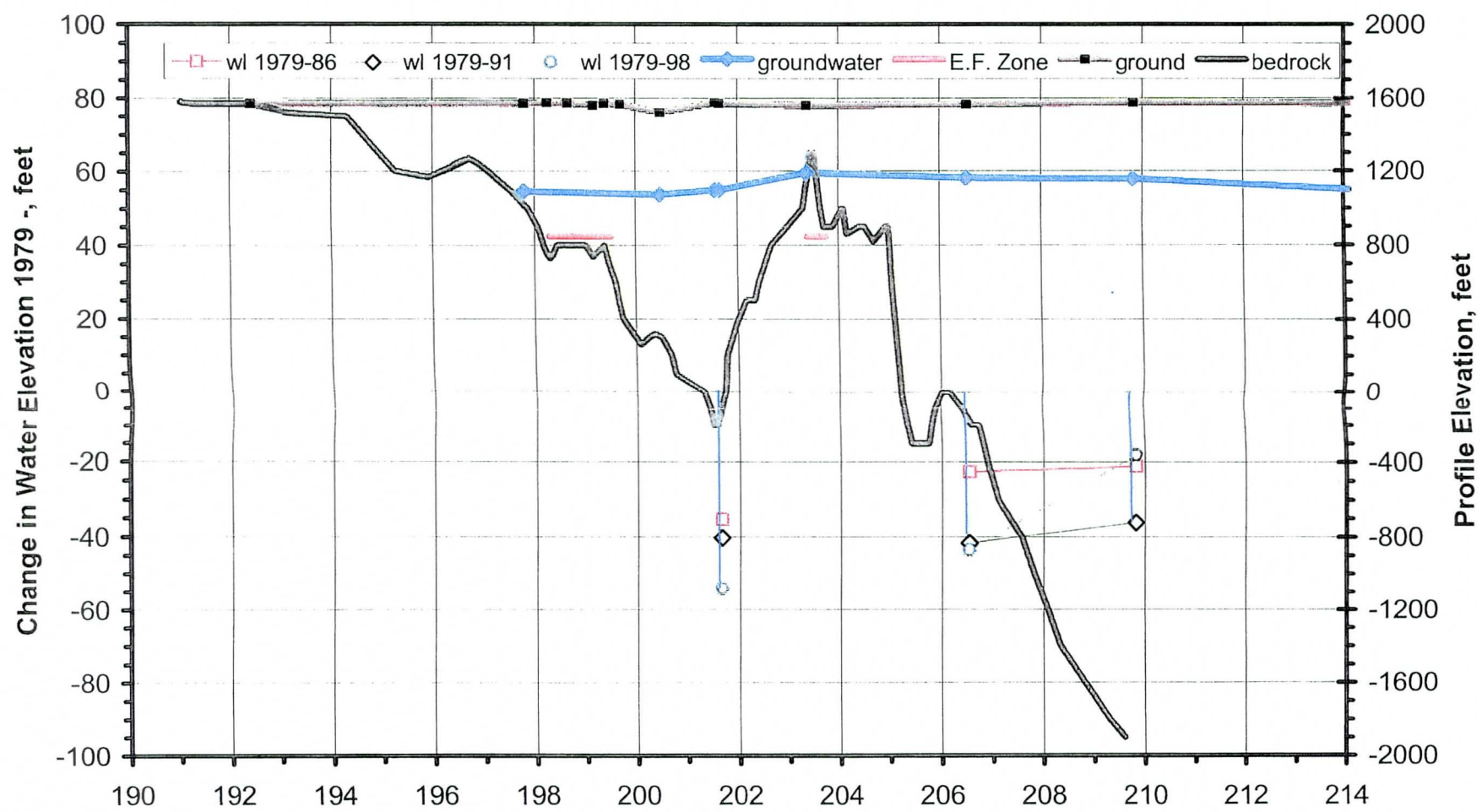

POWERLINE FRS
Estimated Extent of Holocene Soils Below Central Filter George H. Beckwith, P.E. 9-20-01

FIGURE 6.2-1

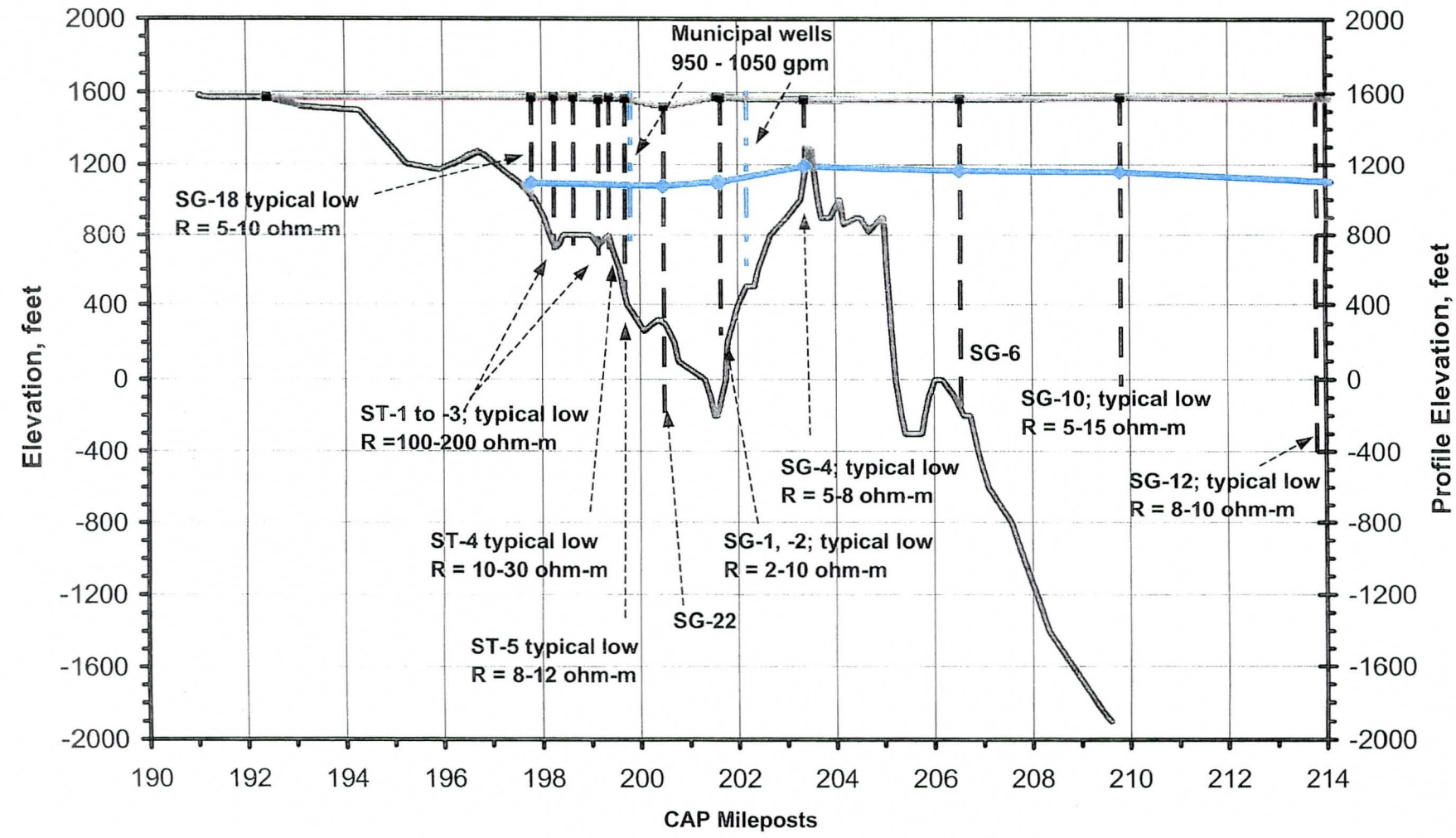
## Survey Data




## Hydrogeologic Profile & Water Level Change

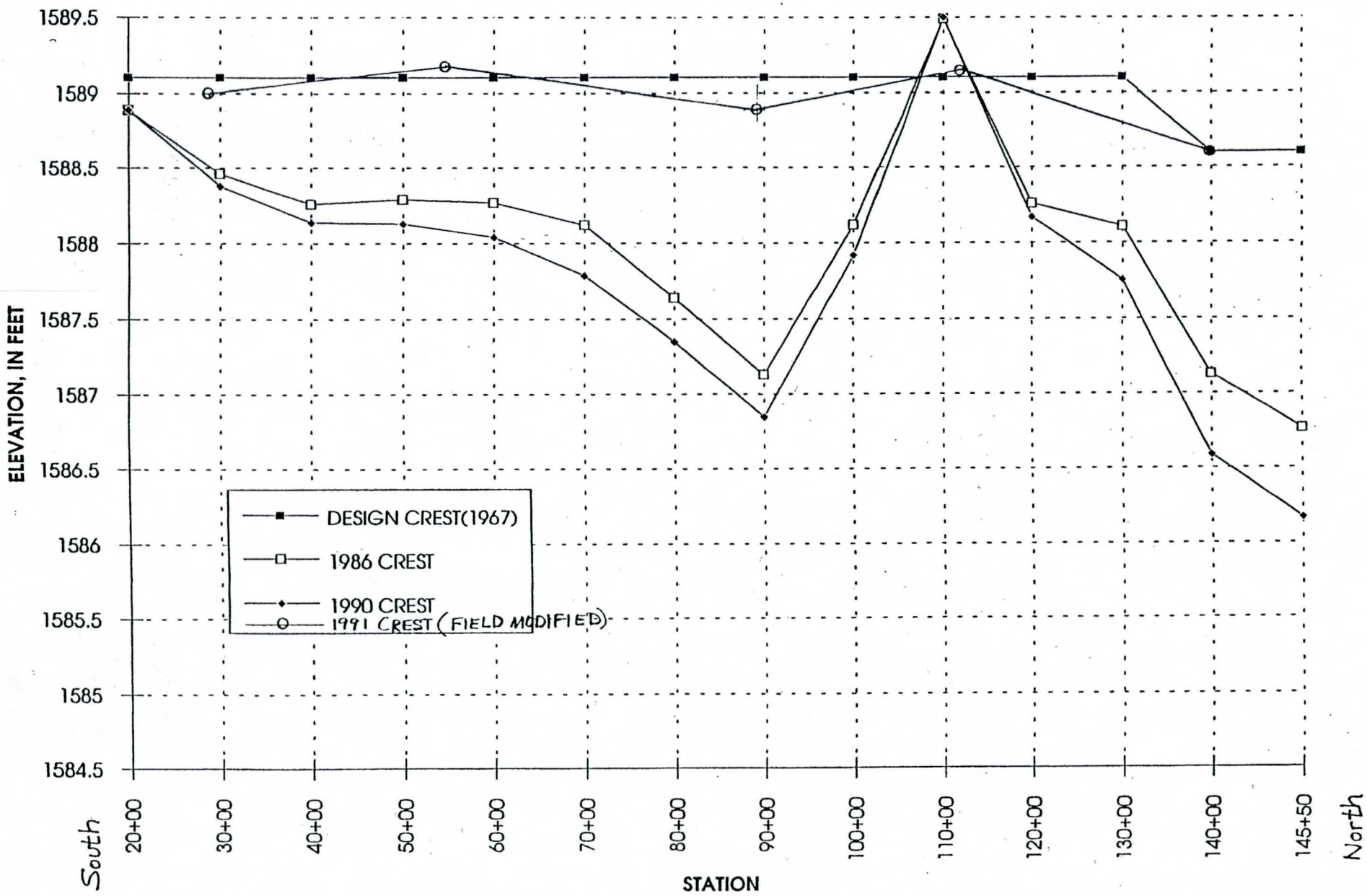



## Deep Well Locations & Data Summary



5/15/95

# POWERLINE FRS CREST SETTLEMENT MONUMENTS



# A BARRIER TO CRACKS IN DRY EARTH DAMS

Susanne Leckband, P.E.

REVISED
July 1984

Abstract: Cracks in dry earth dams were recognized as a serious potential cause of failure by a team of SCS reviewers in 1978. At that time, the only barrier to the cracks that would be flexible and strong enough to maintain a continuous defense against erosion of the cracks appeared to be a vertical or sloping layer of granular material. Filter cloth was mentioned in that report, as was plastic sheeting. Their potential at that time was suspect or unknown.

This granular fill material has been the subject of numerous discussions, calculations and construction claims. It is expensive in most parts of Arizona. Design gradation criteria is not established for crack protection. Analysis of hydraulic behavior with a dry initial condition and multi-directional flow has been, at best, inconclusive. No case history or laboratory test of a granular zone representative of behavior in a dry dam environment is known to this reviewer.

Since the need is to stop flow through a crack for a brief period of time (less than 10 days, by design), one of the other potential barriers mentioned in this report might be simpler and cost-effective.

High density polyethylene (HDPE) is not a new material, but its production in the United States in a sheet form is relatively recent. Of the geomembranes this reviewer is familiar with, HDPE is the least affected by time and exposure and it will stretch the most without failure. This paper suggests that, for dry (limited storage time) floodwater retarding dams, an impermeable diaphragm such as HDPE is a simpler and perhaps more reliable method of crack protection than a zone of granular material.

## Introduction

Because of public and private concern with safety of dams, considerable pressure has been and still is on the Arizona SCS design staff to repair existing dams as quickly as possible. Since the SCS is denied the resources to do any research, an immediate design need, such as these cracked dams, requires assumptions to be made.

Standard review practices have exposed these assumptions to modification according to the reviewers' backgrounds. The reviewers are in a position of authority with respect to approval of the design, we are always under time constraints (generally to obligate funds) and the reviewers have been respected for their knowledge of conventional soil mechanics engineering, therefore, the local design team has not questioned their added criteria.

Each person involved has apparently tried to provide the most safety possible within their own province of design and review. It is time, though, to recognize where we are in the process and make whatever modifications are needed.

## The Crack Study

In 1977, a team of Soil Conservation Service employees from the western region was charged with investigating reported cracks in earth dams near Phoenix, Arizona. The team members were:

C.E. Stearns, State Geologist, SCS, Davis, California (presently WNTC Geologist)

R.J. Smith, State Design Engineer, SCS, Bozeman, Montana (presently State Conservation Engineer, Montana)

J.C. Stevenson, Construction Engineer, SCS, WTSC, Portland, Oregon (presently Head of Engineering, WNTC)

This team physically looked at the dams in the geographic area of concern (Rittenhouse, Vineyard, Powerline, Magma, White Tanks, and Buckeye PL-566 Project dams). Copies of construction information were made available to them for these structures (many boxes of data). They reviewed the results of investigations to locate and identify cracks in several. They consulted with USGS and Bureau of Reclamation Geologists, especially about regional subsidence and fissures. Their conclusions were included in a report "Cracking of Dams in Arizona", April 1978. This report has been the referenced basis of designs for repair of existing dams that had developed cracks as well as for design of new dams.

Their conclusions and recommendations were:

a. Transverse cracks were a greater problem than originally recognized and posed a real hazard to the integrity of the structures.

b. The principal cause of transverse cracks is tension release associated with embankment drying.

c. A dust mulch (common practice at the time) does not prevent cracking.

d. Cracked structures should be "expeditiously" identified and repaired with a graded sand and gravel filter (ASTM C-33 with 45% fine aggregate and 55% coarse aggregate (finer than 1 1/2-inch) that would be installed in a trench parallel with the centerline of the dam.

e. Future designs of dams in hot, arid areas should incorporate features to eliminate or control transverse drying cracks. (On page 18 they call for "a change in the philosophy of earth dam design and construction... in this area.")

f. Section IX calls for monitoring the Salt River Valley and monumenting new structures to keep track of subsidence and any lengthening of individual structures.

A number of potential methods of repair were discussed and reported. These methods included several configurations of graded sand and gravel sections and the possible use of filter cloth. None of the potential methods of repair mentioned or considered a flexible membrane barrier.

The section on design concepts to control cracking did include a "vapor barrier over and around a core section of the embankment". This barrier would consist of "plastic or rubber sheeting or other similar material". The sheeting would be 12-mil or thicker and would require great care in installation to prevent tearing. "Although the vapor barrier will also function as a diaphragm in the embankment, that is not its principal purpose." No other comment is made about a diaphragm or flexible cut-off wall.

The report concludes with statements that a) future designs should include features to effectively eliminate or control cracking problems and b) the Arizona Engineering staff and Design Unit are providing filter drains as an integral part of future structure designs.

Soils

This report had a number of good observations to make. Since that report, data gathered for design of Magma dam repair, further laboratory soil testing and observations during repair of a number of the dams in question have given us information not available to the study team. The study team observed that "most of the soils have low shrink potential". Laboratory tests DO NOT support this observation. Soil classification tests at both Magma and Vineyard dams indicate predominantly CL material and CL-ML with some CH, MH, SC and SM soils. The one sample of (siltstone) foundation material from under Vineyard dam swelled as soon as water was added to the consolidometer (under 2000 #/ft$^2$ confining load).

Review of early geology reports (Vineyard, Rittenhouse, Buckeye and White Tanks dams), shows general field soil classification of SM for the balance of the borrow and shallow foundation soils. There is a definite trend in the laboratory classification of soils from these sites toward CL-ML, Cl and SC as predominant. Another sample of undisturbed soil (from Rittenhouse) swelled in the consolidometer. One of the samples at Fredonia (principal spillway relocation) swelled in the field consolidometer.

The geology report summaries from these sites was definite about borrow materials being SM. No comment was made about conflicting laboratory results. It is possible that the geologist never saw the lab classifications.

A field laboratory was set up in Arizona, probably for construction control, that performed many of the classification tests for the Buckeye investigation. These were nearly all classified SM (even one with 50% fines). Observation of the gradation curves shows that all samples were wet sieved, no hydrometer tests were performed and few Atterburg limits tests were performed. One of the lab technicians, Dave Lambson, was a new employee at that time. He reports that the few Atterburg limits tests that were performed were run immediately after addition of water to the soil samples (no curing). These samples had original moisture contents in the neighborhood of 1% to 4%. This practice continued until our present geologist was concerned by field laboratory classifications of SM when he had identified soils as SC in the field. He requested that the Lincoln Soil Mechanics Laboratory classify some of these same samples. The SC classifications were verified.

Even Buckeye dam, which seems very coarse on the surface, apparently has plastic soils. Samples from the repair trench plotted above the "A-line" when Atterburg limits tests were run on them. (See attached.) Again, these samples were often field classified SM by the A&E inspectors (The Earth Technology Corporation, Buckeye Site I Drain).

Field identification of fine-grained desert soils can be difficult because of the length of time and physical effort required to work water into the dessicated clay aggregates. These aggregates feel like hard grains of sand.

The implication that Arizona soils never have a higher moisture content than that contained at the time of construction is also misleading. Periods of wet weather are erractic in Arizona, but generally occur in the late summer and mid-winter months. Humidity can be high during these periods. The geologist, Aubrey Sanders, observed during repairs of Vineyard dam that cracks in the repair trench were more numerous and of greater magnitude during dry conditions than after periods of wet or humid weather. In fact we were unable to identify several specific severe cracks re-excavated for a study.

Magma dam was constructed in 1964. The first report of cracks in Magma dam was July 30, 1965 (Turner, State Conservation Engineer). He commented on the transverse cracks. W.R. Stanley (WNTC) commented on the longitudinal cracks in Magma dam and the absence of transverse cracks (October 11-12, 1965). (Remember that late summer is one of Arizona's rainy seasons.) Turner wrote to Core (WTSC) November 18, 1965, after reading Stanley's trip report, commenting "It is possible that subsequent rains had cured the transverse cracks." In May of 1972, Benson Scott, responsible for dam safety with the Arizona Water Commission, said there were reaches in the dam crest where numerous transverse cracks had developed and were starting to erode. Some cracks extended more than two feet deep. He said that the cracking should be investigated and repair scheduled. Apparently he was there with Walt Parsons, since the SCS O&M report is for the same date. That report said that several areas along the top of the dam showed evidence of piping from the cracking "that was there two to three years ago". In contrast, D. R. Lawrence and W. C. Jenkins (AWC) inspected Magma Dam in June 1976. They said the embankment was in fair to good condition, though no maintenance had been accomplished since the last inspection (1973?). They said the condition of the dam crest had greatly improved and that it appeared the dam had healed itself. (In 1973 the owner had been told to repair the embankment.) In March 1977, the same two people found cracks up to three feet deep near Station 237+00.

Clay mineralogy tests performed in the area of these two dams indicate that mixed-layer Montmorillonite-Illite clays are common. Both clay minerals are subject to swelling in a humid (not necessarily saturated) environment.

## Construction Moisture

The 1978 report indicated that cracking magnitude was greater where soils were placed wet of optimum. This apparent observation has resulted in designs for nearly all earthfill in Arizona SCS structures calling for soil placement moisture less than optimum, e.g. 3% below to 1% above optimum for a common range.

Magma dam's moisture control specifications called for a "workable mix". Review of the weekly summary of density determination for that job reveals that of 360 samples tested, only 100 (28%) were above optimum moisture, 216 (60%) were below optimum moisture and the balance of 44 (12%) were at optimum moiture content. Bill Cutter, SCS inspector on that job, remembers trouble with wet soils in the early part of that job. The earthfill materials were generally dry as the job progressed.

The only shrinkage limit test reported at Magma was on a soil sample from just downstream of the principal spillway. This soil had a shrinkage limit of 20.3% and a volumetric change of 35.3%. This soil was classified CL-2 with 62% fines, a liquid limit of 36%, a PI of 16 and a positive reaction to HCL (indicates carbonates, usually assumed to be $CaCo_3$).

A similar soil from the borrow area classified CL-2 (sample 111.2) had a liquid limit of 49%, PI of 27, and a maximum dry density of 100 pounds per cubic foot at 21% moisture content (optimum ). Optimum moisture content of the samples tested during construction varied from 11.8 to 21.7%. The shrinkage limit appears to be near or perhaps above the optimum moisture content. How can soils shrink to a smaller volume than their "shrinkage limit"? It appears that this relationship is not well understood.

Saddleback FRS (dam) was recently constructed to specifications that called for 3% dry to 1% wet of optimum moisture content. According to Aubrey Sanders, geologist, this dam has already cracked. The cracks are both transverse and longitudinal. The project was completed April 6, 1982. There are cracks even over locations where the foundation is shallow, with a relatively uniform depth to rock. During construction, compaction densities well over 100% Standard Proctor were recorded. If the investigators' recommendation (1977-78) to compact dry of optimum was the solution to this problem, there should be no cracks or they should be very small. Compaction moisure was probably well below the shrinkage moisture limit. Mr. Sanders says he did not notice cracks in all parts of the dam, but where they were evident, they looked substantially the same as cracks on White Tanks and Vineyard dams.

## Cracks

The studies done prior to actual repair work did not locate cracks penetrating through the foundation of any of the dams investigated. Cracks in the soil surface adjacent to the toe of some of the dams were carefully checked, with no evidence of relationship to the embankments.

The actual repairs have exposed cracks to and into foundation materials under the embankments. The manner of repair has been to 1) open a trench along the centerline of the dam, 2) drag a protective shield for the geologist(s) behind the excavator and ahead of the backfill operation to map the cracks and 3) backfill with a coarse-grained fill material. The geologists have mapped cracks much deeper and much closer together than were ever anticipated by either the 1978 team or the follow-up crack investigations on specific structures.

## Fissures and Subsidence

There is more known about fissures and more fissure cracks have been identified since the Crack Study was completed. The study had a good summary of the situation at that time. It was somewhat severe in its recommendation that any dam with a fissure crack identified within 500 feet be breeched. It seems to this designer that, as with other features, each structure needs to be evaluated on its own. This is not to infer that fissure cracks are not a very serious potential hazard in some parts of Arizona.

Well-Graded Sand and Gravel Zone

The method of repair for cracked dams that, at the time, appeared to be by far the best of the methods available was a graded sand and gravel zone. This type of material has wide acceptance in dam construction as a chimney drain for control of seepage and uplift pressures. It is used as protection from sudden displacement such as is potential in an earthquake. Soil Conservation Service designers are familiar with the design of a chimney drain and there is a body of test data for flow in these materials.

Unfortunately, the testing and performance evaluation of these chimney drains has all been in a saturated environment. The standard design of material for a chimney drain within the SCS is Soil Mechanics Note 1. Recent tests on filter materials that were performed in the Soil Mechanics Laboratory at Lincoln, Nebraska, were done with high head and saturated "embankment" materials. None of this has proven validity in a large-reservoir, low embankment (head), arid environment.

This designer is supportive of chimney drains and their design and use in dams that store water for an appreciable amount of time. The graded sand and gravel zone that the Crack Study recommended, though, was not referred to as a drain and is not needed to function as a drain. Even though at least two of the team members were very familiar with chimney drains and the SCS Soil Mechanics Notes, the terms "chimney drain" and "drainfill" never appear in the report and there is no indication that design should follow Soil Mechanics Notes 1 or 3 as design criteria.

The progression of design criteria for these graded sand and gravel zones for crack protection has been very interesting. The Crack Study recommended a composite of ASTM C-33 fine aggregate (45%) and size number 57 (-1 1/2") (55%) for the granular zone. No mention was made of any other criteria for gradation. This was in 1978.

That same year, the Arizona Design Unit designed the repair for Rittenhouse dam. It was called "Rittenhouse Drain". The gradation specified approximated the gradation recommended in the Crack Study. No compaction was called for. Note

In late 1979, Buckeye Site I dam crack repair design was completed. The design was essentially the same as for Rittenhouse in that a trench was to be excavated along the centerline of the dam and then backfilled with "Rittenhouse filter" material. The repair job was called "Buckeye Site I Drain". The criteria listed for design were: 1) The Crack Study report (1978), 2) the crack location report done by Fugro, Inc., 3) Soil Mechanics Note 3 and 4) "Vertical Drains and Embankment Zones" by Clarence E. Dennis (1971). The specified gradation is identical to that for "Rittenhouse Drain". A new requirement entered, though. The coeficient of permeability could not exceed 250 feet per day at the in-place density. No compaction was required.

Note the implication that the material in the graded sand and gravel zone now meets all of the referenced design tools and that two of those references are specific to designs with steady-state flow. Dennis' report discusses sudden cracks due to earthquakes but he calls for very wide zones with considerably coarser zone material for these ( a minimum $D_{85}$ of 4").

Repairs for White Tanks dams Sites 3 and 4 were designed in the spring of 1981. The job name is "White Tanks Nos. 3 & 4 Drain Repairs". The specified gradation is identical to the first two designs. A maximum permeability of 250 feet per day in place is still

required with no specified compaction. The documentation for this design includes letters from WTSC, Portland, which apparently are the source of the added requirements.

The November 1979, design review report (WNTC) for Buckeye Site 1 Drain comments that "The proposed filter gradation is the same as used on Rittenhouse Dam and may or may not be suitable at this site. The suitability must be verified using the criteria in Soil Mechanics Note No. 3 and the Crack Study team's recommendations (as was done for Rittenhouse Dam). Particular concern is in the $D_{15}$ range of the filter where it is effective against piping of existing base materials. Gradations of borrow materials used on Buckeye Site 1 must be used as base materials in this evaluation."

A letter from the WTSC dated February 6, 1980, concerning Buckeye Site 1 Drain is apparently a follow-up of that design review. The letter is attached. The source of these requirements is not given. They are introduced with the comment, "Our knowledge of flow conditions through cracks in embankments is not complete. However, evidence surfaced to date indicates that the following requirements are essential..." There is no indication of what the evidence is or where it came from.

The letter concludes with the following paragraph:
"In the meantime, we will continue to study this problem to better understand what can happen downstream from the filter trench for future designs and repairs. Until such time as the phenomenon of cracking and its consequences are completely understood, monitoring of such works during periods of hydraulic stress is an essential part of dam safety." (Notice that the reviewer is thinking of this zone as a "filter" now.)

At about this time in the repair design sequence, two new requirements were proposed. They were: 1) the minimum $d_{50}$ size of the filter must be equal to or greater than one-half the crack width; and 2) the minimum $d_{75}$ size of the filter must be equal to or greater than the crack width. This criteria assumes that we know the crack width in advance. Since we generally do not know, and in response to recognition of fine-grained soils in the embankments (base soils), the granular zone material is now being designed by SM 1. Cracks as narrow as 1/4" require additional protection on the downstream side of the repair zone (1/16" at Fredonia).

The next repair design was for Fredonia FRS in northern Arizona. The first sentence in the Design Report for "Fredonia FRS Repair" is: "The final design is in accordance with "The Cracking of Dams in Arizona", April 27, 1978, a report by the crack study team ..." Fredonia dam was repaired with a centerline drain and another trench at the downstream toe to protect against cracks in the embankment foundation that were deeper than the trencher could reach with the centerline trench. The backfill material is not well-graded and is considerably finer than that recommended in the referenced report. It is even farther from SM 3 criteria. Because "The contractor on White Tanks 3 and 4 repair was unable to supply the drainfill to meet the specification", the specification "was considered too narrow and restrictive for the materials at hand" (at White Tanks) and the gradation was broadened. (Actually it was made considerably finer.) Fredonia is roughly 4000 feet higher than the White Tanks sites and 230 miles north, as the crow flies. The site is substantially different than that at White Tanks. The only similarity is that they are both "dry" dams with long, low embankments.

The design for repair of Vineyard Road dam, "Vineyard Road F.R.S. Drain", is substantially the same as all the others. The specifications for gradation of the

granualar material are the same as those at Fredonia dam except the maximum size is slightly larger, as is the $D_{85}$ size. Since this designer reviewed that design, the following comments concern the design charge.

To meet design schedules, this job was sent to the design unit in Wyoming. That design unit, which has no experience with Arizona soils and climate, was essentially directed to produce a repair design by a scheduled deadline and given past designs to use as a guide. No new soil tests were made on the structure. The granular zone was designed by SM 1.

The "design crack" for Vineyard became 1/4-inch. Approximately 85 outlets were anticipated to eliminate cracks larger than 1/4-inch. In fact, over 400 of these outlets are required by that criteria as the cracks were mapped by the geologists during installation of the centerline trench. Some of these cracks were as close as 3 feet from each other. Cracks in this dam do sometimes continue into the natural material underneath the compacted embankment foundation. The granular zone was installed as deep as the trenching equipment could place it in those areas (22 to 24 feet). The maximum height of this dam, which is 5 miles long, is about 17 feet.

One of the most important characteristics this granular zone is supposed to have is that it be composed of self-healing or free-flowing materials that will not sustain a crack. The material installed in Fredonia dam was observed to bridge with approximately 6 feet of overhang as the lower materials unravelled during construction of the outlet trenches. Material stockpiled ready for placement in the trench tended to cake up (solid clods and surface) when dried. According to the acting Project Engineer who observed the stockpiles, the clods collapsed when saturated, therefore, he believed the design purpose was satisfied. Since dams fill with water from the bottom up, these observations are somewhat unnerving when combined. why?

When the granular zone at Vineyard dam was re-excavated for installation of study monitors, it was observed by several people, among them Clifton Deal, WNTC, that the granular material maintained a stable vertical slope.

During these last few years, the formation of a "filter cake" on the upstream side of any granular material with a $D_{15}$ of approximately 0.7 mm has been demonstrated in the Lincoln Soil Mechanics Laboratory by James Sherard and Lorn Dunnigan. As mentioned before, these tests use an upstream slurry and high initial head or water pressure.

The Soil Conservation Service has made two studies to attempt to prove the formation of a "filter cake" on the upstream side of a graded sand-gravel zone that this designer knows of. One study was in Arizona on Vineyard dam in 1983 and water never got close to the protective zone in the 30-day trial. The other study was in Nebraska. Water was ponded near the top of the embankment under consideration and subsequently washed over the top of the settled protective zone.

## Chemistry and Climate

The general explanation for the many transverse cracks in Arizona dams has been that they are "dessication" cracks due to the extremely arid enviroment. The fact that this does not begin to tell the whole story is at least indicated by the July 26, 1983, letter from the Midwest NTC Director to the Arizona State Conservationist about Fredonia dam site. This letter concludes that water loss from gypsum and even more hydrated

salts will need to be considered in calculations of water content in these samples (Fredonia dam).

The letter has much information in it, though two parts of it need to be discussed. One is the table based on a stated assumption that is later referenced as proving the assumption. The other is the statement that any cracks caused by salts would have visible salts on the crack surfaces. Regardless of those issues, it is a significant acknowledgment of the importance of chemistry at that site to engineering soil properties.

Soil chemistry and soil climate (usually the province of special soil science studies) have not been evaluated to see if they play an important role in cracked dams here in Arizona. They have proven to be very important to engineers in cold climates.

Some examples of phenomena that might affect our embankments are:

1. Substantial structural damage has been documented as the result of soil salts (as little as 0.1 % $Na_2SO_4$) expanding with change in temperature near Las Vegas, Nevada. Discussion with Ruben Nelson (retired from the National Soil Survey Laboratory, SCS) suggests that irregular topography in the Fredonia reservoir, which was land leveled during construction, very likly is the result of salts that can migrate through the soil profile in THEIR OWN WATER OF HYDRATION between temperatures of 30° F and 90° F. No watertable is necessary. They tend to migrate up.

2. Soil scientists in other countries have studied profile development directly attributable to daily and seasonal waves of soil temperature. Environmentalists in Arizona have determined that the depth in soils near Tucson to a constant soil temperature is approximately two meters (say 6 feet). This would be the limit of temperature influence on soil profile development, if the former studies are accurate. Profile development involves salts and clays.

3. Soil moisture (even non-saturated) tends to migrate from hot temperatures to cold temperatures. Electro-osmosis, which engineers have recently used for drainage of clays, is the movement of water when an electric current is applied. (In fact, the first Arthur Casagrande Lecture was "Stabilization of Soils by means of Electro-Osmosis, State of the Art", given March 8, 1983, by Dr. Leo Casagrande.) A related phenomenon is the measurable electromotive force developed in the soil when water moves due to a temperature gradient within the soil profile.

4. The USGS has measured soil temperatures of 165°F in the top 1-inch of soil at White Tanks (near the dams). Temperatures adjacent to the ground are often well below freezing in the winter (17°F, for instance, reported to produce harvesters, who need to know when they can pick lettuce). One February day in the desert a temperature near the ground of 23°F was recorded with a high air temperature that same day of over 90°F.

5. National Park Service soil chemists at Tucson have studied the role of soil salts, humidity and temperature in the deterioration of old adobe structures and ancient Indian ruins. The behavior of various clay minerals (expansion, for instance) varies greatly with the cations on the clay surfaces and in any adjacent waters.

## Flow Analysis

Two years ago at the annual Arizona ASCE meeting in Phoenix, a civil engineering professor from the Universitiy of Arizona described the use of the finite element method (FEM) for prediction of flow through porous media. Dr. Chandrakand Desai mentioned a program, SEEP-3D, that would analyze flow three-dimensionally. He made this program available to us for a nominal fee and advised us of how to input data.

Up to this time, Bob Nelson at WNTC had tried to analyze flow through a crack in a dry dam to a repair trench. He has documented many trials and assumptions in an effort to predict how one of these granular zones would perform if water entered from a crack. This determination is important if dams are repaired and constructed with these zones.

Mr. Nelson and this designer worked together on SEEP-3D input with aid from Dr. Desai. It was finally realized that the program requires so many boundaries and assumptions, that it essentially is the same two-dimensional analysis, steady-state flow that the manual calculations perform.

The problem is that most fluid-flow formulas are based on steady-state and/or saturated flow. Since dams in Arizona do not store water, no steady seepage or saturated flow occurs.

Ah, but we have cracks in our dam. How far apart? How wide are they? Do they swell shut? Do materials along the sides of the cracks slake into the water? When the water reaches the granular zone, does it form an impermeable layer (filter face) with slaked material at the surface of the granular zone? If the water moves into the granular zone, does it flow directly across and begin discharging in the crack on the downstream side of the zone? Will the discharge from the granular zone be flowing at an erosive velocity? How much of the flow into the granular zone will cross to the crack and how much will move laterally (both directions) along the centerline trench? How much of the water that enters the centerline trench will be absorbed by the expansive(?) or collapse prone(?) soils underneath the compacted embankment? (Note: Several dams were designed with upstream cut off trenches to protect the foundation from ever becoming saturated. Vineyard dam now has a granular zone that extends into these uncompacted soils below the dam.)

In discussion with Dr. Desai about all of these required assumptions, he suggested just installing an impermeable membrane in the dam. It would then be protected from any sudden failure and the analysis (that would always involve risky assumptions) becomes unnecessary.

## HDPE

In past experience as a field office engineer, plastic liners were used to contain water in irrigation reservoirs. Various membranes have been advertized for several years that have considerably more durability and thickness than those this designer was familiar with.

One of these "new" products (HDPE) was specified for lining Mt. Elbert Forebay by the Bureau of Reclamation. This reservoir required 12.5 million square feet of liner. Static

head could be 76 feet. They specified use of either 80-mil high-density polyethylene lining or 45-mil chlorinated polyethylene lining. The 80-mil material required no additional bedding (just place on prepared bottom or side of reservoir) and 1 foot of earthfill cover. The 45-mil material required 6 inches of bedding and 1½feet of earthfill cover. (The 45-mil chlorinated polyethyene was installed by the contractor.) Ronald K. Frobel (Bureau of Reclamation) has been very involved with studies of this material and is supportive of its use in a long-lived dam. It has been used as a waterproofing material for the facing on the reinforced-earth addition to the top of one of their dams.

The Army Corps of Engineers installed an HDPE geomembrane in a slurry trench to repair Mohicanville Dike No. 2. This is a dike off the end of a dam in Ohio (Huntington, West Virginia District). It only holds water during flooding. The dike is approximately 25 feet high. They used a 100-mil liner, 34 feet deep in the foundation, as a defense against differential settlement cracks that might penetrate the slurry trench. The bentonite in the slurry trench is expected to stop seepage (through peat), but large deformation of the foundation and fill is possible. The HDPE is expected to deform with the embankment. It is expected to perform with no problem with a potential hydrostatic head of 60 feet. According to Larry Franks, Project Soils Engineer at the Huntington District, "If there is a lot of displacement, the liner will stop water even if the trench is breached." The Army used 100-mil material because of their lack of experience with the material and the difficult installation conditions.

High-density polyethylene has been used for many years for weatherproofing and insulating large electric cables with high exposure to sun and weather. In this application HDPE has demonstrated it's stability for a long period of total exposure to sunlight and extreme temperatures. When toxic wastes became a serious issue with numerous organizations willing to spend money, several manufactuers began producing HDPE sheeting to be used to line toxic waste ponds.

High-density polyethylene is inert, developed from a pure polymer with no plasticizers. Approximately 2% carbon black is added to the formula. Its specific gravity is 0.95, tensile stress at yield is over 2700 psi, elongation at yield more than 800% and modulus of elasticity more than 80,000 psi (measured 138,500 psi). The Bureau of Reclamation laboratory tested 100-mil samples supplied by Schlegel Area Sealing Systems, Inc. (Applied Sciences Referral Memorandum No. 79-28). One of the statements made in that memorandum was that "large (1½ to 2-inch) sharp angular aggregate should be eliminated from the subgrade as it will increase the possibility of puncture."

Doubtful. [illegible]

## Crack Protection in Dry Dams

As has been mentioned before, the dams that have been experiencing distress from, apparently, dessication cracks are "dry" dams. They are designed with ungated outlets and a maximum detention time of 10 days (usually shorter) in a 100-year event with an antecedent moisture condition II (moist, but not saturated). These structures often have no water against the embankment (even for short periods of time) for several years in a row. (In part because we rarely have a "moisture condition II".) The ones discussed in this report are very long (5 miles is common) and relatively low in height (commonly 20-25 feet maximum and 12-18 feet average). They are constructed across alluvial fans. Therefore, there are a number of channels entering a structure, not one concentrated flow. They might very well be called diversions rather than dams, except that they do store water briefly since outlet discharge is controlled up to the 100-year event.

No phreatic line can be established in one of these structures, there simply is neither time nor head enough to establish one. It is not possible to saturate any but the very surface embankment soils, upstream slope. If a crack is open enough for floodwaters to enter it, then it is equally possible for those floodwaters to exit the same crack back into the reservoir and out the principal spillway. It appears to this designer that an impermeable barrier would be considerably more sensible under these conditions than a "drain".

As a matter of fact, the Crack Study team apparently thought so, too, but all the materials they reviewed were inflexible and subject to cracking or tearing themselves (soil-cement core, compacted earth core, and 12-mil plastic). The sand and gravel zone was not intended by that team to be a drain. The sand and gravel zone was to provide a flexible barrier to the continuity of the cracks.

HDPE is very strong and very flexible. The size of crack is relatively unimportant. Installed deeply enough (into the core trench or below the downstream toe elevations), any water that got under the membrane would have to travel up to the downstream toe. Granular zones can also be bypassed by a crack underneath. This designer does not believe the granular material would necessarily heal such a crack. Water can travel in any part of a crack. It generally follows a horizontal discontinuity in fissure studies performed by the USGS and the Bureau of Reclamation. It would be unlikely that any installed barrier would stop at one of these natural horizontal discontinuities, especially throughout these long structures.

One of the great advantages of the flexible membrane barrier is that the "design crack" is of no concern. No one has to guess how wide or how long any future crack will be. No one has to decide where that "design crack" width will be exceeded at some future time or, in the case of repair, is already exceeded. (It has been the policy of all repair designers that any crack exceeding the "design crack" width be destroyed by excavation of a permanent trench perpendicular to the centerline granular zone and backfilled with granular material.)

Post-Construction Outlet

## Summary

It has been shown that the currently accepted method of repair of cracked "dry" dams may not be the best method in light of availability of new materials and the actual circumstances encountered in the dams since the original Crack Study.

The application of design criteria based on saturated flow through porous materials (soils) is highly suspect when compared to the physical environment of these "dry" dams.

It has been shown that the cracking mechanism at work in these dams is not well understood. It was the expectation of the Crack Study team and the West National Technical Center engineers that the study would be updated at some future time, since they made reference to the need to develop new design criteria for construction.

## Recommendations

1. The Soil Conservation Service should give serious consideration of HDPE geomembrane as a flexible, impermeable barrier to existing or future cracks in "dry"

dams.

2. The Crack Study should be updated as soon as possible. This would permit confirmation of the parts of the report that have been verified and correction of the apparent circumstances that have been proven untrue or at least misleading.

3. The Soil Conservation Service should find some means for a thorough study of the "dry" dams in Arizona with the intent of clarifying the mechanism(s) causing cracks and developing construction criteria (if possible) to reduce their severity and frequency.

Appendix-References


1. Clays and Clay Technology, proceedings of the First National Conference on Clays and Clay Technology edited by Joseph A. Pask and Mort D. Turner and published by the State of California Division of Mines, Bulletin 169, July 1955.

2. "Cracking of Dams in Arizona", Report of the Soil Conservation Service Crack Study Team, April 27, 1978.

3. Dennis, Clarence E., "Vertical Drains and Embankment Zones", Lincoln Soil Mechanics Laboratory, SCS, April 1971.

4. Dunnigan, Lorn P., Robert J Fredrickson, Clifford P. Flanagan, and Wilfred L. Wortman, "Gering Valley Watershed, Site H, Field Test of a Chim;ney Filter for a Cracked Earthfill Dam", internal report, SCS Soil Mechanics Laboratory, September, 1983.

5. Effects of Temperature and Heat on Engineering Behavior of Soils, Proceedings of an International Conference, Highway Research Board Special Report 103, 1969.

6. Frobel, R.K., "Laboratory Testing of HDPE (High Density Polyethylene) Proposed for Use as a Lining in the Mt. Elbert Forebay Reservoir", Applied Sciences Refereal Memorandum No. 79-1-28, December 19, 1979.

7. Leckband, Susanne M., letter to Dick Raymond, Bureau of Reclamation Geologist on assignment with USGS for Central Arizona Project, November 21, 1983, SCS AZ Files RE: Vineyard FRS Study, (interim information requested).

8. Leckband, Susanne M. "Magma Dam Repair Alternatives", SCS AZ Files RE: Magna Dam Repair, April 22, 1982.

9. Morrison, W.R., E.W. Gray, Jr., D.B. Paul, and R.K. Frobel, "Installation of Flexible Membrane Lining in Mt. Elbert Forebay Reservoir", REC-ERC-82-2, September 1981 (Bureau of Reclamation).

10. Sherard, J.L., L.P. Dunnigan, J.R. Talbot, "Basic Properties of Sand and Gravel Filters", to be published in ASCE Geotechnical Journal June 1984 (April 25, 1983).

11. Sherard, J.L., L.P. Dunnigan, J.R. Talbot, "Filters for Silts and Clays", to be published in ASCE Geotechnical Journal June 1984 (April 25, 1983).

12. Soil Mechanics Note 1, "Tentative Guides for Determing the Gradation of Filter Materials", Lincoln EWP Unit, Soil Conservation Service, May 1, 1968.

13. Soil Mechanics Note 3, "Soil Mechanics Considerations for Embankment Drains", Soil Conservation Service, (Clarence Dennis, Robert Nelson, Roland Phillips and Jack Stevenson), May 1971.

14. Stevenson, Jack C. letter to J. Gordon Odell, NM SCE, November 4, 1983, ENG-Repair of Cracked Dams, SCS.

15. Stout, Maurice, Jr., letter to Verne Bathurst, AZ SC, July 26, 1983, ENG-Design-Fredonia Dam Site, RT83-AZ068, Arizona (SCS).

16. Archived reports on dry dams in Arizona.

    a. "Buckeye Site 1 Drain, AS-Built Report", ERTEC, Inc. Project Number 80-200, March 29, 1981.

    b. Geology Reports from investigations of sites:

       Rittenhouse
       Vineyard
       Buckeye
       White Tanks

    c. Design Reports for sites:

       Rittenhouse Drain
       White Tanks Nos. 3 & 4 Drain Repairs
       Fredonia FRS Repair
       Vineyard Road FRS Repair

    d. Specifications from sites:

       Saddleback FRS
       Rittenhouse Drains
       White Tanks Nos. 3 & 4 Drain Repairs
       Fredonia FRS Repair
       Vineyard Road FRS Repair

HYDRO LIBRARY

# POWER LINE DAM

## Hydrology & Hydraulics & Structure Design

A310.708

Arizona Powerline Dam
Watt 3-30-65

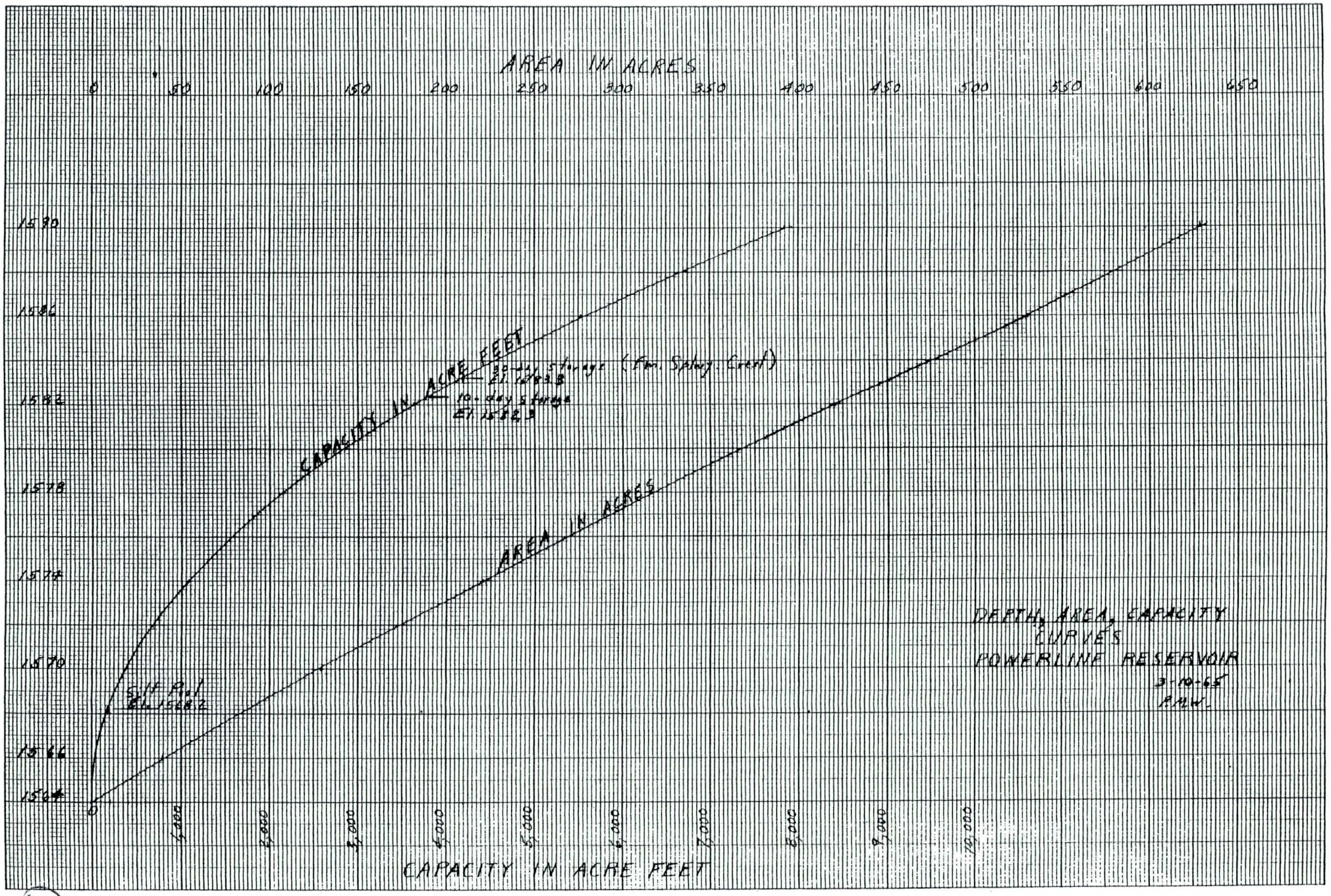
Comparison of Depth Capacity Curve
of Original & Revised Location

| Capacity | Orig Elev | Revised Elev | Diff in WS Elev |
|---|---|---|---|
| 1000 | 72.3 | 73.6 | 1.3 |
| 2000 | 76.2 | 77.5 | 1.3 |
| 3000 | 79.2 | 80.4 | 1.2 |
| 4000 | 81.7 | 82.8 | 1.1 |
| 5000 | 83.8 | 84.9 | 1.1 |
| 6000 | 85.7 | 86.8 | 1.1 |
| 8000 | 89.1 | 90.2 | 1.1 |

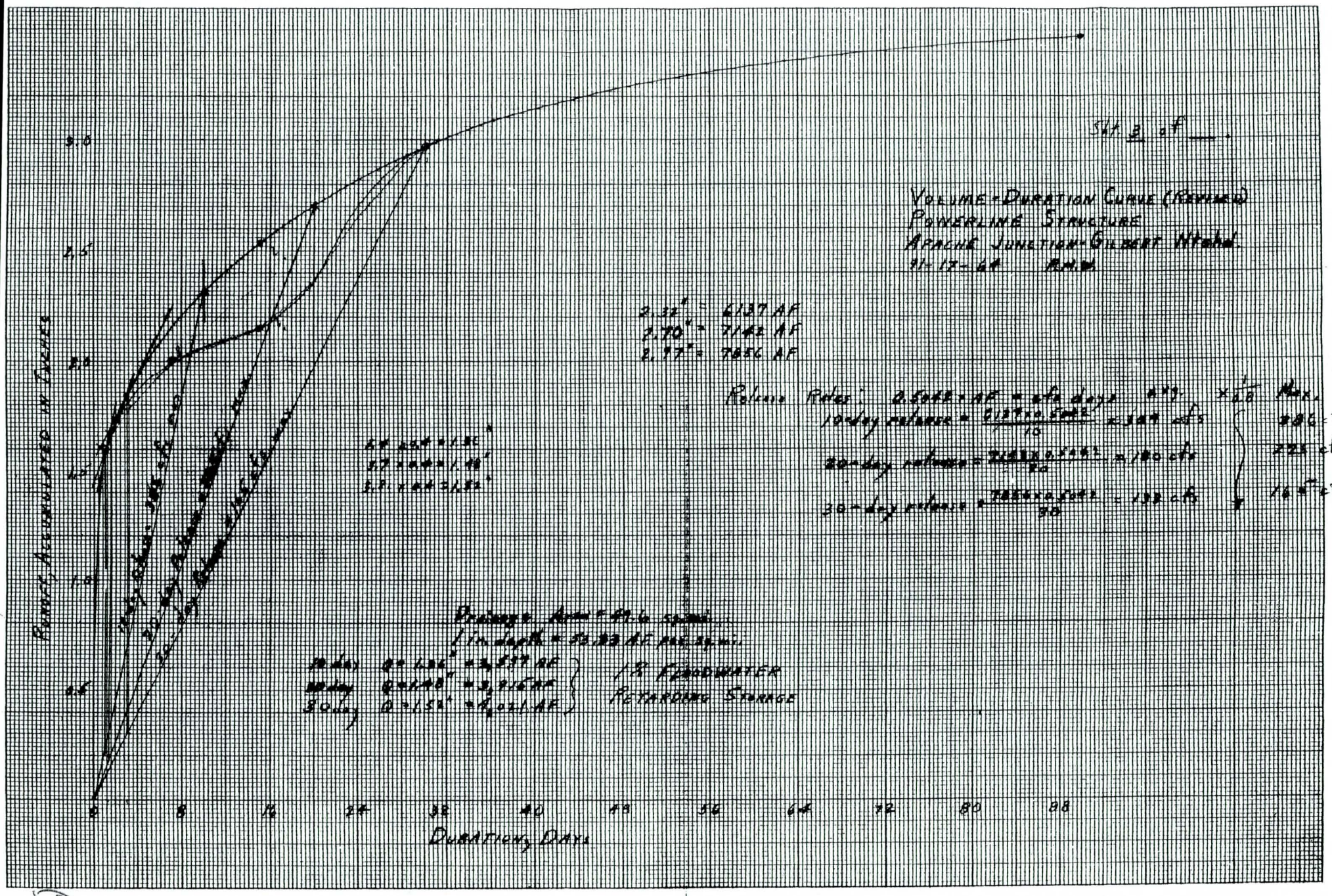
Elev for 4021 flood storage & 175 AcFt silt storag

(*) Revised Location = 1583.3 = Crest Emg St
Original = 1582.2 = " " "
Diff = 1.1

Because the depth capacity curves so closely parrallel each other we will use the same size & depth of emergency spillway as determined in the original flood Routing Freeboard Hydrograph is routed from storage starting at 3667 above which storage curves are parrallel


AREA IN ACRES
0
50
100
150
200
250
300
350
400
450
500
550
600
650
1590
1588
1586
1584
1582
1580
1578
1576
1574
1572
1570
1568
1566
1564
CAPACITY IN ACRE FEET
30-day storage (From Salmy Creek)
El. 1586.5
10-day storage
El. 1582.5
AREA IN ACRES
Sill Pool
El. 1566.2
0
1,000
2,000
3,000
4,000
5,000
6,000
7,000
8,000
9,000
10,000
CAPACITY IN ACRE FEET
DEPTH, AREA, CAPACITY
CURVES
POWERLINE RESERVOIR
3-10-65
RMW

(3)


Sht. 2 of
Volume-Duration Curve (Revised)
Powerline Structure
Apache Junction-Gilbert Wtshd.
11-12-68 RMW
2.22" = 6137 AF
2.70" = 7142 AF
2.97" = 7856 AF
Release Rates:
10-day release = 6137x0.5042 / 10 = 309 cfs
20-day release = 7142x0.5042 / 20 = 180 cfs
30-day release = 7856x0.5042 / 30 = 132 cfs
Max. 286 cfs
225 cfs
16 cfs
Drainage Area = 49.6 sq.mi.
(In depth = 50.33 AF per sq.mi.)
10 day Q = 1.36" = 3,637 AF
20 day Q = 1.40" = 3,516 AF
30 day Q = 1.55" = 4,021 AF
Floodwater Retarding Storage
Peak Discharge in Cfs
Duration, Days
0
8
16
24
32
40
48
56
64
72
80
88
0.6
1.0
1.4
2.0
2.5
3.0

Arizona Powerline Dam

PMW 12-1-64

Principal Spillway Depth-Discharge Curve 7
Computations

The Principal Spillway Discharge is directly proportional to the square root of the head on the pipe. That is,

$$Q = K H^{1/2}$$

The 1% storm results in an emergency spillway crest elevation of. (Refer to Depth-Capacity curve and Volume-Duration Curve);

10-day release = Em. Splwy. El. 1581.1
30-day release = Em. Splwy. El. 1582.2

The Principal Spillway release rates are:

Q, 10-day release = 386 cfs
Q, 30-day release = 165 cfs

The Corresponding Discharge Curves are:

10-day release, $Q = 386$ cfs
$H = 81.1 - 63 = 18.1$ ft.

$$K = \frac{386}{\sqrt{18.1}} = 90.7$$

$$\therefore \underline{Q = 90.7 H^{1/2}} \text{ (10-day)}$$

30-day release, $Q = 165$ cfs
$H = 19.2$ ft.

$$K = \frac{165}{\sqrt{19.2}} = 37.6$$

(*) $\therefore \underline{Q = 37.6 H^{1/2}}$ (30-day) (see page 11)

This is close enough for routing the 6-hr storm thru the emergency spillway.

Arizona Powerline Dam

pmw 12-1-64

Principal Spillway Depth-Discharge Curve 8

Elev. top of pr. splwy. conduit = 1563

| 10-day release $Q = 90.7\ H^{1/2}$ | | | | 30-day release $Q = 37.6\ H^{1/2}$ | | | |
|---|---|---|---|---|---|---|---|
| H | $H^{1/2}$ | Q | Elev. | H | $H^{1/2}$ | Q | Elev. |
| 2 | 1.41 | 128 | 1565 | 2 | 1.41 | 53 | 1565 |
| 4 | 2.0 | 181 | 67 | 4 | 2.0 | 75 | 67 |
| 6 | 2.45 | 222 | 69 | 6 | 2.45 | 92 | 69 |
| 8 | 2.83 | 257 | 71 | 8 | 2.83 | 106 | 71 |
| 10 | 3.16 | 287 | 73 | 10 | 3.16 | 119 | 73 |
| 12 | 3.46 | 314 | 75 | 12 | 3.46 | 130 | 75 |
| 14 | 3.74 | 339 | 77 | 14 | 3.74 | 141 | 77 |
| 16 | 4.0 | 363 | 79 | 16 | 4.0 | 150 | 79 |
| 18 | 4.24 | 384 | 81 | 18 | 4.24 | 159 | 81 |
| 20 | 4.47 | 405 | 83 | 20 | 4.47 | 168 | 83 |
| 22 | 4.69 | 425 | 1585 | 22 | 4.69 | 176 | 1585 |
| 24 | 4.90 | 444 | 87 | 24 | 4.90 | 184 | 87 |
| 26 | 5.10 | 463 | 89 | 26 | 5.10 | 192 | 89 |

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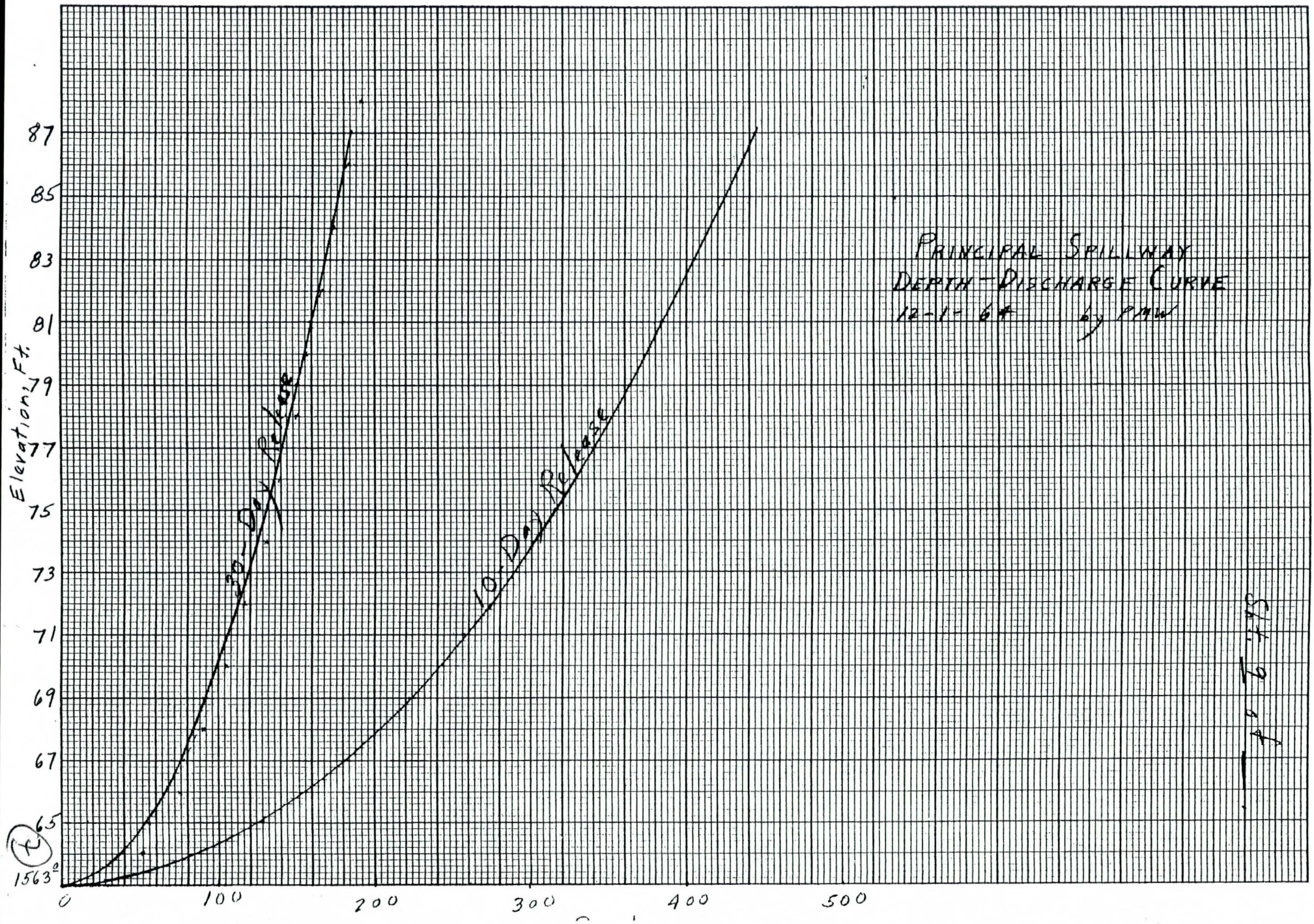



Sheet 1 of 1

Arizona Powerline Dam P

PAW 12-7-64

Elevation Base for Flood Routing 16

For a 10-day release, the starting elevation would be the crest of the principal spillway; in other words, use the elevation of the silt pool. Reason - after 10 days of drawdown the reservoir would be empty.
The drainage area being about the same as the work plan provision, use a silt storage of 175 Acre-ft. The corresponding elevation from the Stage-Storage Curve is 1566.2.

For a 30-day release, the starting elevation would be the elevation after 10 days of drawdown. From the Volume-Duration Curve, the storage after 10 days of drawdown would be: 1.32 in. × 49.6 mi.² × 53.33 AF/mi.²

or

10-day drawdown storage = 3492 A.F.
silt storage = 175 AF
Total storage = 3667 A.F.

Corresponding elevation from the Stage-Storage Curve is 1580.8.

Arizona Powerline Dam

GDH 12-31-69

Flood Routing Results & elevations

| | 1% drawdown tim | |
|---|---|---|
| | 10 day | 30 day |
| Principal Spillway Crest Elevation - - - - - - = | 1566.2 | 1566.2 |
| Emergency Spillway Crest Elevation - - - - - = | 1581.1 | 1582.2 |
| Sediment Volume Ac. feet - - - - - - - = | 175 | 175 |
| Watersurface after 10 days drawdown El. - - = | 1566.2 | 1580.8 |
| Max. W.S. after routing. Emerg. spwy. Hyd. El. - = | | 1583.9 |
| Max. W.S. after routing Fbd. Hyd. - - - El. - = | 1584.7 | 1587.0 |
| Flood water retarding volume, Ac. feet - - - = | 3,600 | 4020 |
| Max. Principal Spwy. release rate - - - - - = | 390 | 165 |

For Revised Location 3-30-65

| | 30 day Rel. |
|---|---|
| Crest Principal Spillway | 1568.2 |
| Crest Emg Spillway | 1583.3 |
| Max W.S. Freeboard Hyd | 1588.1 |
| Max W.S. Emg Sp. Hyd. | 1585.0 |
| Flood Water retarding Vol | 4020 Ac. |

The above revision was caused by moving the dam upstream 200 ft to clear proposed central Arizona aqueduct. Checking the new depth capacity curve both were almost identical except the revised location was 1.1 ft higher. Emergency spillway would be the same. Therefore we raised all elevations 1.1 ft for the new location

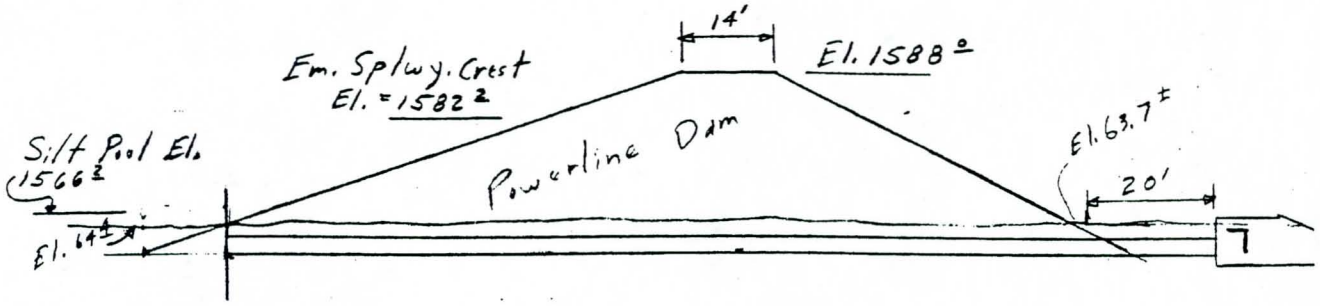
Hewitt

Arizona Powerline Dam

PMW 1-27-65

Principal Spillway Design

1




Silt Pool Elevation = 1566±

Assume the Inlet structure to be built to the top of the silt pool.

Required trash rack area = $\frac{165}{1.0}$ = 165 ft.²

For a 16' height of trashrack, the required perimeter is $\frac{165}{16}$ = 10.3'

Make the riser 3'x 9'.

Area of Riser = 27 ft² = $a_R$

Riser entrance loss

Let $V_R$ = velocity in riser

$V_p$ = velocity in pipe

$a_R$ = area of riser

$a_p$ = area of pipe

$\frac{V_R^2}{V_p^2} = \frac{a_p^2}{a_R^2}$ Try 36" ø outlet pipe. $a_p = 7.07$ ft²

Riser entrance loss $= 0.5 \frac{V_R^2}{2g} = 0.5\left(\frac{7.07}{27}\right)^2 \frac{V_p^2}{2g} = 0.034 \frac{V^2}{2g}$

Bend Loss $= 0.10 \frac{V^2}{2g}$

Pipe Entrance loss = $= 0.200 \frac{V^2}{2g}$

Friction Loss = 0.00428 × 154 $= 0.660 \frac{V^2}{2g}$

Velocity Head = $= 1.00 \frac{V^2}{2g}$

Total H = $1.994 \frac{V^2}{2g}$

$Q = \frac{(7.07)(8.03)}{\sqrt{1.994}\ 1.01} H^{\frac{1}{2}} = 40.2 H^{\frac{1}{2}}$

10

Arizona Powerline Dam

PMW 1-29-65

Principal Spillway Design 3

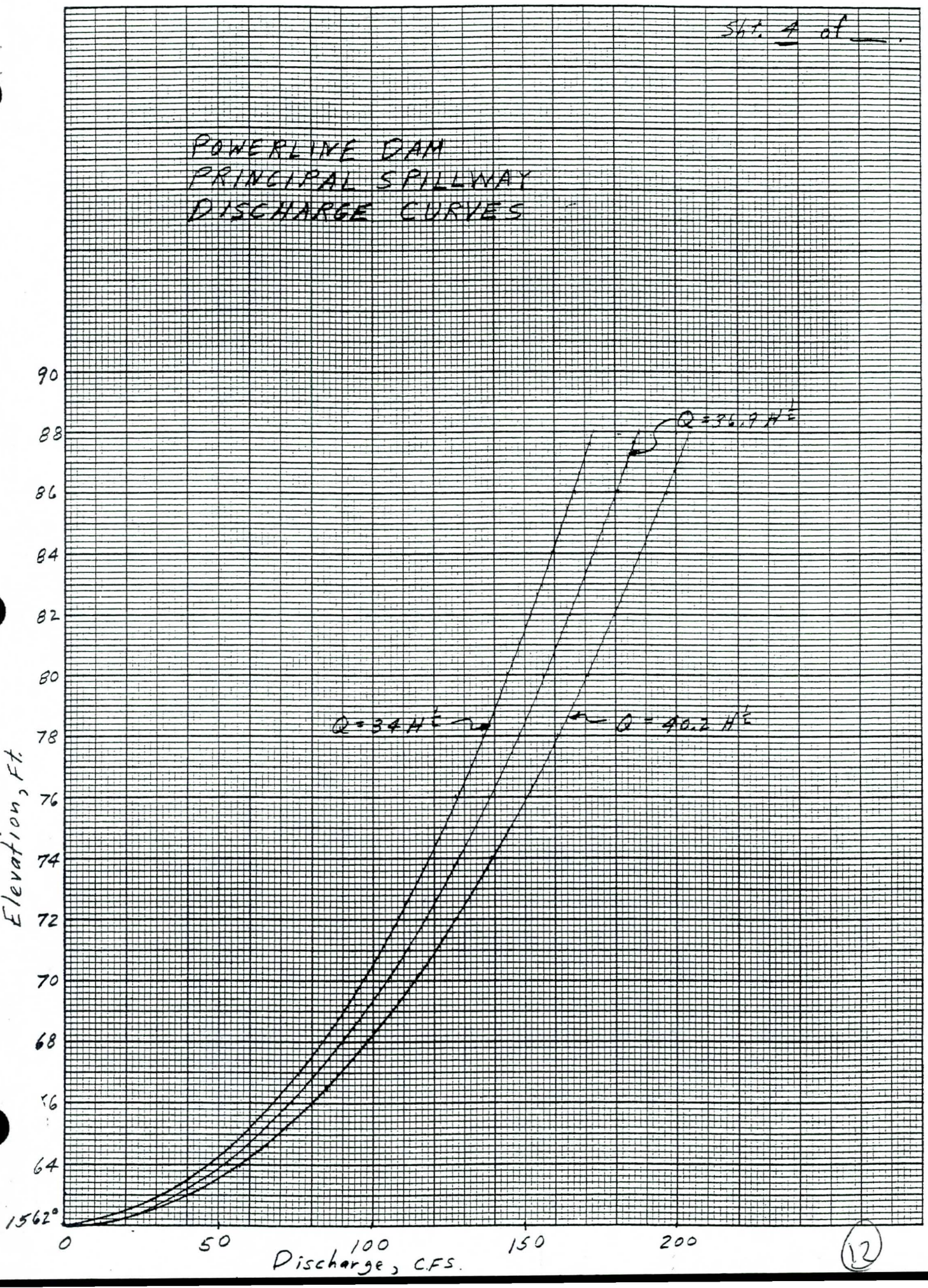
Desired Discharge Formula for Principal Spillway (Top of pipe at outlet = 1562±) is ∴ $K = \frac{165}{\sqrt{20.2}} = 36.9$ ∴ $Q = 36.9\,H^{1/2}$

4.47

Comparison of 33"ϕ and 36"ϕ outlet pipes yield the following discharge relationships:

| Head | | Flood Routing | 33"ϕ pipe | 36"ϕ pipe (*) | Elevation |
|---|---|---|---|---|---|
| $H$ | $H^{1/2}$ | $Q = 36.9\,H^{1/2}$ | $Q = 34\,H^{1/2}$ | $Q = 40.2\,H^{1/2}$ | |
| 2 | 1.41 | 53 | 48 | 57 | 1564 |
| 4 | 2.0 | 74 | 68 | 81 | 66 |
| 6 | 2.45 | 90 | 83 | 98 | 68 |
| 8 | 2.83 | 105 | 96 | 114 | 70 |
| 10 | 3.16 | 117 | 108 | 127 | 72 |
| 12 | 3.46 | 128 | 118 | 139 | 74 |
| 14 | 3.74 | 138 | 127 | 150 | 76 |
| 16 | 4.0 | 148 | 136 | 161 | 78 |
| 18 | 4.24 | 156 | 144 | 171 | 80 |
| 20 | 4.47 | 165 | 152 | 180 | 82 |
| 22 | 4.69 | 173 | 159 | 188 | 84 |
| 24 | 4.90 | 181 | 167 | 197 | 8. |
| 26 | 5.10 | 188 | 173 | 205 | 1588 |

POWERLINE DAM
PRINCIPAL SPILLWAY
DISCHARGE CURVES

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Soil Conservation Service

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BW ___ TC ___
RR ___
MWH ___
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Add to Powerline FM report

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Phoenix, AZ 85012

March 19, 1990

Dan Lawrence, P.E.
Arizona Department of Water Resources
15 South 15th Avenue
Phoenix, Arizona 85007

RE: Powerline FRS Repair, Final Design

Dear Dan:

Submitted herewith for your approval are three (3) copies of the drawings, specifications, and design report for the Powerline FRS Repair, Final Design Phase. Following are responses to the comments in Mr. Jenkins' letter of February 28, 1990.

GENERAL

Regarding drain outlets, the intent is not to construct any drains during this contract. The geologic investigation data gathered during this contract will be analyzed after construction when the need for drain outlets can be evaluated. If it is determined that drain outlets are required, they will be installed under a future contract.

That [illegible] drain [illegible] required

SPECIFICATIONS

1. ✓ Paragraph (8) was added to page 23-7 advising the contractor of ADWR's inspection of the foundation.

2. ✓ The frequency of testing is specified in Section 94, Contractor Inspection. 1 test / 500 yd³ earthfill

3. The relative compaction and moisture requirements are specified in paragraphs (~~6~~ (5)) and (~~7~~ (6)) on page 23-7 of the specifications. — % 7 (6) not sufficient (FINAL OK) Section 6 doesn't include Class C compaction

4. ✓ The drainfill layer thickness of 18 inches specified on page 24-5 governs for this project, as 24-5 gives the site specific requirements relating to drainfill for this project. Sec. 24-3 doesn't

5. ✓ The equipment required for the inspection of the trench is specified in ~~Section 21.12.b~~, Trench Excavation. Not included Section 21-5 ok

6. ✓ The material specification for drainfill is referenced on page 24-1, Materials, and will be included in the contract package. 521-1 used Prelim. Sec. 94-4 discusses drainfill gradation (C136, C117), but not soundness and abrasion tests.

7. ✓ Conventional testing methods for earthfill are exclusively specified in Section 23.10 (page 23-7). ?

This statement [illegible] nuclear devices [illegible] not be used.

The Soil Conservation Service is an agency of the Department of Agriculture

DRAWINGS

1. The design for the excavation of the trench has been changed to extend completely through all compacted earthfill including the deep foundation areas between Station 70+00 and Station 110+00. As stated in the Final Design Report submitted with this letter, the transition zone is not considered a reliable defense against potential fissure cracks.

2. The concrete cap shown on the drawings is existing. The drawings have been corrected to show this.

The requirement of an application and fee was discussed with the Flood Control District of Maricopa County during the monthly coordination meeting held March 15, 1990.

We would appreciate your response to this final design review by April 9, 1990. If you have any questions concerning the design, please contact John Harrington or Tom Jayo of my staff at 640-5152.

Sincerely,

Donald W. Gohmert

DONALD W. GOHMERT
State Conservationist

Enclosures

cc:
Dan Sagramoso, P.E., Chief Engineer and General Manager, FCDMC

United States
Department of
Agriculture

Soil
Conservation
Service

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T.J.
12/4/89

RECEIVED
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ENGINEERING DIV.

Dan Lawrence, P.E.
Arizona Department of Water Resources
Engineering Division
15 S. 15th Avenue
Phoenix, AZ 85007

RE: Geologic and Fissure Review of Powerline FRS and Vicinity

Dear Dan:

On November 21, 1989 a field trip was made to CAP Canal and Ironwood Road to discuss fissures and geology with Bureau of Reclamation geologists John Sandoval and Sam Bartlett. Those at the meeting from SCS were as follows:

John Harrington, State Design Engineer
Aubrey Sanders, State Geologist
Blake Covey, Civil Engineering Technician
Rick Pershken, Geologist Trainee
Tom Jayo, Design Engineer

Bureau geologists showed us Junkers fissure which is a 20 ± year old fissure that intercepts the CAP Canal. The Bureau repaired the fissure by backfilling, capping with soil cement, and mounding compacted material over the fissure approximately one foot above the surrounding ground to prevent water from entering the fissure. The Bureau has replaced several hundred feet of canal in the area with thicker concrete and reinforcing to help in bridging possible future fissure activity.

We looked at a shotcrete overland drainage channel on the east side of the CAP Canal that had numerous repairs made due to cracking. The damaged area lined up with the fissure on the opposite side of the CAP Canal.

We discussed various repair methods with Bureau geologists. The Bureau has considerable experience repairing fissures in Arizona along the CAP. Repairs have been made to the CAP Canal, O&M roads and protection dikes above the CAP. They have tried the following repair methods:

1. Placing a large amount of gravel material over the area where the fissure was expected. This method failed and resulted in the gravel washing into the fissure rather than sealing it.

[illegible] this [illegible] before he left on his leave.
[illegible] agreed w/ the letter's recommn that the problem of earth [illegible] Powerline FRS should be addressed when the fissure [illegible] the FRS. We would then know the nature & extent of damage. [illegible] the meantime, the central drain should be built to take care of the cracks. I suggest we respond on these lines

DKL JBM
NJN DK
TMB DEC
RH MTM GEH
LH
TC
FILE 11.02 AP

2. Driving sheet piling 28 feet ± to refusal perpendicular to fissure. This method does not terminate a fissure, and it is not yet known if it extends deep enough to cutoff flowing water.

3. Trenching and placing geotextile 25 feet ± perpendicular to fissure then backfilling with a bentonite slurry mix. This too has not been field tested, and fissure cracks extend to a depth well below 25 feet.

4. Over excavating five feet below the canal or roads and backfilling with compacted clay material. This method had no effect on the fissures.

They pointed out that all the methods they have tried are only a temporary solution and maintenance of the repair method is necessary to control the fissure. They emphasized that prevention of water entering the fissure at the first signs of fissuring is very important to prevent the fissures from eroding into very wide and deep ravines.

Based on the geology of the area, Hawk Rock is the top of a rock outcrop that slopes to the east approximately for two miles to a depth of 1200 feet then starts to raise again. The depth to bedrock under the FRS appears to be between 400 and 800 feet. Considerable fissuring has occurred within one half mile around Hawk Rock, generally where the depth to bedrock is less than 400 feet. Hawk Rock is within 0.8 mile of the Powerline FRS. Junkers fissure intercepts the CAP Canal just west of Ironwood Road which is within 800 feet of Powerline FRS. Geology maps show there are two concealed faults within one mile of Powerline FRS. SCS personnel discussed the water table fluctuation with Ken Black of ADWR and records show that there was a rapid drop in the water table between 1950 and 1965. It appears the water table recession has slowed considerably and records are not available to assess the recent water table fluctuations. According to a map showing long term water level decline in South Central Arizona by G.L. Wickersham, H.H. Schumann, and R.H. Raymond dated May 1983 the water table in the vicinity of Powerline FRS has declined approximately 125 feet ±.

The location of Powerline FRS in relation to the bedrock that slopes away from Hawk Rock and the lowering of the water table puts it in a fissure prone area. Repair of a known fissure at a dam will require extensive treatment. The repair could be quite expensive, on the order of $100,000, even though confined to an estimated 200 feet of dam length. We conclude that it is not practical to attempt to design for unknown fissures on Powerline FRS. A better solution would be to monitor for the occurrences of fissures and if one intercepts the dam to address the needed treatment at that time.

Based on the information learned from the Bureau of Reclamation's experience with fissures, and the known geology of the area, I am proposing that the repair of Powerline FRS treat only the embankment cracking using a center transition zone similar to the Vineyard Road FRS Repair.

We would appreciate your review and comments on the above proposal.

Sincerely

CHARLES R. ADAMS
State Conservationist

cc:
Ralph M. Arrington, SCE
Dan Sagramoso, P.E., Chief Engineer and General Manager, FCDMC

# FAILURE MODE AND CONSEQUENCE ANALYSIS
# For
# VINEYARD ROAD FLOOD RETARDING STRUCTURE

## PINAL COUNTY, ARIZONA
## NATDAMID AZ 0084

**July 13 2001**

## 1.0 INTRODUCTION

### General Description

Vineyard Road FRS is located about 35 miles east of downtown Phoenix and seven miles southeast of the City of Apache Junction. The project consists of the FRS embankment structure and an emergency spillway. The project is part of the Williams-Chandler Watershed Protection and Flood Prevention Project, which includes the Rittenhouse and Vineyard Road flood retarding structures.

Vineyard Road FRS is classified as a medium sized, significant hazard dam. The reservoir behind the FRS is 637 acres with a capacity of 4,310 acre-feet. Construction of the Vineyard Road FRS was accomplished under contract to Monaco Contractors, Inc. Construction was completed in 1967.

Except for considerable cracking of the structure during and after its first major impoundment in 1972, which necessitated investigations and construction of a defensive design measure (a central filter) repair, the dam has performed satisfactorily to date.

### Dam Data

Dam type: Rolled earthfill
Dam height: 16.5-ft
Dam length: 28,829-ft
Dam crest: width 14-ft; elevation 1579.5-ft
Spillways: Principal - 56-inch RCP; inlet elevation 1563.0-ft; Emergency spillways (two) - 300-ft wide (each) earthlined spillways; crest elevation 1574.8-ft
Freeboard: 4.7-ft
Reservoir Surface: 637-ac at spillway crest
Storage: 4,310-af at spillway crest
Hazard Classification: Significant

### Purpose and Scope

In general, the purpose of the Failure Mode and Consequence Analysis (FMCA) exercise was to:

- Identify potential site-specific failure modes for the dam.
- Discuss non-quantitatively the likelihood of the identified failure modes.

- Determine whether or not and how important failure mechanisms are being monitored
- Examine the consequences of failure and the adverse consequences of successful operation (e.g. – large spillway releases)
- Identify possible risk reduction actions that may be taken to reduce the likelihood of failure or to mitigate adverse consequences
- Determine what information, investigations or analyses may be needed to resolve uncertainties relative to potential failure modes.

(Note: In this phase, the FMCA team only examined the general nature of the "consequences" for the failure modes identified and where appropriate estimated how they may be different than previously anticipated. Greater detail on the estimate of the magnitude of the "consequences" for each significant failure mode will be addressed in the quantitative portion or risk analysis part of the risk assessment for the dam. At this time hopefully more detailed discussion on the emergency response plan for each structure will also be possible.)

**Team Members**

Larry Von Thun, Engineering Consultant and FMCA facilitator
Bob Eichinger, Kimley-Horn and Associates, Inc., Project Manager
Jim Scott, URS Corporation, Principal
Ken Euge, Geological Consultants, Principal
Tom Renckly, Flood Control District of Maricopa County, Project Manager,
Larry Lambert, Flood Control District of Maricopa County, Dam Safety Engineer
George Beckwith, Flood Control District of Maricopa County, Geotechnical Engineer
Justin Beeler, Kimley-Horn and Associates, Inc, Session Recorder
Tim Murphy, Flood Control District of Maricopa County, Civil Engineer (consequences)

## 2.0 MAJOR FINDINGS AND UNDERSTANDINGS GAINED

The following is a summary of the major findings and understandings for Vineyard Road Flood Retarding Structure as a result of the Failure Mode and Consequence Analysis (FMCA). Vineyard Road FRS is one of three adjoining flood retention dams (Rittenhouse and Powerline FRS being the other two dams). Because of concerns about desiccation cracking each dam was modified after construction but in slightly different ways. In carrying out the FMCA for these dams Vineyard Road was done first followed by Rittenhouse and then Powerline. The potential failure modes considered for the dams were of course very similar and it was natural to compare the likelihood of development of these failure modes among the three dams. Thus in the major findings and understandings given below some of the differences are noted as key findings and conditions of general similarity related to potential failure modes are also noted.

The major findings and understandings given below are organized as follows. First the most important design, construction, geologic, and performance differences or unique aspects related to the potential for failure mode development of Vineyard Road FRS are given. Findings related to failure modes or adverse consequences other than overtopping and spillway discharge are given first. Findings related to consequences are given next

followed by action items (risk reduction and investigations). Finally, general findings that are informational and / or generally similar for all three dams are listed. In making comparisons of the dams and the amount of cracking experienced at the time of the repairs it is instructive to note the year of the original construction of each and the year those repairs were made The difference between these two dates [shown in brackets] is the number of years drying had taken place before the central trench was excavated. Each trench was mapped for cracking depth, width and extent. They were Vineyard Road FRS – Constructed 1967, repaired 1983, [16 years], Rittenhouse – Constructed 1969, repaired 1979, [10 years], Powerline – Constructed 1968, repaired 1991 [23 years].

**Key findings / differences related to failure mode development – "static loading failures – seepage erosion – fissuring – foundation erosion –etc."**

1. Potential For Differential Subsidence from North End of Structure to South End of Dam. The FMCA team reviewed the bedrock charts, geotechnical profile, and history of groundwater decline data prepared by AMEC for Vineyard Road FRS. The charts show that at the north end of the structure bedrock is relatively shallow. The depth of alluvium increases rapidly to the south with the southern one-third of the dam underlain by a thick deposit of alluvial sediments. These sediments contain a relatively thick interval of more compressible lakebed clays. Settlement records for the adjacent section of the CAP indicate that differential settlement (subsidence) has occurred with the south half of the dam settling substantially more than the north half. The information summarized by AMEC clearly shows that the subsidence was caused by ground water declines. The structure and reservoir will most likely experience continuing subsidence and that differential subsidence between the north end of the dam and the south end of the dam could potentially cause earth fissures to form beneath the dam. The differential subsidence also has the potential to reduce reservoir capacity
2. Transverse and Longitudinal Crack Characteristics. The cracking experienced at Vineyard Road FRS, which in part results from drying of the embankment soil continues to progress with time as does the drying of the soil (the change in soil moisture pattern is very definitive). The weight of evidence indicates that the primary cause of the cracks is drying shrinkage although some longitudinal cracks on the upstream face appear to have been caused by settlement of the collapsing foundation soils. Cracks observed in the trench mapping extended to depths up to 21-feet, to widths as great as 4-inches, and there were approximately three thousand (3,000) documented cracks logged at the embankment since completion of construction of the central filter (in 1968). There is an important question whether the soil shrinkage due to drying of the embankment has reached it's full potential, or if shrinkage is still proceeding.
3. New Fissure found West of Powerline FRS. AMEC's Phase II investigations included review of previous reported earth fissures and liniments in the vicinity of the Powerline, Vineyard Road, and Rittenhouse FRSs. AMEC has also reviewed recent aerial photographs for possible earth fissures. The review of the photographs and subsequent field visits, a previously identified liniment was determined to be an earth

fissure. The end of the surface expression of this fissure is just west of the CAP about a 1,000 feet west of the northern segment of the Powerline FRS.

4. Collapsing Holocene Soils Exist along Foundation of Dam. Based on a review of foundation soil plots prepared by the District including the recent test borings and pits made as a part of ongoing Phase II geotechnical investigations it was noted that the cross sections indicated the approximate depth and thickness of collapsing erodible Holocene soils in the dam foundation. The collapsing Holocene soils were removed from beneath most of the upstream cutoff trench during the original construction of the dam. However, the recent Phase II geotechnical investigations confirm that these soils are present beneath the entire embankment at certain locations. The collapsing soils are highly susceptible to seepage erosion and collapse settlements upon saturation. Thus the FMCA team identified a potential seepage erosion failure mode passing through the highly erodible Holocene soils just below the embankment. Deep shrinkage cracks extending below the embankment or earth fissures could produce the conditions for this failure mode. See boring logs in Appendix of this section.
5. Estimated Level of Outflow from a Piping /Cracking or Fissure Related Failure. The FMCA team estimated the approximate peak discharge of the dam breach hydrograph from a seepage erosion failure in the dam to be in the range from 1,400-cfs to 5,600-cfs. The reservoir elevation for these events was set at the crest of the emergency spillway. This is considered appropriate, as the time above this level even under the PMF is less than 10 hours. Such a failure could potentially involve a continuous seepage pathway provided by some combination of cracks, fissures and defects in the central filter.
6. Understanding of Filter Characteristics and Role of Filter. The central filter for Vineyard Road FRS was installed by SCS in the early 1980s and was intended to be a crack stopper. The defensive mechanism provided is the formation of a "filter cake" in the initial process of seepage through a transverse crack, thereby preventing of erosion of soils upstream of the filter and minimization of flow into the filter and consequent internal erosion in the downstream part of the embankment. The specified filter meets generally accepted criteria for filtration. The factors that could potentially create defects that would compromise the function of the filter are segregation during construction and cracking. An internal SCS report on the general subject of cracking of dams in Arizona (Leckband, 1984, "A Barrier to Cracks in Dry Dams": see appendix for SCS note of July 1984) indicate that excavations through the central filter of the Vineyard FRS stood vertically for the full height of the dam. This suggests that the filter may be cemented and thereby susceptible to cracking in response to embankment shrinkage, settlement or earth fissuring. This issue deserves further study during the Phase II investigations.
7. Seismic Loading of the Dam was Not Considered as a Major Issue. There were no potential failure modes identified due to seismic loading and no "incipient" static loading potential failure mode that would be triggered by seismic load could be identified. A seismic exposure investigation being performed by AMEC indicates that the peak horizontal acceleration (PHA) at the Vineyard FRS due to the maximum credible earthquake (MCE) is about 0.12g. These small dynamic forces would not cause appreciable deformations of the stiff, unsaturated embankment, so earthquakes do not constitute a significant risk.

8. Dispersivity Tests. Pinhole and SAR test results indicate the embankment soils are non-dispersive. These tests are from the Phase II investigations by AMEC. It should be noted, however, that one of the recent AMEC tests showed the potential for dispersion). See appendix for test results.
9. Good Construction of Fill. The District's evaluation of the field density testing done by the SCS during construction determined that standard deviations for percent compaction and moisture content in relation to optimum were well within generally accepted guidelines. This indicates that the quality control was excellent and that the earthwork complied with specifications.
10. No Drain was provided for the Filter. No provisions for drainage of filter in the event of high hydrostatic heads were included in the design. However, if no defects are present in the filter, the weight of evidence indicates a filter cake would develop and flow into the filter would not be sufficient to require finger drains. The critical question is whether there are defects in the filter. A lack of a drain may create the possibility of high hydrostatic heads in the center portion of the dam.
11. Potential Loss of Central Filter Material Through Crack. Analysis by the District indicates that there is a potential for loss of central filter material by erosion into downstream crack should a crack or other defect in the filter be in direct contact with a system of embankment cracks.

**Key findings / differences related to failure mode development – "flooding, overtopping, spillway discharges."**

12. Settlement Survey Data. Past survey data indicate a lowering of crest elevation and significant variation in elevation along the crest. The dam crest monuments and downstream toe monuments need to be comprehensively reviewed and verified. Surveyed elevations of the monuments located near the crest do not reflect top of dam elevations. Questions were raised regarding the stability of the benchmarks and reference datums used in previous settlement surveys, as well as aerial topographic mapping of the dam. Actual (true) elevations of dam features (crest of dam, emergency spillway crest, principal spillway invert, etc.) need to be established in order to determine whether generalized settlement of the region has taken place or dam settlement has occurred. Also elevation data is needed for sections of the dam next to the emergency spillway. Elevations should be marked on the trash rack of principal spillway and elevations should be shown in tenths on the upper 3 feet of the staff gauge. Historical settlement surveys for the dam and the adjacent reach of the CAP establish the general pattern of subsidence over time, but the data needs to be updated.
13. Overtopping of Vineyard Road FRS. The FMCA team observed that a previous hydrologic and dambreak study for Vineyard Road FRS concluded that the dam would be overtopped during a full PMP/PMF event. However, the height of overtopping is low (less than 0.5-ft) and the duration of overtopping is short (less that five hours).
14. Emergency Spillway Erosion. There is a potential for spillway erosion, although the flow velocities are low. This is not considered as a significant failure mode.

## Consequence Evaluation

15. Importance of Good Consequence Scenarios. The CAP canal should be evaluated relative to its influence on the effects of dam failure and relative to its response to inflows from the dam. Develop an understanding of the importance of the CAP canal and the buffer zone between the dam and CAP canal (and flows into wasteways) to handle and store flows.
16. Dam Breach Characteristics. Owing to the stiff, clayey, unsaturated embankment soils, it is anticipated that the rate of development of a breach at Vineyard Road FRS will be relatively slow. The weight of evidence indicates that the embankment material possesses a relatively high resistance to erosion. . It is expected that breaches involving both overtopping and internal seepage erosion failure modes would be narrow and take a long time to form. Relevant case histories of dam breaches involving similar embankment materials include the Church Rock tailings dam near Gallup, New Mexico and the Clear Creek dam near Winslow, Arizona.
17. Importance of Incremental Damage Analysis. In the process of Risk Analysis, it will be important to route floods of different frequencies (peak and volume) through the reservoir and spillways to estimate the recurrence interval of incipient overtopping. The studies should include comparison of inundation limits from spillway flows versus inundation from combined spillway and overtopping flows and the incremental damage created by overtopping Differences may be small. Thus, the results of this study may be the factor that determines the need to raise the dam.

## Action Items – Risk Reduction or Investigations

18. Risk Reduction Measures. Measures that may be taken by the District at the present time and in the near future to potentially reduce and mange risk include: (1) Ensuring that the crest of the dam is kept level at design elevation; (2) Continue monitoring the impoundment levels during storm events (this includes both remote and as well as on-site monitoring) with observations during and after floods focusing on the potential failure modes identified in this assessment; (3) Provide training to O&M crews regarding identification of earth fissure development and other key observational parameters such as evidence of seepage erosion processes.
19. Lack of Stability Analyses. FMCA recommends that slope stability analyses for the embankment and the central filter be documented. A record of a stability analysis for the original design or the modifications has not been found.

## General Findings That Are Informational And/Or Generally Similar For All Three Dams

20. Differential Subsidence is On-Going. General ground subsidence in the region continues.
21. Ability to Understand the Potential for Fissure Development and How to Detect and Monitor Development of Fissures. The FMCA team discussed the mechanism for potential earth fissure development and concluded that the conditions exist along

certain portions of the dam for earth fissures to develop. The team recognized that settlement surveys can identify areas of earth fissure risk and special instrumentation such as time domain reflectrometry and fiber optics sensors may be capable of detecting earth fissures at the time of formation.

22. Small Floods May Assist in "Detecting" Fissures. The FMCA team recognized that earth fissures form instantaneously in the cemented alluvium below the Holocene surface layer and may extend to hundreds of feet in depth, but may not translate through the Holocene layer to the surface at the time of their formation. Flow in small channels over the alluvial fan surface during small floods may create erosion features in the Holocene soils revealing the fissures. Thus, inspection after small floods may provide a practical means of detection of earth fissures. Small floods, which are much more likely to occur than major floods, which could threaten loss of the dam, may thus provide an early warning for the detection and subsequent repair of an earth fissure.
23. Central Arizona Project Canal. The CAP canal should be evaluated relative to the effects of the various modes of dam failure. The FMCA team concluded that the CAP canal may help reduce the impacts of dam failure by allowing ponding to occur behind the upstream embankment of the canal and reducing the magnitude of the peak breach outflow (this is because the canal section would be the actual "dam failure breach". On the other hand, the CAP canal may under certain circumstances potentially cause an increase in flooding should the canal fail due to a controlled discharge from the dam or a "small" dam failure outflow or an emergency spillway discharge.
24. Groundwater Supply Wells. The FMCA team determined that, depending upon their radius of influence, new groundwater supply wells might increase differential subsidence and the risk of earth fissuring. Further studies are warranted to establish a minimum setback distance for new water supply wells that will prevent significant effects on differential subsidence. It is AMEC's preliminary estimate that wells near the dam and in the general vicinity have a radius of influence of approximately one-mile.
25. Broad Based Value of Risk Assessment. The District observed that the FMCA (risk assessment) discussions have broad-based value in risk reduction as they seemed to apply to all aspects of the dam safety program.
26. Impact of Test Cells. A "long time" was taken to fill the test cells with water (22 hours for Cell 2 and 44.5 hours for Cell 3) – this could have resulted in swelling of the clay soils and thus avoid a rapid flow through a crack capable of seepage erosion. The number of tests run relative to the length of the dam was statistically not adequate. Action Item: Conduct ponding tests for future evaluations.

## 3.0 POTENTIAL FAILURE MODES

Potential failure modes identified by the FMCA team are presented below. The failure modes were placed into one of four categories as follows. Except in special cases, only those potential failure modes in Category I will be considered in the risk (quantitative) analysis phase of the risk assessment exercise.

- Category I – Failure modes of greatest significance

- Category II – Failure modes of lesser significance (but not inconsequential).
- Category III – Failure modes for which insufficient information is presently available to make a judgement on the significance of the failure mode. The development of additional data and information is warranted. Additional records research may be justified.
- Category IV – Failure modes which are not physically possible or which are clearly not credible.

For each of the potential failure modes identified, a Failure Mode Description is briefly described and the factors that make the failure mode more likely (adverse factors) or less likely (positive factors) to occur are listed following the Failure Mode Description. In addition, any identified potential actions for risk reduction for each potential failure are then provided.

## CATEGORY I - FAILURE MODES OF GREATEST SIGNIFICANCE

***Failure from Overtopping of Dam Crest Due To Very Large Flooding Events Approaching the PMF (Category I).***

(Note: This failure mode for Vineyard FRS was highlighted in order to emphasize the importance of taking action relative to obtaining an accurate survey of the elevation of the crest surface, spillway crest and outlet invert and relative to ensuring that the crest is maintained reasonably level at the design crest elevation. However, the failure mode will not be carried into the quantitative analysis because of the remoteness of the floods necessary to produce the possibility of an overtopping failure.
Failure Mode Description: Hydrologic studies conducted on behalf of the Flood Control District for Powerline, Vineyard Road, and Rittenhouse FRS (James M. Montgomery - JMM, June 1985) indicated that there is a potential for Vineyard FRS to be overtopped by the full PMF storm event. The estimated amount of overtopping was reported in the cited report as a maximum height of 0.45 feet with a total overtopping duration of five hours. Overtopping of Vineyard FRS would occur at the low point of the dam crest (which appeared to be significantly below design crest based on available but potentially suspect survey data). As overtopping flows increased erosion of the crest and downstream slope would occur leading to an eventual breach of the dam.

Adverse Factors:

(1) Possible deviation in crest elevation of a minimum of 0.5 feet to a maximum of 1.5 feet.
(2) Two sections of dam 1,000 feet each were designed 1 foot lower (near each abutment).
(3) Reservoir tilting may have lost some storage surcharge.
(4) Transverse cracks would enhance the erodibility.
(5) Evidence of erosion from rills on face of embankment from rainfall.
(6) Additional inflows come from the spillway at the adjacent Rittenhouse FRS reservoir (this flow contribution needs to be confirmed for the flood routing for Vineyard Road FRS that it was accounted).

(7) The critical hydrologic condition may be a sequence of multiple (back to back) storms or the 24-hour event.

Positive Factors:

(1) Hydrologic studies for Vineyard FRS indicate overtopping to occur only for floods very near the full PMF, which would be a very remote event.
(2) The site specific PMF may not be as high as the JMM PMF.
(3) PMF is a rare event.
(4) There are high transmission losses in the upstream watershed.

Potential Actions for data collection and for Risk Reduction:

(1) Confirm dam crest profile elevation.
(2) Verify capacity of Vineyard Road FRS.
(3) Raise and level crest as necessary to return to design elevation.
(4) Construct the buffer zone (area between downstream toe of dam and CAP) and CAP canal to receive and pass discharges from overtopping of Vineyard over the side bank of the canal if necessary.

Consequences:

(1) Buffer zone between dam and CAP canal exists downstream of the dam.
(2) Relatively low level of population below dam.
(3) District has extensive easement upstream and downstream of dam – District can control of development.

***Failure From Potential Earth Fissure(s) Through The Dam Embankment In Association With A Significant Flooding Event (Category I).***

Failure Mode Description: The presence of existing earth fissures in the vicinity of Powerline FRS which is adjacent to Vineyard Road FRS demonstrates the possibility for an earth fissure to manifest itself at the Vineyard dam embankment. An earth fissure could cause a structural failure of the embankment by opening a crack through the structure. Alternatively the fissure could potentially undermine the dam embankment by abruptly causing separation of structure at the foundation with the dam bridging the fissure. If the fissure occurred in association with a large flood the structural failure from the fissure would provide a path for a seepage erosion breach. Potentially there will not be a high level of water impounded during fissure expression. In such a case there would be a minimal loss of water.

Adverse Factors:

(1) High tensile strains in middle region of dam indicating the possibility of fissure development.
(2) Dam is located in region of high-risk zone of subsidence.
(3) Recent (1984) and very recent (2000) evidence of earth fissures in vicinity of Powerline dam, which is adjacent to Vineyard. A fissure developed just west of site near Powerline FRS.
(4) Settlement is continuing at the structure.

(5) Good documentation of fissures and subsidence in the area.
(6) Localized groundwater pumping (deep high capacity wells).
(7) Likely existence of Holocene soils under dam foundation.
(8) Fissure likely not to be seen until flooding occurs.
(9) Potentially more groundwater wells from possible development especially at south end of structure.
(10) Development tends to mask potential fissures.
(11) Possible displacement (but no real evidence here).
(12) Flooding not frequency related.

Positive Factors:
(1) Awareness of existing fissures and fissure development. Can be monitored. District has large easements around structure.
(2) Requires flood to initiate fissuring/potential breach sequence. Small floods may cause fissure to express as surface manifestation without failure.
(3) Central Arizona Project canal provides a distinct, observable marker to help identify fissure development on the downstream side of the dam. Can be used as a "warning" flag.
(4) Probability of fissure developing/manifesting itself for the first time concurrent with a large impoundment is low.
(5) Effect of central filter – reduces impacts of water loss from impoundment.
(6) Fissure takes a lot of water but also could be plugged.
(7) Flow direction uncertain.
(8) Where dam is on resistant soil less potential for erosion.
(9) Further south on Vineyard the lower the chance of fissures because of away from bedrock highs (below ground).

Potential Actions for Risk Reduction:
(1) Recommend a joint agency fissure/subsidence-monitoring program.
(2) Segment dam into smaller dams.
(3) Training O&M crews to know signs of fissure development.
(4) Because small floods may initiate fissuring and thus provide advance warning of the potential presence of a fissure under the dam, inspections after rain events should include not only the dam but also the impoundment area and the areas downstream of the dam.
(5) Adjust easement further out to control development.
(6) Recharge instead of pumping.
(7) Fissures should be monitored if discovered.
(8) Employ CAP canal as a visual indicator of developing fissure.

***Failure From Seepage Erosion Through Transverse Cracks Causing Breach Of Dam In Association With A Major Flooding Event (Category I).***

Failure Mode Description: Potential for impounded water to infiltrate and flow into a transverse crack(s) existing in embankment prior to the major flooding event or a transverse crack that develops in association with the flooding. The transverse crack

allows the entry of a great enough flow of water to initiate seepage erosion and breaching failure. Two modes were identified for failure: Mode A - Failure through a large (wide) transverse crack that extends across the central filter, and Mode B - failure through entry of flow through multiple upstream longitudinal and transverse cracks, water flows to / through a flaw in the central filter and begins seepage erosion (and loss of central filter material) through a downstream transverse crack. Generally, the mechanics of the potential failure need to be linearized such that there is an upstream transverse crack(s) or inflow zone lined-up with a defect in the central filter, which in turn is lined-up with a downstream transverse crack(s).

Adverse Factors:

(1) Filter defect allowing passage of water to produce seepage erosion could be caused by segregation of filter material, cementation of the filter material, internal instability of the filter and/or loss of filter through downstream crack).
(2) Of the three structures at this location Vineyard had the most extensive cracking currently and the most extensive cracking observed in trench mapping.
(3) Potential for cementation of central filter and sustaining a crack. (See appendix for documentation from NRCS during Signal Butte investigations).
(4) Lots of cracks to carry flow to filter (documented 3,000 plus cracks).
(5) Presence of transverse settlement cracks.
(6) Presence of upstream longitudinal cracks which could be feeders to transverse cracks.
(7) Transverse cracks may go to bottom.
(8) Documentation in trench mapping of transverse cracks to 4 inches in width.
(9) Interconnectivity of transverse and longitudinal cracks (evident of other District structures such as Buckeye FRS dams).
(10) No drain for filter.
(11) Hydraulic fracturing by reservoir pressure could widen downstream crack.

Positive Factors:

(1) Formation of filter cake demonstrated in NRCS test cells Vineyard Road FRS.
(2) Coarse filter material placed – harder to erode.
(3) Properties of embankment soils may limit the maximum breach and size of discharge. Low anticipation of large breach outflow. Flows may not be that different from emergency spillway flow.
(4) Time to cause erosion short (time for water in impoundment less than 10 days).
(5) Need a lot of feeders to initiate seepage mechanism – Mode B.
(6) Foundation soils pre-wetted on downstream half of dam section.
(7) Filter meets filter criteria.
(8) Filter placed with tremie – minimize segregation.

(9) More clayey zones and less erodible.
(10) Ponding tests showed no water in central filter.

Potential Actions for Risk Reduction:
(1) Evaluate the need for finger drains.
(2) Monitor cracks and develop crack mapping program.
(3) Train O&M crews on mechanism of transverse and longitudinal crack formation.

Consequences:
(1) Evaluate incremental damages from dambreak scenarios.
(2) Select breach parameters for three modes: fissure, transverse crack, foundation failure.
(3) Estimate flow into interbasin area with and without inflow from Rittenhouse (interbasin is area between Vineyard Road FRS and CAP canal).
(4) Get flow information for CAP canal and operational parameters.
(5) Run dambreak for dam.
(6) Estimate breach parameters for CAP canal.
(7) Run dambreak for CAP canal.
(8) Develop consequences for each scenario.

***Failure From Seepage Erosion Resulting From A Flow Path Through Foundation And Along The Dam/Foundation Contact (Category I).***

(Note: This potential failure mode has three potential failure pathways. Pathways A and B were rated as Category I potential failure modes. Pathway C was rated as a Category II failure mode and is located in the section under Category II).
Failure Mode Description: This potential failure mode is initiated by transverse and longitudinal cracks carrying water to foundation in contact with Holocene soils. Path A is initiated by flow into longitudinal and transverse cracks that do not extend through the central filter but do extend to the foundation materials. Path B is initiated through flow into differential settlement cracks with an upstream source of water (impoundment). Path C is initiated from flow through remnant upstream borrow areas that potentially have exposed gravels through which a seepage pathway develops through buried channels into the foundation Holocene materials. Generally the potential failure mode mechanism is a loss of reservoir via the development of erosion/piping tunnel and a subsequent breach of dam via downstream embankment slope collapse.

Adverse Factors:
(1) Do have materials that are highly erodible (Holocene soils below the cutoff trench – see soil borings in Vineyard Road FRS appendix).
(2) The foundation was not pre-wetted to the extent done for Rittenhouse FRS.
(3) Have a high degree of cracking.
(4) Some silt and fine sand lenses exist.

(5) Have seen longitudinal cracks on upstream slope and toe.
(6) More intense storm grouping may be more critical – may facilitate seepage erosion.
(7) For Path B source of water is impoundment in conjunction with settlement cracks, which may extend to foundation.

Positive Factors:
(1) Need significant storm event.
(2) Time for formation and erosion is long compared to time of impoundment (impoundment time less than 10 days).
(3) Erodible materials only in some locations (20% of the dam approximately).
(4) Central drain interrupts and collects flow.
(5) Need high degree of open fracturing.
(6) Crack has to extend all the way through to foundation and occur in erodible materials and clear to downstream.
(7) For Path B settlement cracks may not extend to foundation.

Potential Actions for Risk Reduction:
(1) Need to evaluate existence of Holocene soils in foundation of dam.
(2) Need to prepare and continue crack mapping.
(3) Remedial measure is to add drain to filter.
(4) Failure mode mainly impacts the lower half of the downstream toe (downstream portion only).

## CATEGORY II – FAILURE MODES OF LESSER SIGNIFICANCE (BUT NOT INCONSEQUENTIAL)

***Failure from Seepage Erosion due to Piping through Foundation (Category II - Considered but not highlighted).***

(Note: This potential failure mode has three potential flow paths. The Failure Mode associated with flow paths A and B was rated as Category I potential failure modes and was presented above. The potential for the Failure Mode associated with flow path C was rated as a Category II failure mode and is presented as follows).
Failure Mode Description: The Failure Mode from pathway C is initiated through remnant upstream borrow areas that potentially have exposed coarse materials (e.g., sands and gravels) in contact with reservoir water which provides a seepage pathway through buried channels to Holocene materials in the dam foundation. Generally the potential failure mode mechanism is a loss of reservoir via the development of a seepage erosion or piping tunnel and a subsequent breach of the dam via downstream embankment slope collapse into the erosion tunnel.

Adverse Factors:
(1) Do have materials that are highly erodible (Holocene soils below the cutoff trench.

(2) The foundation was not pre-wetted to the extent done for Rittenhouse FRS.
(3) Have a high degree of cracking.
(4) Some silt and fine sand lenses exist.
(5) Have seen longitudinal cracks on upstream slope and toe.
(6) More intense storm grouping may be more critical – may facilitate seepage erosion.

Positive Factors:
(1) Need significant storm event to occur.
(2) Time for formation and erosion is long compared to time of impoundment (impoundment time less than 10 days).
(3) Erodible materials only in some locations (20% of the dam approximately).
(4) Need a free exit on the downstream toe or downstream of dam to effect the process.
(5) Hydraulic head difference between emergency spillway and principal spillway is low.

Potential Actions for Risk Reduction:
(1) Need to evaluate existence of Holocene soils in foundation of dam.
(2) Need to prepare crack mapping.
(3) Remedial measure is to add drain to filter.
(4) Failure mode mainly impacts the lower half of the downstream toe (downstream portion only).

***Failure from Potential Erosion of Abutments During Spillway Discharges (Category II - Considered but not highlighted).***

Failure Mode Description: Vineyard Road FRS has two emergency spillways – one located at each abutment. The emergency spillways are earth-lined limited service spillways. High velocity flows in the spillways could potentially erode the abutments of the dam causing loss of reservoir. One erosion process would be direct erosion of the far end of the abutment which forms the approach channel. This embankment is not protected but velocities are less than farther down the spillway. The second erosion process would be erosion of the training dike, which would then allow spillway discharges to begin eroding the abutment of the embankment (which is in effect just the embankment wrapped around to form the spillway channel).

Adverse Factors:
(1) The abutments are not protected from erosion either from slope erosion from rills or from high velocity flows in the spillways. (The FMCA team estimated a range of spillway velocities for different spillway flows-design capacity (6,400 cfs per spillway @ 6.9 ft/s) and up to full capacity from top of dam to emergency spillway crest (12,884 cfs per spillway @ 9.1 ft/s))

(2) Evidence of existing erosion.
(3) Off-road vehicles causing gullying and ruts on abutment slopes exacerbating the erosion potential.

Positive Factors:
(1) Low velocities around inner bend of spillway approach channel.
(2) A low consequence of erosion occurs at upper end.
(3) NRCS may have designed the abutments as sacrificial zone – abutments are one-foot lower that dam crest in elevation.
(4) Training dikes protect abutments.

Potential Actions for Risk Reduction:
(1) Monitor emergency spillway performance during flood events.
(2) Monitor training dikes and abutments during flood events.

***Failure from Potential Embankment Slope Instability (Category II - Considered but not highlighted).***

Failure Mode Description: Potential embankment slope failure due to internal pressure in the central filter.

Adverse Factors:
(1) No drain outlet for central filter. Potentially could cause long duration head and development of a phreatic surface (pore pressures) within downstream slope.
(2) No stability analysis was located from previous studies. (none from original design or from the remediation with a central filter).
(3) Presence of numerous longitudinal and transverse cracks during its history.
(4) No consideration of presence of central filter.
(5) Potential of hydrostatic head in longitudinal crack.
(6) Some sections downstream founded without excavation into native materials (thus sliding could occur on weak uncompacted deposits that were near the ground surface prior to construction).

Positive Factors:
(1) No evidence of past slope instability – no evidence of slope slumping.
(2) Vineyard Road FRS has an upstream cutoff trench.
(3) Low driving head.
(4) Low internal pore pressure.
(5) Trench for central filter stayed open without problems as demonstrated during construction activities for the central filter (25-foot depth trench).
(6) Some impoundment has occurred.
(7) Excellent compaction and quality control for construction.
(8) High cohesion.
(9) Slope generally are adequate at upstream 3:1 and downstream 2:1.

(10) Some settlement has taken place.

Potential Actions for Risk Reduction:
(1) Conduct stability analyses.
(2) Watch for slope distortion - integrate monument data.
(3) Confirm design slope by survey.

***Failure from Piping around Principle Spillway (54-inch Reinforced Concrete Pipe) and Five Abandoned Irrigation Outlets (three 24-inch and two 18-inch RCPs) (Category II - Considered but not highlighted).***

Failure Mode Description: Infiltration of water into embankment material around pipe and irrigation outlets and is carried away due to seepage erosion along the pipe or due to piping leading to formation of an erosional tunnel, caving and formation of a breach failure.

Adverse Factors:
(1) Cutoff collars difficult to compact embankment materials around RCP pipe.
(2) Loose materials beneath pipes.
(3) Possibility of differential settlement/collapsible soils.
(4) P.I.'s relatively high but average 13.
(5) Shrinkage/linkage at the compaction boundary (drying shrinkage could take place and form a crack at the "natural boundary " just at the edge of the cutoff collars where the means of compaction/moisture conditioning may have been different).

Positive Factors:
(1) The principal spillway pipe is located on a concrete cradle.
(2) Five cutoff collars on 20 foot spacing.
(3) Collars interrupt potential shrinkage cracks.
(4) The outlet is ungated - less time for piping to develop.
(5) It would take a long time to develop a seepage erosion flow path or piping flow path with relatively small head.
(6) The central filter was constructed around cradle and pipes.
(7) The outlet was tested several times (from stormwater impoundments) with good performance. No evidence of problem during impoundments.
(8) Reinforced concrete pipe material - good performance with RCP, meets regulations.
(9) Large 54-inch diameter pipe - easier to inspect.
(10) RCP pipes and not CMPs.
(11) Documentation indicates that irrigation pipes are plugged with concrete.

Potential Actions for Risk Reduction:
(1) Video interior of spillway pipe.

(2) Monitor spillway outlet and inspect during regular inspections and during flood events.
(3) Locate and mark abandoned irrigation outlets on embankment slopes.
(4) Verify plugging of irrigation pipes.

***Failure from Seepage Erosion due to Flow/Seepage Erosion through Animal Burrows (Category II - Considered but not highlighted).***

Failure Mode Description: Small animal burrows have been documented in previous inspections and field visits to the dam. The burrows are primarily due to ground squirrels and/or kangaroo rats. The potential failure mode is water impounded on the upstream face of the dam would infiltrate into the animal burrows. The infiltrated water would initiate seepage erosion ultimately leading to an expanded erosion tunnel, collapse of material from above the tunnel and breach on the dam.

Adverse Factors:
(1) Longitudinal and transverse cracks appear to be target zones for burrowing animals due to their being already open.

Positive Factors:
(1) Impounded water is available only for a short time at high elevations in reservoir.
(2) Length and depth of cracks not great enough to go through dam.
(3) District has a rodent abatement and rodent repair program.

Potential Actions for Risk Reduction:
(1) Continue abatement and repair program.

## CATEGORY III– FAILURE MODES FOR WHICH INSUFFICIENT INFORMATION IS PRESENTLY AVAILABLE FOR MAKING ENGINEERING JUDGEMENTS

No Category III potential failure modes were identified by the FMCA team for Vineyard Road FRS at this time.

## CATEGORY IV – FAILURE MODES WHICH ARE NOT PHYSICALLY POSSIBLE OR WHICH ARE NOT CLEARLY CREDIBLE

***Failure from Potential Erosion of Emergency Spillway During Spillway Discharges (Category IV).***

Failure Mode Description: The emergency spillway is a limited service earth-lined spillway. Spillway discharges from extreme events could potentially damage the spillway floor and/or initiate headcut erosion into reservoir causing loss of reservoir. There would be little adverse consequences from this erosion– the main potential adverse

consequence is relative to reduced flow capacity – this may increase overtopping potential --- this seems extremely unlikely to be significant factor.
Note: Vineyard Road FRS has two emergency spillways, one located at each abutment.

Adverse Factors:
(1) The discharge velocity from the PMF event is estimated at 6.9 feet per second.
(2) No control sills to interrupt potential head cut.
(3) Damage to spillway basin and training walls at higher flows.
(4) No erosion protection for end of either left or right abutments.

Positive Factors:
(1) No drop at end of spillways to initiate erosion.
(2) Founded on strongly cemented clayey sand at level approach section of right spillway. (This is noted in Powerline FRS construction sheet 6 of 10 dated December 31 1964 – Powerline FRS and Vineyard Road FRS right emergency spillway are located on same area).
(3) Discharges in spillway are directed away from dam.
(4) Powerline and Vineyard Road principal outlets tailwater at downstream end of right emergency spillway - creates energy dissipation.
(5) Relatively short duration of flows in spillway.
(6) Left spillway ends with flat area then a rise to CAP canal – no drop to initiate headcut.
(7) Right spillway – outlet channel provides a drop but would be full of water from principal spillway or emergency spillway itself.

Potential Actions for Risk Reduction:
(1) Monitor emergency spillway performance during flood events.
(2) Monitor training dikes and left abutment during flood events.

***Failure from Potential Seismic Activity Impacting Structure (Category IV).***

Failure Mode Description: A seismic event occurs that impacts structure by opening or expanding existing transverse cracks (this then leads to the earlier described transverse crack failure mode).

Adverse Factors:
(1) Natural variations in geology (rock profile, soil depth) amplify ground motions to cause localized higher acceleration values.
(2) Brittle nature of dam embankment.

Positive Factors:
(1) Theoretical analysis for small strain- showed no sign of adverse effects from seismic shaking.
(2) Dam is already segmented and broken up.
(3) Flood event with earthquake is very rare event for consideration.

(4) Opportunity to view in advance.
(5) No shallow groundwater – no liquefaction impact.

Potential Actions for Risk Reduction:
(1) Examine and inspect dam and outlets and surrounding ground after earthquake (greater than a minimum of 0.05g) and high potential fissure areas.

## 4.0 SUMMARY AND CONCLUSIONS

Vineyard Road FRS was constructed pursuant to a relatively modern dam design. Construction appears to have been without any particular issues. The dam has performed normally and satisfactorily for 34 years except for cracking after the first major impoundment and the continuation of cracking since that time. The structure is satisfactorily maintained and monitored.

However, it is prudent to recognize that there exist for all dams specific ways that failure could come about that warrant attention and diligent monitoring. The identification of a condition or process as a "potential failure mode" does not imply that the dam is about to fail or even necessarily that there is a dam safety deficiency at the site. Rather it identifies physically possible conditions or processes (generally with a remote but still credible chance of occurrence) that persons associated with owning, inspecting, analyzing and operating the dam should be aware. Some of the potential failure modes are highlighted (or prioritized) for attention of the dam owners and operators. They are highlighted because the specific conditions at the dam and appurtenant structures are such that these failure modes are physically possible and are considered the most realistic and most credible potential failure modes definable at the site.

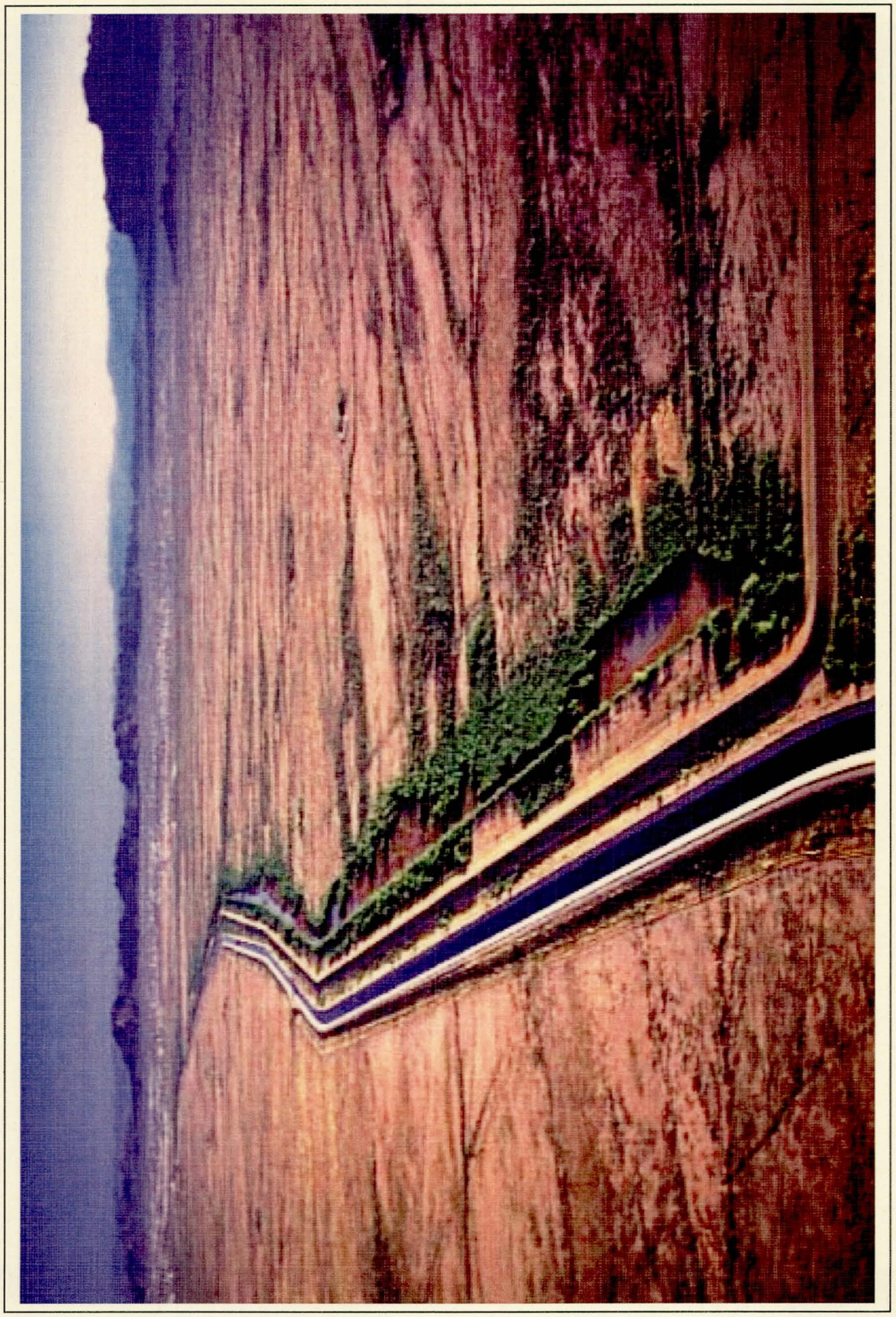
Four Category I potential failure modes were identified by the FMCA team. The Category I failure mode related to Overtopping due to a flood event approaching the PMF will not be carried into the quantitative analysis portion of the risk assessments at this time as it is considered to be a very remote possibility. The Category I potential failure modes identified that were considered most significant and appropriate for consideration in the risk assessment were failure due to an earth fissure, failure due to seepage erosion through transverse crack(s) causing a breach, and failure from seepage erosion through Holocene soils left in the foundation. The likelihood of these failure modes, however, is dependent on the presence or rapid development of a seepage erosion path at the time of major flooding from an earth fissure, from a transverse crack through the central filter (or a flaw in the filter), or from a tunnel through the Holocene foundation deposits. The length of time for any of these failure mechanism to develop is considered to be long compared to the time that water is impounded in the reservoir (less than 10 days) but they are all physically possible.

Awareness, monitoring, and understanding transverse crack development, and earth fissure development, and training (classroom training) of maintenance and operation teams about the possibility of these potential failure modes are key risk reduction actions

that appear warranted. Training should include before, after, and during inspections/flood events. Other risk reduction measures include a joint agency fissure/subsidence-monitoring program, and/or evaluation of segmenting the dam into smaller dams.

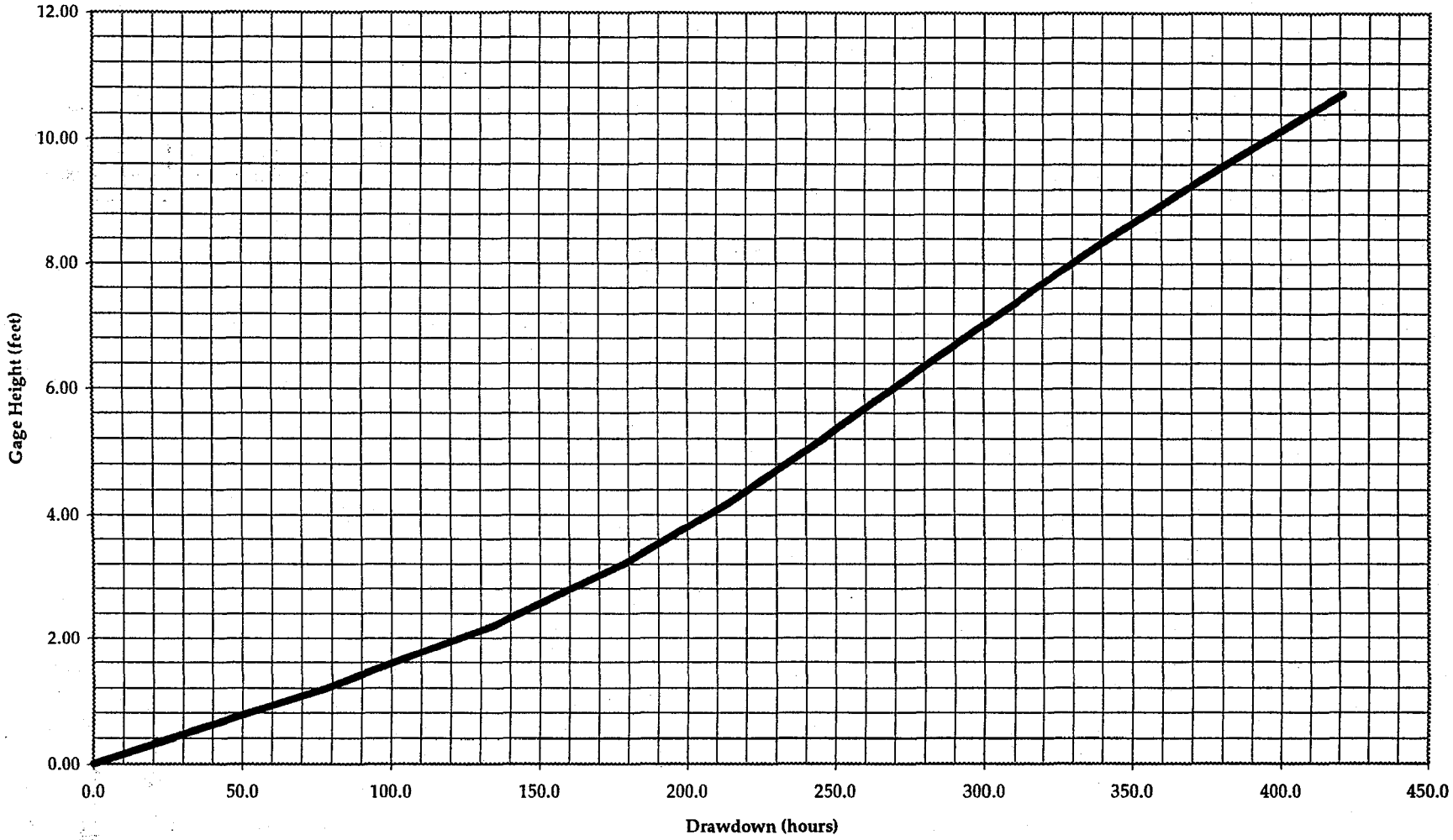
There are not a large number of people and structures at risk in the flood path in the event of a spillway discharge or dam failure by breach. There could be a likely chance that casualties could occur due to a dambreak or spillway discharge. A very significant consequence of dam failure would be the adverse impact on the Central Arizona Project canal. Conversely, depending on the magnitude of the dam failure outflow, the canal could mitigate the adverse consequences of dam failure. This appears to be particularly true for any seepage erosion or piping type failure. It is highly recommended that an individual emergency action plan be developed for Vineyard Road FRS.

## VINEYARD FRS APPENDIX - FMCA REFERENCE MATERIALS



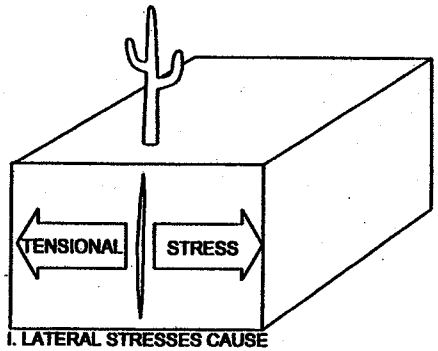
Vineyard Road FRS south end looking north.
Photo date: October 2001 Photograph Copyright Kimley-Horn and Associates, Inc.

## Vineyard FRS Drawdown

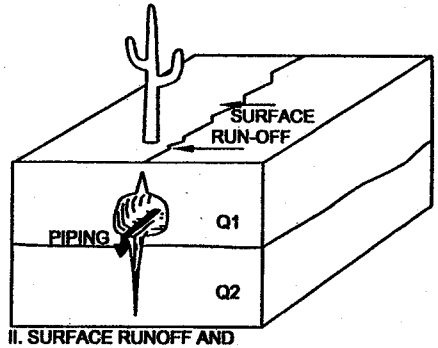



Prepared by DEG
April 17, 1997

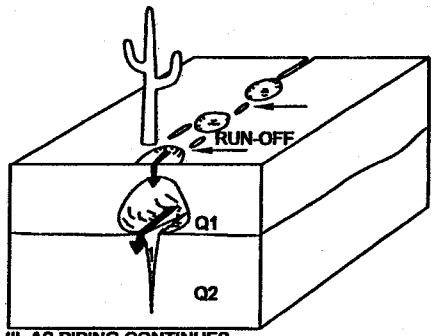
Source: McLain Harbers topography, FCD Contract 93-51



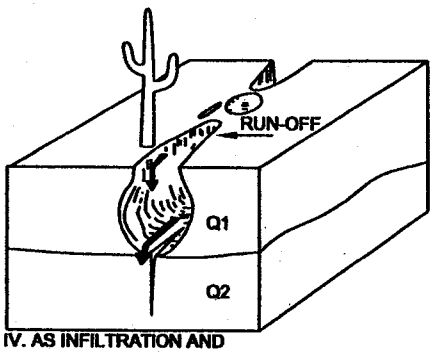

I. LATERAL STRESSES CAUSE TENSION CRACKING



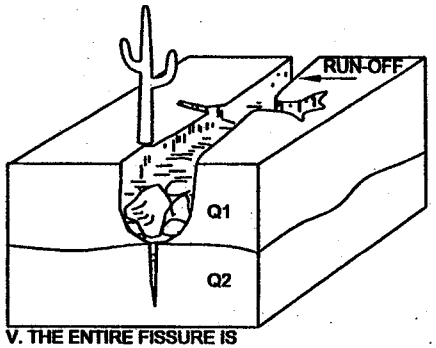

II. SURFACE RUNOFF AND INFILTRATION ENLARGE CRACK THROUGH SUBSURFACE PIPING



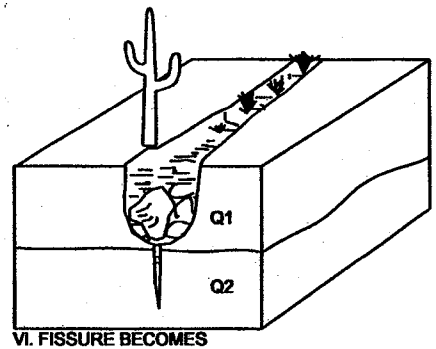

III. AS PIPING CONTINUES, FISSURE BEGINS TO APPEAR AT SURFACE AS SERIES OF POT HOLES AND SMALL CRACKS




IV. AS INFILTRATION AND EROSION CONTINUE, FISSURE ENLARGES AND COMPLETELY OPENS TO SURFACE AS TUNNEL ROOF COLLAPSES




V. THE ENTIRE FISSURE IS OPENED TO THE SURFACE AND ENLARGEMENT CONTINUES AS FISSURED WALLS ARE WIDENED; EXTENSIVE SLUMPING AND SIDE STREAM GULLYING OCCUR




VI. FISSURE BECOMES FILLED WITH SLUMP AND RUN-OFF DEBRIS AND IS MARKED BY VEGETATION LINEAMENT AND SLIGHT SURFACE DEPRESSION; IT MAY BECOME REACTIVATED UPON RENEWAL OF TENSILE STRESS

MODIFIED AFTER BELL, 1981

LEGEND

Q1 - HOLOCENE SOILS WITH STAGE I CEMENTATION
Q2 - PREHOLOCENE SOILS WITH STAGE II CEMENTATION

| | |
|---|---|
| JOB NO. | 0-117-001122 |
| DESIGN: | RW |
| DRAWN: | DJL |
| DATE: | 6/01 |
| SCALE: | NTS |

FIGURE 7-1

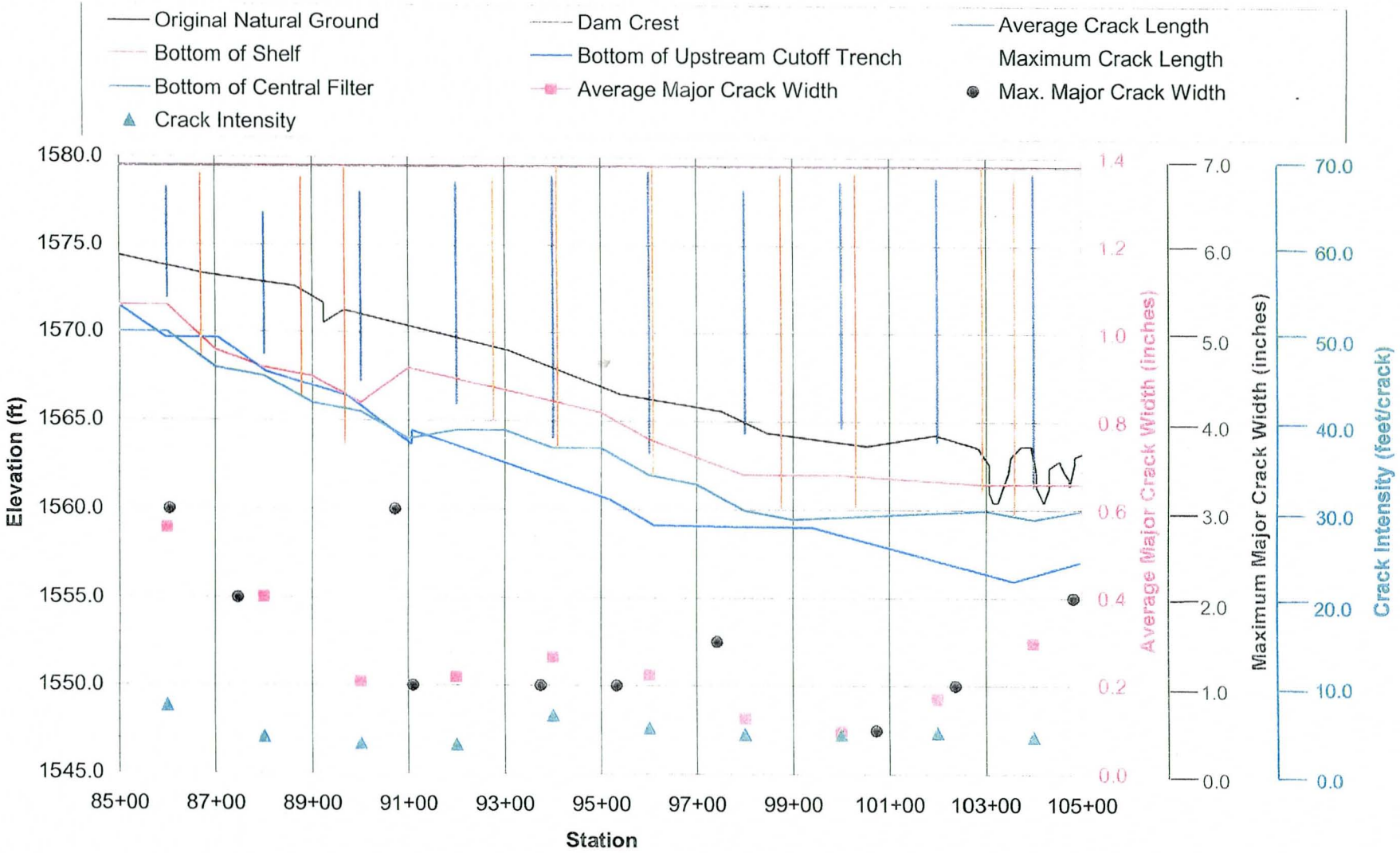
STAGES OF FISSURE DEVELOPMENT

amec

G:\Engineering-Development\2000Projects\0-117-001122 Structures Assessment-Ph II\Task 2 - Vineyard FRS\Cad\FISSURE.dwg

Flood Control District of Maricopa County
Structures Assessment Phase II
Investigation of Cracking
AMEC Job No. 0-117-001122
FCD Contract No. 2000C006
Assignment 2, Vineyard FRS

## Vineyard FRS Investigation of Cracking



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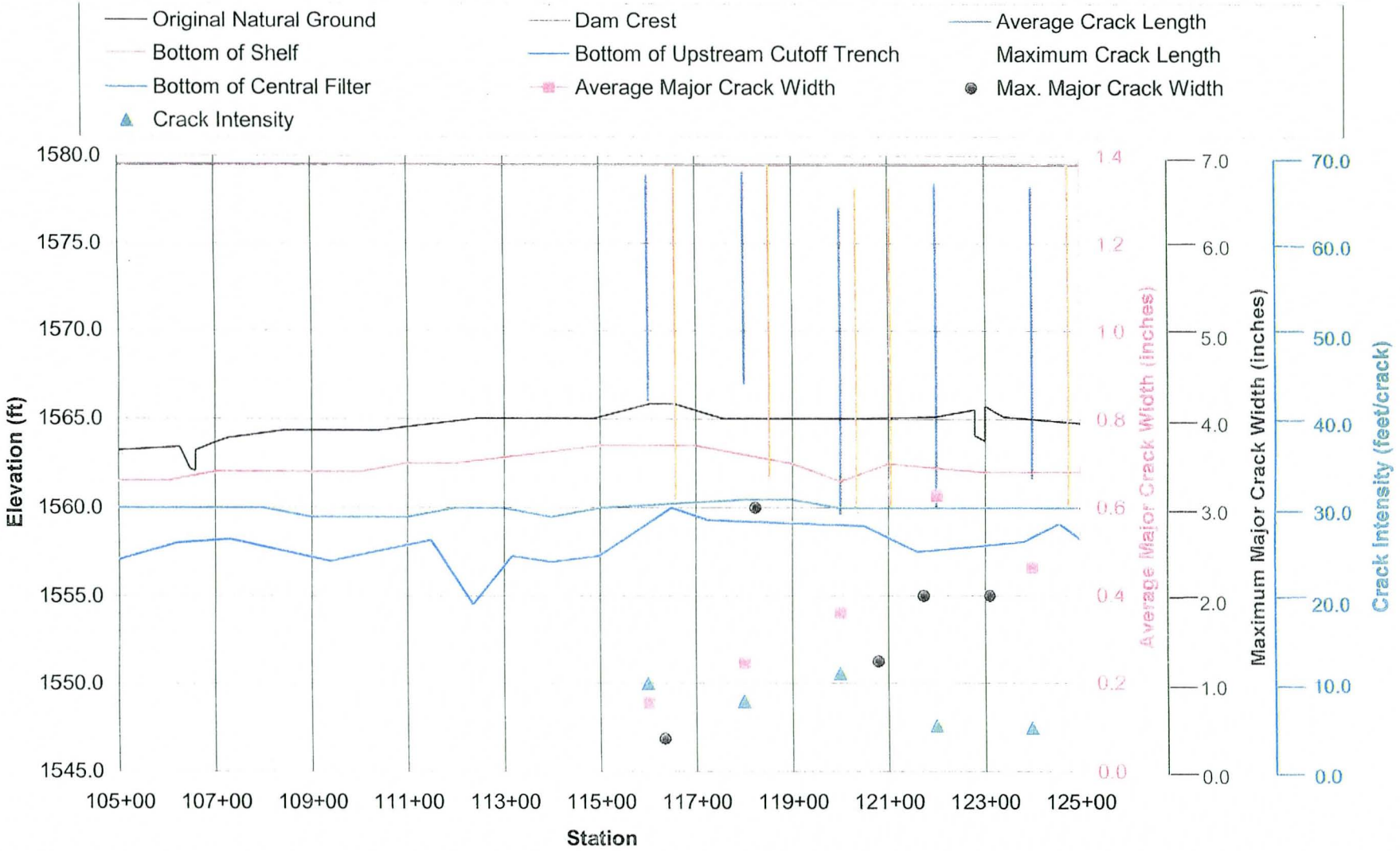

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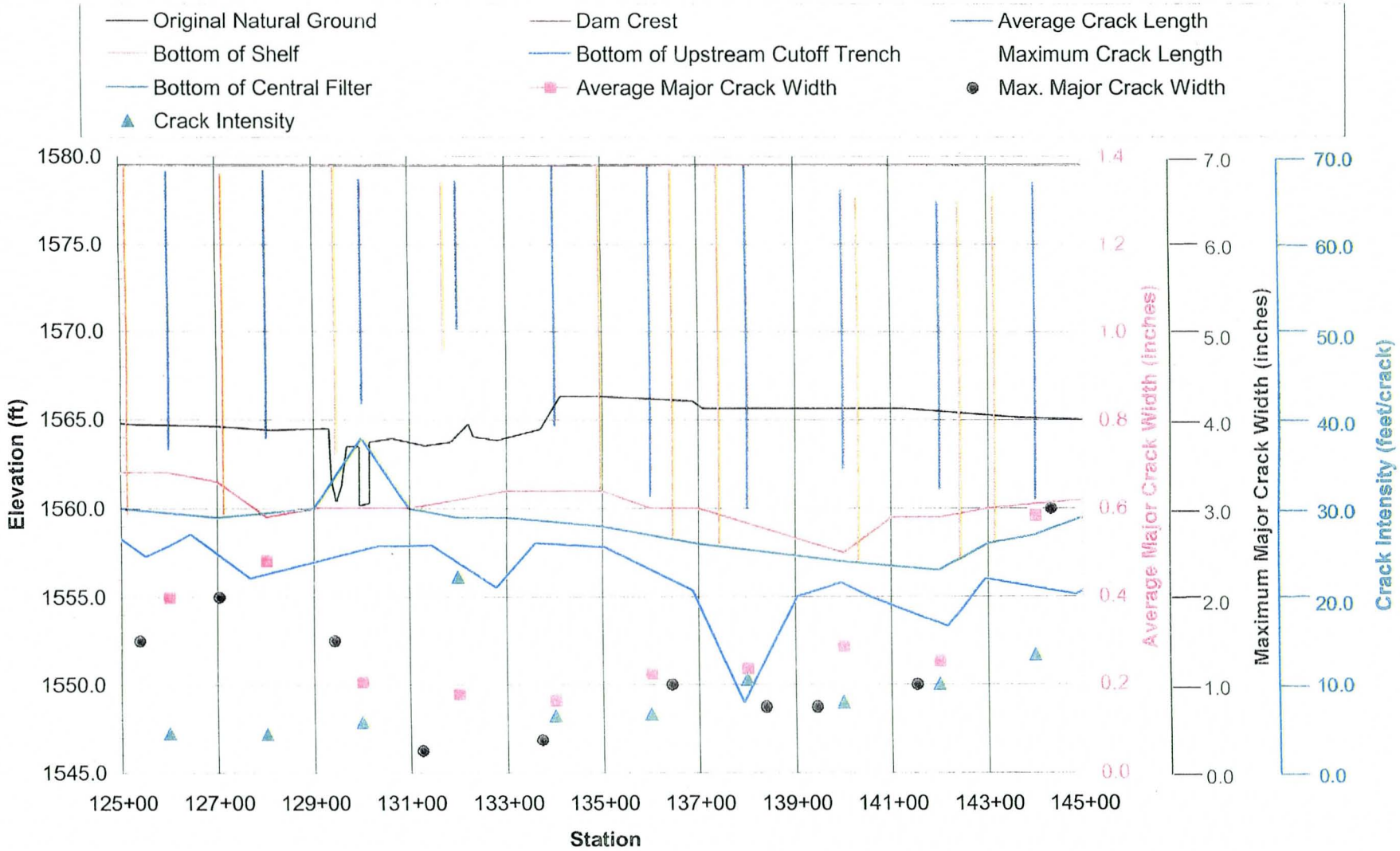

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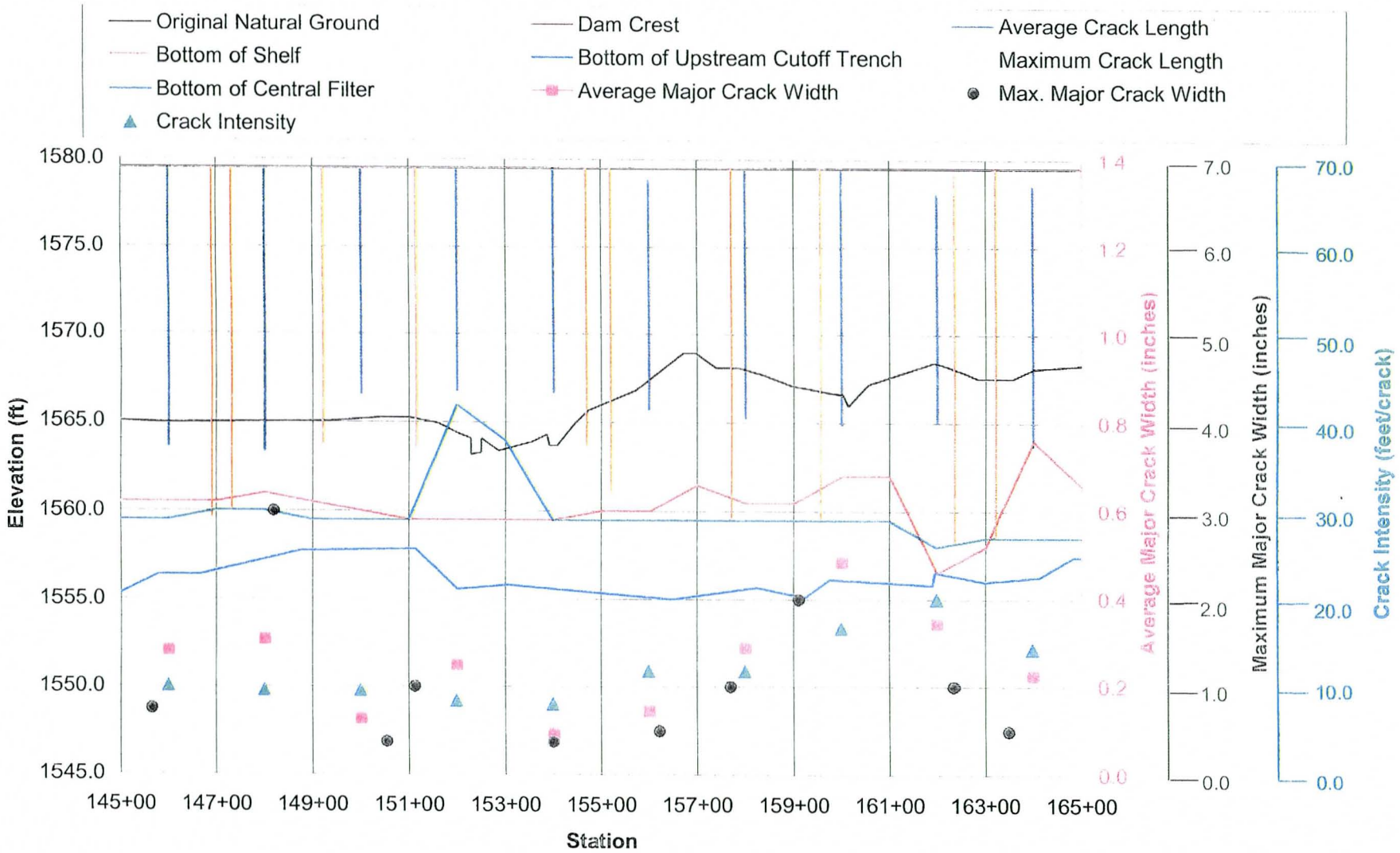

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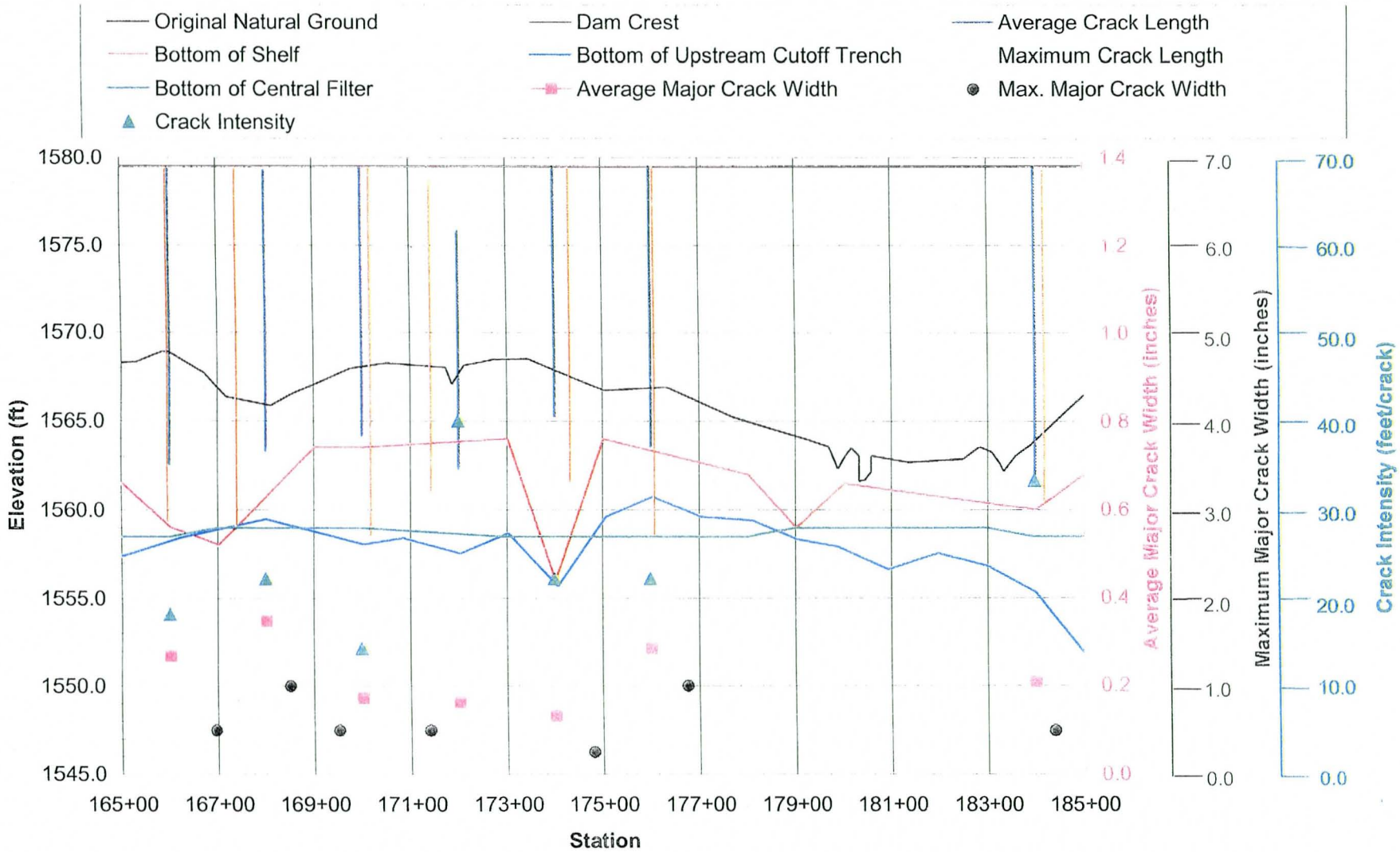

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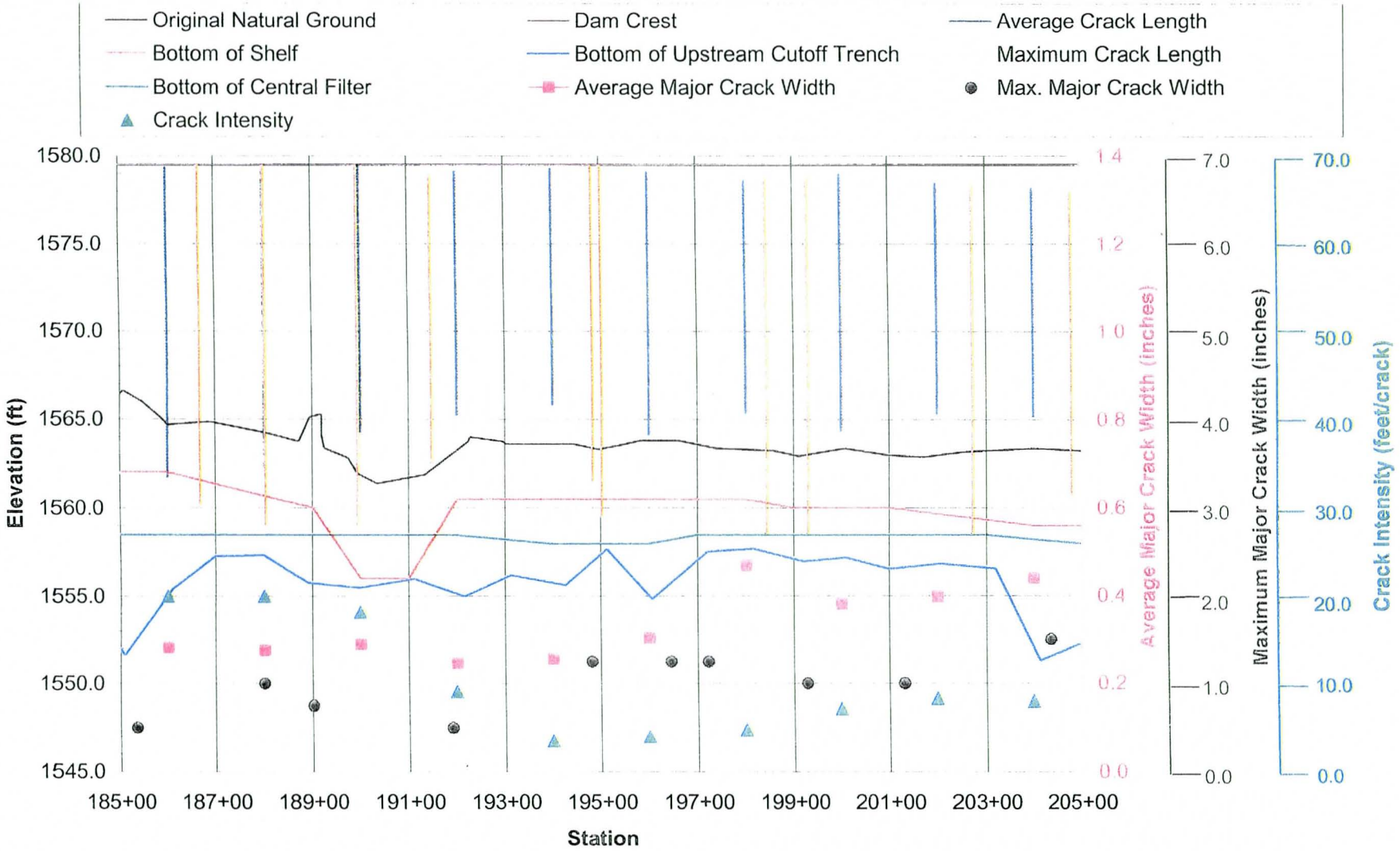

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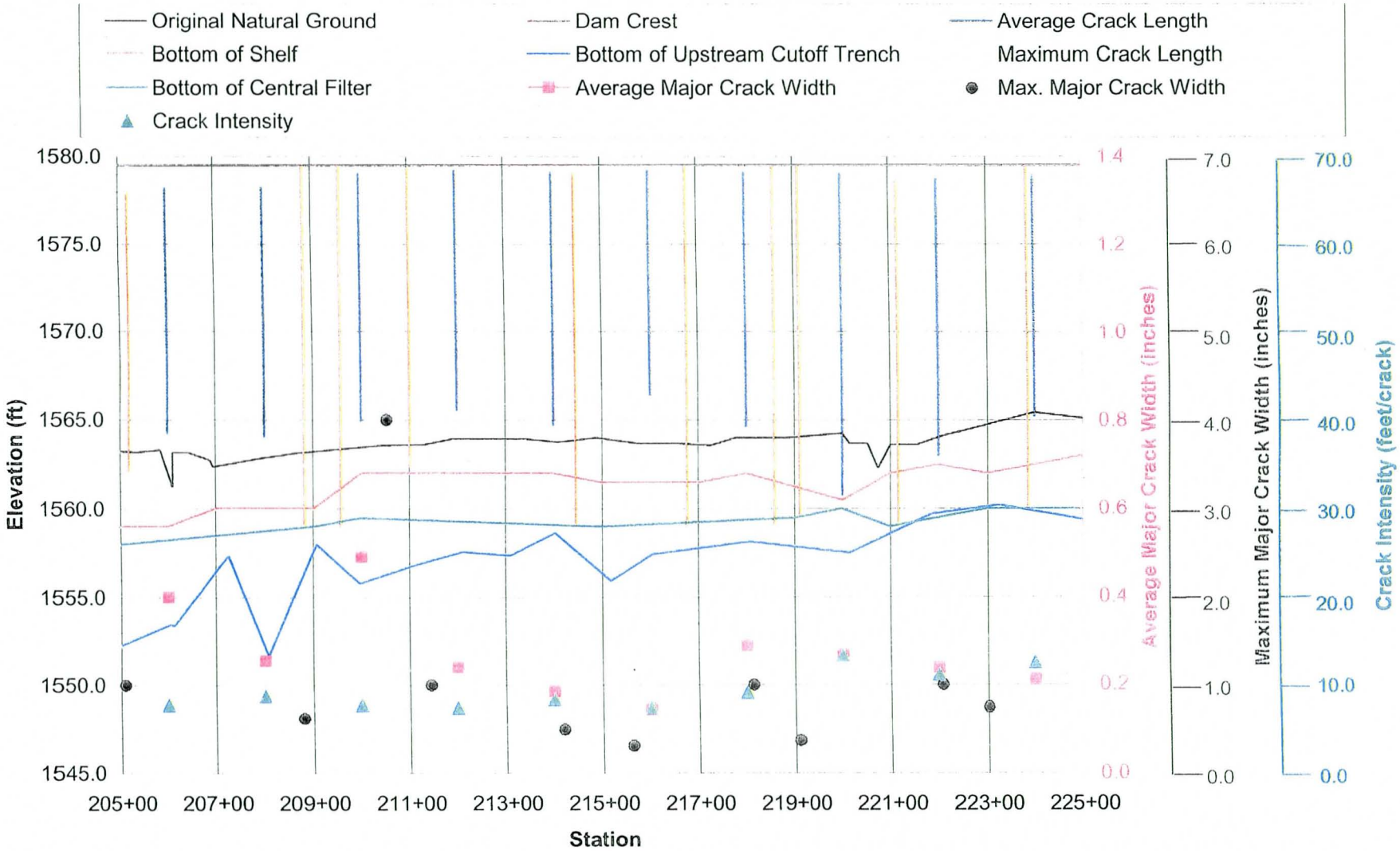
Assignment 2, Vineyard FRS

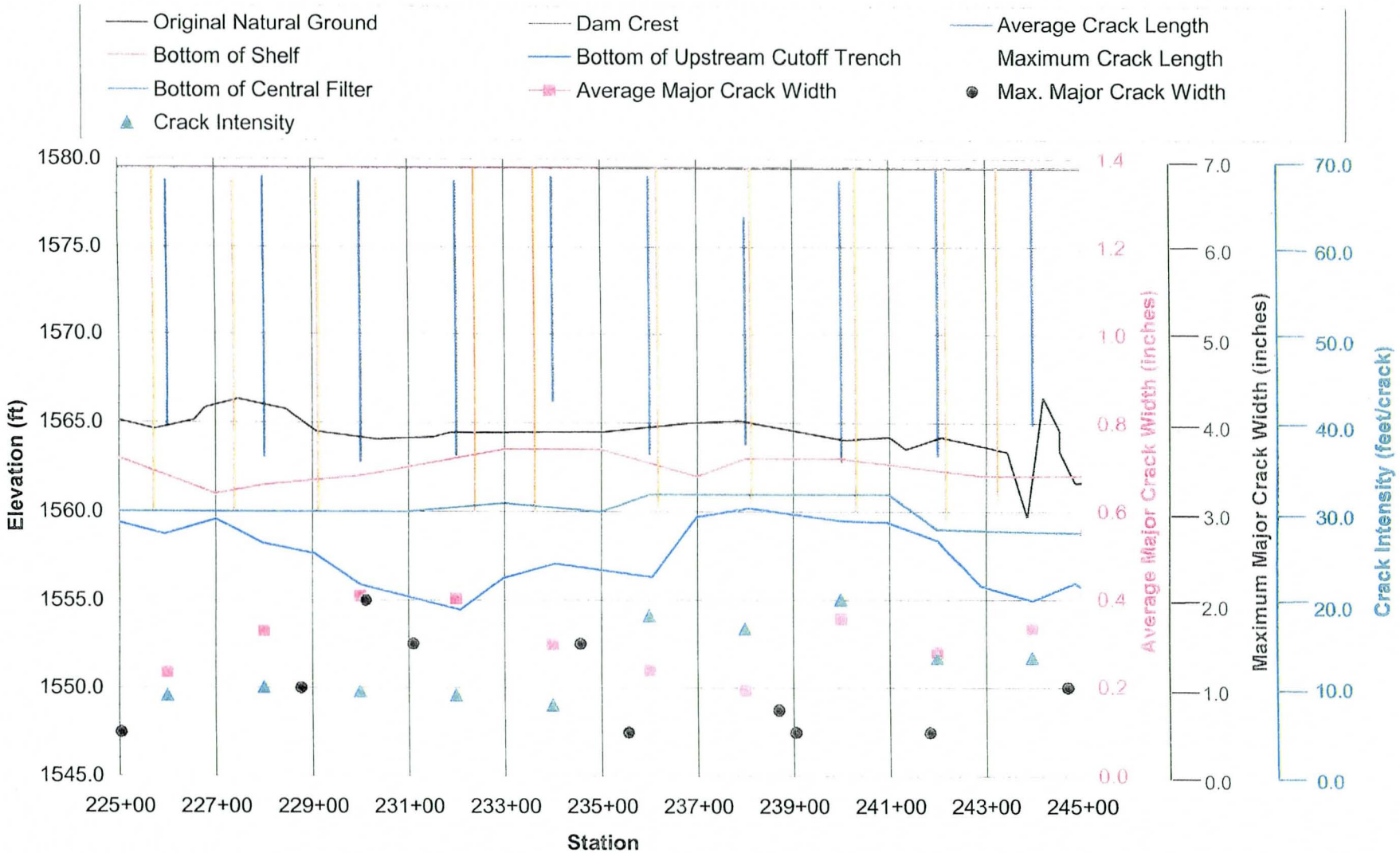
## Vineyard FRS Investigation of Cracking



Flood Control District of Maricopa County
Structures Assessment Phase II
Investigation of Cracking
AMEC Job No. 0-117-001122
FCD Contract No. 2000C006
Assignment 2, Vineyard FRS

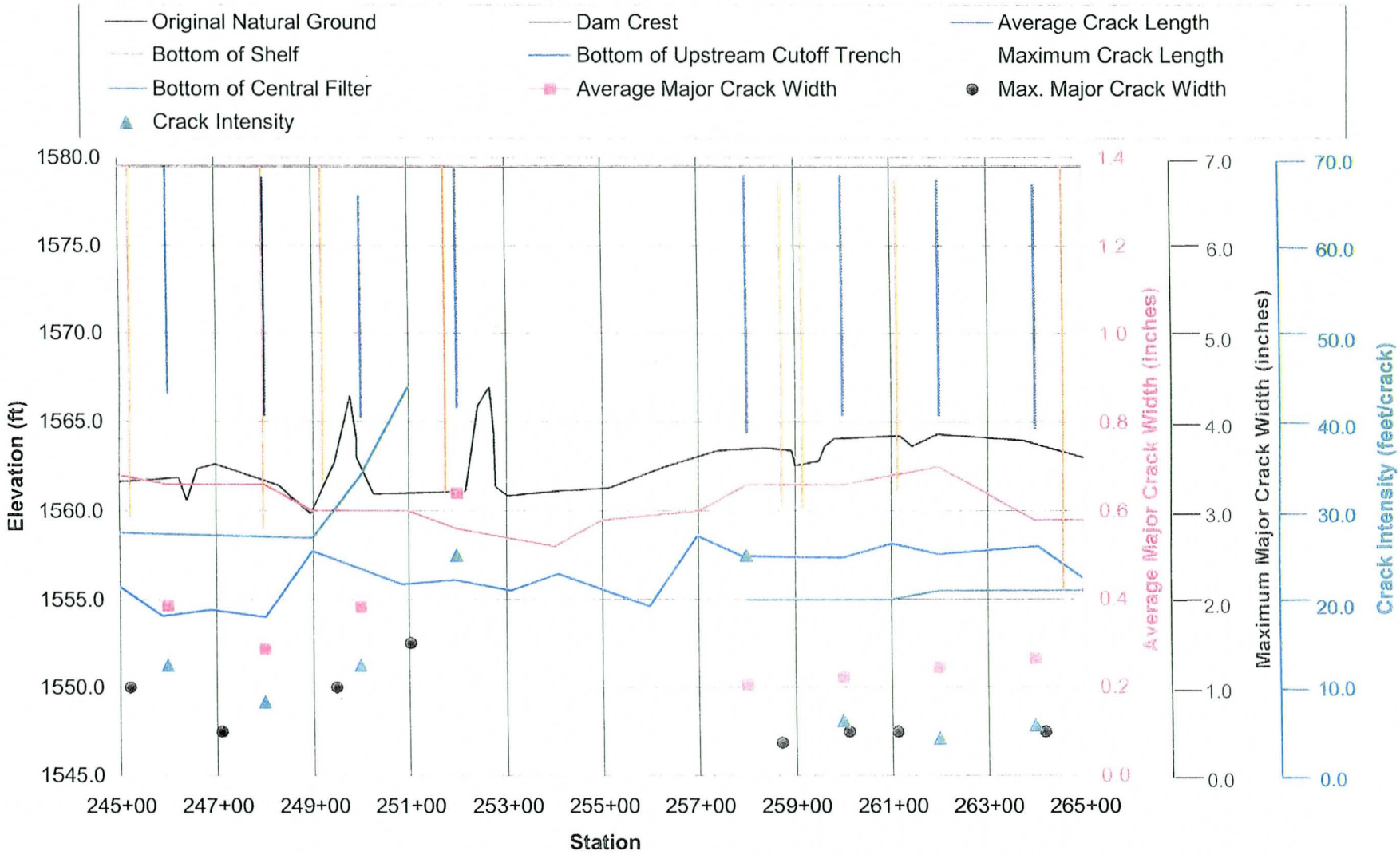
amec

## Vineyard FRS Investigation of Cracking



Flood Control District of Maricopa County
Structures Assessment Phase II
Investigation of Cracking
AMEC Job No. 0-117-001122
FCD Contract No. 2000C006
Assignment 2, Vineyard FRS

## Vineyard FRS Investigation of Cracking



Flood Control District of Maricopa County
Structures Assessment Phase II
Investigation of Cracking
AMEC Job No. 0-117-001122
FCD Contract No. 2000C006
Assignment 2, Vineyard FRS

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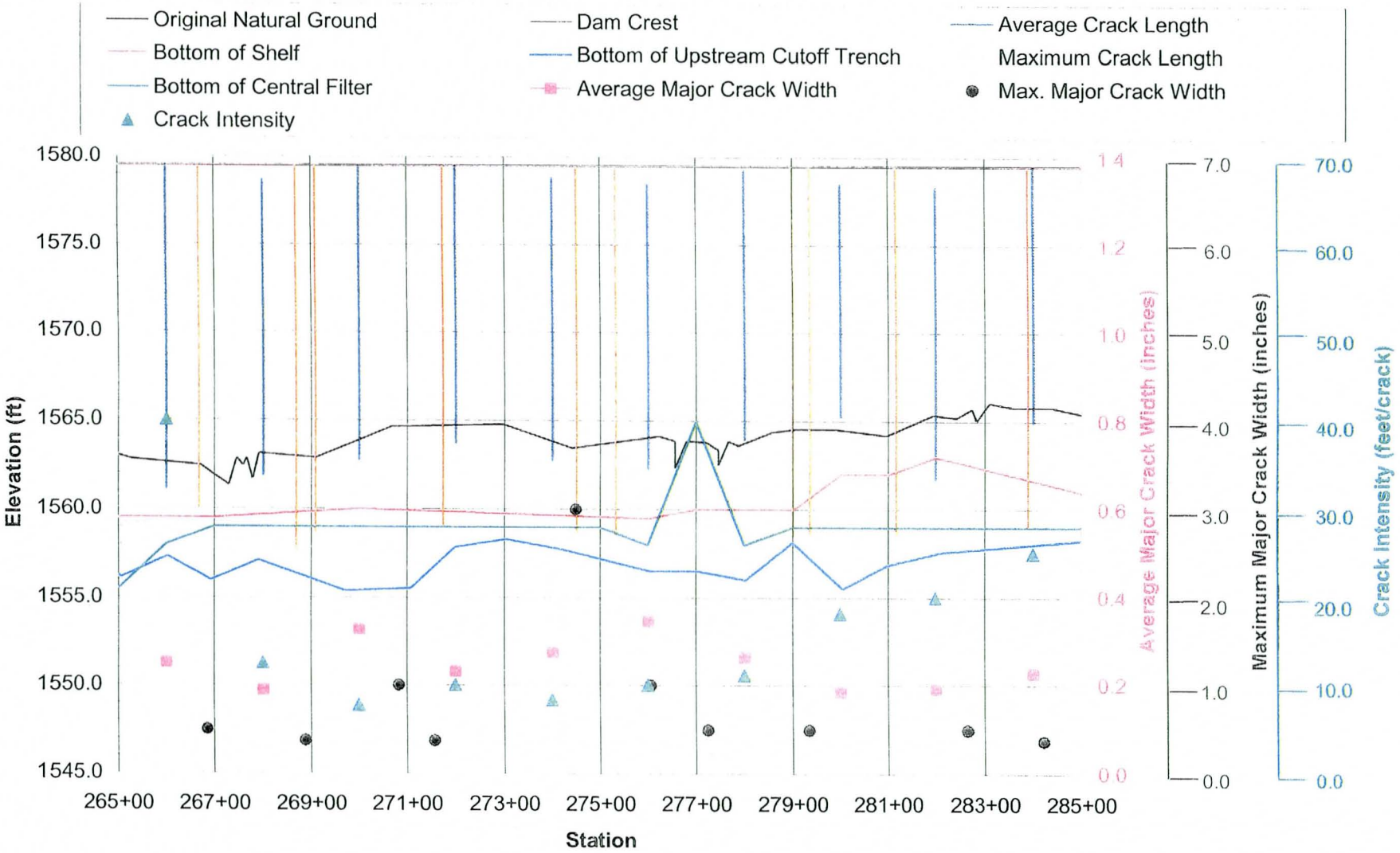
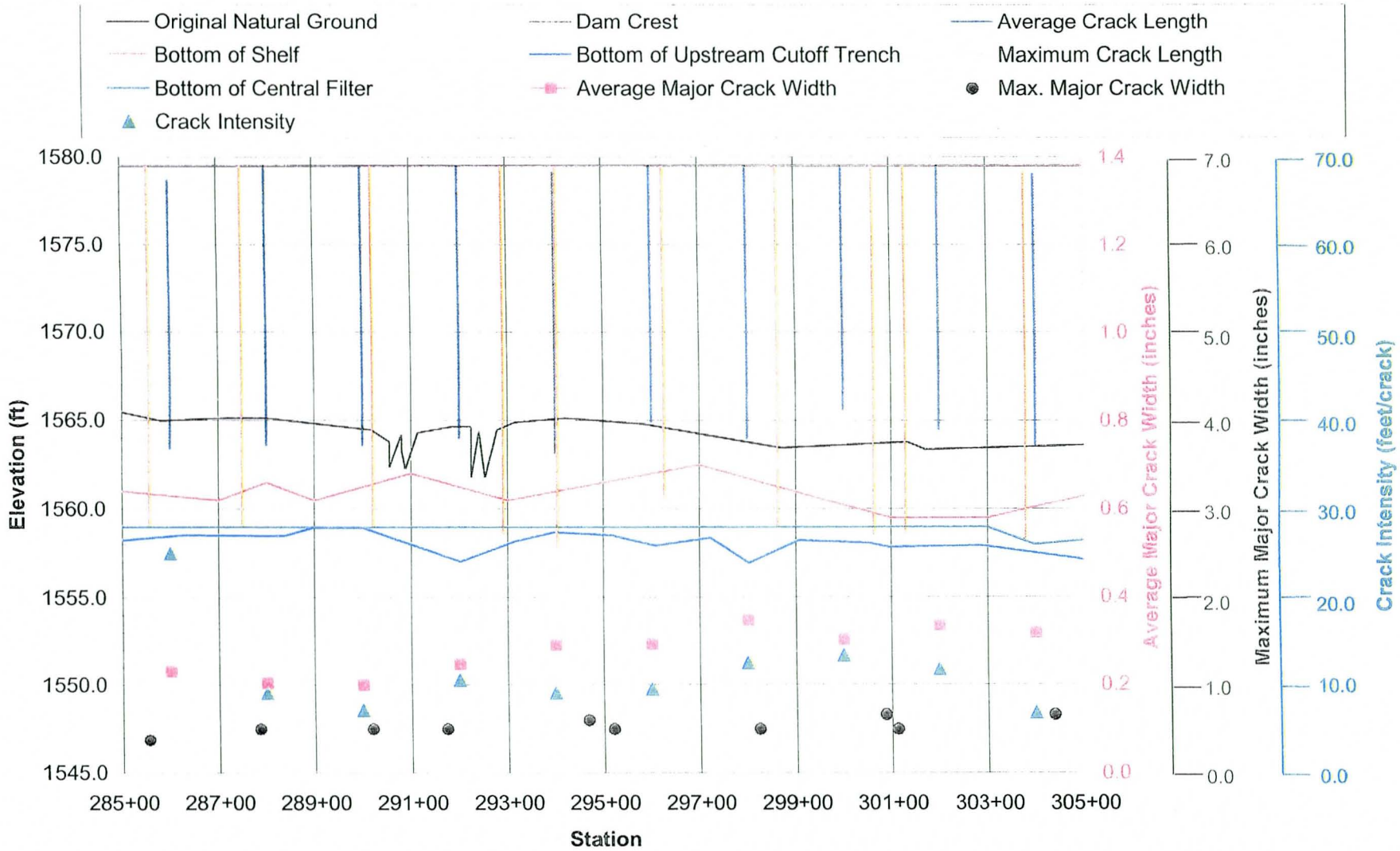


Flood Control District of Maricopa County
Structures Assessment Phase II
Investigation of Cracking
AMEC Job No. 0-117-001122
FCD Contract No. 2000C006
Assignment 2, Vineyard FRS

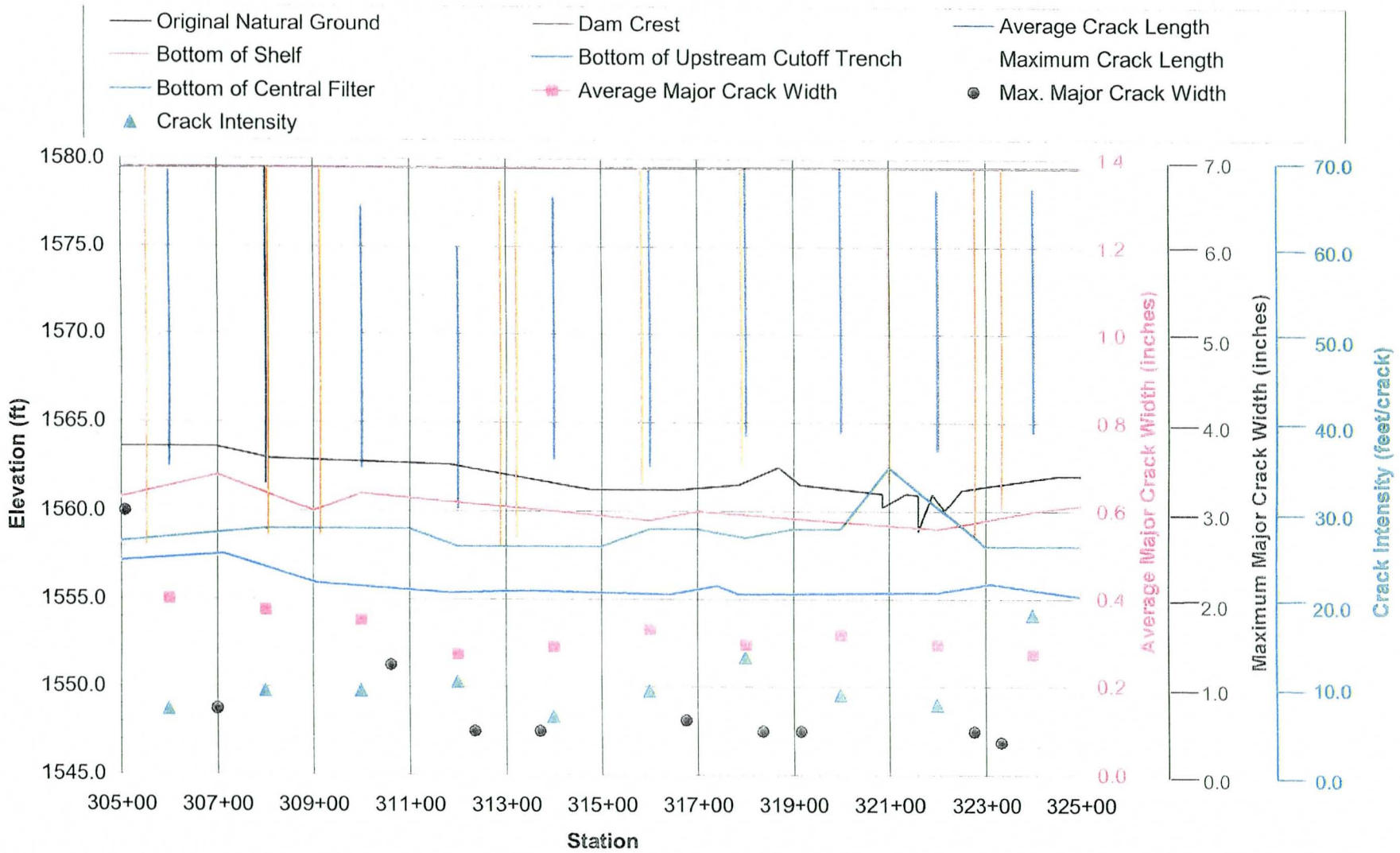
amec

## Vineyard FRS Investigation of Cracking



Flood Control District of Maricopa County
Structures Assessment Phase II
Investigation of Cracking
AMEC Job No. 0-117-001122
FCD Contract No. 2000C006
Assignment 2, Vineyard FRS

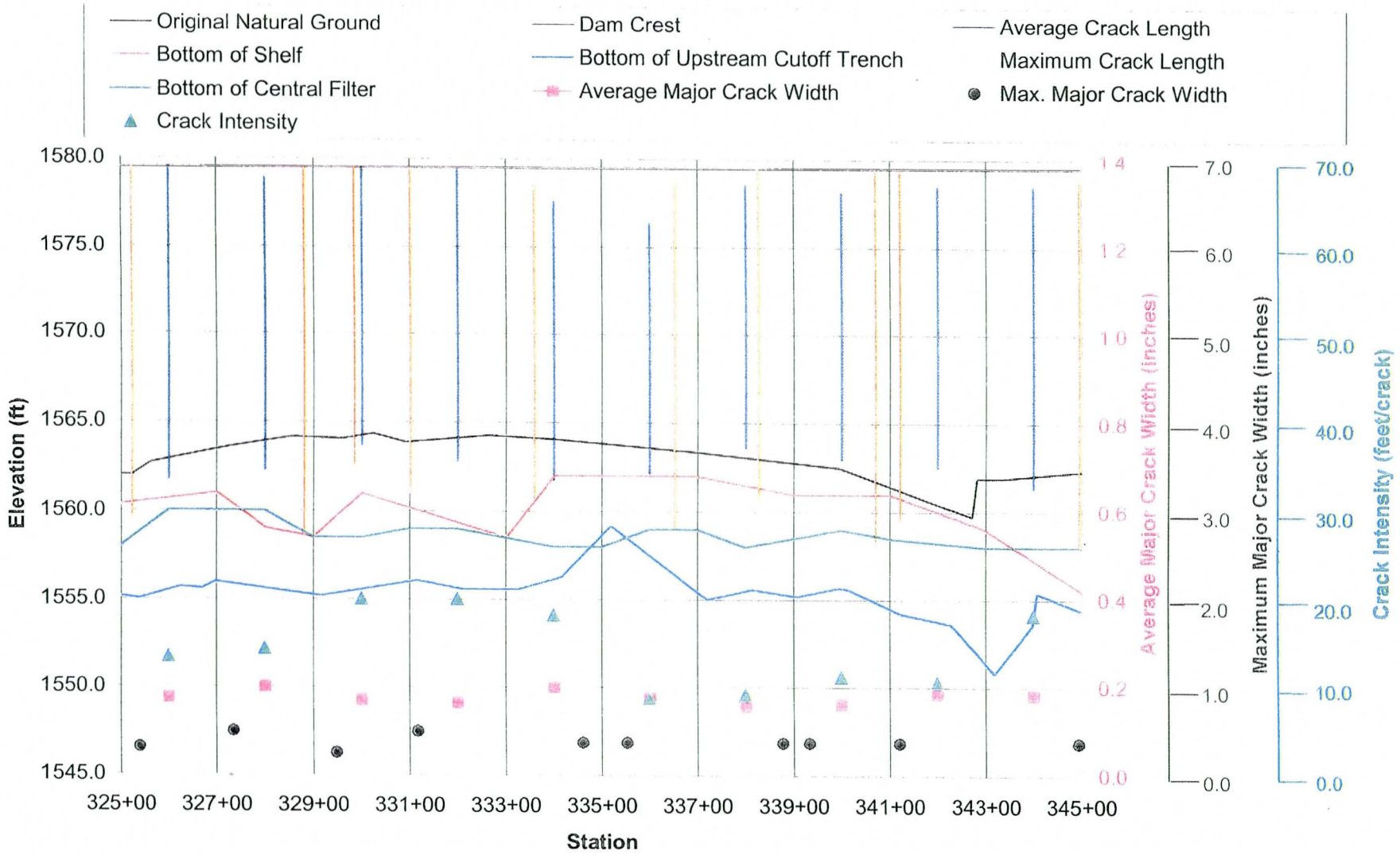
## Vineyard FRS Investigation of Cracking



Flood Control District of Maricopa County
Structures Assessment Phase II
Investigation of Cracking
AMEC Job No. 0-117-001122
FCD Contract No. 2000C006
Assignment 2, Vineyard FRS

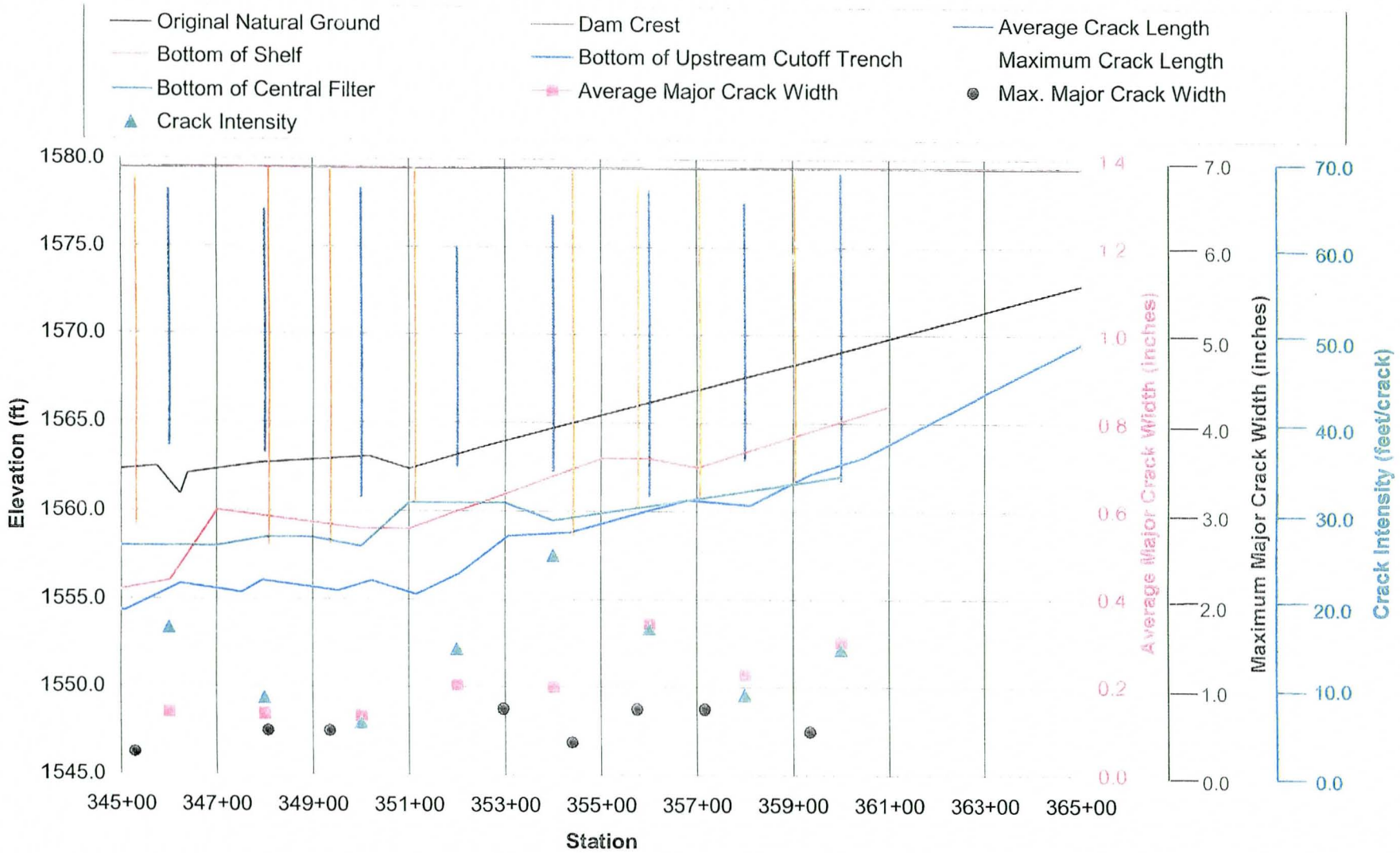
amec

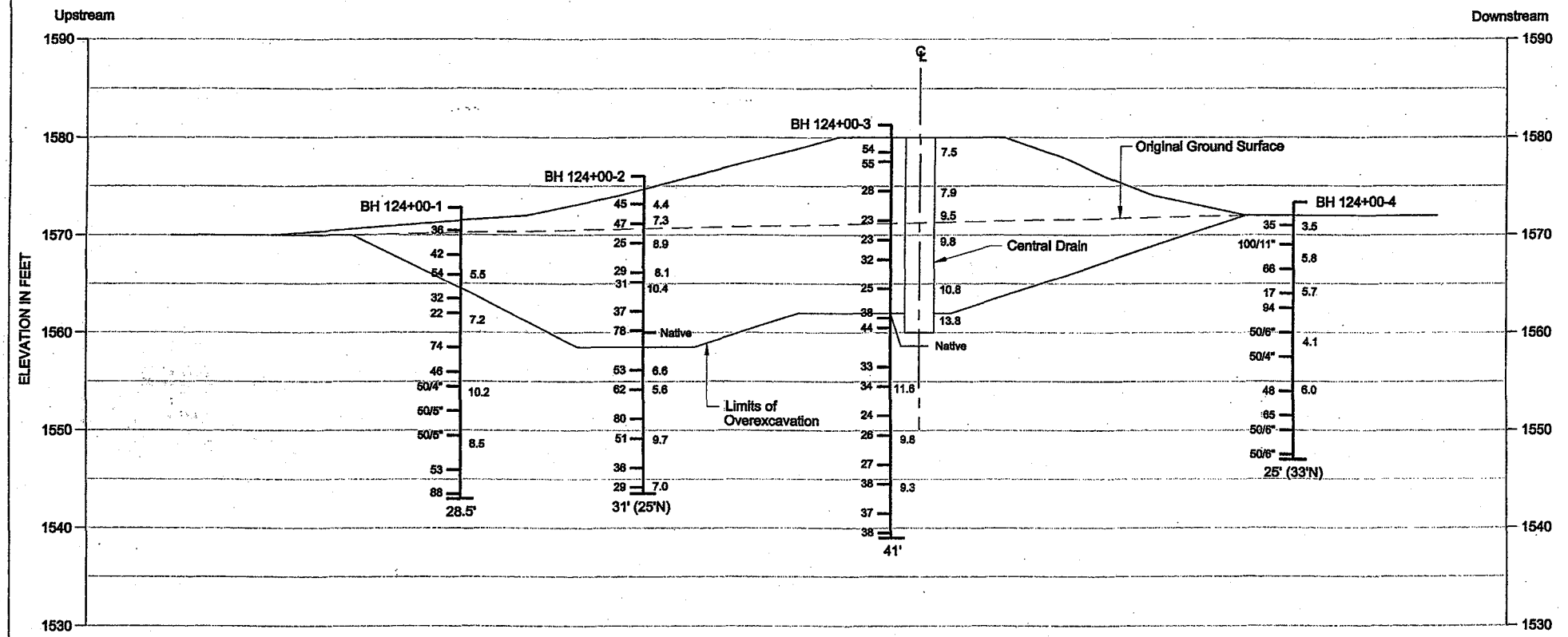
## Vineyard FRS Investigation of Cracking



Flood Control District of Maricopa County
Structures Assessment Phase II
Investigation of Cracking
AMEC Job No. 0-117-001122
FCD Contract No. 2000C006
Assignment 2, Vineyard FRS

## Vineyard FRS Investigation of Cracking






CROSS-SECTION: Station 124+00

SCALE: 1" = 10'

## Notes:

31' (25'N)

Boring Depth, feet

Location Relative to Cross-Section Station

50/6" | 4.1

Blowcount (SPT of Ring Sampler)

Moisture Content

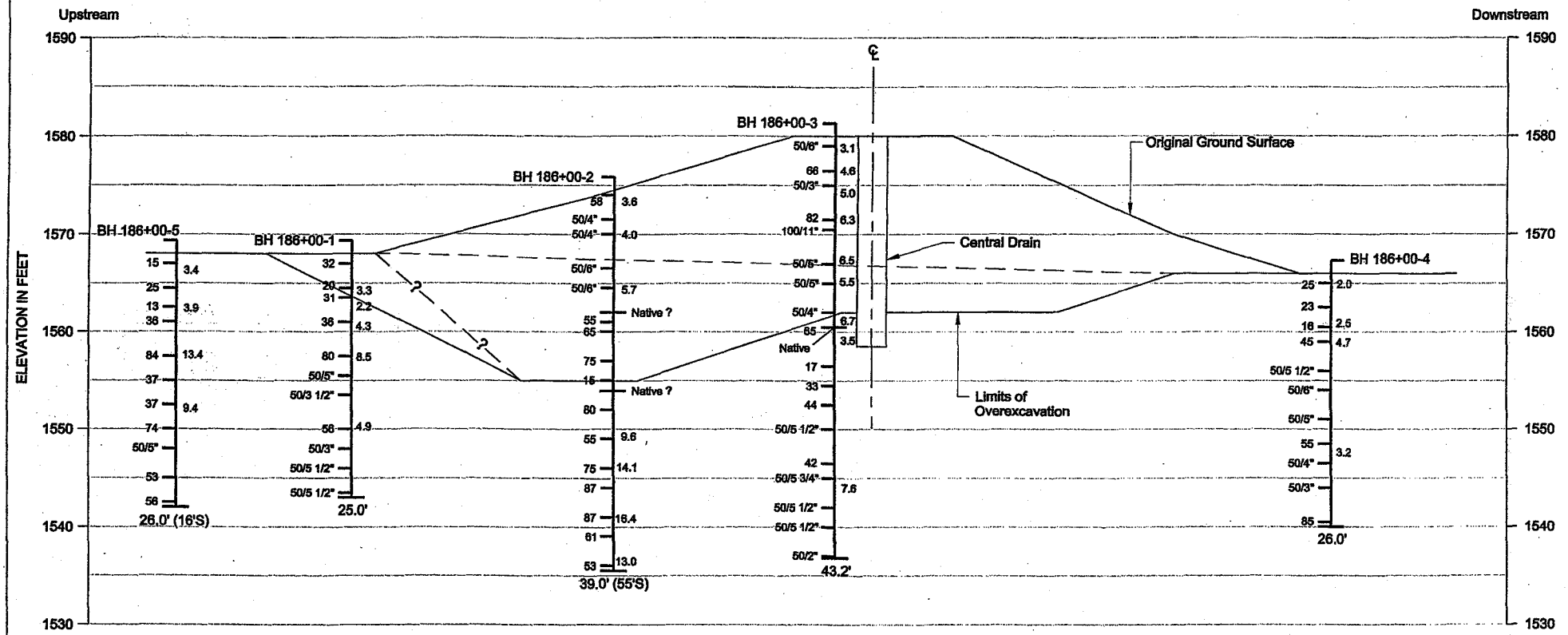
Native indicates delineation between native and embankment soils based on field descriptions.

| | |
|---|---|
| JOB NO. | 0-117-001122 Task 2 |
| DESIGN: | LAH |
| DRAWN: | GWH |
| DATE: | 7/2001 |
| SCALE: | 1"=10' |

VINEYARD FRS
INVESTIGATION OF CRACKING

CROSS-SECTIONS SHOWING
BORING LOCATIONS
SHEET 1 OF 4

amec




## CROSS-SECTION: Station 186+00

SCALE: 1" = 10'

## Notes:

31' (25'N)

Boring Depth, feet

Location Relative to Cross-Section Station

50/6" | 4.1

Blowcount (SPT of Ring Sampler)

Moisture Content

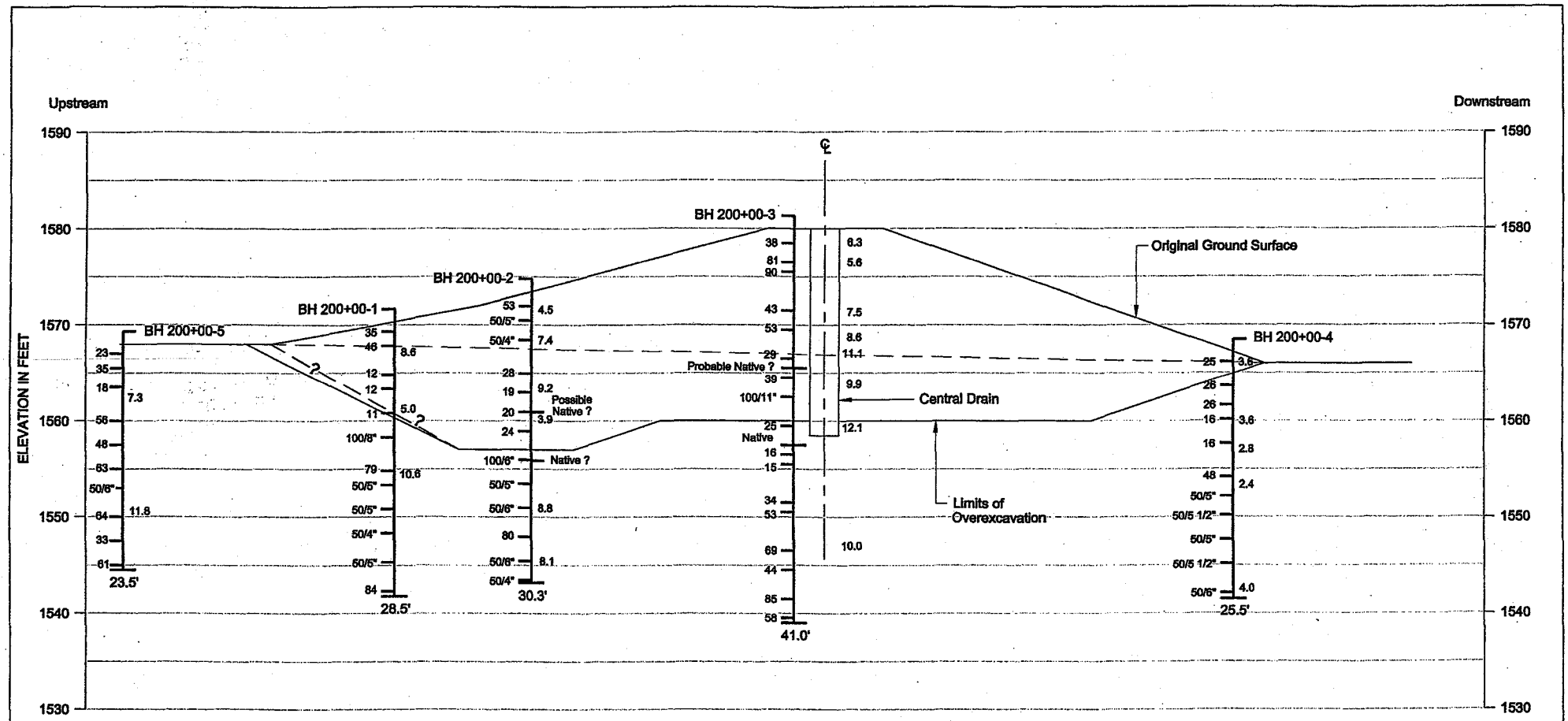
Native indicates delineation between native and embankment soils based on field descriptions.

| | |
|---|---|
| JOB NO. | 0-117-001122 Task 2 |
| DESIGN: | LAH |
| DRAWN: | GWH |
| DATE: | 7/2001 |
| SCALE: | 1"=10' |

VINEYARD FRS
INVESTIGATION OF CRACKING

CROSS-SECTIONS SHOWING
BORING LOCATIONS
SHEET 2 OF 4

amec




## CROSS-SECTION: Station 200+00

SCALE: 1" = 10'

## Notes:

31' (25'N)
Boring Depth, feet
Location Relative to Cross-Section Station

50/6" — 4.1
Blowcount (SPT of Ring Sampler)
Moisture Content

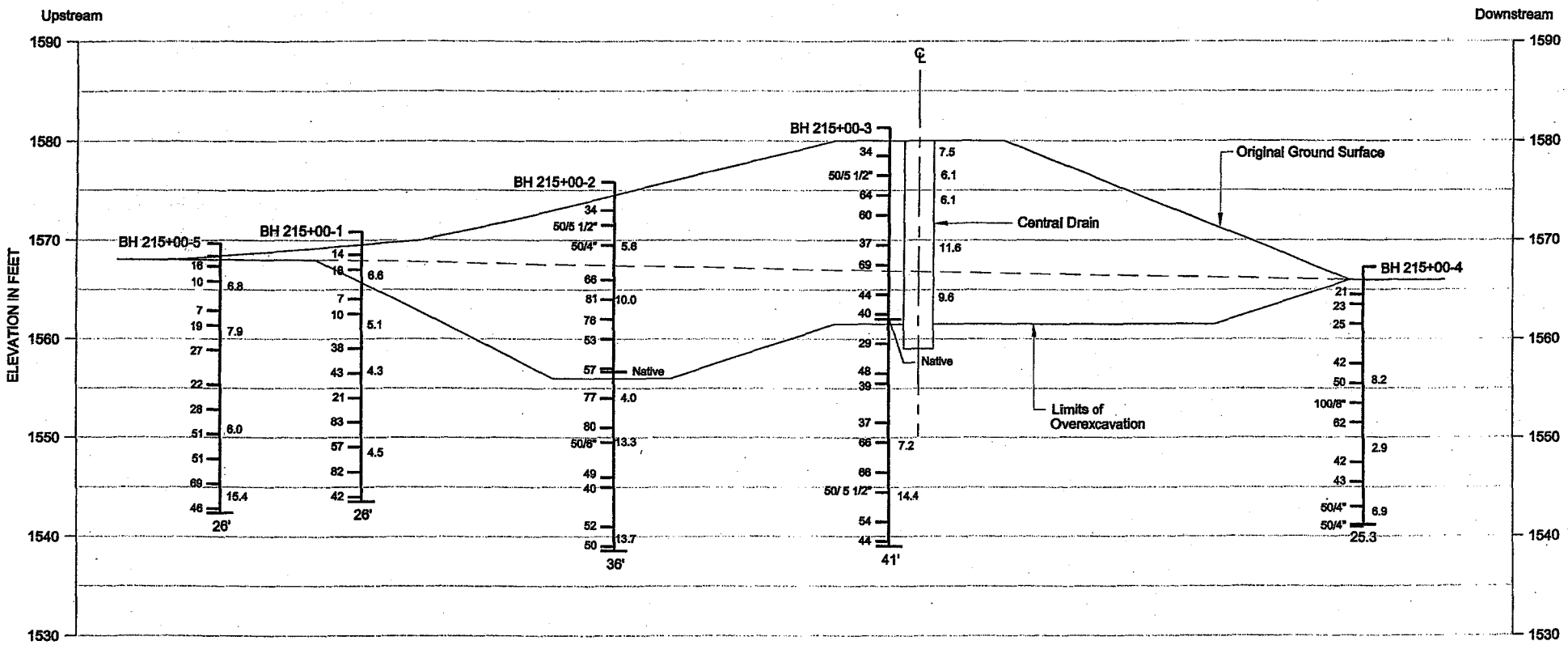
Native indicates delineation between native and embankment soils based on field descriptions.

| | |
|---|---|
| JOB NO. | 0-117-001122 Task 2 |
| DESIGN: | LAH |
| DRAWN: | GWH |
| DATE: | 7/2001 |
| SCALE: | 1"=10' |

VINEYARD FRS
INVESTIGATION OF CRACKING

CROSS-SECTIONS SHOWING BORING LOCATIONS
SHEET 3 OF 4

amec




## CROSS-SECTION: Station 215+00

SCALE: 1" = 10'

## Notes:

31' (25'N)

Boring Depth, feet

Location Relative to Cross-Section Station

50/6" — 4.1

Blowcount (SPT of Ring Sampler)

Moisture Content

Native indicates delineation between native and embankment soils based on field descriptions.

| | |
|---|---|
| JOB NO. | 0-117-001122 Task 2 |
| DESIGN: | LAH |
| DRAWN: | GWH |
| DATE: | 7/2001 |
| SCALE: | 1"=10' |

VINEYARD FRS
INVESTIGATION OF CRACKING

CROSS-SECTIONS SHOWING BORING LOCATIONS
SHEET 4 OF 4

amec

**PROJECT** Vineyard FRS

amec

**JOB NO.** 0-117-001122 Task 2 **DATE** 6/5/01

**LOCATION** Sta. 124+00, 48' R
**RIG TYPE** CME-75
**BORING TYPE** 6 5/8" Hollow Stem Auger
**SURFACE ELEV.**
**DATUM**

| Depth in Feet | Blows Per Foot | Graphical Log | Sample | Sample Type | Blow Count / % Recovery | Dry Density lbs. per Cubic ft. | Moisture Content Percent of Dry Weight | Unified Soil Classification | REMARKS | VISUAL CLASSIFICATION |
|---|---|---|---|---|---|---|---|---|---|---|
| 0 | | | | S | 12-16-20 | | | SC | slightly moist<br>very firm | **CLAYEY SAND**, trace of silt, trace of fine grained, subrounded to subangular gravel, predominantly fine grained, subrounded to subangular, low plasticity<br><br>Stage I cementation (weak HCL reaction, filaments), moderate brown (5YR, 4/4) |
| | | | | S | 32-22-20 | | | | | |
| 5 | | | | S | 24-24-30 | | | | | |
| | | | | | | | | SM | slightly moist<br>very firm to firm | **SILTY SAND**, predominantly fine grained, subrounded to subangular, low plasticity<br><br>Stage I cementation, (filaments, weak HCL reaction), moderate brown (5YR, 5/4)<br><br>note: trace of clay from 6'6" to 9'; light brown (5YR, 5/6) below 9' |
| | | | | S | 14-16-16 | | | | | |
| 10 | | | | U | 22 | | | | | |
| | | | | S | 16-24-50 | | | | | |
| | | | | | | | | SM | slightly moist<br>hard to very firm | **SILTY SAND**, trace to some fine grained, subrounded to subangular gravel, predominantly fine grained, subrounded to subangular, low plasticity<br><br>Stage II cementation (nodules, CaCO3 laminations, moderate HCL reaction), pale brown (5YR, 6/2)<br><br>note: medium to coarse grained sand layer from 14' to 15'<br><br>Stage III cementation below 17'6" (nodules, strong HCL reaction, thin CaCo3 layers)<br><br>Stage II cementation below 24' (filaments, weak to moderate HCL reaction), light brown (5YR, 5/4) |
| 15 | | | | S | 18-20-26 | | | | | |
| | | | | S | 22-50/4" | | | | | |
| 20 | | | | S | 24-50/5" | | | | | |
| | | | | S | 30-50/5" | | | | | |
| 25 | | | | S | 17-19-34 | | | | | |

GROUNDWATER

| DEPTH(ft) | HOUR | DATE |
|---|---|---|
| | none | |
| | | |
| | | |
| | | |

SAMPLE TYPE
A - Drill cuttings; NR - No Recovery
S - 2" O.D. 1.38" I.D. tube sample
U - 3" O.D. 2.42" I.D. tube sample
T - 1" O.D. thin-walled tube sample
D - 2.5" O.D. 1.9" I.D. tube sample
C - California tube sample



LOG OF TEST BORING NO. BH124+00-1

BORING LOG 1171122B.GPJ AGRA_PHX.GDT 6/25/01

**PROJECT** Vineyard FRS

**JOB NO.** 0-117-001122 Task 2 **DATE** 6/5/01

**LOCATION** Sta. 124+00, 48' R
**RIG TYPE** CME-75
**BORING TYPE** 6 5/8" Hollow Stem Auger
**SURFACE ELEV.**
**DATUM**

| Depth in Feet | Blows Per Foot | Graphical Log | Sample | Sample Type | Blow Count / % Recovery | Dry Density lbs. per Cubic ft. | Moisture Content Percent of Dry Weight | Unified Soil Classification | REMARKS | VISUAL CLASSIFICATION |
|---|---|---|---|---|---|---|---|---|---|---|
| 25 | | | | | | | | SM | | **SILTY SAND**, continued |
| | | | | | | | | | | note: light brown (5YR, 5/6) below 27' |
| | | | | S | 22-40-48 | | | | | |
| | | | | | | | | | | Stopped Auger at 27' |
| | | | | | | | | | | Stopped Sampler at 28'6" |
| 30 | | | | | | | | | | |
| 35 | | | | | | | | | | |
| 40 | | | | | | | | | | |
| 45 | | | | | | | | | | |
| 50 | | | | | | | | | | |

GROUNDWATER

| DEPTH(ft) | HOUR | DATE |
|---|---|---|
| | none | |
| | | |
| | | |
| | | |

SAMPLE TYPE
A - Drill cuttings; NR - No Recovery
S - 2" O.D. 1.38" I.D. tube sample
U - 3" O.D. 2.42" I.D. tube sample
T - 1" O.D. thin-walled tube sample
D - 2.5" O.D. 1.9" I.D. tube sample
C - California tube sample



LOG OF TEST BORING NO. BH124+00-1

BORING LOG 1171122B.GPJ AGRA_PHX.GDT 6/25/01

**PROJECT** Vineyard FRS

amec

**JOB NO.** 0-117-001122 Task 2 **DATE** 6/22/01

**LOCATION** Sta. 124+25, 29' R
**RIG TYPE** CME-75
**BORING TYPE** 6 5/8" Hollow Stem Auger
**SURFACE ELEV.**
**DATUM**

| Depth in Feet | Drill Rate Min/ft. | Graphical Log | Sample | Sample Type | Blow Count / % Recovery | Dry Density lbs. per Cubic ft. | Moisture Content Percent of Dry Weight | Unified Soil Classification | REMARKS | VISUAL CLASSIFICATION |
|---|---|---|---|---|---|---|---|---|---|---|
| 0 | | | | | | | | SC | slightly moist | Man-made FILL |
| | | | | S | 27-23-22 | | | | firm to very firm | **CLAYEY SAND**, some well graded gravel from 0 to 1', predominantly medium to coarse grained sand, subangular to subrounded, weak HCL reaction, medium plasticity, light brown (5YR, 6/4) |
| | | | | S | 11-17-30 | | | | | note: predominantly fine to medium grained sand below 2'6" with occasional coarse grained lenses with trace of fine grained gravel |
| 5 | | | | S | 9-12-13 | | | | | note: small root fragment at 10' |
| | | | | | | | | | moist at 7'6" | |
| | | | | S | 9-13-16 | | | | | |
| 10 | | | | U | 31 | | | | | |
| | | | | U | 37 | | | | | |
| 15 | | | | U | 78 | | | CL | slightly moist | NATIVE<br>**SANDY CLAY**, some silt, predominantly fine to medium grained sand, medium plasticity |
| | | | | | | | | | hard | Stage II cementation below 14'6" (considerable soft CaCO3 nodules up to 1" diameter, strong HCL reaction), light brown (5YR, 6/4) |
| | | | | S | 15-23-30 | | | | | note: increase in silt content & trace of gravel at 17'6" |
| 20 | | | | S | 17-30-32 | | | SM | slightly moist | **SILTY SAND**, considerable clay & trace of predominantly fine grained gravel, predominantly medium to coarse grained, subangular to subrounded, low to medium plasticity |
| | | | | | | | | | hard | Stage II cementation (strong HCL reaction, considerable various-sized nodules up to 3/4" in diameter), light brown (5YR, 6/4) |
| | | | | S | 23-40-40 | | | CL | slightly moist | **SANDY CLAY**, predominantly medium to coarse grained sand & some silt, subangular to subrounded, medium plasticity |
| | | | | | 15-23-28 | | | | hard | |
| 25 | | | | S | | | | | | |

GROUNDWATER

| DEPTH(ft) | HOUR | DATE |
|---|---|---|
| | none | |
| | | |
| | | |
| | | |

SAMPLE TYPE
A - Drill cuttings; NR - No Recovery
S - 2" O.D. 1.38" I.D. tube sample
U - 3" O.D. 2.42" I.D. tube sample
T - 1" O.D. thin-walled tube sample
D - 2.5" O.D. 1.9" I.D. tube sample
C - 2.5" O.D. 2.0" I.D. tube sample



LOG OF TEST BORING NO. BH124+00-2

BORING LOG 1171122B.GPJ AGRA_PHX.GDT 6/29/01

**PROJECT** Vineyard FRS

**JOB NO.** 0-117-001122 Task 2 **DATE** 6/22/01

**LOCATION** Sta. 124+25, 29' R
**RIG TYPE** CME-75
**BORING TYPE** 6 5/8" Hollow Stem Auger
**SURFACE ELEV.**
**DATUM**

| Depth in Feet | Drill Rate Min/ft. | Graphical Log | Sample | Sample Type | Blow Count / % Recovery | Dry Density lbs. per Cubic ft. | Moisture Content Percent of Dry Weight | Unified Soil Classification | REMARKS | VISUAL CLASSIFICATION |
|---|---|---|---|---|---|---|---|---|---|---|
| 25 | | | | | | | | CL | slightly moist<br>very firm to hard | **SANDY CLAY**, continued, considerable silt & predominantly medium grained sand<br>Stage III cementation (considerable small soft CaCO3 nodules starting to coalesce with considerable filaments, strong HCL reaction), light brown (5YR, 6/4)<br>note: silty clay below 27'6" with trace of sand |
| | | | | S | 15-16-20 | | | | | |
| 30 | | | | S | 9-12-17 | | | SC | slightly moist<br>firm | **CLAYEY SAND**, considerable silt & trace of gravel, predominantly medium to coarse grained sand, subangular to subrounded, medium plasticity<br>Stage II cementation (some small, soft CaCO3 nodules, trace of filaments, strong HCL reaction), light brown (5YR, 6/4)<br>Stopped Auger at 29'6"<br>Stopped sampler at 31' |
| 35 | | | | | | | | | | |
| 40 | | | | | | | | | | |
| 45 | | | | | | | | | | |
| 50 | | | | | | | | | | |

GROUNDWATER

| DEPTH(ft) | HOUR | DATE |
|---|---|---|
| | none | |
| | | |
| | | |
| | | |

SAMPLE TYPE
A - Drill cuttings; NR - No Recovery
S - 2" O.D. 1.38" I.D. tube sample
U - 3" O.D. 2.42" I.D. tube sample
T - 1" O.D. thin-walled tube sample
D - 2.5" O.D. 1.9" I.D. tube sample
C - 2.5" O.D. 2.0" I.D. tube sample



LOG OF TEST BORING NO. BH124+00-2

BORING LOG 1171122B.GPJ AGRA_PHX.GDT 6/29/01

**PROJECT** Vineyard FRS

amec

**JOB NO.** 0-117-001122 Task 2 **DATE** 6/15/01

**LOCATION** Sta. 124+00, 3' R
**RIG TYPE** CME-75
**BORING TYPE** 6 5/8" Hollow Stem Auger
**SURFACE ELEV.**
**DATUM**

| Depth in Feet | Drill Rate Min/ft. | Graphical Log | Sample | Sample Type | Blow Count / % Recovery | Dry Density lbs. per Cubic ft. | Moisture Content Percent of Dry Weight | Unified Soil Classification | REMARKS | VISUAL CLASSIFICATION |
|---|---|---|---|---|---|---|---|---|---|---|
| 0 | | | | | | | | SC with CL zones | slightly moist<br>firm to hard | Man-made FILL<br>**CLAYEY SAND with SANDY CLAY zones**, considerable well graded gravel on the ground surface, predominantly fine to medium grained sand, subangular to subrounded, medium plasticity, light brown (5YR, 6/4)<br>note: light brown (5YR, 5/6) below 7'6"<br>note: trace of fine grained gravel below 9'6"<br>note: sandy clay zones up to 6" to 12" thick |
| | | | | S | 23-32-22 | | | | | |
| | | | | U | 55 | | | | | |
| 5 | | | | S | 14-14-14 | | | | | |
| | | | | S | 11-12-11 | | | | | |
| 10 | | | | S | 9-11-12 | | | | | |
| | | | | U | 32 | | | | | |
| 15 | | | | S | 10-12-13 | | | | | |
| | | | | S | 14-19-19 | | | SC | moist to slightly moist<br>firm to hard | NATIVE<br>**CLAYEY SAND**, considerable clay in lenses, predominantly fine grained sand with medium grained zones, medium plasticity<br>Stage II cementation (considerable CaCO3 filaments, strong HCL reaction in zones), moderate brown (5YR, 4/4) below 20'<br>note: some silt below 22'6" |
| 20 | | | | U | 44 | | | | | |
| | | | | S | 15-16-17 | | | | | |
| | | | | S | 12-16-18 | | | | | |
| 25 | | | | | | | | | | |

GROUNDWATER

| | DEPTH(ft) | HOUR | DATE |
|---|---|---|---|
| ▽ | | none | |
| ▼ | | | |
| ▽ | | | |
| ▼ | | | |

SAMPLE TYPE
A - Drill cuttings; NR - No Recovery
S - 2" O.D. 1.38" I.D. tube sample
U - 3" O.D. 2.42" I.D. tube sample
T - 1" O.D. thin-walled tube sample
D - 2.5" O.D. 1.9" I.D. tube sample
C - 2.5" O.D. 2.0" I.D. tube sample



LOG OF TEST BORING NO. BH124+00-3

BORING LOG 1171122B.GPJ AGRA_PHX.GDT 6/29/01

**PROJECT** Vineyard FRS

**JOB NO.** 0-117-001122 Task 2 **DATE** 6/15/01

**LOCATION** Sta. 124+00, 3' R
**RIG TYPE** CME-75
**BORING TYPE** 6 5/8" Hollow Stem Auger
**SURFACE ELEV.**
**DATUM**

| Depth in Feet | Drill Rate Min/ft. | Graphical Log | Sample | Sample Type | Blow Count / % Recovery | Dry Density lbs. per Cubic ft. | Moisture Content Percent of Dry Weight | Unified Soil Classification | REMARKS | VISUAL CLASSIFICATION |
|---|---|---|---|---|---|---|---|---|---|---|
| 25 | | | X | | | | | CL | slightly moist<br>firm to very firm | **SILTY CLAY**, considerable sand in zones, medium plasticity<br>Stage II cementation (considerable CaCO3 nodules), moderate brown (5YR, 4/4) |
| | | | X | S | 6-9-15 | | | | | |
| | | | | | | | | SM | slightly moist<br>firm | **SILTY SAND**, predominantly fine to medium grained, subangular to subrounded, nonplastic, light brown (5YR, 6/4)<br>note: trace of gravel in zones with medium to coarse grained sand lenses below 29'6" |
| 30 | | | X | S | 7-13-13 | | | | | |
| | | | X | | | | | CL | slightly moist<br>firm to very firm | **SILTY CLAY**, trace of sand in lenses, medium plasticity<br>Stage III cementation (considerable CaCO3, nodules, strong HCL reaction), light brown (5YR, 6/4) |
| | | | X | S | 10-14-13 | | | | | |
| 35 | | | X | S | 12-18-20 | | | | | |
| | | | X | S | 12-18-19 | | | | | |
| 40 | | | X | S | 12-18-20 | | | | | |
| 45 | | | | | | | | | | Stopped Auger at 39'6"<br>Stopped Sampler at 41' |
| 50 | | | | | | | | | | |

GROUNDWATER

| | DEPTH(ft) | HOUR | DATE |
|---|---|---|---|
| ▽ | | none | |
| ▼ | | | |
| ▽ | | | |
| ▼ | | | |

SAMPLE TYPE
A - Drill cuttings; NR - No Recovery
S - 2" O.D. 1.38" I.D. tube sample
U - 3" O.D. 2.42" I.D. tube sample
T - 1" O.D. thin-walled tube sample
D - 2.5" O.D. 1.9" I.D. tube sample
C - 2.5" O.D. 2.0" I.D. tube sample



LOG OF TEST BORING NO. BH124+00-3

BORING LOG 1171122B.GPJ AGRA_PHX.GDT 6/29/01

**PROJECT** Vineyard FRS

amec

**JOB NO.** 0-117-001122 Task 2 **DATE** 6/6/01

**LOCATION** Sta. 124+33, 40' L
**RIG TYPE** CME-75
**BORING TYPE** 6 5/8" Hollow Stem Auger
**SURFACE ELEV.**
**DATUM**

| Depth in Feet | Blows Per Foot | Graphical Log | Sample | Sample Type | Blow Count / % Recovery | Dry Density lbs. per Cubic ft. | Moisture Content Percent of Dry Weight | Unified Soil Classification | REMARKS | VISUAL CLASSIFICATION |
|---|---|---|---|---|---|---|---|---|---|---|
| 0 | | | | S | 17-16-19 | | | SC | slightly moist<br>very firm | **CLAYEY SAND**, predominantly fine grained, subrounded to subangular, low to medium plasticity<br>Stage I cementation (weak HCL reaction, filaments), moderate brown (5YR, 4/4)<br>note: uncemented above 0.5' & small diameter roots present |
| | | | | U | 100/11" | | | SM | | |
| | | | | | | | | SC | slightly moist<br>hard | **SILTY SAND**, trace of clay, predominantly fine grained, subrounded to subangular, low plasticity<br>Stage I cementation (weak HCL reaction), light brown (5YR, 5/6) |
| 5 | | | | S | 15-28-38 | | | | | |
| | | | | | | | | SM | slightly moist<br>hard | **CLAYEY SAND**, predominantly fine grained, subrounded to subangular, low to medium plasticity<br>Stage I cementation (filaments, weak HCL reaction), moderate brown (5YR, 4/4) |
| | | | | S | 7-8-9 | | | | | |
| | | | | | | | | SM | slightly moist<br>firm | **SILTY SAND**, trace of fine grained, subrounded to subangular gravel, predominantly fine grained, subrounded to subangular, low plasticity<br>Stage I cemented (filaments, moderate HCL reaction), light brown (5YR, 5/6) |
| 10 | | | | U | 94 | | | | | |
| | | | | | | | | SP | slightly moist<br>hard | **SILTY SAND**, predominantly fine grained, subrounded to subangular, low plasticity<br>Stage II cementation (1/4" nodules & grain coating), light brown (5YR, 5/4) |
| | | | | S | 15-50/6" | | | | | |
| | | | | | | | | SM | slightly moist<br>dense | **SAND**, trace of fine grained, subrounded to subangular gravel, trace of silt, predominantly medium to coarse grained, subrounded to subangular, nonplastic<br>Stage I cementation (weak HCL reaction), light brown (5YR, 5/4) |
| 15 | | | | S | 25-50/4" | | | | | |
| | | | | S | 13-23-25 | | | | slightly moist<br>very firm to hard | **SILTY SAND**, trace of fine grained, subangular to subrounded gravel, predominantly fine grained, subrounded to subangular, low plasticity<br>Stage II cemenation (nodules, filaments), light brown (5YR, 5/4) |
| 20 | | | | S | 29-35-30 | | | SP | slightly moist<br>very dense | **SAND**, trace to some predominantly fine grained, subrounded to subangular gravel, trace of silt, well graded, subrounded to subangular, nonplastic<br>Stage I cementation, light brown (5YR, 6/4) |
| | | | | | | | | SM | | |
| | | | | S | 25-50/6" | | | | slightly moist<br>hard | **SILTY SAND**, trace of fine grained, subrounded to subangular gravel, predominantly fine grained, subrounded to subangular, low plasticity<br>Stage II cementation (nodules, moderate HCL reaction), light brown (5YR, 5/4)<br>Stopped Auger at 24'6", Stopped Sampler at 25' |
| 25 | | | | S | 50/6" | | | | | |

GROUNDWATER

| | DEPTH(ft) | HOUR | DATE |
|---|---|---|---|
| ∇ | | none | |
| ▼ | | | |
| ∇ | | | |
| ▼ | | | |

SAMPLE TYPE
A - Drill cuttings; NR - No Recovery
S - 2" O.D. 1.38" I.D. tube sample
U - 3" O.D. 2.42" I.D. tube sample
T - 1" O.D. thin-walled tube sample
D - 2.5" O.D. 1.9" I.D. tube sample
C - California tube sample



# LOG OF TEST BORING NO. BH124+00-4

BORING LOG 1171122B.GPJ AGRA_PHX.GDT 6/25/01

**PROJECT** Vineyard FRS

amec

**JOB NO.** 0-117-001122 Task 2 **DATE** 6/5/01

**LOCATION** Sta. 139+00, 63' R
**RIG TYPE** CME-75
**BORING TYPE** 6 5/8" Hollow Stem Auger
**SURFACE ELEV.**
**DATUM**

| Depth in Feet | Drill Rate Min/ft. | Graphical Log | Sample | Sample Type | Blow Count / % Recovery | Dry Density lbs. per Cubic ft. | Moisture Content Percent of Dry Weight | Unified Soil Classification | REMARKS | VISUAL CLASSIFICATION |
|---|---|---|---|---|---|---|---|---|---|---|
| 0 | | | | S | 8-8-11 | | | SC | slightly moist<br>firm | **CLAYEY SAND**, predominantly medium to fine grained, subrounded to subangular, uncemented, low to medium plasticity, moderate brown (5YR, 4/4)<br>note: plant roots up to 0.5" |
| | | | | | | | | SM | slightly moist<br>moderately firm | **SILTY SAND**, predominantly fine grained, subrounded to subangular, low plasticity<br>Stage I cementation (filaments), light brown (5YR, 5/4)<br>note: sand & gravel layer from 10'6" to 11' |
| | | | | S | 5-6-7 | | | | | |
| 5 | | | | U | 14 | | | | | |
| | | | | U | 12 | | | | | |
| 10 | | | | S | 5-4-12 | | | | | |
| | | | | S | 9-9-9 | | | | | |
| | | | | | | | | SP | slightly moist<br>medium dense | **SAND**, trace to some subrounded to subangular gravel, trace of silt, predominantly medium to coarse grained, subrounded to subangular, uncemented, nonplastic, light brown (5YR, 5/6) |
| 15 | | | | S | 14-28-44 | | | SM | slightly moist<br>hard to very firm | **SILTY SAND**, trace of fine grained, subangular to subrounded gravel, predominantly fine grained, subangular to subrounded, low plasticity to nonplastic<br>Stage III cementation (large nodules, CaCO3 seams & layers), light brown (5YR, 5/6)<br>note: light brown (5YR, 6/4) below 17'6"<br>Stage II cementation (moderate HCL reaction), grayish orange pink (5YR, 7/2) below 23' |
| | | | | S | 48-50/4" | | | | | |
| 20 | | | | S | 50/5 1/2" | | | | | |
| | | | | S | 34-50/5 1/2" | | | | | |
| 25 | | | | S | 23-26-23 | | | | | |

GROUNDWATER

| | DEPTH(ft) | HOUR | DATE |
|---|---|---|---|
| ▽ | | none | |
| ▼ | | | |
| ▽ | | | |
| ▼ | | | |

SAMPLE TYPE
A - Drill cuttings; NR - No Recovery
S - 2" O.D. 1.38" I.D. tube sample
U - 3" O.D. 2.42" I.D. tube sample
T - 1" O.D. thin-walled tube sample
D - 2.5" O.D. 1.9" I.D. tube sample
C - 2.5" O.D. 2.0" I.D. tube sample



LOG OF TEST BORING NO. BH139+00-1

BORING LOG 1171122B.GPJ AGRA_PHX.GDT 6/29/01

**PROJECT** Vineyard FRS

amec

**JOB NO.** 0-117-001122 Task 2 **DATE** 6/5/01

**LOCATION** Sta. 139+00, 63' R
**RIG TYPE** CME-75
**BORING TYPE** 6 5/8" Hollow Stem Auger
**SURFACE ELEV.**
**DATUM**

| Depth in Feet | Drill Rate Min/ft. | Graphical Log | Sample | Sample Type | Blow Count / % Recovery | Dry Density lbs. per Cubic ft. | Moisture Content Percent of Dry Weight | Unified Soil Classification | REMARKS | VISUAL CLASSIFICATION |
|---|---|---|---|---|---|---|---|---|---|---|
| 25 | | | X | | | | | SP | slightly moist<br>medium dense | **SAND**, trace of silt, some predominantly fine grained, subangular to subrounded gravel, well graded, subrounded to subangular, nonplastic<br>note: predominantly medium to coarse grained sand below 26'<br>Stage I cementation (weak HCL reaction), light brown (5YR, 5/6) below 26' |
| | | | X | S | 14-10-18 | | | | | |
| | | | X | S | 26-45-50/3 1/2" | | | | | |
| 30 | | | | | | | | SM | slightly moist<br>hard | **SILTY SAND**, predominantly medium to fine grained, subrounded to subangular, low plasticity<br>Stage III cementation (strong HCL reaction, CaCO3 coated gravel), pale brown (5YR, 6/2)<br>Stopped Auger at 29'6"<br>Sampler refused at 31' |
| 35 | | | | | | | | | | |
| 40 | | | | | | | | | | |
| 45 | | | | | | | | | | |
| 50 | | | | | | | | | | |

GROUNDWATER

| | DEPTH(ft) | HOUR | DATE |
|---|---|---|---|
| ▽ | | none | |
| ▼ | | | |
| ▽ | | | |
| ▼ | | | |

SAMPLE TYPE
A - Drill cuttings; NR - No Recovery
S - 2" O.D. 1.38" I.D. tube sample
U - 3" O.D. 2.42" I.D. tube sample
T - 1" O.D. thin-walled tube sample
D - 2.5" O.D. 1.9" I.D. tube sample
C - 2.5" O.D. 2.0" I.D. tube sample



LOG OF TEST BORING NO. BH139+00-1

BORING LOG 1171122B.GPJ AGRA_PHX.GDT 6/29/01

**PROJECT** Vineyard FRS

amec

**JOB NO.** 0-117-001122 Task 2 **DATE** 6/5/01

**LOCATION** Sta. 170+00, 45' R
**RIG TYPE** CME-75
**BORING TYPE** 6 5/8" Hollow Stem Auger
**SURFACE ELEV.**
**DATUM**

| Depth in Feet | Drill Rate Min/ft. | Graphical Log | Sample | Sample Type | Blow Count / % Recovery | Dry Density lbs. per Cubic ft. | Moisture Content Percent of Dry Weight | Unified Soil Classification | REMARKS | VISUAL CLASSIFICATION |
|---|---|---|---|---|---|---|---|---|---|---|
| 0 | | | | S | 18-16-14 | | | SC | slightly moist<br>firm | **CLAYEY SAND**, predominantly fine grained, subrounded to subangular, uncemented, low plasticity, moderate yellowish brown (10YR, 5/4)<br>note: predominantly well graded sand from 0 to 0.5' |
| | | | | | | | | SM | slightly moist<br>firm to moderately firm | **SILTY SAND**, trace of fine grained, subrounded to subangular gravel, predominantly fine grained, subangular to subrounded, low plasticity<br>Stage I cementation (weak HCL reaction), low plasticity, light brown (5YR, 6/4)<br>note: predominantly medium to coarse grained sand from 7'6" to 8'6" |
| | | | | S | 13-15-13 | | | | | |
| 5 | | | | S | 8-10-11 | | | | | |
| | | | | U | 21 | | | | | |
| 10 | | | | U | 21 | | | | | |
| | | | | | | | | SM | slightly moist<br>firm to hard | **SILTY SAND**, trace of fine grained, subrounded to subangular gravel, predominantly fine grained, subangular to subrounded, low plasticity<br>Stage II cementation (moderate HCL reaction, small nodules), pale yellowish brown (10YR, 6/2)<br>note: pink grayish orange (5YR, 7/2), predominantly well graded sand at 15'<br>Stage III cemention (strong HCL reaction, CaCO3 seams) below 15'<br>note: predominantly medium to fine grained sand from 19' to 22' |
| | | | | S | 12-13-11 | | | | | |
| 15 | | | | S | 7-50/5 1/2" | | | | | |
| | | | | S | 50/4" | | | | | |
| 20 | | | | S | 28-44-50/4 1/2" | | | | | |
| | | | | S | 28-50/5 1/2" | | | | | |
| | | | | S | 27-50/4 1/2" | | | | | |
| 25 | | | | | | | | | | |

GROUNDWATER

| DEPTH(ft) | HOUR | DATE |
|---|---|---|
| | none | |
| | | |
| | | |
| | | |

SAMPLE TYPE
A - Drill cuttings; NR - No Recovery
S - 2" O.D. 1.38" I.D. tube sample
U - 3" O.D. 2.42" I.D. tube sample
T - 1" O.D. thin-walled tube sample
D - 2.5" O.D. 1.9" I.D. tube sample
C - 2.5" O.D. 2.0" I.D. tube sample



LOG OF TEST BORING NO. BH170+00-1

BORING LOG 1171122B.GPJ AGRA_PHX.GDT 6/29/01

**PROJECT** Vineyard FRS

**JOB NO.** 0-117-001122 Task 2 **DATE** 6/5/01

**LOCATION** Sta. 170+00, 45' R
**RIG TYPE** CME-75
**BORING TYPE** 6 5/8" Hollow Stem Auger
**SURFACE ELEV.**
**DATUM**

| Depth in Feet | Drill Rate Min/ft. | Graphical Log | Sample | Sample Type | Blow Count / % Recovery | Dry Density lbs. per Cubic ft. | Moisture Content Percent of Dry Weight | Unified Soil Classification | REMARKS | VISUAL CLASSIFICATION |
|---|---|---|---|---|---|---|---|---|---|---|
| 25 | | | ☒ | | | | | SM | | **SILTY SAND**, continued |
| | | | | | | | | | | Stopped Auger at 24'6" |
| | | | | | | | | | | Sampler refused at 25'6" |
| 30 | | | | | | | | | | |
| 35 | | | | | | | | | | |
| 40 | | | | | | | | | | |
| 45 | | | | | | | | | | |
| 50 | | | | | | | | | | |

GROUNDWATER

| | DEPTH(ft) | HOUR | DATE |
|---|---|---|---|
| ▽ | | none | |
| ▼ | | | |
| ▽ | | | |
| ▼ | | | |

SAMPLE TYPE
A - Drill cuttings; NR - No Recovery
S - 2" O.D. 1.38" I.D. tube sample
U - 3" O.D. 2.42" I.D. tube sample
T - 1" O.D. thin-walled tube sample
D - 2.5" O.D. 1.9" I.D. tube sample
C - 2.5" O.D. 2.0" I.D. tube sample



LOG OF TEST BORING NO. BH170+00-1

BORING LOG 1171122B.GPJ AGRA_PHX.GDT 6/29/01

**PROJECT** Vineyard FRS

**JOB NO.** 0-117-001122 Task 2 **DATE** 6/5/01

**LOCATION** Sta. 186+00, 55' R
**RIG TYPE** CME-75
**BORING TYPE** 6 5/8" Hollow Stem Auger
**SURFACE ELEV.**
**DATUM**

| Depth in Feet | Drill Rate Min/ft. | Graphical Log | Sample | Sample Type | Blow Count / % Recovery | Dry Density lbs. per Cubic ft. | Moisture Content Percent of Dry Weight | Unified Soil Classification | REMARKS | VISUAL CLASSIFICATION |
|---|---|---|---|---|---|---|---|---|---|---|
| 0 | | | | S | 28-16-16 | | | SM | slightly moist<br>very firm | **SILTY SAND**, trace of fine grained, subrounded to subangular gravel, predominantly fine grained, subrounded to subangular, uncemented, low plasticity, moderate brown (5YR, 4/4)<br><br>note: small diameter roots in upper 1' |
| | | | | | | | | SC | slightly moist<br>firm | **CLAYEY SAND**, trace of predominantlyu fine grained, subrounded to subangular gravel, predominantly fine grained, subrounded to subangular, low to medium plasticity<br><br>Stage I cementation (weak HCL reation, filaments) moderate yelowish brown (10YR, 5/4) |
| | | | | S | 4-6-14 | | | | | |
| | | | | | | | | SP | slightly moist<br>medium dense | **GRAVELLY SAND**, trace of clay, predominantly fine grained, subrounded to subangular gravel, predominantly medium to coarse grained, subrounded to subangular, nonplastic<br><br>Stage I cementation (weak HCL reaction, filaments), moderate brown (10YR, 4/6)<br><br>note: thin (0.01') lenses of medium to coarse grained sand |
| 5 | | | | U | 31 | | | | | |
| | | | | U | 36 | | | | | |
| | | | | | | | | SC | slightly moist<br>firm | **CLAYEY SAND**, trace of silt, predominantly medium to fine grained, subrounded to subangular, low plasticity<br><br>Stage I cementation (weak HCL reaction, filaments), moderate brown (5YR, 4/4)<br><br>note: thin (0.01') lenses of medium to coarse grained sand |
| | | | | S | 18-30-50 | | | | | |
| 10 | | | | | | | | SC | slightly moist<br>hard | **CLAYEY SAND**, trace of silt, predominantly fine grained, subrounded to subangular, low plasticity<br><br>Stage II cementation (nodules, grain coatings), moderate brown (5YR, 4/4) to light brown (5YR, 5/6) |
| | | | | | | | | SM | slightly moist<br>hard | **SILTY SAND**, trace of clay, trace to some fine grained, subrounded to subangular gravel, predominantly fine grained, subrounded to subangular, low plasticity<br><br>Stage III cementation (thin CaCO3 layers, strong HCL reaction), grayish orange pink (5YR, 7/2)<br><br>note: medium to coarse grained sand from 16'6" to 18' exhibiting Stage II characteristics, moderate brown (5YR, 4/4)<br><br>Stopped Auger at 24'6"<br>Stopped Sampler at 25' |
| | | | | S | 29-50/5" | | | | | |
| | | | | S | 35-50/3 1/2" | | | | | |
| 15 | | | | | | | | | | |
| | | | | S | 29-26-30 | | | | | |
| | | | | S | 42-50/3" | | | | | |
| 20 | | | | | | | | | | |
| | | | | S | 50/5 1/2" | | | | | |
| | | | | S | 50/5 1/2" | | | | | |
| 25 | | | | | | | | | | |

GROUNDWATER

| DEPTH(ft) | HOUR | DATE |
|---|---|---|
| | none | |
| | | |
| | | |
| | | |

SAMPLE TYPE
A - Drill cuttings; NR - No Recovery
S - 2" O.D. 1.38" I.D. tube sample
U - 3" O.D. 2.42" I.D. tube sample
T - 1" O.D. thin-walled tube sample
D - 2.5" O.D. 1.9" I.D. tube sample
C - 2.5" O.D. 2.0" I.D. tube sample



LOG OF TEST BORING NO. BH186+00-1

BORING LOG 1171122B.GPJ AGRA_PHX.GDT 6/29/01

**PROJECT** Vineyard FRS

amec

**JOB NO.** 0-117-001122 Task 2 **DATE** 6-21-01 & 6-22-01

**LOCATION** Sta. 185+45, 27' R
**RIG TYPE** CME-75
**BORING TYPE** 6 5/8" Hollow Stem Auger
**SURFACE ELEV.**
**DATUM**

| Depth in Feet | Drill Rate Min/ft. | Graphical Log | Sample | Sample Type | Blow Count / % Recovery | Dry Density lbs. per Cubic ft. | Moisture Content Percent of Dry Weight | Unified Soil Classification | REMARKS | VISUAL CLASSIFICATION |
|---|---|---|---|---|---|---|---|---|---|---|
| 0 | | | | | | | | SM | slightly moist | Man-made FILL |
| | | | | U | 58 | | | | hard | **SILTY SAND**, some clay & well graded gravel, well graded sand, subangular to subrounded, weak HCL reaction, low plasticity, light brown (5YR, 6/4) |
| | | | | S | 33-50/4" | | | | | note: considerable gravel from 1' to 2'6" with increase in clay at 2'6" |
| 5 | | | | S | 50/4" | | | SC | slightly moist | Man-made FILL |
| | | | | | | | | | hard | **CLAYEY SAND**, some silt, predominantly medium to coarse grained, subangular to subrounded, weak HCL reaction, medium plasticity, light brown (5YR, 5/6) |
| | | | | S | 24-50/6" | | | | moist at 8' | note: well graded sand below 9'6", with occasional thin sandy clay lenses |
| 10 | | | | S | 20-40-50/6" | | | | | |
| | | | | S | 10-25-30 | | | SM | slightly moist | NATIVE |
| | | | | | | | | | | **SILTY SAND**, trace of predominantly coarse grained gravel, predominantly medium to coarse grained sand, subangular, low plasticity |
| 15 | | | | U | 65 | | | | | Stage I cementation (weak HCL reaction), light brown (5YR, 5/6) |
| | | | | | | | | SP | | note: increase in fine grained gravel & some clay below 14'6" |
| | | | | U | 75 | | | | slightly moist | **GRAVELLY SAND**, some silt & clay, predominantly fine grained gravel & medium to coarse grained sand, subangular to subrounded, uncemented, nonplastic to low plasticity |
| | | | | | | | | | hard | Stage I cementation, light brown (5YR, 5/6) |
| 20 | | | | U | 15 | | | SM | slightly moist | **SILTY SAND**, trace of predominantly fine grained gravel, predominantly medium to coarse grained sand, subangular to subrounded, low plasticity |
| | | | | | | | | | moderately firm | Stage I cementation, moderate brown (5YR, 4/4) |
| | | | | U | 80 | | | SC | slightly moist | **CLAYEY SAND**, predominantly medium to coarse grained in zones (well graded overall), subangular to subrounded, medium plasticity |
| | | | | S | 15-20-35 | | | | | |
| 25 | | | | | | | | | | |

GROUNDWATER

| DEPTH(ft) | HOUR | DATE |
|---|---|---|
| | none | |
| | | |
| | | |
| | | |

SAMPLE TYPE
A - Drill cuttings; NR - No Recovery
S - 2" O.D. 1.38" I.D. tube sample
U - 3" O.D. 2.42" I.D. tube sample
T - 1" O.D. thin-walled tube sample
D - 2.5" O.D. 1.9" I.D. tube sample
C - 2.5" O.D. 2.0" I.D. tube sample



LOG OF TEST BORING NO. BH186+00-2

BORING LOG 1171122B.GPJ AGRA_PHX.GDT 6/29/01

**PROJECT** Vineyard FRS

amec

**JOB NO.** 0-117-001122 Task 2 **DATE** 6-21-01 & 6-22-01

**LOCATION** Sta. 185+45, 27' R
**RIG TYPE** CME-75
**BORING TYPE** 6 5/8" Hollow Stem Auger
**SURFACE ELEV.**
**DATUM**

| Depth in Feet | Drill Rate Min/ft. | Graphical Log | Sample | Sample Type | Blow Count / % Recovery | Dry Density lbs. per Cubic ft. | Moisture Content Percent of Dry Weight | Unified Soil Classification | REMARKS | VISUAL CLASSIFICATION |
|---|---|---|---|---|---|---|---|---|---|---|
| 25 | | | | | | | | SC with CL zones | slightly moist<br>hard | **CLAYEY SAND**, continued, occasional thin silty clay lenses below 25', medium plasticity<br>Stage II cementation (strong HCL reaction, some soft CaCO3 nodules), light brown (5YR, 6/4)<br>note: some to trace of fine grained gravel below 27'6"<br>Stage III cementation (considerable large CaCO3 nodules, strong HCL reaction), light brown (5YR, 6/4) below 28' |
| | | | | S | 27-35-40 | | | | | |
| 30 | | | | S | 23-37-50 | | | CL | slightly moist<br>hard | **SANDY CLAY locally grading to SILTY CLAY**, some silt, predominantly medium grained sand, medium to high plasticity<br>Stage III cementation (considerable soft, large CaCO3 nodules), light brown (5YR, 5/6), occasional black manganese-oxide stain around particle borders<br>note: silty clay with trace of sand below 32'6"<br>note: some short CaCO3 filaments & considerable small nodules below 34'6" |
| | | | | S | 16-40-47 | | | | | |
| 35 | | | | S | 13-20-41 | | | | | |
| | | | | S | 19-24-29 | | | | | |
| 40 | | | | | | | | | | Stopped Auger at 37'6"<br>Stopped sampler at 39' |
| 45 | | | | | | | | | | |
| 50 | | | | | | | | | | |

GROUNDWATER

| | DEPTH(ft) | HOUR | DATE |
|---|---|---|---|
| ▽ | | none | |
| ▼ | | | |
| ▽ | | | |
| ▼ | | | |

SAMPLE TYPE
A - Drill cuttings; NR - No Recovery
S - 2" O.D. 1.38" I.D. tube sample
U - 3" O.D. 2.42" I.D. tube sample
T - 1" O.D. thin-walled tube sample
D - 2.5" O.D. 1.9" I.D. tube sample
C - 2.5" O.D. 2.0" I.D. tube sample



LOG OF TEST BORING NO. BH186+00-2

BORING LOG 1171122B.GPJ AGRA_PHX.GDT 6/29/01

**PROJECT** Vineyard FRS

amec

**JOB NO.** 0-117-001122 Task 2 **DATE** 6/14/01

**LOCATION** Sta. 186+00, 4' R
**RIG TYPE** CME-75
**BORING TYPE** 6 5/8" Hollow Stem Auger
**SURFACE ELEV.**
**DATUM**

| Depth in Feet | Drill Rate Min/ft. | Graphical Log | Sample | Sample Type | Blow Count / % Recovery | Dry Density lbs. per Cubic ft. | Moisture Content Percent of Dry Weight | Unified Soil Classification | REMARKS | VISUAL CLASSIFICATION |
|---|---|---|---|---|---|---|---|---|---|---|
| 0 | | | | | | | | SM | slightly moist<br>hard | Man-made FILL<br>**SILTY SAND**, some clay below 2'6", considerable gravel on the ground surface, predominantly fine to medium grained, subangular to subrounded, low plasticity, light brown (5YR, 5/6) |
| | | | | S | 22-50/6" | | | | | |
| | | | | S | 15-32-34 | | | | | |
| 5 | | | | S | 32-50/3" | | | SC | slightly moist<br>hard | Man-made FILL<br>**CLAYEY SAND**, trace of fine grained gravel, predominantly fine to medium grained, subangular to subrounded, medium plasticity, light brown (5YR, 6/4)<br>note: considerable silt below 17'6" |
| | | | | S | 20-44-38 | | | | | |
| 10 | | | | U | 100/11" | | | | | |
| | | | | S | 36-50/5" | | | | | |
| 15 | | | | S | 17-35-50/5" | | | | | |
| | | | | S | 41-42-50/4" | | | | | |
| 20 | | | | U | 65 | | | SP | slightly moist<br>hard | NATIVE<br>**SAND**, trace of predominantly coarse grained gravel & clay, predominantly medium to coarse grained sand, subangular to subrounded, uncemented to weakly cemented, nonplastic to low plasticity, light brown (5YR, 5/6) |
| | | | | S | 7-8-9 | | | SM | slightly moist<br>firm to very firm | **SILTY SAND**, predominantly fine grained, subangular to subrounded, nonplastic to low plasticity,<br>note: possible Stage II cementation at 22'6" (weak HCL reaction), light brown (5YR, 6/4) |
| | | | | S | 7-12-21 | | | | | |
| 25 | | | | | | | | | | |

GROUNDWATER

| DEPTH(ft) | HOUR | DATE |
|---|---|---|
| | none | |
| | | |
| | | |
| | | |

SAMPLE TYPE
A - Drill cuttings; NR - No Recovery
S - 2" O.D. 1.38" I.D. tube sample
U - 3" O.D. 2.42" I.D. tube sample
T - 1" O.D. thin-walled tube sample
D - 2.5" O.D. 1.9" I.D. tube sample
C - California tube sample



LOG OF TEST BORING NO. BH186+00-3

BORING LOG 1171122B.GPJ AGRA_PHX.GDT 6/28/01

**PROJECT** Vineyard FRS

amec

**JOB NO.** 0-117-001122 Task 2 **DATE** 6/14/01

**LOCATION** Sta. 186+00, 4' R
**RIG TYPE** CME-75
**BORING TYPE** 6 5/8" Hollow Stem Auger
**SURFACE ELEV.**
**DATUM**

| Depth in Feet | Drill Rate Min/ft. | Graphical Log | Sample | Sample Type | Blow Count / % Recovery | Dry Density lbs. per Cubic ft. | Moisture Content Percent of Dry Weight | Unified Soil Classification | REMARKS | VISUAL CLASSIFICATION |
|---|---|---|---|---|---|---|---|---|---|---|
| 25 | | | | | | | | | | |
| | | | | | | | | SC with CL zones | slightly moist<br>very firm to hard | **CLAYEY SAND with SANDY CLAY zones**, trace of fine grained gravel below 32'6", predominantly medium to coarse grained sand, subangular to subrounded, medium plasticity<br><br>Stage II cementation (nodules, strong HCL reaction), light brown (5YR, 6/4) below 25'6" |
| | | | | U | 44 | | | | | |
| 30 | | | | S | 16-33-50/ 5 1/2" | | | | | |
| | | | | S | 18-20-22 | | | | | |
| 35 | | | | S | 7-30-50/ 5 3/4" | | | SM | slightly moist<br>hard | **SILTY SAND**, some clay, predominantly fine to medium grained, subangular to subrounded, nonplastic to low plasticity<br><br>Stage II cementation (strong HCL reaction in zones, considerable CaCO3 around gravel borders), light brown (5YR, 6/4)<br><br>note: clayey sand lenses below 37'6" |
| | | | | S | 21-50/ 5 1/2" | | | | | |
| 40 | | | | S | 17-50/ 5 1/2" | | | | | |
| | | | | S | 48-50/2" | | | | | |
| 45 | | | | | | | | | | Stopped Auger at 42'6"<br>Stopped Sampler at 43'2" |
| 50 | | | | | | | | | | |

GROUNDWATER

| DEPTH(ft) | HOUR | DATE |
|---|---|---|
| | none | |
| | | |
| | | |
| | | |

SAMPLE TYPE
A - Drill cuttings; NR - No Recovery
S - 2" O.D. 1.38" I.D. tube sample
U - 3" O.D. 2.42" I.D. tube sample
T - 1" O.D. thin-walled tube sample
D - 2.5" O.D. 1.9" I.D. tube sample
C - California tube sample



LOG OF TEST BORING NO. BH186+00-3

BORING LOG 1171122B.GPJ AGRA_PHX.GDT 6/28/01

**PROJECT** Vineyard FRS

amec

**JOB NO.** 0-117-001122 Task 2 **DATE** 6/6/01

**LOCATION** Sta. 186+00, 50' L
**RIG TYPE** CME-75
**BORING TYPE** 6 5/8" Hollow Stem Auger
**SURFACE ELEV.**
**DATUM**

| Depth in Feet | Drill Rate Min/ft. | Graphical Log | Sample | Sample Type | Blow Count / % Recovery | Dry Density lbs. per Cubic ft. | Moisture Content Percent of Dry Weight | Unified Soil Classification | REMARKS | VISUAL CLASSIFICATION |
|---|---|---|---|---|---|---|---|---|---|---|
| 0 | | | X | S | 22-13-12 | | | SM | slightly moist<br>firm | **SILTY SAND**, predominantly medium to fine grained, subrounded to subangular, uncemented, low plasticity, light brown (5YR, 5/6) |
| | | | X | S | 6-10-13 | | | SP-SM | slightly moist<br>medium dense | **SAND TO SILTY SAND**, trace to some fine grained, subrounded to subangular gravel, predominantly medium to coarse grained, subrounded to subangular, uncemented, nonplastic, moderate brown (5YR, 4/4)<br><br>Stage I cementation (thin & discontinuous, CaCO3 coated grains) below 5' |
| 5 | | | X | S | 8-8-8 | | | | | |
| | | | | | | | | SP | slightly moist<br>medium dense | **SAND**, trace to some fine grained, subrounded to subangular gravel, predominantly medium to coarse grained, subrounded to subangular, nonplastic<br><br>Stage I cementation (weak HCL reaction, thin coatings), light brown (5YR, 5/6) |
| | | | | U | 45 | | | | | |
| | | | | | | | | SM | | |
| 10 | | | X | S | 25-50/5 1/2" | | | SM | slightly moist<br>very firm | **SILTY SAND**, predominantly fine grained, subrounded to subangular, low plasticity<br><br>Stage II cementation (moderate HCL reaction, nodules, thin layers of CaCO3), light brown (5YR, 5/4) |
| | | | X | S | 50/6" | | | | slightly moist<br>hard | **SILTY SAND**, predominantly fine grained, subrounded to subangular, low plasticity<br><br>Stage III cementation (nodules, thin CaCO3 lenses), light brown (5YR, 6/6)<br><br>note: increase in medium to coarse grained sand from 16'6" to 18'6" & from 24' to 26' |
| 15 | | | X | S | 42-50/5" | | | | | |
| | | | X | S | 24-26-29 | | | | | |
| 20 | | | X | S | 50/4" NR | | | | | |
| | | | X | S | 50/3" | | | | | |
| 25 | | | X | S | 32-36-49 | | | | | |

GROUNDWATER

| DEPTH(ft) | HOUR | DATE |
|---|---|---|
| | none | |
| | | |
| | | |
| | | |

SAMPLE TYPE
A - Drill cuttings; NR - No Recovery
S - 2" O.D. 1.38" I.D. tube sample
U - 3" O.D. 2.42" I.D. tube sample
T - 1" O.D. thin-walled tube sample
D - 2.5" O.D. 1.9" I.D. tube sample
C - 2.5" O.D. 2.0" I.D. tube sample



LOG OF TEST BORING NO. BH186+00-4

BORING LOG 1171122B.GPJ AGRA_PHX.GDT 6/29/01

**PROJECT** Vineyard FRS

amec

**JOB NO.** 0-117-001122 Task 2 **DATE** 6/6/01

**LOCATION** Sta. 186+00, 50' L
**RIG TYPE** CME-75
**BORING TYPE** 6 5/8" Hollow Stem Auger
**SURFACE ELEV.**
**DATUM**

| Depth in Feet | Drill Rate Min/ft. | Graphical Log | Sample | Sample Type | Blow Count / % Recovery | Dry Density lbs. per Cubic ft. | Moisture Content Percent of Dry Weight | Unified Soil Classification | REMARKS | VISUAL CLASSIFICATION |
|---|---|---|---|---|---|---|---|---|---|---|
| 25 | | | X | | | | | SM | | **SILTY SAND**, continued |
| | | | | | | | | | | Stopped Auger at 24'6"<br>Stopped Sampler at 26' |
| 30 | | | | | | | | | | |
| 35 | | | | | | | | | | |
| 40 | | | | | | | | | | |
| 45 | | | | | | | | | | |
| 50 | | | | | | | | | | |

BORING LOG 1171122B.GPJ AGRA_PHX.GDT 6/29/01

GROUNDWATER

| | DEPTH(ft) | HOUR | DATE |
|---|---|---|---|
| ▽ | | none | |
| ▼ | | | |
| ▽ | | | |
| ▼ | | | |

SAMPLE TYPE
A - Drill cuttings; NR - No Recovery
S - 2" O.D. 1.38" I.D. tube sample
U - 3" O.D. 2.42" I.D. tube sample
T - 1" O.D. thin-walled tube sample
D - 2.5" O.D. 1.9" I.D. tube sample
C - 2.5" O.D. 2.0" I.D. tube sample



LOG OF TEST BORING NO. BH186+00-4

**PROJECT** Vineyard FRS

amec

**JOB NO.** 0-117-001122 Task 2 **DATE** 6/5/01

**LOCATION** Sta. 185+94, 73' R
**RIG TYPE** CME-75
**BORING TYPE** 6 5/8" Hollow Stem Auger
**SURFACE ELEV.**
**DATUM**

| Depth in Feet | Drill Rate Min/ft. | Graphical Log | Sample | Sample Type | Blow Count / % Recovery | Dry Density lbs. per Cubic ft. | Moisture Content Percent of Dry Weight | Unified Soil Classification | REMARKS | VISUAL CLASSIFICATION |
|---|---|---|---|---|---|---|---|---|---|---|
| 0 | | | | S | 9-8-7 | | | SM | slightly moist<br>firm | **SILTY SAND**, trace of clay, trace of fine grained, subrounded to subangular gravel, predominantly fine grained, subrounded to subangular, uncemented, low plasticity, light brown (5YR, 5/6)<br><br>note: moderate yellowish brown (10YR, 5/4) in upper 6" |
| | | | | S | 8-13-12 | | | SP-SM | slightly moist<br>medium dense | **GRAVELLY SAND**, trace to some silt, predominantly fine grained, subrounded to subangular gravel, predominantly medium to coarse grained, subrounded to subangular, nonplastic<br><br>Stage I cementation (weak HCL reaction), moderate reddish brown (10YR, 4/6) |
| 5 | | | | S | 6-6-7 | | | | | |
| | | | | U | 36 | | | | | |
| 10 | | | | S | 18-42-42 | | | SC | slightly moist<br>hard | **CLAYEY SAND**, predominantly fine grained, subrounded to subangular, low to medium plasticity<br><br>Stage II cementation (nodules, thin zones of CaCO3), moderate brown (5YR, 4/4) |
| | | | | | | | | SM | slightly moist<br>very firm | **SILTY SAND**, trace of fine grained, subrounded to subangular gravel, predominantly fine grained, subangular to subrounded, low plasticity<br><br>Stage II cementation (small nodules, interconnected filaments), moderate yellowish brown (10YR, 5/4) |
| | | | | S | 10-16-21 | | | | | |
| 15 | | | | S | 13-19-18 | | | | | |
| | | | | | | | | SP-SM | slightly moist<br>very dense | **GRAVELLY SAND**, trace to some silt, fine grained, subrounded to subangular gravel, well graded, subrounded to subangular, nonplastic<br><br>Stage II cementation (some grain coatings & moderate HCL reaction), light brown (5YR, 6/4) |
| | | | | S | 30-38-36 | | | | | |
| | | | | | | | | SM | slightly moist<br>hard | **SILTY SAND**, trace of fine grained, subrounded to subangular gravel, predominantly fine grained, subrounded to subangular, low plasticity<br><br>Stage II cementation (nodules, small CaCO3 zones), moderate yellowish brown (10YR, 5/4)<br><br>note: light brown (5YR, 6/4) below 22' |
| 20 | | | | S | 24-38-50/5" | | | | | |
| | | | | S | 10-20-33 | | | | | |
| | | | | S | 14-28-28 | | | | | |
| 25 | | | | | | | | | | |

GROUNDWATER

| DEPTH(ft) | HOUR | DATE |
|---|---|---|
| | none | |
| | | |
| | | |
| | | |

SAMPLE TYPE
A - Drill cuttings; NR - No Recovery
S - 2" O.D. 1.38" I.D. tube sample
U - 3" O.D. 2.42" I.D. tube sample
T - 1" O.D. thin-walled tube sample
D - 2.5" O.D. 1.9" I.D. tube sample
C - 2.5" O.D. 2.0" I.D. tube sample



LOG OF TEST BORING NO. BH186+00-5

BORING LOG 1171122B.GPJ AGRA_PHX.GDT 6/29/01

**PROJECT** Vineyard FRS

amec

**JOB NO.** 0-117-001122 Task 2 **DATE** 6/5/01

**LOCATION** Sta. 185+94, 73' R
**RIG TYPE** CME-75
**BORING TYPE** 6 5/8" Hollow Stem Auger
**SURFACE ELEV.**
**DATUM**

| Depth in Feet | Drill Rate Min/ft. | Graphical Log | Sample | Sample Type | Blow Count / % Recovery | Dry Density lbs. per Cubic ft. | Moisture Content Percent of Dry Weight | Unified Soil Classification | REMARKS | VISUAL CLASSIFICATION |
|---|---|---|---|---|---|---|---|---|---|---|
| 25 | | | | | | | | SM | | **SILTY SAND**, continued |
| | | | | | | | | | | Stopped Auger at 24'6" |
| | | | | | | | | | | Stopped Sampler at 26' |
| 30 | | | | | | | | | | |
| 35 | | | | | | | | | | |
| 40 | | | | | | | | | | |
| 45 | | | | | | | | | | |
| 50 | | | | | | | | | | |

BORING LOG 1171122B.GPJ AGRA_PHX.GDT 6/29/01

GROUNDWATER

| DEPTH(ft) | HOUR | DATE |
|---|---|---|
| | none | |
| | | |
| | | |
| | | |

SAMPLE TYPE
A - Drill cuttings; NR - No Recovery
S - 2" O.D. 1.38" I.D. tube sample
U - 3" O.D. 2.42" I.D. tube sample
T - 1" O.D. thin-walled tube sample
D - 2.5" O.D. 1.9" I.D. tube sample
C - 2.5" O.D. 2.0" I.D. tube sample



LOG OF TEST BORING NO. BH186+00-5

**PROJECT** Vineyard FRS

amec

**JOB NO.** 0-117-001122 Task 2 **DATE** 6/4/01

**LOCATION** Sta. 200+00, 52' R
**RIG TYPE** CME-75
**BORING TYPE** 6 5/8" Hollow Stem Auger
**SURFACE ELEV.**
**DATUM**

| Depth in Feet | Blows Per Foot | Graphical Log | Sample | Sample Type | Blow Count / % Recovery | Dry Density lbs. per Cubic ft. | Moisture Content Percent of Dry Weight | Unified Soil Classification | REMARKS | VISUAL CLASSIFICATION |
|---|---|---|---|---|---|---|---|---|---|---|
| 0 | | | X | S | 14-18-17 | | | SM | slightly moist<br>very firm | **SILTY SAND**, predominantly fine grained, subrounded to subangular, low plasticity, moderate yellowish brown (10YR, 5/4) transitioning to light brown (5YR, 5/6) below 0.5'<br>note: uncemented above 0.5'<br>Stage I cementation below 0.5'<br>note: trace of clay & moderate brown (5YR, 4/4) color below 2'6" |
| | | | | U | 46 | | | | | |
| | | | | | | | | SM | | |
| 5 | | | X | S | 5-6-6 | | | | slightly moist<br>moderately firm to soft | **SILTY SAND**, predominantly fine grained, subrounded to subangular, low plasticity<br>Stage I cementation, moderate brown (5YR, 4/4) |
| | | | | U | 12 | | | | | |
| 10 | | | | U | 11 | | | | | |
| | | | | | | | | SM | slightly moist<br>hard | **SILTY SAND**, trace of subrounded to subangular gravel, predominantly fine grained, subangular to subrounded, low plasticity<br>Stage I cementation, moderate brown (5YR, 4/4)<br>note: increase in gravel from 13' to 14'<br>Stage II cementation & light brown (5YR, 5/6) below 15'6"<br>Stage III cementation below 17' |
| | | | | U | 100/8" | | | | | |
| 15 | | | X | S | 14-31-48 | | | | | |
| | | | X | S | 50/5" | | | | | |
| 20 | | | X | S | 50/5" | | | | | |
| | | | X | S | 50/4" | | | | | |
| 25 | | | X | S | 40-50/5" | | | | | |

BORING LOG 117122B.GPJ AGRA_PHX.GDT 6/25/01

GROUNDWATER

| DEPTH(ft) | HOUR | DATE |
|---|---|---|
| | none | |
| | | |
| | | |
| | | |

SAMPLE TYPE
A - Drill cuttings; NR - No Recovery
S - 2" O.D. 1.38" I.D. tube sample
U - 3" O.D. 2.42" I.D. tube sample
T - 1" O.D. thin-walled tube sample
D - 2.5" O.D. 1.9" I.D. tube sample
C - California tube sample



LOG OF TEST BORING NO. BH200+00-1

**PROJECT** Vineyard FRS

amec

**JOB NO.** 0-117-001122 Task 2 **DATE** 6/4/01

**LOCATION** Sta. 200+00, 52' R
**RIG TYPE** CME-75
**BORING TYPE** 6 5/8" Hollow Stem Auger
**SURFACE ELEV.**
**DATUM**

| Depth in Feet | Blows Per Foot | Graphical Log | Sample | Sample Type | Blow Count / % Recovery | Dry Density lbs. per Cubic ft. | Moisture Content Percent of Dry Weight | Unified Soil Classification | REMARKS | VISUAL CLASSIFICATION |
|---|---|---|---|---|---|---|---|---|---|---|
| 25 | | | | | | | | SM | | **SILTY SAND**, continued |
| | | | | S | 22-34-50 | | | | | |
| | | | | | | | | | | Stopped Auger at 27'<br>Stopped Sampler at 28'6" |
| 30 | | | | | | | | | | |
| 35 | | | | | | | | | | |
| 40 | | | | | | | | | | |
| 45 | | | | | | | | | | |
| 50 | | | | | | | | | | |

BORING LOG 1171122B.GPJ AGRA_PHX.GDT 6/25/01

GROUNDWATER

| DEPTH(ft) | HOUR | DATE |
|---|---|---|
| | none | |
| | | |
| | | |
| | | |

SAMPLE TYPE
A - Drill cuttings; NR - No Recovery
S - 2" O.D. 1.38" I.D. tube sample
U - 3" O.D. 2.42" I.D. tube sample
T - 1" O.D. thin-walled tube sample
D - 2.5" O.D. 1.9" I.D. tube sample
C - California tube sample



LOG OF TEST BORING NO. BH200+00-1

**PROJECT** Vineyard FRS

amec

**JOB NO.** 0-117-001122 Task 2 **DATE** 6/21/01

**LOCATION** Sta. 200+00, 31' R
**RIG TYPE** CME-75
**BORING TYPE** 6 5/8" Hollow Stem Auger
**SURFACE ELEV.**
**DATUM**

| Depth in Feet | Drill Rate Min/ft. | Graphical Log | Sample | Sample Type | Blow Count / % Recovery | Dry Density lbs. per Cubic ft. | Moisture Content Percent of Dry Weight | Unified Soil Classification | REMARKS | VISUAL CLASSIFICATION |
|---|---|---|---|---|---|---|---|---|---|---|
| 0 | | | | | | | | SC | slightly moist<br>firm to hard | Man-made FILL<br>**CLAYEY SAND**, trace of well graded gravel on the ground surface, some silt, well graded sand, subangular to subrounded, low to medium plasticity, light brown (5YR, 6/4)<br><br>note: predominantly medium to coarse sand at 7'6", with trace of fine grained gravel, moderate brown (5YR, 4/4) |
| | | | | S | 21-23-30 | | | | | |
| | | | | S | 25-50/5" | | | | | |
| 5 | | | | S | 15-30-50/4" | | | | | |
| | | | | S | 14-14-14 | | | | | |
| 10 | | | | S | 9-8-11 | | | | | |
| | | | | U | 20 | | | SM | slightly moist<br>firm | Probable NATIVE<br>**SILTY SAND**, predominantly fine grained, subangular to subrounded, uncemented, nonplastic<br><br>note: probable Stage I cementation (weak HCL reaction), light brown (5YR, 6/4) |
| 15 | | | | U | 24 | | | | | |
| | | | | U | 100/6" | | | SC with CL lenses | slightly moist<br>hard | **CLAYEY SAND**, trace of predominantly fine grained gravel, predominantly medium to coarse grained, subangular to subrounded, low to medium plasticity<br><br>Stage II cementation (strong HCL reaction, some soft CaCO3 nodules), light brown (5YR, 6/4)<br><br>note: thin sandy clay lenses below 22'6" |
| 20 | | | | S | 22-50/5" | | | | | |
| | | | | S | 50/6" | | | | | |
| 25 | | | | S | 25-40-40 | | | | | |

GROUNDWATER

| | DEPTH(ft) | HOUR | DATE |
|---|---|---|---|
| ▽ | | none | |
| ▼ | | | |
| ▽ | | | |
| ▼ | | | |

SAMPLE TYPE
A - Drill cuttings; NR - No Recovery
S - 2" O.D. 1.38" I.D. tube sample
U - 3" O.D. 2.42" I.D. tube sample
T - 1" O.D. thin-walled tube sample
D - 2.5" O.D. 1.9" I.D. tube sample
C - 2.5" O.D. 2.0" I.D. tube sample



# LOG OF TEST BORING NO. BH200+00-2

BORING LOG 1171122B.GPJ AGRA_PHX.GDT 6/29/01

**PROJECT** Vineyard FRS

**JOB NO.** 0-117-001122 Task 2 **DATE** 6/21/01

**LOCATION** Sta. 200+00, 31' R
**RIG TYPE** CME-75
**BORING TYPE** 6 5/8" Hollow Stem Auger
**SURFACE ELEV.**
**DATUM**

| Depth in Feet | Drill Rate Min/ft. | Graphical Log | Sample | Sample Type | Blow Count / % Recovery | Dry Density lbs. per Cubic ft. | Moisture Content Percent of Dry Weight | Unified Soil Classification | REMARKS | VISUAL CLASSIFICATION |
|---|---|---|---|---|---|---|---|---|---|---|
| 25 | | | X | | | | | SC | slightly moist<br>hard | **CLAYEY SAND**, continued |
| | | | X | S | 25-50/6" | | | CL | slightly moist<br>hard | **SANDY CLAY**, considerable silt, predominantly fine grained sand, low to medium plasticity<br>Stage II cementation (strong HCL reaction, some small, soft CaCO3 nodules)<br>note: sample hammer broke at swivel connection & fell downhole, HSA advanced to 35', bentonite added to plug auger & retrive hammer |
| 30 | | | X | S | 35-50/4" | | | | | Stopped Auger at 29'6"<br>Sampler refused at 30'4" |
| 35 | | | | | | | | | | |
| 40 | | | | | | | | | | |
| 45 | | | | | | | | | | |
| 50 | | | | | | | | | | |

GROUNDWATER

| | DEPTH(ft) | HOUR | DATE |
|---|---|---|---|
| ▽ | | none | |
| ▼ | | | |
| ▽ | | | |
| ▼ | | | |

SAMPLE TYPE
A - Drill cuttings; NR - No Recovery
S - 2" O.D. 1.38" I.D. tube sample
U - 3" O.D. 2.42" I.D. tube sample
T - 1" O.D. thin-walled tube sample
D - 2.5" O.D. 1.9" I.D. tube sample
C - 2.5" O.D. 2.0" I.D. tube sample



LOG OF TEST BORING NO. BH200+00-2

BORING LOG 1171122B.GPJ AGRA_PHX.GDT 6/29/01

**PROJECT** Vineyard FRS

amec

**JOB NO.** 0-117-001122 Task 2 **DATE** 6/14/01

**LOCATION** Sta. 200+00, 3' R
**RIG TYPE** CME-75
**BORING TYPE** 6 5/8" Hollow Stem Auger
**SURFACE ELEV.**
**DATUM**

| Depth in Feet | Drill Rate Min/ft. | Graphical Log | Sample | Sample Type | Blow Count / % Recovery | Dry Density lbs. per Cubic ft. | Moisture Content Percent of Dry Weight | Unified Soil Classification | REMARKS | VISUAL CLASSIFICATION |
|---|---|---|---|---|---|---|---|---|---|---|
| 0 | | | | | | | | SC | slightly moist<br>firm to hard | Man-made FILL<br>**CLAYEY SAND with SILTY SAND in lenses**, considerable predominantly coarse grained gravel on the ground surface, predominantly fine to medium grained sand, subangular to subrounded, low to medium plasticity, light brown (5YR, 6/4)<br>note: trace of fine grained gravel below 4'6"<br>note: considerable clay & moderate brown (5YR, 4/4) from 9'6" to 11' with increase in moisture content |
| | | | | S | 15-15-23 | | | | | |
| | | | | S | 18-37-44 | | | | | |
| 5 | | | | U | 90 | | | | | |
| | | | | S | 12-23-20 | | | | | |
| 10 | | | | S | 10-23-30 | | | | | |
| | | | | S | 11-15-14 | | | | | |
| 15 | | | | S | 12-18-21 | | | CL | slightly moist to moist<br>firm to hard | Possible NATIVE<br>**SANDY CLAY**, some silt, predominantly fine to medium grained sand, medium plasticity, moderate brown (5YR, 4/4)<br>note: grading to clayey sand<br>Stage II cementation (weak HCL reaction, short discontinuous CaCO3 filaments), moderate brown (5YR, 4/4) below 20'6"<br>note: considerable filaments & strong HCL reaction at 22' |
| | | | | U | 100/11" | | | | | |
| 20 | | | | S | 8-11-14 | | | | | |
| | | | | S | 5-8-8 | | | SM | slightly moist moderately firm to firm | NATIVE<br>**SILTY SAND**, predominantly fine to medium grained with occasional coarse grained lenses, subangular to subrounded, nonplastic |
| 25 | | | | U | 15 | | | | | |

GROUNDWATER

| DEPTH(ft) | HOUR | DATE |
|---|---|---|
| | none | |
| | | |
| | | |
| | | |

SAMPLE TYPE
A - Drill cuttings; NR - No Recovery
S - 2" O.D. 1.38" I.D. tube sample
U - 3" O.D. 2.42" I.D. tube sample
T - 1" O.D. thin-walled tube sample
D - 2.5" O.D. 1.9" I.D. tube sample
C - 2.5" O.D. 2.0" I.D. tube sample



LOG OF TEST BORING NO. BH200+00-3

BORING LOG 1171122B.GPJ AGRA_PHX.GDT 6/29/01

**PROJECT** Vineyard FRS

amec

**JOB NO.** 0-117-001122 Task 2 **DATE** 6/14/01

**LOCATION** Sta. 200+00, 3' R
**RIG TYPE** CME-75
**BORING TYPE** 6 5/8" Hollow Stem Auger
**SURFACE ELEV.**
**DATUM**

| Depth in Feet | Drill Rate Min/ft. | Graphical Log | Sample | Sample Type | Blow Count / % Recovery | Dry Density lbs. per Cubic ft. | Moisture Content Percent of Dry Weight | Unified Soil Classification | REMARKS | VISUAL CLASSIFICATION |
|---|---|---|---|---|---|---|---|---|---|---|
| 25 | | | | | | | | SM | slightly moist<br>moderately firm to hard | **SILTY SAND**, continued<br>Stage II cementation (considerable CaCO3 filaments, strong HCL reaction)<br>note: grading to clayey sand with some fine grained gravel below 27'6"<br>note: some coarse grained gravel below 29'6" |
| | | | | S | 13-17-17 | | | | | |
| 30 | | | | U | 53 | | | | | |
| | | | | S | 21-32-37 | | | SC | slightly moist<br>hard | **CLAYEY SAND**, predominantly fine to medium grained, subangular to subrounded, low to medium plasticity<br>Stage II cementation (strong HCL reaction), light brown (5YR, 6/4) |
| 35 | | | | S | 14-19-25 | | | CL | slightly moist<br>hard | **SANDY CLAY**, some silt, predominantly fine grained sand, medium plasticity<br>Stage II cementation (strong HCL reaction), light brown (5YR, 5/6) |
| | | | | S | 19-36-49 | | | | | |
| 40 | | | | S | 23-33-25 | | | SC | slightly moist<br>hard | **CLAYEY SAND**, some predominantly coarse grained gravel, predominantly medium to coarse grained sand, subangular to subrounded, medium plasticity<br>Stage II cementation (strong HCL reaction), light brown (5YR, 5/6) |
| | | | | | | | | | | Stopped Auger at 39'6"<br>Stopped Sampler at 41' |
| 45 | | | | | | | | | | |
| 50 | | | | | | | | | | |

GROUNDWATER

| | DEPTH(ft) | HOUR | DATE |
|---|---|---|---|
| ▽ | | none | |
| ▼ | | | |
| ▽ | | | |
| ▼ | | | |

SAMPLE TYPE
A - Drill cuttings; NR - No Recovery
S - 2" O.D. 1.38" I.D. tube sample
U - 3" O.D. 2.42" I.D. tube sample
T - 1" O.D. thin-walled tube sample
D - 2.5" O.D. 1.9" I.D. tube sample
C - 2.5" O.D. 2.0" I.D. tube sample



LOG OF TEST BORING NO. BH200+00-3

BORING LOG 1171122B.GPJ AGRA_PHX.GDT 6/29/01

**PROJECT** Vineyard FRS

amec

**JOB NO.** 0-117-001122 Task 2 **DATE** 6/6/01

**LOCATION** Sta. 200+00, 45' L
**RIG TYPE** CME-75
**BORING TYPE** 6 5/8" Hollow Stem Auger
**SURFACE ELEV.**
**DATUM**

| Depth in Feet | Drill Rate Min/ft. | Graphical Log | Sample | Sample Type | Blow Count / % Recovery | Dry Density lbs. per Cubic ft. | Moisture Content Percent of Dry Weight | Unified Soil Classification | REMARKS | VISUAL CLASSIFICATION |
|---|---|---|---|---|---|---|---|---|---|---|
| 0 | | | | S | 7-9-16 | | | SC | slightly moist<br>firm | **CLAYEY SAND**, trace of fine grained, subrounded to subangular gravel, predominantly fine grained, subrounded to subangular, uncemented, low to medium plasticity, moderate brown (5YR, 4/4) |
| | | | | S | 11-12-14 | | | SM | slightly moist<br>moderately firm to firm | **SILTY SAND**, trace of clay, predominantly fine grained, subrounded to subangular, low plasticity<br>Stage I cementation (weak HCL reaction, trace filaments), light brown (5YR, 5/6) |
| 5 | | | | S | 11-12-14 | | | | | |
| | | | | U | 16 | | | | | |
| 10 | | | | U | 16 | | | | | |
| | | | | S | 17-20-28 | | | SP | slightly moist<br>dense | **GRAVELLY SAND**, trace of silt, predominantly fine grained, subrounded to subangular gravel, predominantly medium to coarse grained, subangular to subrounded, nonplastic<br>Stage II cementation (coatings on grains, moderate HCL reaction), moderate reddish brown (10R, 4/6) |
| 15 | | | | S | 28-50/5" | | | SM | slightly moist<br>hard | **SILTY SAND**, trace of clay, predominantly fine grained, subrounded to subangular, low plasticity<br>Stage II cemenation (nodules, thin, partial CaCO3 laminations), light brown (5YR, 6/6) |
| | | | | S | 50/5 1/2" | | | SM | slightly moist<br>hard | **SILTY SAND**, predominantly fine grained, subrounded to subangular, low plasticity<br>Stage III cementation (CaCO3 laminations, nodules), grayish orange pink (5YR 7/2) |
| 20 | | | | S | 50/5" | | | | | |
| | | | | S | 50/5 1/2" | | | | | |
| 25 | | | | S | 36-50/6" | | | | | |

GROUNDWATER

| DEPTH(ft) | HOUR | DATE |
|---|---|---|
| | none | |
| | | |
| | | |
| | | |

SAMPLE TYPE
A - Drill cuttings; NR - No Recovery
S - 2" O.D. 1.38" I.D. tube sample
U - 3" O.D. 2.42" I.D. tube sample
T - 1" O.D. thin-walled tube sample
D - 2.5" O.D. 1.9" I.D. tube sample
C - 2.5" O.D. 2.0" I.D. tube sample



LOG OF TEST BORING NO. BH200+00-4

BORING LOG 1171122B.GPJ AGRA_PHX.GDT 6/29/01

**PROJECT** Vineyard FRS

amec

**JOB NO.** 0-117-001122 Task 2 **DATE** 6/6/01

**LOCATION** Sta. 200+00, 45' L
**RIG TYPE** CME-75
**BORING TYPE** 6 5/8" Hollow Stem Auger
**SURFACE ELEV.**
**DATUM**

| Depth in Feet | Drill Rate Min/ft. | Graphical Log | Sample | Sample Type | Blow Count / % Recovery | Dry Density lbs. per Cubic ft. | Moisture Content Percent of Dry Weight | Unified Soil Classification | REMARKS | VISUAL CLASSIFICATION |
|---|---|---|---|---|---|---|---|---|---|---|
| 25 | | | X | | | | | SM | | **SILTY SAND**, continued |
| | | | | | | | | | | Stopped Auger at 24'6" |
| | | | | | | | | | | Sampler refused at 24'6" |
| 30 | | | | | | | | | | |
| 35 | | | | | | | | | | |
| 40 | | | | | | | | | | |
| 45 | | | | | | | | | | |
| 50 | | | | | | | | | | |

GROUNDWATER

| DEPTH(ft) | HOUR | DATE |
|---|---|---|
| | none | |
| | | |
| | | |
| | | |

SAMPLE TYPE
A - Drill cuttings; NR - No Recovery
S - 2" O.D. 1.38" I.D. tube sample
U - 3" O.D. 2.42" I.D. tube sample
T - 1" O.D. thin-walled tube sample
D - 2.5" O.D. 1.9" I.D. tube sample
C - 2.5" O.D. 2.0" I.D. tube sample



LOG OF TEST BORING NO. BH200+00-4

BORING LOG 1171122B.GPJ AGRA_PHX.GDT 6/29/01

**PROJECT** Vineyard FRS

amec

**JOB NO.** 0-117-001122 Task 2 **DATE** 6/4/01

**LOCATION** Sta. 200+00, 74' R
**RIG TYPE** CME-75
**BORING TYPE** 6 5/8" Hollow Stem Auger
**SURFACE ELEV.**
**DATUM**

| Depth in Feet | Blows Per Foot | Graphical Log | Sample | Sample Type | Blow Count / % Recovery | Dry Density lbs. per Cubic ft. | Moisture Content Percent of Dry Weight | Unified Soil Classification | REMARKS | VISUAL CLASSIFICATION |
|---|---|---|---|---|---|---|---|---|---|---|
| 0 | | | | S | 17-12-11 | | | SM | slightly moist<br>firm | **SILTY SAND**, trace of fine grained, subrounded to subangular gravel, predominantly medium to fine grained, subrounded to subangular, uncemented, low plasticity, moderate yellowish brown (10YR, 5/6) |
| | | | | | | | | SC | slightly moist<br>firm | **CLAYEY SAND**, predominantly fine grained, subrounded to subangular, low to medium plasticity<br>Stage I cementation, moderate brown (5YR, 4/4) |
| | | | | U | 35 | | | SM | slightly moist<br>moderately firm | **SILTY SAND**, trace of clay & fine grained gravel, predominantly fine grained, subrounded to subangular, low plasticity<br>Stage I cementation, moderate brown (5YR, 4/4) |
| 5 | | | | U | 18 | | | | | |
| | | | | S | 12-27-29 | | | SM | slightly moist<br>hard to very firm | **SILTY SAND**, predominantly fine grained, subrounded to subangular, low plasticity<br>Stage II cementation, moderate brown (5YR, 4/4)<br>note: medium to coarse grained sand from 11' to 12'<br>note: between moderate reddish brown (10R, 4/6) & moderate brown (5YR, 4/4) from 12' to 13'<br>note: grayish orange pink (5YR, 7/2) from 13' to 14'<br>note: some gravel from 16' to 17'<br>note: small diameter roots at 17'6"<br>Stage I cementation below 19'<br>note: medium to coarse grained sand from 22'6" to 23' |
| 10 | | | | S | 14-24-24 | | | | | |
| | | | | S | 16-31-32 | | | | | |
| 15 | | | | S | 32-50/6" | | | | | |
| | | | | S | 17-32-32 | | | | | |
| 20 | | | | S | 17-16-17 | | | | | |
| | | | | S | 25-32-29 | | | | | |
| 25 | | | | | | | | | | Stopped Auger at 22'<br>Stopped Sampler at 23'6" |

BORING LOG 1171122B.GPJ AGRA_PHX.GDT 6/25/01

GROUNDWATER

| DEPTH(ft) | HOUR | DATE |
|---|---|---|
| | none | |
| | | |
| | | |
| | | |

SAMPLE TYPE
A - Drill cuttings; NR - No Recovery
S - 2" O.D. 1.38" I.D. tube sample
U - 3" O.D. 2.42" I.D. tube sample
T - 1" O.D. thin-walled tube sample
D - 2.5" O.D. 1.9" I.D. tube sample
C - California tube sample



LOG OF TEST BORING NO. BH200+00-5

**PROJECT** Vineyard FRS

amec

**JOB NO.** 0-117-001122 Task 2 **DATE** 6/4/01

**LOCATION** Sta. 215+00, 51' R
**RIG TYPE** CME-75
**BORING TYPE** 6 5/8" Hollow Stem Auger
**SURFACE ELEV.**
**DATUM**

| Depth in Feet | Blows Per Foot | Graphical Log | Sample | Sample Type | Blow Count / % Recovery | Dry Density lbs. per Cubic ft. | Moisture Content Percent of Dry Weight | Unified Soil Classification | REMARKS | VISUAL CLASSIFICATION |
|---|---|---|---|---|---|---|---|---|---|---|
| 0 | | | | S | 6-7-7 | | | SM | slightly moist | **SILTY SAND**, trace of clay, trace of fine grained, subrounded to subangular gravel, predominantly fine grained, subrounded to subangular, uncemented, low plasticity, light brown (5YR, 5/6) |
| | | | | | | | | SC | | **CLAYEY SAND**, trace of silt, trace of subrounded to subangular gravel, predominantly fine grained, subrounded to subangular, low to medium plasticity |
| | | | | U | 18 | | | | slightly moist to moist | Stage I cementation, low to medium plasticity, moderate yellowish brown (10YR, 5/4) |
| | | | | | | | | SM | moderately firm | note: no cementation, moderate brown (5YR, 4/4) & no gravel from 2'6" to 4' |
| 5 | | | | S | 4-3-4 | | | | slightly moist | **SILTY SAND**, trace of clay, predominantly fine grained, subrounded to subangular, low plasticity |
| | | | | U | 10 | | | SM-SP | soft | Stage I cementation, light brown (5YR, 5/6) |
| | | | | | | | | | slightly moist to moist | **SILTY SAND TO SAND**, trace to some predominantly fine grained gravel, predominantly medium to coarse grained, subangular to subrounded, low plasticity to nonplastic |
| 10 | | | | S | 6-15-23 | | | SM | soft | Stage I cementation, moderate reddish brown (10R, 4/6) |
| | | | | S | 12-21-22 | | | | slightly moist | **SILTY SAND**, trace of subrounded to subangular gravel, predominantly medium to fine grained, low plasticity to nonplastic |
| | | | | | | | | | very firm to firm | Stage I cementation, light brown (5YR, 5/6) |
| | | | | | | | | | | note: color change to light brown (5YR, 6/4) below 14'6" |
| 15 | | | | S | 7-9-12 | | | | | |
| | | | | | | | | SP | slightly moist | **GRAVELLY SAND**, trace of silt, predominantly fine grained, subrounded to rounded gravel, well graded, subrounded to rounded, nonplastic |
| | | | | S | 20-33-50 | | | | very dense | Stage II cementation, light brown (5YR, 6/4) |
| | | | | | | | | | | note: increased gravel content from 20' to 21'6" |
| 20 | | | | S | 16-29-28 | | | | | |
| | | | | | | | | SM | slightly moist to moist | **SILTY SAND**, trace of fine grained, subrounded to subangular gravel, predominantly medium to fine grained, subrounded to subangular, low plasticity |
| | | | | S | 19-38-44 | | | | very firm to hard | Stage III cementation, light brown (5YR, 6/4) |
| | | | | S | 12-20-22 | | | | | |
| 25 | | | | | | | | | | |

BORING LOG 1171122B.GPJ AGRA_PHX.GDT 6/25/01

GROUNDWATER

| DEPTH(ft) | HOUR | DATE |
|---|---|---|
| | none | |
| | | |
| | | |
| | | |

SAMPLE TYPE
A - Drill cuttings; NR - No Recovery
S - 2" O.D. 1.38" I.D. tube sample
U - 3" O.D. 2.42" I.D. tube sample
T - 1" O.D. thin-walled tube sample
D - 2.5" O.D. 1.9" I.D. tube sample
C - California tube sample



LOG OF TEST BORING NO. BH215+00-1

**PROJECT** Vineyard FRS

**JOB NO.** 0-117-001122 Task 2 **DATE** 6/4/01

**LOCATION** Sta. 215+00, 51' R
**RIG TYPE** CME-75
**BORING TYPE** 6 5/8" Hollow Stem Auger
**SURFACE ELEV.**
**DATUM**

| Depth in Feet | Blows Per Foot | Graphical Log | Sample | Sample Type | Blow Count / % Recovery | Dry Density lbs. per Cubic ft. | Moisture Content Percent of Dry Weight | Unified Soil Classification | REMARKS | VISUAL CLASSIFICATION |
|---|---|---|---|---|---|---|---|---|---|---|
| 25 | | | X | | | | | SM | | **SILTY SAND**, continued<br>Stage II cementation below 24'6"<br>note: small diameter roots at 25', color chage to light brown (5YR, 5/6)<br>Stopped Auger at 24'6"<br>Stopped Sampler at 26' |
| 30 | | | | | | | | | | |
| 35 | | | | | | | | | | |
| 40 | | | | | | | | | | |
| 45 | | | | | | | | | | |
| 50 | | | | | | | | | | |

GROUNDWATER

| | DEPTH(ft) | HOUR | DATE |
|---|---|---|---|
| ▽ | | none | |
| ▼ | | | |
| ▽ | | | |
| ▼ | | | |

SAMPLE TYPE
A - Drill cuttings; NR - No Recovery
S - 2" O.D. 1.38" I.D. tube sample
U - 3" O.D. 2.42" I.D. tube sample
T - 1" O.D. thin-walled tube sample
D - 2.5" O.D. 1.9" I.D. tube sample
C - California tube sample



LOG OF TEST BORING NO. BH215+00-1

BORING LOG 1171122B.GPJ AGRA_PHX.GDT 6/25/01

**PROJECT** Vineyard FRS

amec

**JOB NO.** 0-117-001122 Task 2 **DATE** 6/21/01

**LOCATION** Sta. 215+00, 31' R
**RIG TYPE** CME-75
**BORING TYPE** 6 5/8" Hollow Stem Auger
**SURFACE ELEV.**
**DATUM**

| Depth in Feet | Drill Rate Min/ft. | Graphical Log | Sample | Sample Type | Blow Count / % Recovery | Dry Density lbs. per Cubic ft. | Moisture Content Percent of Dry Weight | Unified Soil Classification | REMARKS | VISUAL CLASSIFICATION |
|---|---|---|---|---|---|---|---|---|---|---|
| 0 | | | | | | | | SC | slightly moist<br>very firm to hard | Man-made FILL<br>**CLAYEY SAND**, some well graded gravel on the ground surface, predominantly medium to coarse grained sand, subangular to subrounded, trace of lime, medium plasticity, light brown (5YR, 6/4) |
| | | | | S | 9-13-21 | | | | | |
| | | | | S | 29-50/5 1/2" | | | | | note: moderate brown (5YR, 4/4) at 8' |
| | | | | | | | | | | note: occaisonal thin lime-rich lenses below 7'6" & reworked nodules |
| 5 | | | | S | 31-50/4" | | | | | note: considerable coarse grained sand particles & some fine grained gravel below 14'6" |
| | | | | S | 20-36-30 | | | | | |
| 10 | | | | S | 20-31-50 | | | | | |
| | | | | U | 76 | | | | | |
| 15 | | | | U | 53 | | | | | |
| | | | | U | 57 | | | | | |
| | | | | | | | | CL | slightly moist<br>hard | NATIVE<br>**SANDY CLAY**, predominantly medium to medium grained sand, medium plasticity<br>Stage II cementation (considerable small, soft nodules, trace of short filaments), light brown (5YR, 6/4) |
| 20 | | | | S | 20-35-42 | | | SM | slightly moist<br>hard | **SILTY SAND**, trace to some predominantly fine grained gravel, predominantly medium to coarse grained, subangular to subrounded, low plasticity<br>Stage II cementation (considerable CaCO3 coating on gravel borders), light brown (5YR, 6/4) |
| | | | | S | 21-45-35 | | | SC | | |
| 25 | | | | S | 15-50/6" | | | | | |

BORING LOG 1171122B.GPJ AGRA_PHX.GDT 6/28/01

GROUNDWATER

| DEPTH(ft) | HOUR | DATE |
|---|---|---|
| | none | |
| | | |
| | | |
| | | |

SAMPLE TYPE
A - Drill cuttings; NR - No Recovery
S - 2" O.D. 1.38" I.D. tube sample
U - 3" O.D. 2.42" I.D. tube sample
T - 1" O.D. thin-walled tube sample
D - 2.5" O.D. 1.9" I.D. tube sample
C - California tube sample



LOG OF TEST BORING NO. BH215+00-2

**PROJECT** Vineyard FRS

amec

**JOB NO.** 0-117-001122 Task 2 **DATE** 6/21/01

**LOCATION** Sta. 215+00, 31' R
**RIG TYPE** CME-75
**BORING TYPE** 6 5/8" Hollow Stem Auger
**SURFACE ELEV.**
**DATUM**

| Depth in Feet | Drill Rate Min/ft. | Graphical Log | Sample | Sample Type | Blow Count / % Recovery | Dry Density lbs. per Cubic ft. | Moisture Content Percent of Dry Weight | Unified Soil Classification | REMARKS | VISUAL CLASSIFICATION |
|---|---|---|---|---|---|---|---|---|---|---|
| 25 | | | X | | | | | SC | slightly moist<br>hard | **CLAYEY SAND**, considerable silt in zones, trace of predominantly fine grained gravel & occasional coarse grained gravel lenses, predominantly medium to coarse grained sand, subangular to subrounded, medium plasticity<br><br>Stage II cementation (some filaments & soft nodules, strong HCL reaction), light brown (5YR, 5/6) |
| | | | X | S | 15-23-26 | | | | | |
| 30 | | | | U | 40 | | | CL with SC lenses | slightly moist to moist<br>very firm to hard | **SILTY CLAY**, trace of predominantly medium grained sand, medium plasticity<br><br>Stage II cementation, (some CaCO3 filaments, strong HCL reaction), light brown (5YR, 5/6)<br><br>note: trace of predominantly fine grained gravel below 27'6"<br><br>note: thin clayey sand lenses below 29'6"<br><br>Stage III cementation (strong HCL reaction, consideable CaCO3, soft nodules) below 32'6"<br><br>note: trace of predominantly fine grained gravel below 34'6" |
| | | | X | S | 17-27-25 | | | | | |
| 35 | | | X | S | 10-20-30 | | | | | |
| 40 | | | | | | | | | | Stopped Auger at 34'6"<br>Stopped Sampler at 36' |
| 45 | | | | | | | | | | |
| 50 | | | | | | | | | | |

GROUNDWATER

| | DEPTH(ft) | HOUR | DATE |
|---|---|---|---|
| ▽ | | none | |
| ▼ | | | |
| ▽ | | | |
| ▼ | | | |

SAMPLE TYPE
A - Drill cuttings; NR - No Recovery
S - 2" O.D. 1.38" I.D. tube sample
U - 3" O.D. 2.42" I.D. tube sample
T - 1" O.D. thin-walled tube sample
D - 2.5" O.D. 1.9" I.D. tube sample
C - California tube sample



LOG OF TEST BORING NO. BH215+00-2

BORING LOG 1171122B.GPJ AGRA_PHX.GDT 6/28/01

**PROJECT** Vineyard FRS

amec

**JOB NO.** 0-117-001122 Task 2 **DATE** 6/14/01

**LOCATION** Sta. 215+00, 3' R
**RIG TYPE** CME-75
**BORING TYPE** 6 5/8" Hollow Stem Auger
**SURFACE ELEV.**
**DATUM**

| Depth in Feet | Drill Rate Min/ft. | Graphical Log | Sample | Sample Type | Blow Count / % Recovery | Dry Density lbs. per Cubic ft. | Moisture Content Percent of Dry Weight | Unified Soil Classification | REMARKS | VISUAL CLASSIFICATION |
|---|---|---|---|---|---|---|---|---|---|---|
| 0 | | | | | | | | SC | slightly moist<br>very firm to hard | Man-made FILL (crest of dam)<br>**CLAYEY SAND**, some silt, considerable predominantly coarse grained gravel on the ground surface, predominantly fine to medium grained, subangular to subrounded, weak HCL reaction, medium plasticity<br>note: moderate brown (5YR, 4/4)<br>note: some silty sand lenses below 4'6"<br>note: slight moisture increase at 7'6"<br>note: increase in sand grain-size below 7'6" with some coarse grained particles<br>note: trace of fine grained gravel below 14'6", considerable coarse grained sand below 17'6" |
| | | | | S | 13-15-19 | | | | | |
| | | | | S | 16-38-50/5 1/2" | | | | | |
| 5 | | | | S | 22-27-37 | | | | | |
| | | | | U | 60 | | | | | |
| 10 | | | | S | 16-16-21 | | | | | |
| | | | | U | 69 | | | | | |
| 15 | | | | S | 12-20-24 | | | | | |
| | | | | U | 40 | | | | | |
| | | | | | | | | SC | slightly moist<br>firm to hard | NATIVE<br>**CLAYEY SAND**, some silt, predominantly fine to medium grained, subangular to subrounded, medium plasticity<br>Stage II cementation (weak HCL reaction), light brown (5YR, 5/6) at 20'<br>note: moderate to strong HCL reaction below 22'6" |
| 20 | | | | S | 21-17-12 | | | | | |
| | | | | S | 11-16-32 | | | | | |
| 25 | | | | U | 39 | | | | | |

BORING LOG 1171122B.GPJ AGRA_PHX.GDT 6/28/01

GROUNDWATER

| DEPTH(ft) | HOUR | DATE |
|---|---|---|
| | none | |
| | | |
| | | |
| | | |

SAMPLE TYPE
A - Drill cuttings; NR - No Recovery
S - 2" O.D. 1.38" I.D. tube sample
U - 3" O.D. 2.42" I.D. tube sample
T - 1" O.D. thin-walled tube sample
D - 2.5" O.D. 1.9" I.D. tube sample
C - California tube sample



LOG OF TEST BORING NO. BH215+00-3

**PROJECT** Vineyard FRS

amec

**JOB NO.** 0-117-001122 Task 2 **DATE** 6/14/01

**LOCATION** Sta. 215+00, 3' R
**RIG TYPE** CME-75
**BORING TYPE** 6 5/8" Hollow Stem Auger
**SURFACE ELEV.**
**DATUM**

| Depth in Feet | Drill Rate Min/ft. | Graphical Log | Sample | Sample Type | Blow Count / % Recovery | Dry Density lbs. per Cubic ft. | Moisture Content Percent of Dry Weight | Unified Soil Classification | REMARKS | VISUAL CLASSIFICATION |
|---|---|---|---|---|---|---|---|---|---|---|
| 25 | | | | | | | | SC | slightly moist<br>very firm to hard | **CLAYEY SAND**, considerable silt with some silty sand lenses, predominantly fine grained sand, subangular to subrounded, low to medium plasticity<br><br>Stage II cementation (moderate to strong HCL reaction, considerable small diameter nodules, thin CaCO3 lenses), light brown (5YR, 5/6) below 27'6" |
| | | | | S | 9-19-18 | | | | | |
| 30 | | | | S | 16-38-28 | | | | | |
| | | | | S | 11-30-36 | | | SM | slightly moist<br>hard | **SILTY SAND**, trace to some clay & trace of fine grained gravel, predominantly fine to medium grained sand, subangular to subrounded, low plasticity<br><br>Stage II cementation (moderate to strong HCL reaction in lenses), light brown (5YR, 6/4 with 5YR, 5/6 lenses) |
| 35 | | | | S | 21-39-50/5 1/2" | | | CL | slightly moist<br>very firm to hard | **SILTY TO SANDY CLAY**, predominantly fine grained sand, medium plasticity<br><br>Stage III cementation (nodules & considerable CaCO3 filaments, strong HCL reaction), light brown (5YR, 6/4), some black manganese-oxide staining |
| | | | | S | 16-26-28 | | | | | |
| 40 | | | | S | 14-20-24 | | | | | |
| 45 | | | | | | | | | | Stopped Auger at 39'6"<br>Stopped Sampler at 41' |
| 50 | | | | | | | | | | |

GROUNDWATER

| DEPTH(ft) | HOUR | DATE |
|---|---|---|
| | none | |
| | | |
| | | |
| | | |

SAMPLE TYPE
A - Drill cuttings; NR - No Recovery
S - 2" O.D. 1.38" I.D. tube sample
U - 3" O.D. 2.42" I.D. tube sample
T - 1" O.D. thin-walled tube sample
D - 2.5" O.D. 1.9" I.D. tube sample
C - California tube sample



LOG OF TEST BORING NO. BH215+00-3

RING LOG 1171122B.GPJ AGRA_PHX.GDT 6/28/01

**PROJECT** Vineyard FRS

amec

**JOB NO.** 0-117-001122 Task 2 **DATE** 6/15/01

**LOCATION** Sta. 215+00
**RIG TYPE** CME-75
**BORING TYPE** 6 5/8" Hollow Stem Auger
**SURFACE ELEV.**
**DATUM**

| Depth in Feet | Drill Rate Min/ft. | Graphical Log | Sample | Sample Type | Blow Count / % Recovery | Dry Density lbs. per Cubic ft. | Moisture Content Percent of Dry Weight | Unified Soil Classification | REMARKS | VISUAL CLASSIFICATION |
|---|---|---|---|---|---|---|---|---|---|---|
| 0 | | | | | | | | ML | slightly moist<br>firm | **SANDY SILT**, trace of gravel, predominantly fine to medium grained sand, nonplastic to low plasticity<br>Stage II cementation (weak HCL reaction), light brown (5YR, 6/4) from 0 to 2'6"<br>note: grading to clayey silt below 2'6"<br>Stage II cementation (moderate HCL reaction, some short filaments), light brown (5YR, 6/4) below 2'6" |
| | | | | S | 7-10-11 | | | | | |
| | | | | U | 23 | | | | | |
| 5 | | | | U | 25 | | | SM | slightly moist<br>firm | **SILTY SAND**, some clay, well graded in lenses, subangular to subrounded, low plasticity<br>Stage II cementation (moderate HCL reaction), light brown (5YR, 5/6) |
| | | | | S | 13-19-23 | | | ML | slightly moist<br>very firm | **CLAYEY SILT**, trace of fine grained sand, medium plasticity<br>Stage II cementation (some filaments, moderate HCL reaction in zones), light brown (5YR, 5/6) |
| 10 | | | | S | 14-28-22 | | | CL | slightly moist<br>hard | **SANDY CLAY**, predominantly fine to medium grained sand, medium plasticity<br>Stage III cementation (considerable filaments, strong HCL reaction), light to moderate brown (5YR, 5/6 to 5YR, 4/4) |
| | | | | U | 100/8" | | | | | |
| 15 | | | | U | 62 | | | | | |
| | | | | | | | | SM/GM | slightly moist<br>medium dense to dense | **SILTY SAND & GRAVEL**, predominantly coarse grained gravel from 16' to 17' & decreasing particle size below 17', well graded sand, subangular to subrounded, uncemented to weakly cemented (weak HCL reaction), nonplastic, light brown (5YR, 6/4) |
| | | | | S | 24-19-23 | | | | | |
| 20 | | | | S | 15-20-23 | | | | | |
| | | | | S | 42-50/4" | | | CL | slightly moist<br>hard | **SANDY CLAY**, predominantly fine to medium grained sand, medium plasticity<br>Stage III cementation (nodules, strong HCL reaction) |
| | | | | | 27-50/4" | | | | | |
| | | | | S | | | | | | |
| 25 | | | | | | | | | | |

GROUNDWATER

| DEPTH(ft) | HOUR | DATE |
|---|---|---|
| | none | |
| | | |
| | | |
| | | |

SAMPLE TYPE
A - Drill cuttings; NR - No Recovery
S - 2" O.D. 1.38" I.D. tube sample
U - 3" O.D. 2.42" I.D. tube sample
T - 1" O.D. thin-walled tube sample
D - 2.5" O.D. 1.9" I.D. tube sample
C - 2.5" O.D. 2.0" I.D. tube sample



LOG OF TEST BORING NO. BH215+00-4

BORING LOG 1171122B.GPJ AGRA_PHX.GDT 6/29/01

**PROJECT** Vineyard FRS

amec

**JOB NO.** 0-117-001122 Task 2 **DATE** 6/15/01

**LOCATION** Sta. 215+00
**RIG TYPE** CME-75
**BORING TYPE** 6 5/8" Hollow Stem Auger
**SURFACE ELEV.**
**DATUM**

| Depth in Feet | Drill Rate Min/ft. | Graphical Log | Sample | Sample Type | Blow Count / % Recovery | Dry Density lbs. per Cubic ft. | Moisture Content Percent of Dry Weight | Unified Soil Classification | REMARKS | VISUAL CLASSIFICATION |
|---|---|---|---|---|---|---|---|---|---|---|
| 25 | | | | | | | | | | |
| | | | | | | | | | | Stopped Auger at 24'6" |
| | | | | | | | | | | Sampler refused at 25'4" |
| 30 | | | | | | | | | | |
| 35 | | | | | | | | | | |
| 40 | | | | | | | | | | |
| 45 | | | | | | | | | | |
| 50 | | | | | | | | | | |

BORING LOG 1171122B.GPJ AGRA_PHX.GDT 6/29/01

GROUNDWATER

| DEPTH(ft) | HOUR | DATE |
|---|---|---|
| | none | |
| | | |
| | | |
| | | |

SAMPLE TYPE
A - Drill cuttings; NR - No Recovery
S - 2" O.D. 1.38" I.D. tube sample
U - 3" O.D. 2.42" I.D. tube sample
T - 1" O.D. thin-walled tube sample
D - 2.5" O.D. 1.9" I.D. tube sample
C - 2.5" O.D. 2.0" I.D. tube sample



LOG OF TEST BORING NO. BH215+00-4

**PROJECT** Vineyard FRS

amec

**JOB NO.** 0-117-001122 Task 2 **DATE** 6/4/01

**LOCATION** Sta. 215+07, 71' R
**RIG TYPE** CME-75
**BORING TYPE** 6 5/8" Hollow Stem Auger
**SURFACE ELEV.**
**DATUM**

| Depth in Feet | Drill Rate Min/ft. | Graphical Log | Sample | Sample Type | Blow Count / % Recovery | Dry Density lbs. per Cubic ft. | Moisture Content Percent of Dry Weight | Unified Soil Classification | REMARKS | VISUAL CLASSIFICATION |
|---|---|---|---|---|---|---|---|---|---|---|
| 0 | | | | S | 6-8-8 | | | SC | slightly moist<br>firm | **CLAYEY SAND**, trace of silt, predominantly medium to fine grained, subrounded to subangular, uncemented, low plasticity, moderate brown (5YR, 4/4)<br>note: small diameter roots in upper 0.5' |
| | | | | | | | | SM | slightly moist<br>soft to moderately firm | **SILTY SAND**, trace of clay, predominantly fine grained, subrounded to subangular<br>Stage I cementation, moderate brown (5YR, 4/4)<br>note: small diameter roots<br>note: trace of fine grained, subrounded to subangular gravel below 4' |
| | | | | U | 10 | | | | | |
| 5 | | | | S | 3-4-3 | | | | | |
| | | | | U | 19 | | | | | |
| 10 | | | | U | 27 | | | SM | slightly moist<br>firm | **SILTY SAND**, trace of fine grained subrounded to subangular gravel, predominantly medium to fine grained, subrounded to subangular, low plasticity<br>Stage I cementation, moderate light brown (5YR, 5/6)<br>note: gravel layer from 12' to 12'6" |
| | | | | S | 11-10-12 | | | | | |
| 15 | | | | S | 8-13-15 | | | SM | slightly moist<br>firm | **SILTY SAND**, predominantly medium to fine grained, subrounded to subangular, low plasticity<br>Stage II cementation, moderate brown (5YR, 4/4) |
| 20 | | | | S | 11-21-30 | | | SM | slightly moist<br>hard to very firm | **SILTY SAND**, occasional cobble, trace to some predominantly fine grained, subangular to subrounded gravel, well graded, subrounded to subangular, low plasticity<br>Stage II cementation, light brown (5YR, 6/4)<br>note: small diameter roots<br>Stage I cementation characteristics from 20' to 22' |
| | | | | S | 21-23-28 | | | | | |
| | | | | S | 23-28-41 | | | | | |
| 25 | | | | S | 13-17-29 | | | | | |

GROUNDWATER

| DEPTH(ft) | HOUR | DATE |
|---|---|---|
| | none | |
| | | |
| | | |
| | | |

SAMPLE TYPE
A - Drill cuttings; NR - No Recovery
S - 2" O.D. 1.38" I.D. tube sample
U - 3" O.D. 2.42" I.D. tube sample
T - 1" O.D. thin-walled tube sample
D - 2.5" O.D. 1.9" I.D. tube sample
C - 2.5" O.D. 2.0" I.D. tube sample



LOG OF TEST BORING NO. BH215+00-5

BORING LOG 1171122B.GPJ AGRA_PHX.GDT 6/29/01

**PROJECT** Vineyard FRS

**JOB NO.** 0-117-001122 Task 2 **DATE** 6/4/01

**LOCATION** Sta. 215+07, 71' R
**RIG TYPE** CME-75
**BORING TYPE** 6 5/8" Hollow Stem Auger
**SURFACE ELEV.**
**DATUM**

| Depth in Feet | Drill Rate Min/ft. | Graphical Log | Sample | Sample Type | Blow Count / % Recovery | Dry Density lbs. per Cubic ft. | Moisture Content Percent of Dry Weight | Unified Soil Classification | REMARKS | VISUAL CLASSIFICATION |
|---|---|---|---|---|---|---|---|---|---|---|
| 25 | | | X | | | | | SM | | **SILTY SAND**, continued |
| | | | | | | | | | | Stopped Auger at 24'6"<br>Stopped Sampler at 26' |
| 30 | | | | | | | | | | |
| 35 | | | | | | | | | | |
| 40 | | | | | | | | | | |
| 45 | | | | | | | | | | |
| 50 | | | | | | | | | | |

GROUNDWATER

| DEPTH(ft) | HOUR | DATE |
|---|---|---|
| | none | |
| | | |
| | | |
| | | |

SAMPLE TYPE
A - Drill cuttings; NR - No Recovery
S - 2" O.D. 1.38" I.D. tube sample
U - 3" O.D. 2.42" I.D. tube sample
T - 1" O.D. thin-walled tube sample
D - 2.5" O.D. 1.9" I.D. tube sample
C - 2.5" O.D. 2.0" I.D. tube sample



LOG OF TEST BORING NO. BH215+00-5

BORING LOG 1171122B.GPJ AGRA_PHX.GDT 6/29/01

# MEMORANDUM

**DATE:** 9 July 2001

**TO:** Tom Renckly, FCDMC.
Larry Lambert, FCDMC
Bob Eichenger, Kimley-Horn and Associates, Inc.
Larry Von Thun, Consulting Engineer
Jim Scott, URS Corporation
Ken Euge, Geological Consultants

**FROM:** George Beckwith, FCDMC

**RE: SUMMARY & ANALYSIS OF GEOTECHNICAL DATA, VINYARD FRS, RISK ASSESSMENT/RISK ANALYSIS, STRUCTURES ASSESSMENT DAM SAFETY PROGRAM, FLOOD CONTROL DISTRICT OF MARICOPA COUNTY.**

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## 1.0 INTRODUCTION

Presented in this memorandum is an assessment of the geotechnical properties of the Vineyard FRS embankment and foundation and of the character and intensity of the cracking within the embankment and upper part of the foundation. The work includes analysis of the filtration and related properties of the Central Filter. The memorandum was prepared to serve as a resource document for the Risk Analysis Team in the Risk Assessment on 11 July 2001.

Information reviewed includes: a) geotechnical data developed by the U. S. Soil Conservation Service (SCS) during the original planning, design and construction of the project in the 1960's; b) later work by the SCS (now the Natural Resources Conservation Service or NRCS) and it's consultants for evaluation of cracking and design, construction and evaluation of the Central Drain in the 1980's and; c) present work by AMEC.

Documents submitted separately to the Risk Analysis Team by AMEC that were considered in this evaluation include:

- The draft of the report of Investigation of Ground Subsidence and Earth Fissuring (Volume I) dated 2 July 2001.
- Maps showing active and potentially active faults and contours of peak horizontal ground accelerations prepared by AMEC in an Investigation of Seismic Hazard that is in progress.
- Cross sections of the Vineyard FRS at Stations 124+00, 186+00, 198+00 and 215+00 where an extensive geotechnical investigation was performed.

# FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

PROJECT VINEYARD FRS PAGE 1 OF 4

DETAIL FLOW THROUGH CRACKS COMPUTED GHB DATE 7.18.01

CHECKED BY _____ DATE _____

COMPUTATION OF DISCHARGE PER LINEAL FT. & SEEPAGE VELOCITY

| b (ft.) | e (in) | d/h | m | k (ft./sec) | h (ft) | q (cfs) | n (porosity) | V (ft./sec) |
|---|---|---|---|---|---|---|---|---|
| 6.0 | 0.04 | 2.4 | 0.29 | 0.002 | 20 | 0.012 | 0.0005 | 24 |
| | | 5.53 | 0.09 | 0.002 | 11.3 | 0.002 | 0.0005 | 4 |
| 6.0 | 0.20 | 2.4 | 0.29 | 0.328 | 20 | 1.902 | 0.0028 | 679 |
| | | 5.53 | 0.09 | 0.328 | 11.3 | 0.333 | 0.0028 | 118 |
| 6.0 | 0.39 | 2.4 | 0.29 | 2.62 | 20 | 15.20 | 0.0054 | 2814 |
| | | 5.53 | 0.09 | 2.62 | 11.3 | 2.66 | 0.0054 | 492 |
| 12.0 | 0.04 | 2.4 | 0.29 | 0.0013 | 20 | 0.003 | 0.0002 | 40 |
| | | 5.53 | 0.09 | 0.0013 | 11.3 | 0.001 | 0.0002 | 5 |
| 12.0 | 0.20 | 2.4 | 0.29 | 0.168 | 20 | 0.974 | 0.0013 | 749 |
| | | 5.53 | 0.09 | 0.168 | 11.3 | 0.171 | 0.0013 | 131 |
| 12.0 | 0.39 | 2.4 | 0.29 | 1.31 | 20 | 7.60 | 0.0027 | 2814 |
| | | 5.53 | 0.09 | 1.31 | 11.3 | 1.33 | 0.0027 | 492 |

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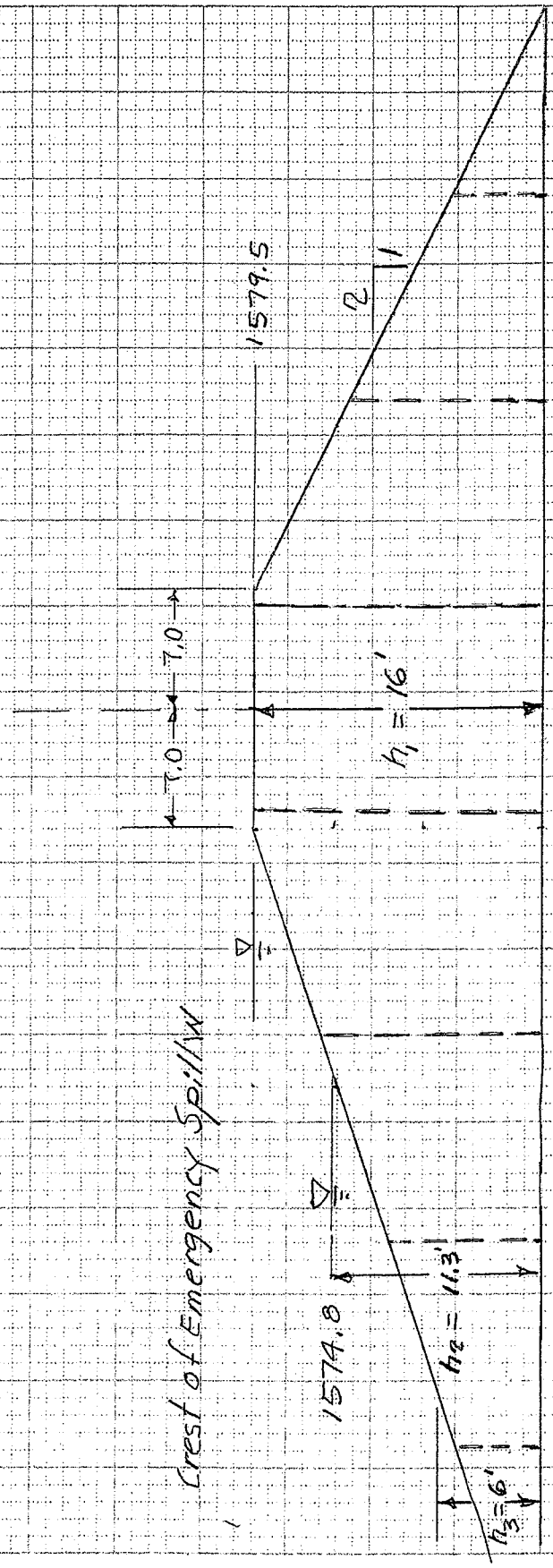
# FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

PROJECT VINEYARD FRS PAGE 1 OF 4

DETAIL FLOW THROUGH CRACKS COMPUTED GHB DATE 7.18.0

CHECKED BY _____ DATE _____




$A = 64 \times 20 = 1280$ sq. ft.

$V_T = 64 \times 20 \times 12 = 15360$ cu. ft.

For $b = 12'$; $e = 0.04'' = 0.0033'$ $V_1 = 4.2$ cu. ft. $n = 0.0002$

$e = 0.20'' = 0.0166'$ $V_2 = 21.2$ cu. ft. $n = 0.0013$

$e = 0.39'' = 0.0325$ $V_3 = 41.6$ cu. ft. $n = 0.0027$

For $b = 6.0'$ $n = 0.0005$ $e = 0.0033'$

$n = 0.0028$ $e = 0.0166'$

$n = 0.0054$ $e = 0.0325$

# FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

PROJECT VINEYARD FRS PAGE 2 OF 4

DETAIL FLOW THROUGH CRACKS COMPUTED GHB DATE 7.18.01

CHECKED BY _____ DATE _____

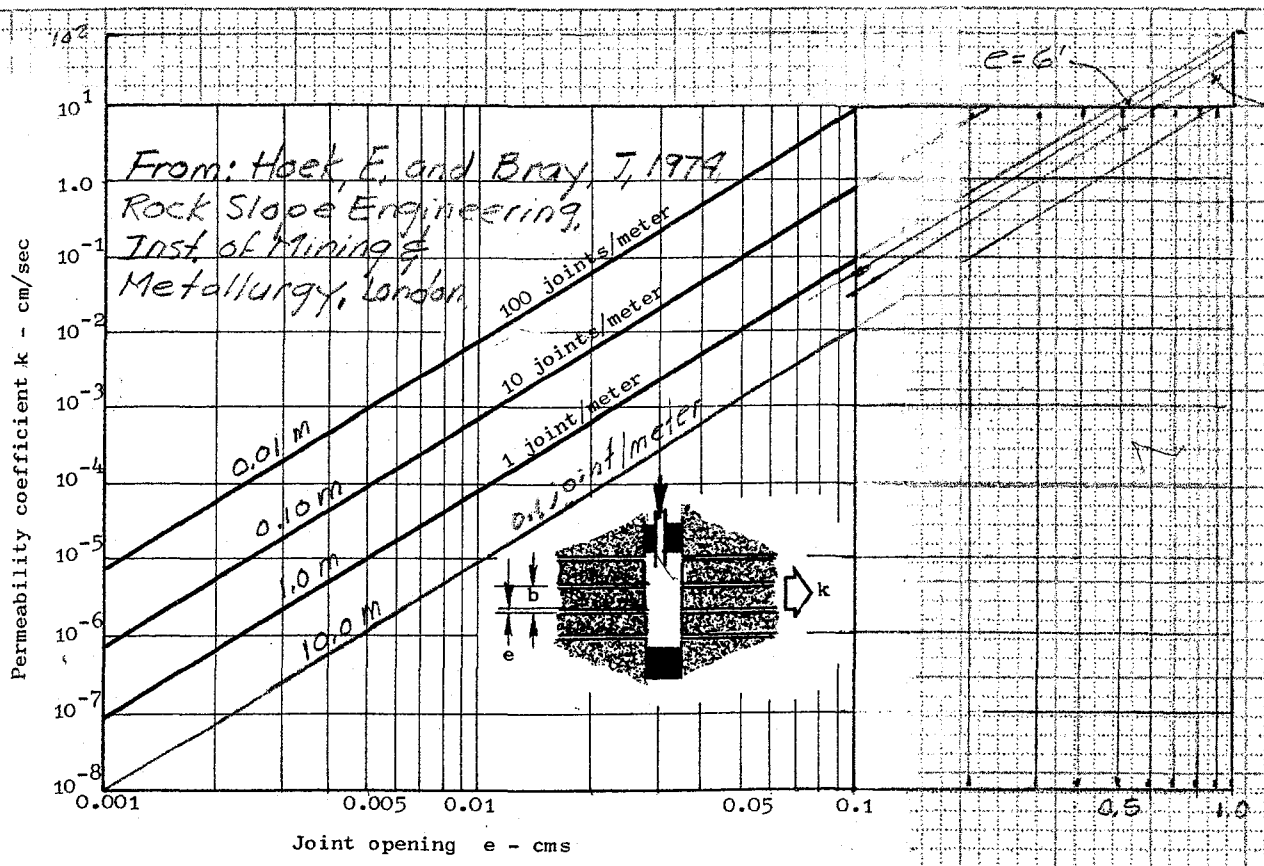



Figure 52 : Influence of joint opening e and joint spacing b on the permeability coefficient k in the direction of a set of smooth parallel joints in a rock mass.

e = 0.1 cm = 0.04 in
e = 0.5 cm = 0.20 in
e = 1.0 cm = 0.39 in

b = 12.0 ft = 3.65 m
b = 6.0 ft = 1.82 m

| e (in) | b (ft) | k cm/sec | k feet/sec |
|---|---|---|---|
| 0.04 | 6.0 | 0.07 | 0.002 |
| 0.20 | 6.0 | 10 | 0.328 |
| 0.39 | 6.0 | 80 | 2.62 |
| 0.04 | 12.0 | 0.04 | 0.0013 |
| 0.20 | 12.0 | 6 | 0.168 |
| 0.39 | 12.0 | 40 | 1.31 |

# FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

PROJECT VINEYARD FRS PAGE 3 OF 4

DETAIL FLOW COMPUTED GHB DATE 7.18.01

CHECKED BY DATE

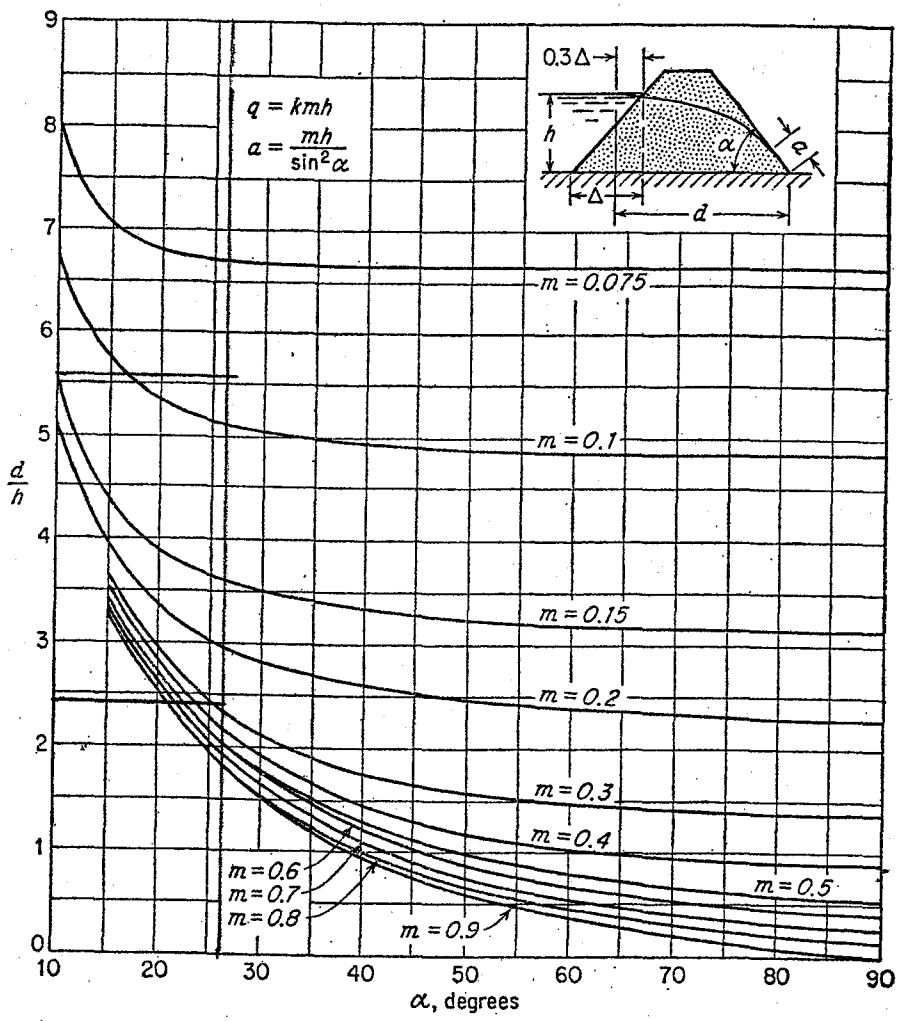



FIG. 2-16. (*Based on Gilboy* [42].)

**Example 2-2.** Determine the quantity of seepage for 100 ft of the dam section shown in Fig. 2-17. $k = 0.002$ ft/min.

From: Harr, M.E, 1962, Groundwater and Seepage, McGraw Hill Book Company, New York

For 2:1 slope: $\alpha = 26.6°$

For Dam crest: $\frac{d}{h} = \frac{48}{20} = 2.4$

For Emergency Spillway: $\frac{d}{h} = \frac{62.5}{11.3} = 5.53$

For 6.0' storage: $\frac{81.5}{6} = 13.5$

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- Logs of test borings and pits made at the above cross sections and of single borings drilled at Stations 139+00 and 170+00.
- Limited test data from the AMEC borings and test pits.
- Fourteen profile sheets showing data on the frequency, length and width of the transverse cracks through the dam logged by the SCS during the construction of the Central Filter.

## 2.0 CHARACTERISTICS OF EMBANKMENT & FOUNDATION.

### 2.1 Geomorphology & Geotechnical Profile

The dam site is underlain by an apron of highly stratified and cross-bedded soils formed by coalescing alluvial fans which extend from the front of the Superstition Mountains across the reservoir and dam site. This surface layer, which varied from about 7.5 to 16.5 feet in thickness in the AMEC borings, is underlain by Late Pleistocene alluvium.

Deposition of the low-density alluvial fan soils occurred during the Holocene era (during about the past 11,000 years) in semi-arid climatic conditions similar to the present. The deposition involved the complex hydrological processes of alluvial fans including sediment transport in channels, general flooding of the fans creating avulsion of the braided channels and debris flows or mudflows associated with large, infrequent floods (Blissenback, 1954; Beckwith and Hansen, 1989; Harvey, 1992). These processes explain the lenticular inclusions, high degree of stratification and erratic variation of soil types within the surface layer. As indicated by the detailed information presented in Section 2.3, the soils are predominately sandy clays and clayey sands, but contain lessor amounts of silty sands, sandy silts and sandy gravels.

The surface deposit possesses a high potential for collapse settlement. Deposits of this kind typically undergo from 2 to 6 percent of vertical strain upon wetting under their self-weight or low superimposed loads (Beckwith and Hansen, 1989). Even the sandy gravels often have the potential for appreciable collapse settlements (Rollins et al, 1994).

The underlying Late Pleistocene alluviam is composed of dense or hard, highly stratified silty sands, clayey sands, gravelly sands, sandy clays and silty clays. In contrast to the surface layer, these soils possesses substantial calcareous cementation. Experience has demonstrated that soils with this degree of cementation are essentially incompressible from the viewpoint of contribution to settlement of low embankments (Beckwith and Hansen, 1982).

There is a sharp contrast between the degree of cementation of the Holocene surface soils and underlying alluvium. The former is uncemented or has discontinuous Stage I calcareous cementation while the underlying Late Pleistocene soils possess Stage II or III cementation. The degree of cementation is noted on the AMEC logs using standard soil science terminology. This information is valuable for distinguishing between the two

units, because both have scattered gravel particles which distort standard penetration blow-counts.

## 2.2 Embankment Geometry

The four AMEC cross sections illustrate the geometry of the 5.46-mile long, 16.5-foot homogeneous embankment which was constructed almost entirely of soils from the surface layer. The embankment soils came from both the borrow ditch upstream of the dam and from sub-excavation of the foundation soils. Important features include:

- An upstream cutoff trench that appears to extend through the surface layer into the underlying cemented deposits along much of the alignment.
- A "shelf", extending from the upstream cutoff trench to somewhat downstream of the dam centerline. The specifications called for the bottom of the shelf to be a minimum of 4 feet below original natural grade.
- A 3-foot wide Central Filter, that extends to between 19 to 21 feet below the dam crest along most of the alignment, or to about 2.5 to 4.5 feet below the original natural grade. The filter was constructed in the 1980's after extensive cracking of the embankment was discovered and evaluated.
- An upstream slope of 3:1 (horizontal to vertical) and a downstream slope of 2:1.
- A crest width of 14 feet.

Exploratory borings were drilled by AMEC in June 2001 at locations where longitudinal cracks were noted at about the mid-point of the upstream slope. Borings were drilled upstream of the dam, at the upstream toe, on the upstream slope at the centerline of the upstream cutoff trench, on the dam crest just upstream of the Central Filter and at the downstream toe. Boring locations and depths were selected to investigate the cause or causes of the longitudinal cracks, the extent and characteristics of the Holocene surface layer, the characteristics of the underlying Late Pleistocene alluvium and the extent that the embankment extends below original natural grade.

The borings confirmed that sub-excavation of upstream cutoff trench and shelf area was performed essentially as indicated by the as-built plans. The cutoff and shelf extended to the Late Pleistocene alluvium at Stations 124+00 and 215+00. At Station 186+00, there appears to be 10 feet of the collapsible surface layer beneath the cutoff and 3 feet beneath the shelf. There appears to be 5 feet of the surface layer beneath the cutoff at Station 200+00. Thus, some thickness of the Holocene surface layer was present beneath the embankment in 3 of the 8 borings drilled through the cutoff and shelf.

Wide variation in the thickness of the surface layer over short distances suggests that the top of the Late Pleistocene alluvium was an irregular erosion surface. The direction of the drainage channels may have been different than the present drainage system.

## 3.2 Geotechnical Properties.

Figure 1 shows the range of gradation of the embankment soils and the gradation limits of the Central Filter. The Atterberg limits tests on the embankment soils are plotted on the plasticity chart on Figure 2. Eight Atterberg limits tests for which the samples were determined to be non-plastic samples are not plotted on Figure 2. Thus, the embankment soils range from non plastic or slightly plastic sandy silts and silty fine sands, to medium plastic sandy clays and clayey sands. The more clayey soils predominate. Because the embankment was constructed almost entirely with soils from shallow excavations beneath or upstream of the dam, the test data shown on Figures 1 and 2 are representative of the properties of the Holocene surface layer.

Project specifications called for the embankment to be compacted to a minimum of 95 percent of maximum dry density (MDD) as determined by ASTM D698 with the moisture content during compaction being maintained within two percent of the optimum moisture content (OMC). Figure 3 shows the relationship percent of maximum dry density of the field density tests versus elevation. Figure 4 shows the relationship between variation of field moisture content of the embankment from the OMC and elevation.

Based on statistical analysis of field compaction testing of finer grained soils on 72 BOR dams, standards deviations of 3.0 pcf for variation of percent compaction and 1.5 for variation of field moisture content from the OMC were recommended for earthwork quality control (Hilf, 1991). As indicated on Figures 3 and 4, standard deviations for the Vineyard FRS compaction test are 2.5 pcf and 1.3 percent moisture in terms of dry weight. The data shows that good quality control practices were followed and that a high quality, relatively uniform embankment was attained.

Figure 5 shows the relationship between degree of saturation and depth below the dam surface. Both data from the 1967-68 construction testing and the Central Drain construction in 1984-85 are included on Figure 5. Figure 6 shows the relationship between moisture content and depth below the surface. Test data from the original construction testing in 1967-68, construction of the Central Drain in 184-85 and the present AMEC geotechnical investigation are shown on Figure. The data are presented in terms of depth below the embankment surface in Figures 5 and 6 because depth influences the processes of unsaturated flow, seasonal moisture variation near the surface, deep infiltration and drying shrinkage at depth (Morris et al, 1992; Fredlund and Rahardjo, 1993).

As indicated by Figure 5 and 6, a large amount of drying of the embankment soils has taken place between 1967 and 2001. It appears that the average amount of drying is approximately 6 or 7 percent. Overall, the 1984-85 moisture contents appear to fall between the 1967-68 and 2001 values suggesting that the drying process has continued over the entire 33 year period.

To evaluate the distribution of soil types within the embankment, the location of field density tests with four different ranges of MDD are plotted on Figure 7 in terms of station and elevation. Atterberg limit tests were not run on the soils for which the ASTM

D698 moisture-density relations tests (Proctors) were performed for construction. However, Atterberg limits were performed for moisture-density relations tests run during design studies. The data shows that the higher MDD's correspond to mixtures of well graded sand and clay while the lower MDD's correspond to non plastic or slightly plastic fine sand-silt mixtures. Figure 7 indicates that the embankment is relative uniform with perhaps a slight overall tendency toward less plastic lower-density soils in the southern half.

## 3.0 ANALYSIS OF CENTRAL FILTER

The results of filter analysis for the Central Filter is described in this section. The filter was originally designed as an element in a system that included finger drains at each crack 0.25-inch or greater in width. The initial phase of construction revealed so large a number of these cracks that it was judged to be impractical to install finger drains at each crack. Moreover, the cracks were found to be considerably deeper than anticipated. Thus, the finger drains were eliminated and the filter was extended to 19 to 21 feet below the dam crest.

Without the finger drains, the sole function of the Central Filter is to build a filter cake in the event of seepage progressing from the upstream face of the dam to the filter. Thus, the capacity of the filter to seal cracks in the upstream part of the homogeneous embankment was analyzed. Various failure modes involving internal erosion processes are discussed by Von Thun (1996). The analysis focused on Mode 3 (seepage erosion) as opposed to Mode 1 (piping). Seepage erosion processes involve continuous pathways through the dam and/or it's foundation as opposed to piping that is initiated at the downstream face or toe and progresses upstream. (Von Thun, 1996; Milligan, 1996).

A seepage erosion pathway extending through an interconnected network of embankment cracks and a defect in the central drain appears to be credible. Piping seems far less likely, given the short time of impoundment of floodwaters in the reservoir. The 54-inch reinforced concrete principal spillway conduit is designed for a reservoir draw-down time of 10 days. A series of large floods in close sequence (and perhaps blockage of the primary spillway) could result in considerably longer period of impoundment. However the potential time of significant head behind the dam appears to be small in relation to the time necessary for piping to progress to the point of dam breach initiation.

A description of the methodology used in analysis and a discussion of results are presented in the following sections.

### 3.1 Approach

Methodology in Chapter 26 of the SCS National Engineering Handbook, *Gradation Design of Sand and Gravel Filters* (SCS, 1994) was used in the analysis. These procedures are the product of comprehensive research by the SCS (Sherard et al, 1984a, 1984b; Sherard and Dunnigan, 1985, 1989) and are essentially the same as those of the Bureau of Reclamation (BOR, 1987) and U. S. Army Corps of Engineers (USACOE, 2000). They have been endorsed by various organizations involved with the design,

construction and operation of dams including the International Commission on Large Dams (ICOLD, 1994) and Canadian Dam Safety Association (Milligan, 1996).

Filtration properties and criteria to prevent gap-graded filters and minimize segregation were evaluated with the methods in SCS Chapter 26. More recent work by Foster and Fell (2001) was also considered. Procedures discussed by Kenney and Lau (1984) and Skempton and Brogan (1994) were employed to analyze the internal stability. This procedure evaluates whether the pore size of the Central Filter aggregate is sufficient to allow the finer fraction to be washed out through the granular skeleton of coarser particles.

The potential for the Central Filter aggregate to be washed into the larger downstream embankment cracks was also evaluated using the SCS Chapter 26 criterion for perforated pipe and other SCS criteria (Leckband, 1984).

### 3.1 Base Soil Category (BSC) of Embankment Soils

SCS Chapter 26 subdivides base materials into four categories for filtration analysis. The homogeneous embankment soils (Figures 1 and 2) fall into BSC 3; sands, silts, clays and silty & clayey sands with 40 to 85 percent passing the no. 200 sieve.

### 3.2 Filtration Analysis.

Filtration requirements are defined in terms of D15F. For the coarse band of the Central Filter, D15F= 0.82 mm while D15F= 0.11 mm for the fine band. A D15F equal to or less than 0.7 mm is required for BSC 3. Thus, the fine band meets the criterion and the coarse band is marginal. In extensive tests on BSC 3 soils, Foster and Fell (2001) found the no-erosion boundary to be from 0.7 to 1.7 mm. They confirmed the criteria of SCS Chapter 26, but found it to be generally conservative. Thus, the Central Filter is judged to satisfy filtration requirements.

### 3.3 Filter Band Limits to Prevent Gap Grading.

For the particle sizes of the filter, for which 60 percent by weight or less are finer, the ratio of the course band size to the fine band size should be 5 or less. Calculated ratios are 5.3 to 7.3. Secondly, the coefficient of uniformity (CU) should be equal to or less than 6. The calculated CU-values are 15.6 for the coarse band 16.7 for the fine band. Thus, the Central Filter does not meet SCS Chapter 26 criteria for prevention of gap grading.

Analysis of the internal stability by the Kenney and Lau (1984) procedure is presented on Figure 8. The results suggest that the Central Filter is internally unstable.

### 3.4 Criterion for Segregation

SCS Chapter 26 includes a criterion to minimize segregation during construction. This criterion stipulates maximum D90F sizes for various ranges of the minimum D10F. The minimum specified D10F is 0.09 mm, so the maximum D90F should be no more than 20 mm. The actual value is 22 mm or slightly out of conformance with the segregation criterion.

### 3.5 Potential for Erosion of Central Filter into Downstream Cracks

In their evaluation of cracking of the Vineyard FRS and other dams, the SCS proposed that the D50F should be equal or greater than one-half the crack width and that the D75F should be equal to or greater than the crack width (Leckband, 1984). The fine band of the specified filter meets these criteria for cracks up to 0.12 inch in width and the coarse band for cracks up to 0.36 inch in width.

SCS Chapter 26 criteria for filters in contact with perforated drain pipe stipulate that, for non-critical drains, D85F must be greater than or equal to the perforation size. If this criterion is applied for downstream cracks, the fine band satisfies the requirement for cracks up to 0.24 inch and the coarse band for cracks up to 0.78 inch. SCS Chapter 26 further stipulates that, for critical drains (those subject to surging or gradient reversal), D15F must be equal or greater to the perforation size. If this criterion is applied for downstream cracks, the fine band satisfies the requirement for cracks up to about 0.005 inch and the coarse band for cracks up to about 0.03 inch.

As illustrated by the figures summarizing crack data, there are numerous cracks in the embankment with widths ranging from 1 to 3 inches, some of which extend to depths as great as 21 feet below the dam crest. The analysis indicates that there is a high potential for the central drain aggregate to wash into downstream cracks.

## 4.0 DISCUSSION OF RESULTS

### 4.1 Earth Fissuring Risks & Processes

Significant differential settlement (or subsidence in geological terms), in response to ground water declines, has occurred along the northern portion of Vineyard FRS resulting in accumulation of tensile strains. Various studies including the present photo-geologic work by AMEC have not identified earth fissures beneath the dam. Extensive earth fissuring is, however, present about one mile northwest of the dam and significant differential subsidence and increase in horizontal tensile strain is continuing along approximately the north half of he embankment.

Jachens and Holzer (1982) estimated that tensile strains at the threshold of earth fissuring are in the range of 0.02 to 0.06 for soils similar to the Late Pleistocene alluvium at the site. Leonards and Narain (1962) report the results of bending tests on compacted soil beams performed to determine tensile strain at cracking. Five of the six soils tested were silty and clayey sands similar to the Vineyard FRS embankment materials. Tests were performed at about the OMC and somewhat below the MDD. Tensile strains at cracking

ranging from 0.05 to 0.18 were obtained. They compared well with computed tensile strains in cracked dams. The present desiccated embankment probably has a lower strain threshold for formation of new cracks than determined in most of the Leonards and Narain tests. It is apparent that both the embankment and Late Pleistocene alluvium are relatively stiff and brittle and would experience earth fissuring at low tensile strains.

The weight of evidence indicate that there is some significant degree of risk of earth fissuring along approximately the northern one-half of the dam.

Earth fissures are typically about ¾ -inch in width at the time of formation and are believed to extend hundreds of feet in depth into the brittle cemented alluvium below the Holocene surface layer. Earth fissures are often difficult to detect after formation because they do not translate through the low-density Holocene surface layer. They also may not translate through the loose zone at the surface of dam slopes created by biological action and other surface processes or the granular surfacing on the crests of the dams. Earth fissures may be manifested in the embankment by the opening of several existing shrinkage cracks rather than formation of a continuous new crack. If this is the case, they might be particularly difficult to identify. After the initial brittle fracture, fissures continue to widen in response to further ground water decline, subsidence and accumulation of horizontal strains.

### 4.2 Causes of Embankment Cracking

The AMEC investigations indicate that the longitudinal cracks on the upstream face of the dam were caused by collapse settlements in the prism of Holocene surface soils upstream of the cutoff trench. The settlements probably occurred during the flooding in October 1972 that apparently filled the reservoir and may have resulted in emergency spillway discharge. A review of data on the rate of infiltration into collapsing soils (Hansen et al, 1989) indicated that rates are typically in the range of 0.5 to 3.0 feet per day. It is likely that water was impounded at the upstream toe of the dam for at least 10 days in October 1972. That would have probably been sufficient time for the soils to become wetted to a degree that would induce collapse settlement.

Given the large amount of drying of the embankment (Figures 5 and 6) and the fact that most of the collapsing soils were removed from beneath the upstream portion of the dam, the weight of evidence indicates that most of the cracks are shrinkage cracks. However, a substantial thickness of collapsing soil remains beneath the cutoff trench and shelf in at least a few areas. It is possible that seepage has extended beneath the dam through more permeable lenses (such as the soils from 16.5 to 21.5 feet in Boring No. BH215+00-1) in the collapsing layer created differential transverse settlement cracks through the dam. Such cracks might be wider and have greater continuity than shrinkage cracks.

### 4.3 Potential Seepage Erosion Pathways & Erosion Resistance

In their studies in the 1980's, the SCS analyzed the depth of potential drying shrinkage cracks using a simplified method based on the estimated modulus of deformation of the

unsaturated soil and the changes in suction (negative pore pressure) due to desiccation. The method is described by Fredlund and Rahardjo (1993). Crack depths of 21 feet were computed which correspond closely to the actual maximum crack depth observed during construction of the Central Filter. Postulation of drying shrinkage as the cause of the cracks seems consistent with unsaturated soil mechanics theory (Morris et al, 1992; Fleureau et al, 1993: Fredlund and Rahardjo, 1993).

According to the SCS (1985), 2336 transverse and diagonal cracks were observed during construction of the Central Filter in 1984-85. The average spacing of cracks for the entire dam is 12.3 feet, but as the figures illustrating the crack data show, cracking is much more intense in some segments.

The degree of continuity and interconnectivity of the crack system is not known, but there is clearly a risk that interconnected open cracks form pathways through the dam on both sides of the Central Filter. Whether such a crack system would form a continuous path for seepage erosion through the embankment depends on whether there are defects in the Central Filter. It seems credible that such defects could be rapidly formed by transport of the finer fraction of internally unstable filter aggregate and progressively washing aggregate into wide downstream cracks.

Skempton and Brogan (1994) state that the hydraulic gradient required to initiate internal instability in horizontal flow through broadly graded aggregates is typically about 0.16. Were seepage to flow through open cracks to the upstream face of the Central Filter with little head loss, it appears that hydraulic gradients of 3 to 4 would be induced across the 3-foot thick filter.

It appears possible that segregated zones could contribute to the formation of defects in the filter. It is also possible that the filter or portions thereof are cemented and have the potential to translate shrinkage/settlement cracks or earth fissures. According to Leckband (1984), it was reported that the Vineyard FRS filter stood vertically for it's full height in excavations made during the 1989 construction.

Desiccation of the embankment has produced high negative pore pressures and tensile stresses in the uncracked segments of the embankment substantially reducing minor principal stresses. Thus, it seems possible that embankment cracks could be rapidly opened or expanded by hydraulic fracturing (Sherard, 1973, 1985; Lo and Kaniru, 1990) during the initial surge of water into open cracks. It also seems possible that drying shrinkage and hydraulic fracturing processes could contribute to the formation of seepage erosion pathways along the primary spillway conduit.

As shown on Figure 2, most of the embankment material falls into Resistance Categories 1 and 2 of Sherard (1953), although some samples are classified as Category 3 (the least resistant to erosion). It is likely that the Sherard system underestimates the erosion resistance of the embankment soils because of the cohesion created by the high negative pore pressures and some amount of re-cementation by calcareous agents.

It appears that all of the embankment soils have a relatively high degree of resistance to erosion. Summaries of the global hydraulic gradients required to initiate internal erosion (Van Zyl and Harr, 1981; Meyer et al, 1994) suggest that hydraulic gradients in the range of 0.4 to 0.5 would be necessary to initiate seepage erosion and dam breach formation processes in the embankment.

In contrast, at least some of the soils of the low-density Holocene surface layer that remain beneath the dam foundation (such as the silty fine sand beneath the cutoff trench from 12.5 to 17.5 feet in Boring No. BH200+00-2) fall into Sherard Resistance Category 3 and have very high susceptibility to erosion. It appears that seepage erosion could be induced in these soils at hydraulic gradients as low as about 0.12. A very high susceptibility to erosion of these soils is demonstrated by the processes of their erosion into earth fissures.

As shown on the figures summarizing crack data, a considerable number of cracks extend to the bottom of the embankment and may be in direct contact with highly erodible silty fine sands and sandy silts of the Holocene surface layer. Thus, seepage erosion pathways along the embankment-foundation interface passing below the Central Filter also seem credible.

## REFERENCES

Beckwith, G. H., and Hansen, L. A., 1989, *Identification and Characterization of the Collapsing Soils of the Western United States,* Foundation Engineering: Current Principles and Practices, F. H. Kulhawy, Ed., Vol. I, ASCE, New York.

Beckwith, G. H., and Hansen, L. A., 1982, *Calcareous Soils of the Southwestern United States,* ASTM STP 777, K. R. Demers & R. C. Chaney, Eds., American Society for Testing and Materials, Philadelphia.

Blissenback, E., 1954, *The Geology of Alluvial Fans in Semi-Arid Regions,* Bulletin of the Geological Society of America, Vol. 66, February.

Bureau of Reclamation (BOR), 1987, *Chapter 5 Filters,* Design Standards 13, Embankment Dams, U. S. Department of Interior, Denver.

Foster, M., and Fell, R., 2001, *Assessing Embankment Dam Filters That Do Not Satisfy Design Criteria,* Journal of Geotechnical and Environmental Engineering, ASCE, Vol. 127, No. 5, May.

Fleureau, J. M., Kheirbe-Saoud, S., Soemitro, R., S., and Taibi, S., 1993, *Behavior of Clayed Soils on Drying-Wetting Paths,* Canadian Geotechnical Journal, Vol. 30.

Fredlund, D G., and Rahardjo, H., 1993, *Soil Mechanics for Unsaturated Soils,* John Wiley & Sons, Inc., New York.

Hansen, L. A.; Booth, R. D. and Beckwith, G. H., 1989, Characterization *of a Site Underlain by Deep Collapsing Soils,* Foundation Engineering: Current Principles and Practices, F. H. Kulhawy, Ed., Vol. I, ASCE, New York.

Harvey, A. M., 1992, *The Occurrence and Role of Arid Zone Alluvial Fans*, Arid Zone Geomorphology.

Hilf, 1991, *Compacted Fill,* Foundation Engineering Handbook, Second Edition, Hsai-Yang Fang, Ed., Van Nostrand Reinhold, New York.

International Commission on Large Dams (ICOLD), 1994, *Embankment Dam Filters and Drains,* Bulletin 95, Paris.

Jachens, R. C., and Holzer, T. L., 1982, *Differential Compaction Mechanisms for Earth Fissuring Near Casa Grande, Arizona,* Geological Society of America Bulletin, Vol. 93.

Kenny, T. C., and Lau, D., 1984, *Internal Stability of Granular Filters,* Canadian Geotechnical Journal, Vol. 22.

Leckband, S., 1984, *A Barrier to Cracks in Dry Earth Dams,* SCS, Phoenix, Arizona.

Leonards, G. A., and Narain, J., 1963, *Flexibility of Clay and the Cracking of Earth Dams,* Journal of Soils and Foundations Division, ASCE, Vol. 89, No. SM2, March.

Lo, K. Y., and Kaniaru, K., 1990, *Hydraulic Fracture of Earth and Rock-fill Dams,* Canadian Geotechnical Journal, Vol. 27.

Meyer, W, Schuster, R. L., and Sabol, M. A., 1994, *Potential for Seepage Erosion of a Landslide Dam,* Journal of Geotechnical Engineering, ASCE, Vol. 120, No. 7, July

Milligan, V., 1996, *Thoughts on the Development of Filter Design Concepts and Their Current Application,* Joint 1996 CDSA/CANCOLD Embankment Design Workshop, Niagra Falls, Ontario.

Morris, P. H., Graham, J., and Williams, D. J., 1992, *Cracking in Drying Soils,* Canadian Geotechnical Journal, Vol. 29.

Rollins, K. L., Rollins, R. A., Smith, T. D. and Beckwith, G. H., 1994, *Identification and Characterization of Collapsible Gravels,* Journal of Geotechnical Engineering, ASCE, Vol. 120, No. 3, March.

Soil Conservation Service (SCS), 1994, *Chapter 26, Gradation Design of Sand and Gravel Filters,* Part 633, National Engineering Handbook, U. S. Department of Agriculture, May.

SCS, 1985, *Memo to Ralph M. Arrington, State Conservation Engineer (Phoenix) from Jack C. Stevenson, Head, Engineering Staff (Portland); Subject: ENG-Williams-*

*Chandler Watershed, Vineyard Road FRS Field Trail-Crack Study, Arizona,* 5 November 1985.

Sherard, J. L., and Dunnigan, L. P., 1989, *Critical Filters for Impervious Soils,* Journal of Geotechnical Engineering, ASCE, Vol. 115, No. 7, July

Sherard, J. L., and Dunnigan, L. P., 1985, *Filters and Leakage Control of Embankment Dams,* Seepage and Leakage from Embankment Dams and Impoundments, R. L. Volpe & W. E. Kelly, Eds., American Society of Civil Engineers, New York.

Sherard, J. L., Dunnigan, L. P., and Talbot, J. R., 1984a, *Filters for Clays and Silts,* Journal of the Geotechnical Engineering Division, ASCE, June.

Sherard, J. L., Dunnigan, L. P. and Talbot, J. R., 1984b, *Basic Properties of Sand and Gravel Filters,* Journal of the Geotechnical Engineering Division, ASCE, June.

Sherard, J. L., Woodward, R. J., Gizienski, S. F., and Clevenger, W. A.,1963, *Earth and Earth-Rock Dams, Engineering Problems of Design and Construction,* John Wiley & Sons, Inc., New York.

Sherard, J. L., 1985, *Hydraulic Fracturing of Embankment Dams,* Seepage and Leakage from Dams and Impoundments, R. L. Volpe & W. E. Kelly, Eds., American Society of Civil Engineers, New York..

Sherard, J. L., 1979, *Sinkholes in Dams of Coarse Broadly Graded Soils,* Thirteenth International Congress on Large Dams, Vol. II, ICOLD, New Delhi, India.

Sherard, J. L., 1973, *Embankment Dam Cracking,* Embankment Dam Engineering, John Wiley & Sons, Inc., New York.

Sherard, J. L., 1953, *Influence of Soil Properies and Construction Methods on the Performance of Homogeneous Earth Dams,* Technical Memorandum 645, U. S. Department of Interior, BOR.

Skempton, A. W., and Brogan, J. M., 1994, *Experiments With Piping of Sandy Gravels,* Geotechnique, Vol. 44, No. 3

Stearns, C. E., Smith, R. J., and Stevenson, J. C., 1978, *Cracking of Dams in Arizona,* Report of the Soil Conservation Service Crack Study Team, April

U. S. Army, Corps of Engineers (USACOE), 2000, *Appendix D, Filter Design,* EM 1110-2-1913, U. S. Department of the Army..

Van Thun, J. L., 1996, *Understanding Seepage and Piping Failures- the No. 1 Dam Safety Problem of the West,* ADSO Western Regional Conference, Lake Tahoe, Nevada, April.

Van Thun, J. L.,1985, *Application of Statistical Data from Dam Failures and Accidents to Risk-Based Decision Analysis on Existing Dams,* BOR, Engineering and Research Center, Denver Colorado, October.

Van Zyl, D., and Harr, M. E., 1981, *Seepage Erosion Analysis of Structures,* Tenth International Conference , Soil Mechanics and Foundation Engineering, Vol. 1, A. A. Balkema/Rotterdam.

**FIGURE 1**

Gradation of Embankment Soils & Central Drain Aggregate

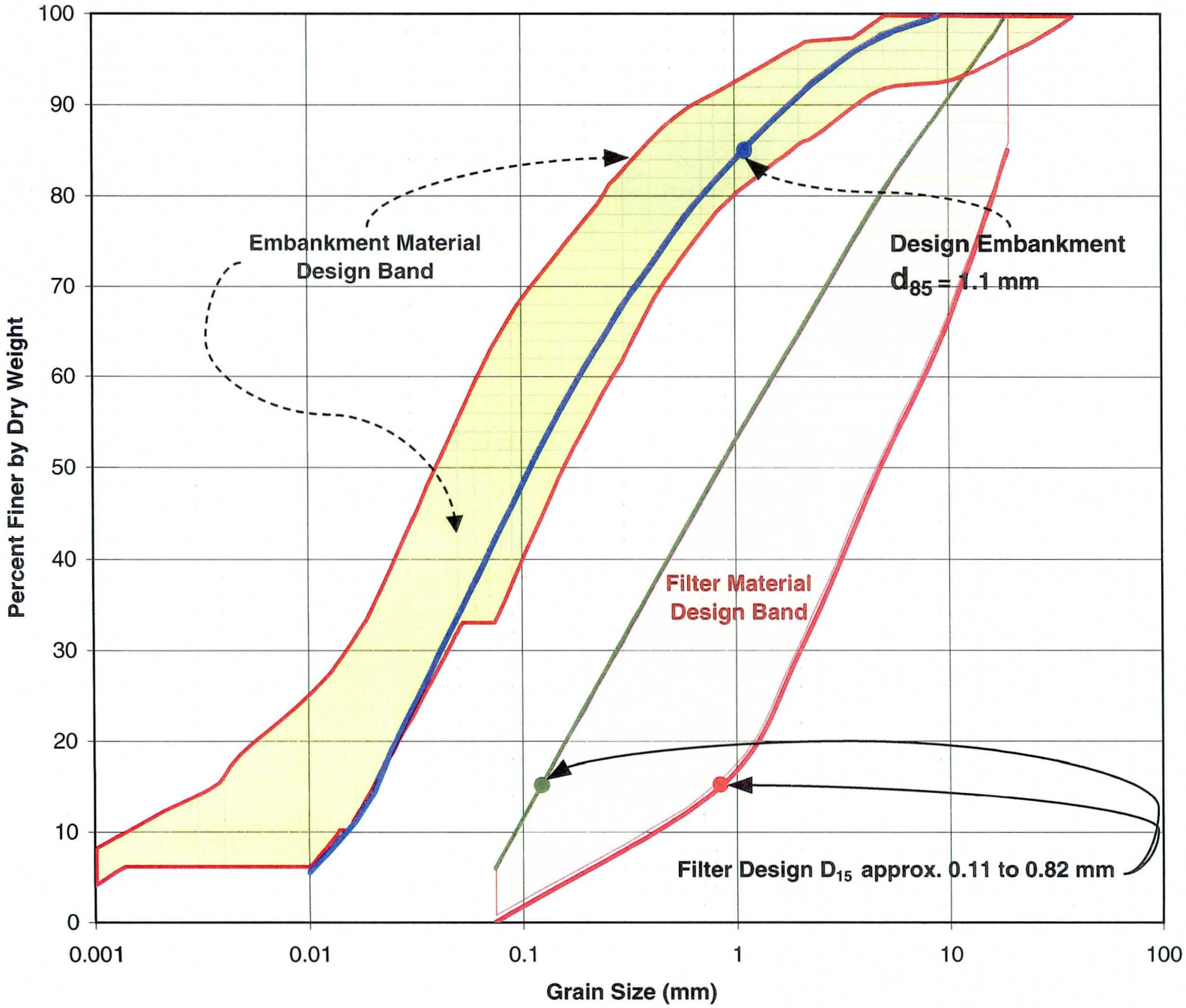


**FIGURE 2**

Vineyard FRS Embankment Material Properties

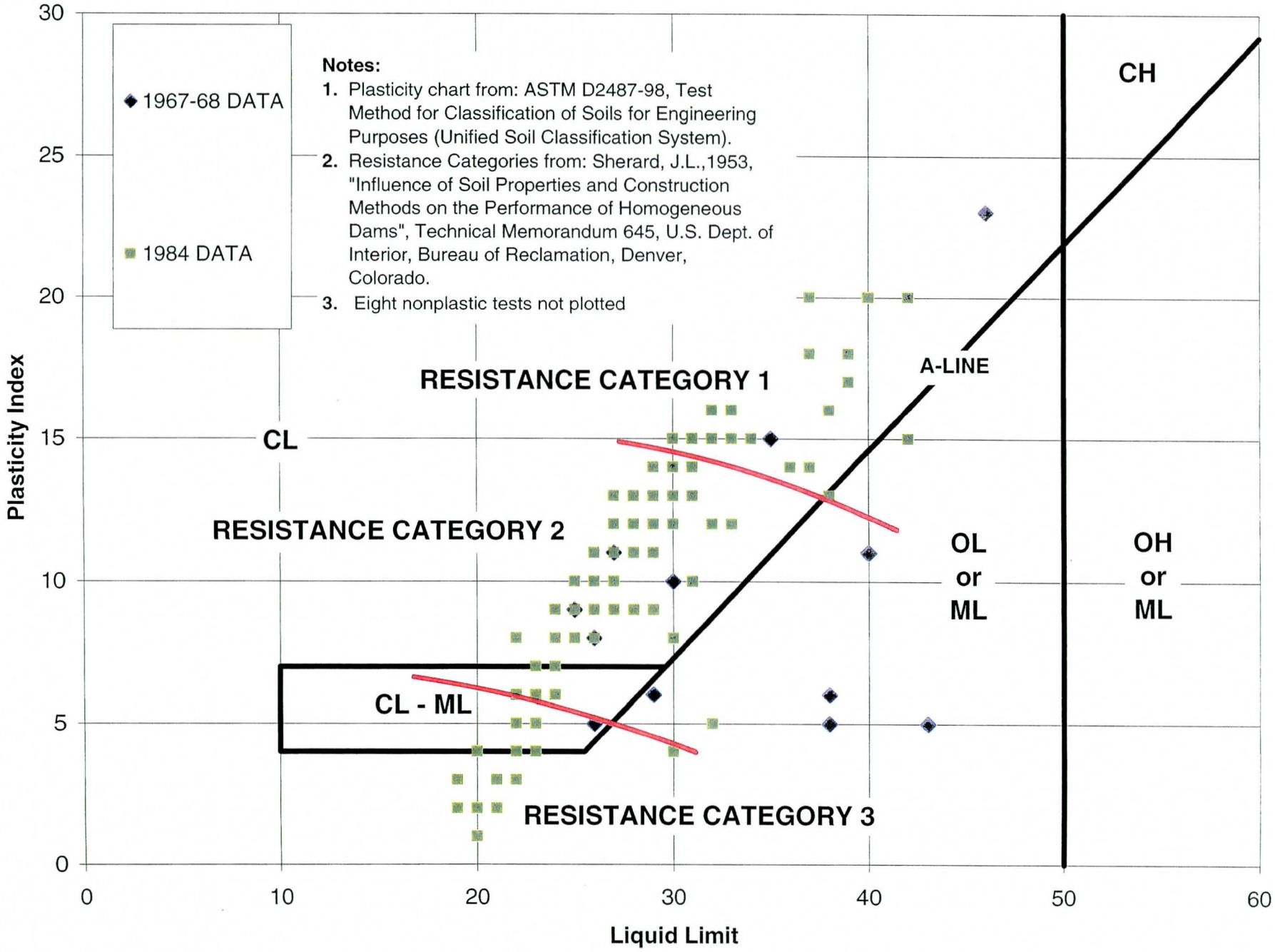


**FIGURE 3**

Percent Compaction -vs- Elevation, Vineyard FRS Embankment

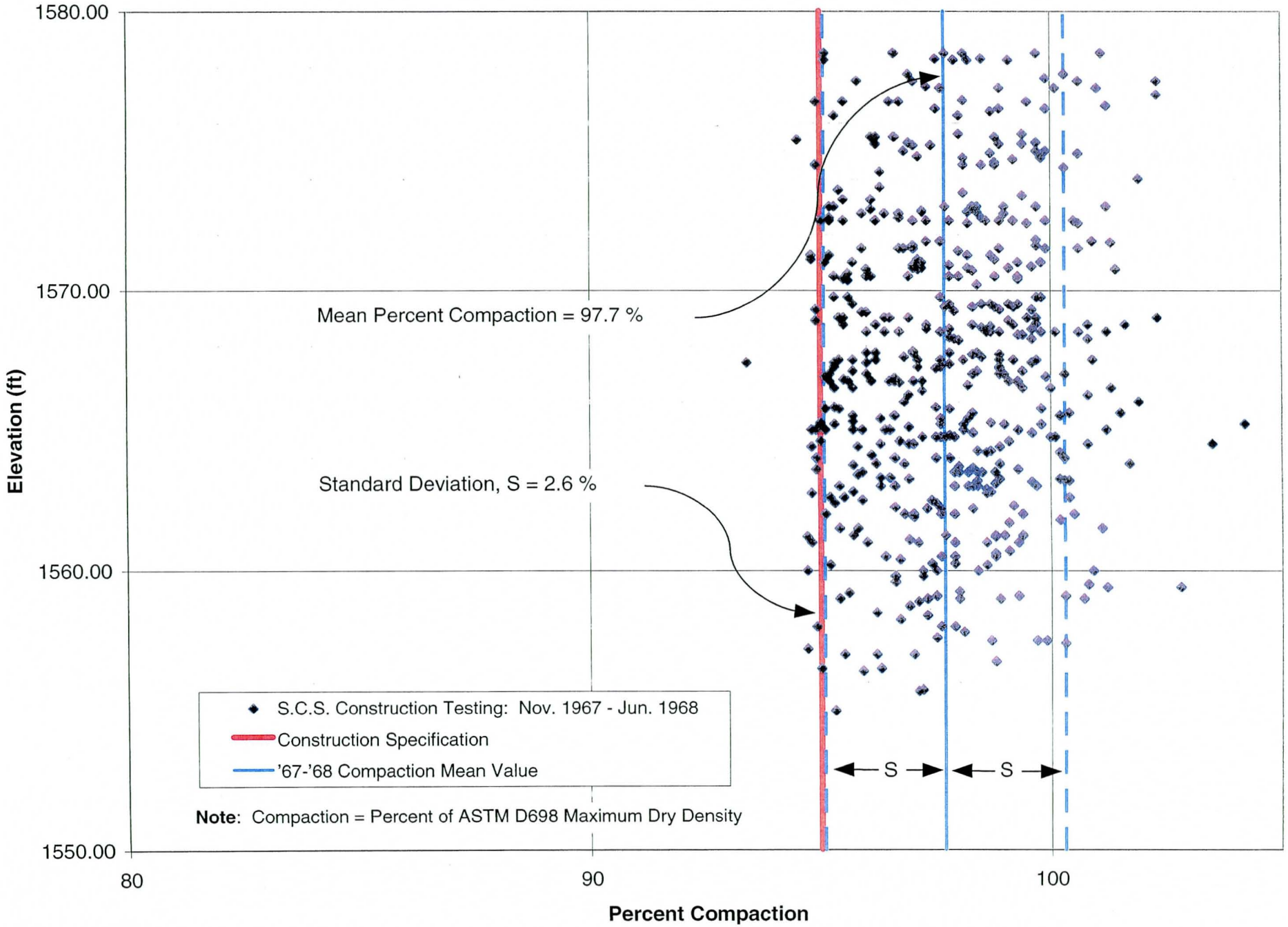


**FIGURE 4**

Variation of Field Moisture Content from Optimum Moisture Content, Vineyard FRS Embankment

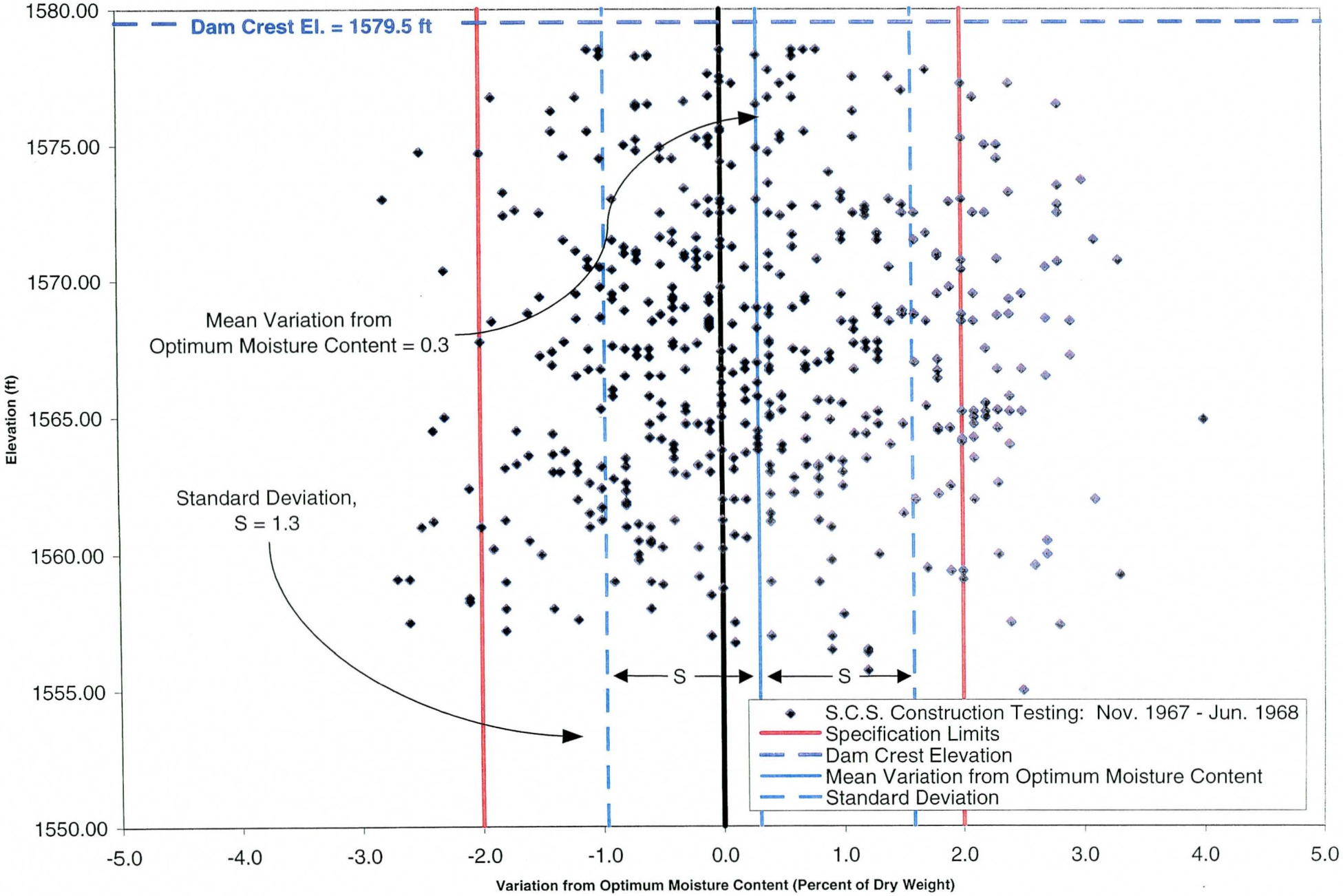


**FIGURE 5**

Degree of Saturation -vs- Depth, Vineyard FRS Embankment & Foundation

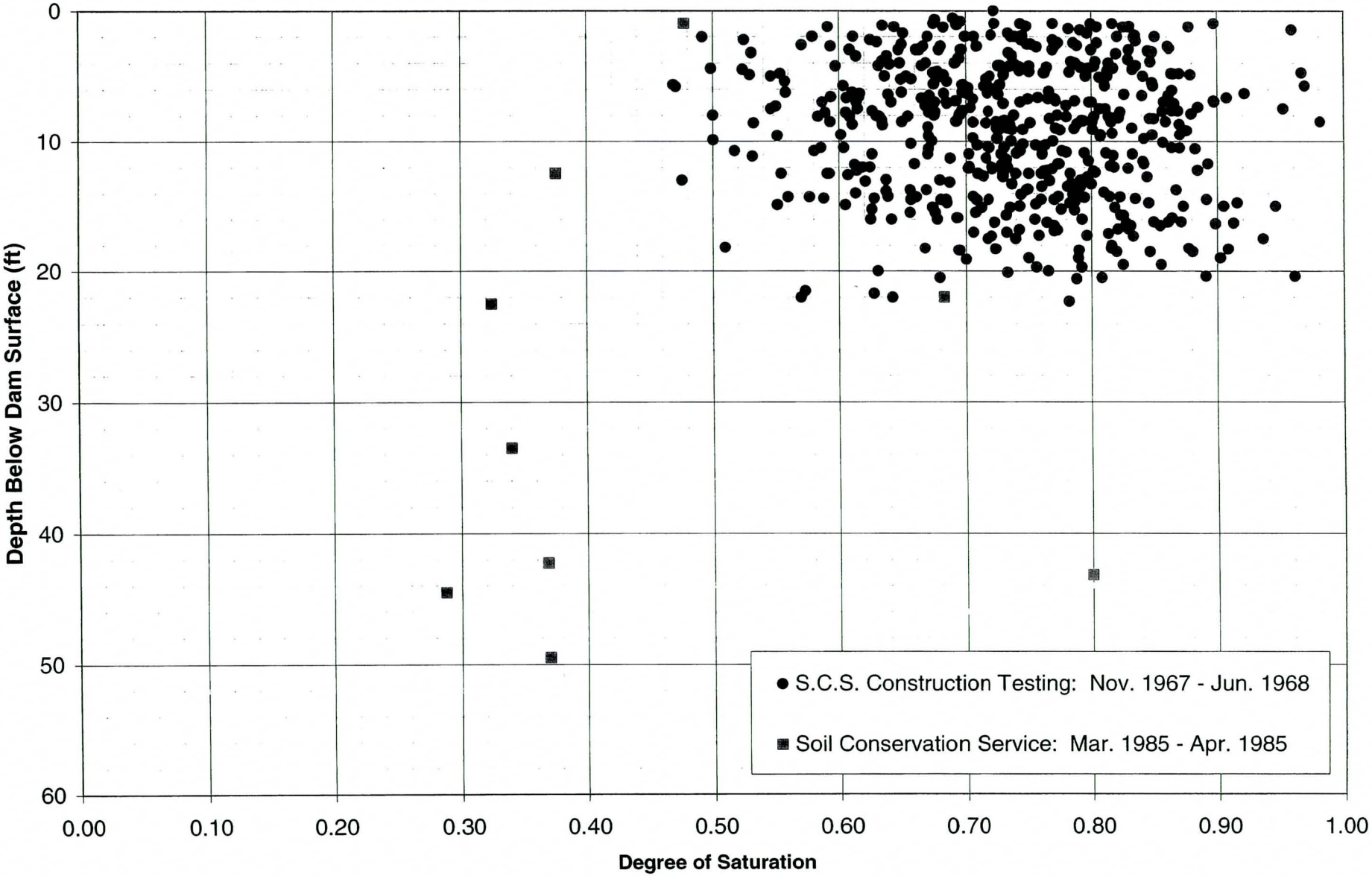


**Figure 6**

Moisture Content -vs- Depth, Vineyard FRS Embankment & Foundation

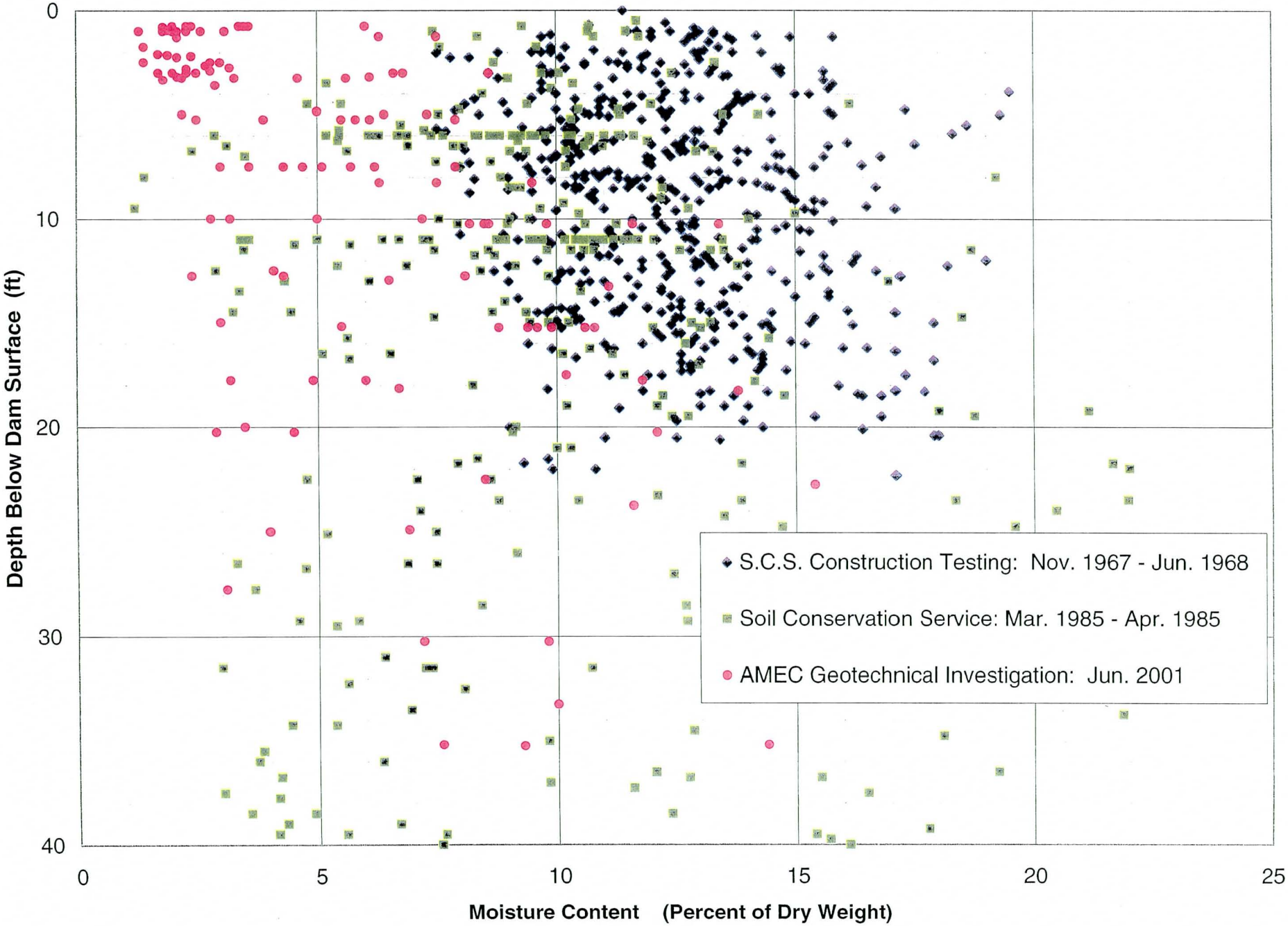


**FIGURE 7**

Station & Elevation of In-Place Density Tests, Vineyard FRS Embankment

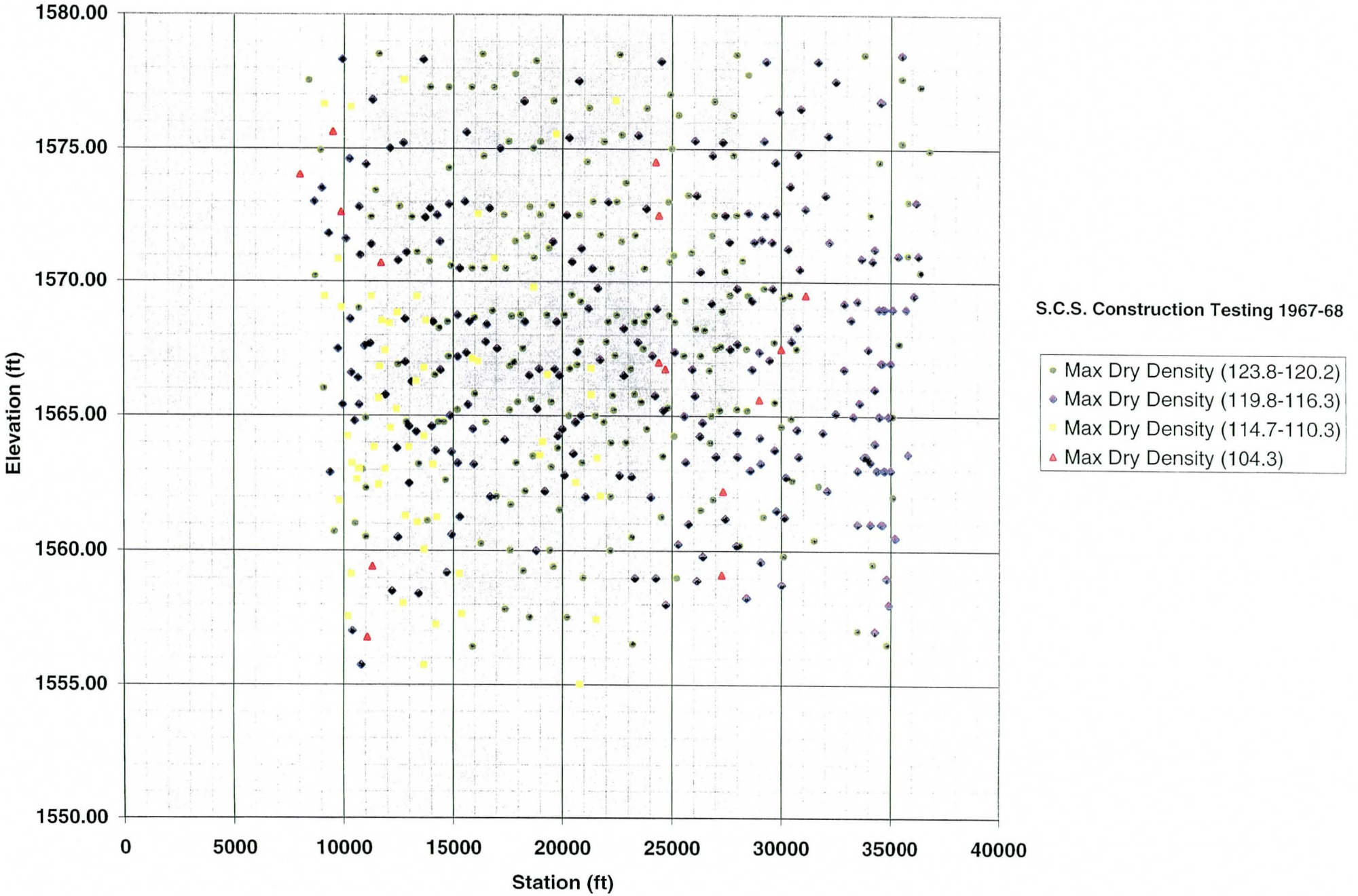
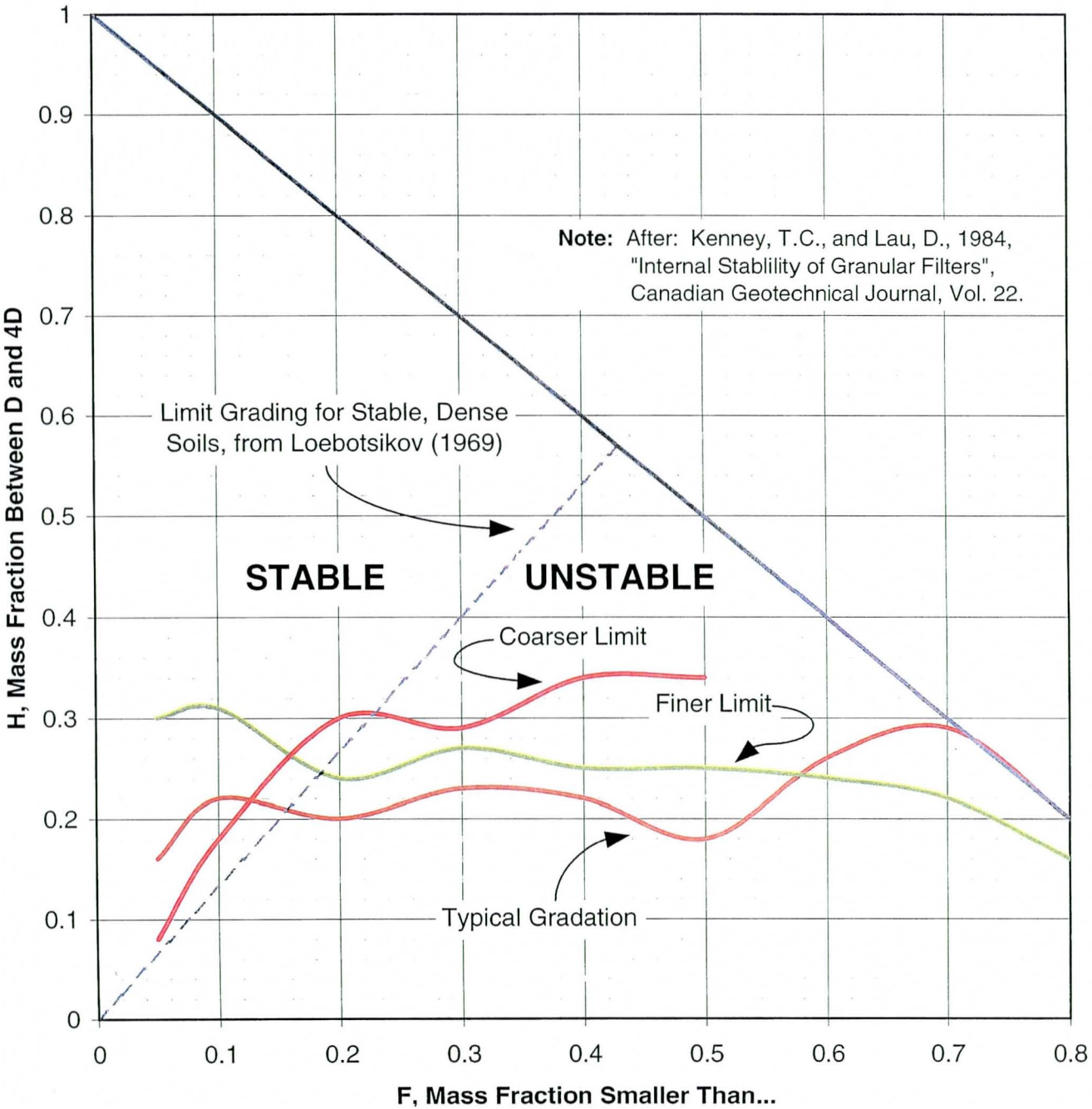


**FIGURE 8**

Internal Stability Analysis,Vineyard FRS Central Filter



# FAILURE MODE AND CONSEQUENCE ANALYSIS
# For
# RITTENHOUSE FLOOD RETARDING STRUCTURE

## PINAL COUNTY, ARIZONA
## NATDAMID AZ 0085

**July 12 2001**

## 1.0 INTRODUCTION

### General Description

Rittenhouse FRS is located about 40 miles east of downtown Phoenix and twelve miles southeast of the City of Apache Junction. The project consists of the FRS embankment structure and an emergency spillway. The project is part of the Williams-Chandler Watershed Protection and Flood Prevention Project, which includes the Rittenhouse and Vineyard Road flood retarding structures.

Rittenhouse FRS is classified as a medium sized, significant hazard dam. The reservoir behind the FRS is 660 acres with a capacity of 4,060 acre-feet. Construction of the Rittenhouse FRS was accomplished under contract to William Pulice Construction Company. Construction of the FRS and appurtenant structures were completed in 1969.

Except for considerable cracking of the structure during and after its first major impoundment in 1972, which necessitated investigations and construction of a defensive design measure (a central filter) repair the dam has performed satisfactorily to date

### Dam Data

Dam type: Rolled earthfill
Dam height: 24.3-ft
Dam length: 19,008-ft
Dam crest: width 14-ft; elevation 1602.3-ft
Spillways: Principal - 33-inch RCP; inlet elevation 1578.1-ft; Emergency spillway - 600-ft wide earthlined spillway; crest elevation 1597.6-ft
Freeboard: 4.7-ft
Reservoir Surface: 660-ac at spillway crest
Storage: 4,060-af at spillway crest
Hazard Classification: Significant

### Purpose and Scope

In general, the purpose of the Failure Mode and Consequence Analysis (FMCA) exercise was to:

- Identify potential site-specific failure modes for the dam.

- Discuss non-quantitatively the likelihood of the identified failure modes.
- Determine whether or not and how important failure mechanisms are being monitored
- Examine the consequences of failure and the adverse consequences of successful operation (e.g. – large spillway releases)
- Identify possible risk reduction actions that may be taken to reduce the likelihood of failure or to mitigate adverse consequences
- Determine what information, investigations or analyses may be needed to resolve uncertainties relative to potential failure modes.

(Note: In this phase, the FMCA team only examined the general nature of the "consequences" for the failure modes identified and where appropriate estimated how they may be different than previously anticipated . Greater detail on the estimate of the magnitude of the "consequences" for each significant failure mode will be addressed in the quantitative portion or risk analysis part of the risk assessment for the dam. At this time hopefully more detailed discussion on the emergency response plan for each structure will also be possible.)

**Team Members**

Larry Von Thun, Engineering Consultant and FMCA facilitator
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George Beckwith, Flood Control District of Maricopa County, Geotechnical Engineer
Justin Beeler, Kimley-Horn and Associates, Inc, Session Recorder
Tim Murphy, Flood Control District of Maricopa County, Civil Engineer (consequences)

## 2.0 MAJOR FINDINGS AND UNDERSTANDINGS GAINED

The following is a summary of the major findings and understandings for Rittenhouse Flood Retarding Structure as a result of the Failure Mode and Consequence Analysis (FMCA). Rittenhouse FRS is one of three adjoining flood retention dams (Vineyard Road and Powerline FRS being the other two dams). Because of concerns about drying shrinkage and desiccation cracking each dam was modified after construction but in slightly different ways. In carrying out the FMCA for these dams Vineyard Road FRS was done first followed by Rittenhouse and then Powerline FRS. The potential failure modes considered for the dams were of course very similar and it was natural to compare the likelihood of development of these failure modes among the three dams. Thus in the major findings and understandings given below some of the differences are noted as key findings and conditions of general similarity related to potential failure modes are also noted.

The major findings and understandings given below are organized as follows. First the most important design, construction, geologic, and performance differences or unique aspects related to the potential for failure mode development of Rittenhouse FRS are

given. Findings related to failure modes or adverse consequences other than overtopping and spillway discharge are given first. Findings related to consequences are given next followed by action items (risk reduction and investigations). Finally, general findings that are informational and / or generally similar for all three dams are listed. In making comparisons of the dams and the amount of cracking experienced at the time of the repairs it is instructive to note the year of the original construction of each and the year those repairs were made. The difference between these two dates [shown in brackets] is the number of years drying had taken place before the central trench was excavated. Each trench was mapped for cracking depth, width and extent,. They were Vineyard Road FRS – Constructed 1967, repaired 1983, [16 years], Rittenhouse – Constructed 1969, repaired 1979, [10 years], Powerline – Constructed 1968, repaired 1991 [23 years].

**Key findings / differences related to failure mode development – "static loading failures – seepage erosion – fissuring – foundation erosion –etc."**

- The Central Filter Repair Was Not Made Over The Entire Length Of The Dam. The filter drain is not continuous along the entire length of the dam being absent on the south end of the dam.
- The Central Filter repair did not Extend to the bottom of the Foundation. The fact that the filter was not taken to the full depth of the foundation opens up the potential for a transverse crack to extend through the full section of the dam at or near the base of the dam where the head would be the greatest (and the potential for water would be the highest) but the crack opening would tend to be the smallest based on the trench mapping of cracks.
- Transverse and Longitudinal Crack Characteristics. The cracks at Rittenhouse FRS appeared to be the most significant of the three dams in 1972 when the problem first came to light. However, the trench mapping of cracks during the repair in 1979 showed that the cracks in Rittenhouse were considerably shallower and narrower than in Vineyard Road FRS that was repaired in 1983. While not as extensive as at Vineyard FRS it still should be noted that numerous cracks were observed in the trench mapping at Rittenhouse to significant depths. A plausible reason for the difference in the extent of cracking would be due to the additional drying time (6 years) which Vineyard had before repair took place. However, based on current observations, ongoing cracking at Vineyard generally seems to be more extensive than that of the Rittenhouse FRS embankment. Maximum crack width was 0.6 inch and crack depth 14 feet at the Rittenhouse FRS as compared to a maximum width of 4 inches and a maximum depth of 21 feet for the Vineyard FRS. The average spacing of transverse and diagonal cracks is about 12 feet for both dams. The explanation for the difference in cracking is uncertain. It may be due to differences in the length of time between construction and installation of the central filters, differences in the shrink-swell properties and sand/gravel and clay content of the embankments or some combination of these and other factors. Limited classification test data suggests that the Rittenhouse FRS embankment is less plastic and coarser, but the data is inconclusive. Further study is needed to resolve the critical question of whether the cracks in the Rittenhouse FRS have grown appreciably since the central drain was constructed.

- Original Material Logging was Not Necessarily Valid. The Material logging (classification of soils) in the borrow and in the foundation generally indicated the presence of coarse grain materials (gravel) at all the structures. Such material would not have been very susceptible to desiccation cracking. The soil classification may have been conducted by testing personnel not familiar with local soils/nomenclature. As a result the original logging does not appear to be good enough to differentiate between the soils used in the construction of the dams and current testing is needed to tell what the real nature of the fills are like and whether there is a material difference in the soils at each dam. The material logging may be inaccurate in consideration of the lack of a detailed description of cementation in the logs and lack of definition in the logs of the contact between the Holocene surface layer and underlying more cemented Late Pleistocene deposits
- Understanding of Filter Characteristics & Function of Filter. The central filter for Rittenhouse FRS was installed by SCS in 1979 and was intended by the SCS to function as a "crack stopper". The defensive mechanism provided is the formation of a "filter cake" in the initial process of seepage through a transverse or diagonal crack. This would prevent erosion of the soils upstream of the filter and minimize flow into the filter and consequent erosion of soils in the downstream part of the embankment. However, as noted below, the central filter only provides partial protection of the embankment. The specified filter meets generally accepted criteria for filtration. Factors that could potentially create defects that would compromise the function of filter are segregation during construction and cracking. An internal SCS report on the general subject of cracking of dams in Arizona (Leckband, 1984; "A Barrier to Cracks in Dry Dams") indicates that the central filter of the Vineyard FRS stood vertically in temporary excavations that extended for the full height of the dam. This suggests that the filter may be cemented and thereby susceptible to cracking in response to shrinkage or settlement. This possibility leads directly to the postulation of a seepage erosion potential failure mode even where a central filter has been constructed.
- Longer Drawdown Time. Rittenhouse FRS has a design drawdown time of 30-days, which is longer than the 10-day design of the Vineyard Road FRS. This primary difference is due to the sizing of the outlet works pipe (principal spillway). The significance of this finding is that there is a considerably longer time (20 more days) at a higher head to allow development of a seepage erosion flow path and retention of enough head to accomplish erosion if the flow path was established. The FMCA team noted that even though the storage time is 3 times longer, the available time to develop flow paths and initiate seepage erosion is still very short.
- Reduced Potential for Fissuring. There is a reduced potential for earth fissuring at Rittenhouse FRS based on the local geology and lack of strain potential based on settlement profile. (Don't have differential settlement – no strain).
- Reduced Potential for Subsidence related cracking. There is a reduced potential for subsidence related cracking at Rittenhouse FRS based on the local geology and lack of strain potential based on geologic profile. (Don't have subsidence – no strain).
- Collapsing Holocene Soils Exist in the Foundation of Dam. The FMCA team reviewed information on the geotechnical properties of the foundation soils presented

in the various reports by the SCS. This data indicates that a surface layer of collapsible Holocene soils is present at the site and is underlain by dense cemented Late Pleistocene deposits. The collapsible layer was completely removed from beneath most of the upstream cutoff trench during the original construction. However, the available evidence suggests that finer-grained collapsing soils may be present beneath the entire embankment in localized areas. These materials are highly susceptible to seepage erosion and collapse settlements upon wetting. This material is more susceptible to seepage erosion than the compacted embankment fill. A potential failure mode involves eroding this material at the contact of the dam and foundation. At Vineyard drilling along 4 cross sections revealed that this material was left in place. There is no direct evidence that such Holocene earth material was left in at Rittenhouse and that it extends the full length of the foundation. However that possibility exists.

- Seismic Loading of the Dam was not considered as a Major Issue. There were no potential failure modes identified due to seismic loading and no "incipient" static loading potential failure mode that would be triggered by seismic load could be identified. The seismic exposure evaluation indicates that the peak horizontal acceleration (PHA) at the Rittenhouse FRS due to the maximum credible earthquake (MCE) is about 0.12g. These small dynamic forces would not cause appreciable deformations of the stiff unsaturated embankment, so earthquakes do not constitute a significant risk.
- Dispersivity Tests. Pinhole and SAR test results indicate the embankment soils are non-dispersive.
- Good Construction of Fill. Quality control was good/excellent. Good compaction of fill.
- Generally, the Finger Drains may be Ineffective. The finger drains interconnected to the central filter/drain do not appear to have the capacity to control seepage from open upstream cracks, prevent build-up of heads in the central filter drain and control seepage through and erosion of downstream cracks. The thirteen finger drains are spaced on 1000-ft centers while the hydraulic conductivity of the central filter/drain aggregate is only about 250 feet per year. The generalized spacing of drains would have little effect on a nearby, randomly located crack or fissure.
- Potential for Transverse Crack to occur where no Finger Drain exists. There is a high potential for any new transverse crack to intersect the central filter where there is no finger drain outlet.
- Potential Loss of Central Filter Material through a Crack. The potential exists that there may be a loss of central filter material through a large downstream crack should such a crack promulgate to the filter and a driving head exists in the central filter. The maximum crack width at the time of the central filter-drain construction in 1979 was only 0.6 inches. If the cracks have not expanded to greater widths, this potential would not exist. However, if the have expanded to the range of widths present at the Vineyard FRS, the central filter-drain aggregate would be susceptible to erosion into downstream cracks.
- Foundation was Pre-Wetted. The foundation soils were pre-wetted to minimize collapse potential. The pre-wetting was performed over the area extending from the crest to the downstream toe. This may have positively affected the number and

spacing of longitudinal cracks. It may have been why more cracking is being observed at Vineyard than at Rittenhouse. However it is believed that no pre-wetting was done at Powerline.

- Not As Much Foundation Excavation. Not as much foundation over-excavation was performed at Rittenhouse as was done for Vineyard FRS. However, more foundation overexcavation was done in field than called for in design.
- Small Outlet Principal Outlet Pipe. The 33-inch RCP that serves as the principal spillway for the Rittenhouse FRS is smaller that the 36-inch principal spillway pipe at the Powerline FRS and the 56-inch principal spillway pipe at the Vineyard FRS. This results in a drawdown time of 30 days as compared to drawdown times of 10 days for the Vineyard FRS and Powerline FRS. The smaller diameter pipe at Rittenhouse increases drawdown time and potential for plugging.
- Dam has been Tested. Rittenhouse FRS has experienced a significant impoundment, possibly to the emergency spillway crest elevation based on October 1972 flood photographs on file with the District. This is important to note that the dam has experienced at least one significant impoundment. (NOTE: The shrinkage cracks were in their early stages of development in Oct. 1972).
- Central Filter is More Broadly Graded. The Rittenhouse central filter is more broadly graded and has a larger maximum particle size than the Vineyard FRS filter. Thus, this filter is more susceptible to segregation and internal instability due to broader gradation and larger maximum particle sizes. There was one recorded incident of filter material rejection by an inspector during construction.

**Key findings / differences related to failure mode development – "flooding, overtopping, spillway discharges."**

- Overtopping of Rittenhouse FRS. The FMCA team observed that a previous hydrologic and dambreak study for Rittenhouse FRS concluded that the dam would be overtopped during a full PMP/PMF event. However, the depth of overtopping is low (approximately 0.5-ft) and the duration of overtopping is short. This finding however is based on design elevations and the assumption of a level crest and is thus dependent on what the actual crest elevation of the dam is and what the current reservoir capacity is (effect of regional subsidence versus dam settlement)
- Settlement Survey Data. The survey data completed in the past for the dam monuments crest shows wide variations in crest elevations. Thus the data for the crest and downstream toe monuments needs to be comprehensively reviewed and verified. Surveyed elevations of the monuments located near the crest do not reflect top of dam elevations. Questions were raised as to the benchmarks and reference datums used in previous settlement surveys and aerial topographic mapping of the dam. True elevations of dam features (crest of dam, emergency spillway crest, principal spillway, etc.) need to be established. Elevation data for sections of dam next to emergency spillway need to be acquired and elevations marked on trash rack of principal spillway and elevations should be shown in tenths on the upper 3 feet of the staff gage. The survey data for the adjacent section of the CAP Aqueduct needs to be integrated package of survey information for risk assessment.

- Potential Failure of Rittenhouse FRS into Vineyard Road FRS. A failure of the north abutment of Rittenhouse FRS will discharge floodwaters into Vineyard Road FRS. Further investigation of this consequence should be undertaken since the structures are on adjacent watersheds.
- Lowered Abutment Sections. The abutments are 1 foot lower than the dam crest elevations in the original design according to the construction as-built drawings. This may be a "fuse plug" type design feature.
- Confirmation of Inflows to Reservoir. Hydrological analysis should be performed to confirm the hydrographs of the PMF, ½ PMF, 100-year and other appropriate events, determine the frequency of occurrence of incipient overtopping and define the downstream areas of inundation and other consequences of the various floods. Alluvial fan avulsion and deposit processes may need to be considered in this analysis.
- Erosion of Dam Embankment (Left Abutment) due to Emergency Spillway Discharges. There are high flow velocities for large spillway discharges from the spillway at Rittenhouse (estimated flow velocities at 10 – 12 feet per second). These are higher flow velocities than at Vineyard FRS spillways and there is a potential for spillway flows to impinge on the dam due to the narrowing of the spillway channel and because the training walls terminate before the end of the dam. (NOTE: The SCS test pits suggest that cemented soils are present at spillway grade along portions of the spillway alignment. These soils are probably highly resistant to erosion, but the SCS logs do not conclusively demonstrate the characteristics of these materials. The dam break analysis performed by Montgomery in 1989 showed a very severe spillway erosion event with the peak of the outflow hydrograph being essentially the same as the dam breach case. The peak discharge was about 1.33 times the peak of the PMF inflow hydrograph. More study is needed to confirm that there is no risk of a serious spillway erosion failure mode).

## Consequence Evaluation

- Dambreach Characteristics. Owing to the stiff, unsaturated embankment soils, it is anticipated the rate of breach formation at Rittenhouse FRS would be relatively slow. The weight of evidence indicates that the embankment material has a relatively high resistance to erosion.. It is expected that breaches for both the overtopping and internal seepage erosion failure modes would be narrow and take a relatively long time to form. Relevant case histories of breaches of low homogeneous dams involving similar embankment materials include the Church Rock tailings dam near Gallup, New Mexico and the Clear Creek dam near Winslow, Arizona.
- Importance of Good Consequence Scenarios. In order to get a good understanding of the consequences of large spillway discharges, a potential overtopping failure or a potential seepage erosion breach there is a need to develop an understanding of the importance of the buffer zone between the dam and the CAP canal (and flows into the wasteways). At Rittenhouse there is a large buffer zone downstream of the dam between the dam and the CAP canal. This buffer zone would receive the breach flows from the dam and from there flows would discharge into the Queen Creek wasteway and if great enough they would enter the CAP canal. Thus the actual peak

discharge flows affecting downstream areas may well be controlled by the breach parameters of the CAP canal or of the wasteway.

- Importance of Incremental Damage Analysis. It will be important to route floods of different frequencies (peak and volume) through reservoir and spillway to estimate the recurrence interval at incipient overtopping. The studies should include comparison of inundation limits from spillway flows with inundation from combined overtopping and spillway flows and the incremental damage created by overtopping. Differences may be small so that the results of this study may provide the basis for determination of whether raising of the dam is needed (for larger events).

**Action Items – Risk Reduction or Investigations**

- Risk Reduction Measures. The FMCA team recommends several measures for risk reduction and management that may be taken by the District in the immediate future. These measures include: (1) Keep the crest of the dam level at the design elevation ; (2) Monitor the impoundment levels during storm events (this includes both remote and as well as on-site monitoring); (3) Provide training to O&M crews regarding identification of earth fissure development and other key observational parameters such as indications of seepage erosion; (4) Operation of irrigation outlets – close gates except as needed.
- Confirm Utility Crossing. The details of the telephone utility crossing of dam at Station 141+00 should be confirmed. Document on dam as-built drawings.
- Lack of Material Testing on Key Parameters. The FMCA team has found that there is relatively few grain size analysis and Atterberg limits test available for the embankment fill and borrow. Thus the characteristics of the embankment material is uncertain.
- Lack of Stability Analyses. The FMCA team recommends that slope stability analyses for the embankment and the central filter be documented. A record of a stability analysis for the original design or the modifications has not been found.

**General Findings That Are Informational And / Or Generally Similar For All Three Dams (Powerline, Vineyard Road, and Rittenhouse FRSs)**

- Potential For Uniform Subsidence Induced by Ground Water Declines. Information summarized by AMEC in their "Investigation of Ground Subsidence and Earth Fissures" indicates that the dam is underlain by thick deposits of alluvial sediments over 1200 feet in depth which contain a relatively thick interval of clayey lakebed deposits. Substantial groundwater declines and moderate ground subsidence in the area of the Rittenhouse FRS are probable in the future. Survey records for the adjacent section of the CAP Aqueduct indicate that almost uniform subsidence of about 1.6 feet has occurred along the Rittenhouse FRS since it was constructed in 1968. It is the opinion of the FMCA team that future subsidence of the dam will be moderate and relatively uniform and that the risk of earth fissuring is low. There is a remote possibility that abrupt differential subsidence and fissuring could be induced in the future by a high rate of localized ground water pumping. However, any such trend would be identified by the crest surveys well before significant risk developed.

- Differential Subsidence is On-Going. Subsidence in the region continues.
- New Fissure found Just North and West of Powerline FRS. AMEC's Phase II investigations included review of previous reported earth fissures in the vicinity of the Powerline, Vineyard, and Rittenhouse FRSs. AMEC has also reviewed recent aerial photographs for possible earth fissures. The review of the photographs and subsequent field visit has discovered a new previously unobserved or unreported earth fissure near the north end of Powerline FRS. (Note: The new fissure is well removed from Rittenhouse and there is a low risk of fissuring as compared to the high risk at Powerline.
- Ability to Understand Fissure Development Potential Rational and the Potential to Detect and Monitor Development of Fissures. The FMCA team discussed the general mechanism for potential earth fissure development and concluded that the conditions exist at the regionally, and at the dam site and the immediate vicinity for earth fissures to develop. The team recognized that there are early warning indicators that can be observed in the field that may be linked to the potential formation of an earth fissure.
- Small Floods May " Reveal or Detect" the Presence of Fissures. The FMCA team realized that water in the form of stormwater as overland flow or flooding is often associated with development of earth fissures or revealing the existence of already developed fissures. These floodwaters cause the fissure to manifest itself as surface expressions. Earth fissure development as the result of surface water flow is not tied to a storm event frequency. Small floods, which are much more likely to occur than major floods, which could threaten loss of the dam, may thus provide an early warning for the detection and subsequent repair of an earth fissure.
- Groundwater Supply Wells. The FMCA team determined that new groundwater supply wells may increase differential subsidence and the risk of earth fissuring. Further studies are warranted to establish a minimum setback distance that will not significant increase risks of fissure development. It is AMEC's preliminary estimate that the radius of influence of high capacity water supply wells is approximately one mile. Groundwater supply wells are not as important a factor influencing risks related to subsidence for Rittenhouse FRS as they are for the Powerline FRS and Vineyard Road FRS.
- Broad Based Value of Risk Assessment. It is the opinion of the FMCA team that the broad-based risk assessment process was of particular value to the District's Structures Assessment Dam Safety Program. In it's comprehensive review of the technical and operational issues, it added value to the program well beyond it's objective of identification of failure modes and quantification of risks. The process better defined priorities for the next stages of the program, and, with the broad experience of the team, provided peer review of areas of technical study critical to the program. It is believed that the Risk Analysis process will lead to more efficient allocation of resources in Phase I and II engineering and optimization of the emergency action plans, dam inspection procedures and design of remedial measures.
- Impact of Vineyard Road FRS Test Cells. A long time was taken to fill the test cells at Vineyard Road FRS with water (22 hours for Cell 2 and 44.5 hours for Cell 3)– this could have resulted in swelling of the clay soils and thus avoided a rapid flow through a crack capable of seepage erosion. The number of tests run relative to the

length of the dam was statistically not adequate for Vineyard FRS. (Note the filter at Rittenhouse FRS does extend for the full length of the dam or to the full depth of the compacted embankment soils. Thus, ponding tests are less relevant to Rittenhouse than for Vineyard and Powerline).

- Central Arizona Project Canal. The CAP canal should be evaluated relative to its influence on the effects of various modes of Rittenhouse dam failure and relative to its response to inflows from the dam. The ability of the CAP canal to handle/store flows resulting from failure at Powerline seems to be considerably less than at Vineyard and Rittenhouse. This is especially true due to the canal flow restrictions at the overchute area and at the bridge just downstream. However both of these restrictions could be countered to take advantage of the canal in attenuating / lessening dam failure peak outflow flooding.

## 3.0 POTENTIAL FAILURE MODES

Potential failure modes identified by the FMCA team are presented below. The failure modes were placed into one of four categories listed below. Except in special cases only those potential failure modes in Category I will be considered in the risk analysis phase of the risk assessment exercise.

- Category I – Failure modes of greatest significance.
- Category II – Failure modes of lesser significance (but not inconsequential).
- Category III – Failure modes for which insufficient information is presently available to allow the team to make a judgement on the significance of the failure mode. The development of additional data and information is warranted. Additional records research may be justified.
- Category IV – Failure modes which are not physically possible or which are clearly not credible.

For each of the potential failure modes identified, a Failure Mode Description is briefly described and the factors that make the failure mode more likely (adverse factors) or less likely (positive factors) to occur are listed following the Failure Mode Description. In addition, any identified potential actions for risk reduction for each potential failure are then provided.

## CATEGORY I - FAILURE MODES OF GREATEST SIGNIFICANCE

### *Failure From Overtopping Of Dam Crest Due To PMF (Category I).*

(Note: This failure mode for Rittenhouse FRS was highlighted in order to emphasize the importance of taking action relative to obtaining an accurate survey of the elevation of the crest surface, spillway crest and outlet invert and relative to ensuring that the crest is maintained reasonably level at the design crest elevation. However, the failure mode will not be carried into the quantitative analysis because of the remoteness of the floods necessary to produce the possibility of an overtopping failure)

Failure Mode Description: Hydrologic studies conducted on behalf of the Flood Control District for Powerline, Vineyard Road, and Rittenhouse FRS (James M. Montgomery - JMM, June 1985) indicated that there is a potential for Rittenhouse FRS to be overtopped by the full PMF storm event. The estimated amount of overtopping based on the design crest elevation was reported in the cited report as a maximum of 0.45 feet with a total duration of five hours. Overtopping of Rittenhouse FRS would occur at the low point of the dam crest. Indications from available survey data are that the crest is significantly below design elevation and quite irregular in elevation with definite low points. Flow would overtop the dam at that point and as flows increased erosion of the crest and downstream slope would occur potentially leading to a breach of the dam.

Adverse Factors:

(1) Hydrologic studies for Rittenhouse FRS indicate overtopping to occur for the full PMF.
(2) Possible deviation in crest elevation from a minimum of 0.5 feet to a maximum of 1.5 feet.
(3) Reservoir tilting not as evident as on for Rittenhouse FRS as on Vineyard FRS.
(4) Transverse cracks would enhance erodibility of embankment.
(5) Evidence of erosion from rills on face of embankment from rainfall.
(6) Smaller outlet pipe – increased chance of plugging and greater drawdown time.
(7) Only one emergency spillway (compared to Vineyard Road FRS – 2).
(8) CAP canal is 30 feet lower than dam (item 8 is not adverse or positive – just factual – flows would still have to pass into the CAP canal before causing damages downstream).

Positive Factors:

(1) PMF an extreme event.
(2) A site specific PMF may not be as high as the JMM PMF.
(3) There are high transmission losses in the upstream watershed.
(4) Short duration of overtopping.
(5) No contribution from another reservoir (as on Vineyard Road FRS).
(6) Dam failure breach outflow characteristics would likely be controlled by CAP canal failure or Queen Creek wash channel failure – which are smaller structures and would result in lesser peak outflow values.

Potential Actions for Risk Reduction:

(1) Confirm dam crest profile elevation.
(2) Verify capacity of Rittenhouse FRS.
(3) Evaluate the role of the CAP canal and the Queen Creek Wash in the spillway discharge and dam breach scenarios.

Consequences:

(1) Buffer land zone between dam and CAP canal large over length of the dam.

(2) No greater population.
(3) 24-hour and/or multiple event may be more critical.
(4) Relatively low level of population below dam.
(5) District has extensive easement upstream and downstream of dam - control of development.
(6) Irrigation outlets can be used in extreme events to discharge water.
(7) Flow direction is away from dam for emergency spillway discharges.

***Failure From Seepage Erosion Through Transverse Cracks Causing Breach Of Dam In Association With A Major Flooding Event (Category I).***

Failure Mode Description: Potential for impounded water to infiltrate and flow into a transverse crack(s) existing in embankment prior to the major flooding event or a transverse crack that develops in association with the flooding. The transverse crack allows the entry of a great enough flow of water to initiate seepage erosion and breaching failure. Two modes were identified for failure: Mode A - Failure through a large (wide) transverse crack that extends across the central filter, and Mode B - failure through entry of flow through multiple upstream longitudinal and transverse cracks, water flows to / through a flaw in the central filter and begins seepage erosion (and loss of central filter material) through a downstream transverse crack. Generally, the mechanics of the potential failure need to be linearized such that there is an upstream transverse crack(s) or inflow zone lined-up with a defect in the central filter, which in turn is lined-up with a downstream transverse crack(s).

Adverse Factors:

(1) Filter defect allowing passage of water to produce seepage erosion could be caused by segregation of filter material, cementation of the filter material, internal instability of the filter and/or loss of filter through downstream crack).
(2) Evidence of placement that could produce segregation of central filter material – found some segregation (removed and replaced with correct material during construction activities for central filter (this was documented in NRCS construction records).
(3) Potential for cementation of central filter and sustaining a crack.
(4) Lots of cracks to carry flow to filter.
(5) Presence of transverse settlement cracks.
(6) Presence of upstream longitudinal cracks which could act to be feeders to transverse cracks.
(7) Transverse cracks may extend below filter depth or go to bottom to contact.
(8) Downstream crack could be widened by hydraulic fracturing by reservoir pressure.
(9) Interconnectivity of transverse and longitudinal cracks (evident of other District structures such as Buckeye FRSs).
(10) Filter does not go to foundation.

(11) There are sections of the dam constructed without filter. Filter not present in abutments.
(12) Finger drains are generally ineffective unless located at a transverse crack.
(13) Central filter is 2.5 feet wide instead of 3.0 feet as in Vineyard Road and Powerline FRSs.
(14) No large ponding tests as conducted at Vineyard.
(15) Relatively longer drawdown time – 30 days.
(16) Evidence of water flow from cracks to downstream toe.

Positive Factors:
(1) Formation of filter cake demonstrated in NRCS test cells as Vineyard Road FRS.
(2) Coarse filter material placed – harder to erode.
(3) A trench investigation that mapped existing cracks was done for Rittenhouse and Vineyard Road FRSs.
(4) Time to cause erosion short (time for water in impoundment in less than 10 days).
(5) Need a lot of feeders to initiate seepage mechanism – Mode B.
(6) Foundation soils pre-wetted on downstream half of dam section.
(7) Filter meets filter criteria.
(8) Filter placed with tremie – minimize segregation.
(9) More clayey zones and less erodible.
(10) Has forty finger drains each three feet wide - 13 drains spaced every 1000 feet and 27 auxiliary drains (located at cracks).
(11) Properties of embankment soils may limit the maximum breach and size of discharge. Low anticipation of large breach outflow. Flows may not be that different from emergency spillway flows.

Potential Actions for Risk Reduction:
(1) Monitor cracks and develop crack mapping program.
(2) Train O&M crews to on mechanism of transverse and longitudinal crack formation.

***Failure From Seepage Erosion Resulting From A Flow Path Through Foundation And Along The Dam /Foundation Contact (Category I).***

(Note: This potential failure mode has three potential pathways. Pathways A and B were rated as Category I potential failure modes. Pathway C was rated as a Category II failure mode and is located in the section under Category II).
Failure Mode Description: : This potential failure mode is initiated by transverse and longitudinal cracks carrying water to foundation in contact with Holocene soils. Path A is initiated by flow into longitudinal and transverse cracks that do not extend through the central filter but do extend to the foundation materials. Path B is initiated through flow into differential settlement cracks with an upstream source of water (impoundment). Path C is initiated from flow through remnant upstream borrow areas that potentially have exposed gravels through which a seepage pathway develops through buried channels into

the foundation Holocene materials. Generally the potential failure mode mechanism is a loss of reservoir via the development of a erosional / piping tunnel and a subsequent breach of dam via downstream embankment slope collapse.

Adverse Factors:

(1) Filter does not extend to foundation.
(2) Higher impoundment head.
(3) Have a high degree of cracking.
(4) Some silt and fine sand lenses exist.
(5) Have seen longitudinal cracks on upstream slope and toe.
(6) More intense storm grouping may be more critical – may facilitate seepage erosion.
(7) For Mode B source of water is impoundment in conjunction with settlement cracks, which may extend to foundation.
(8) Longer impoundment time – 30 day drawdown for any event equaling or exceeding a 100 year event.

Positive Factors:

(1) Need significant storm event.
(2) No direct evidence of Holocene soils under Rittenhouse. However, generally shallower excavation. Drill hole logs similar with Vineyard.
(3) Perhaps shallower excavation in borrow areas.
(4) Central drain interrupts and collects flow.
(5) Need high degree of open fracturing.
(6) Crack has to extend all the way through to foundation and occur in erodible materials and clear to downstream.
(7) For Mode B settlement cracks may not extend to foundation.
(8) Downstream foundation was pre-wetted during construction.
(9) Possibly less cracking than Vineyard – to supply flow adequate to initiate failure mode.

Potential Actions for Risk Reduction:

(1) Need to evaluate existence of Holocene soils in foundation of dam.
(2) Need to prepare and continue crack mapping.
(3) Evaluate finger drains.
(4) Failure mode mainly impacts the lower half of the downstream toe (downstream portion only).

## CATEGORY II – FAILURE MODES OF LESSER SIGNIFICANCE (BUT NOT INCONSEQUENTIAL)

***Failure From Potential Erosion Of Left Abutment Of The Dam During Spillway Discharges (Category II - Considered But Not Highlighted).***

Failure Mode Description: Rittenhouse FRS has one emergency spillway – located just to the left of the left abutment of the dam The emergency spillway is a earth-lined limited

service spillway. A spillway training wall protects most of the left abutment of the dam. However, the spillway narrows a bit just before its terminus and the training wall ends before reaching the main dam / end of the left abutment. Thus it is possible that spillway flows could deflect towards the corner of the dam (junction of the left sidewall (abutment) and the main (straight) portion of the dam. High velocity flows in the spillway could potentially erode the training wall and the left abutment / main dam causing loss of reservoir (see plan view of Dam – Rittenhouse FRS Appendix).

Adverse Factors:

(1) Training dike does not extend to the main section of the dam as it does for Vineyard and Powerline.
(2) The abutment is not protected from erosion either from slope erosion from rills or from high velocity flows in the spillway.
(3) Evidence of existing erosion at left abutment and on embankment slopes.
(4) High velocity flows in spillway channel – approximately 10 to 12 feet per second. for major (design) spillway discharge events.
(5) Training wall is close to dam and abutment.
(6) Spillway section narrows which increases velocity.
(7) Off-road vehicles causing gullying and ruts on abutment slopes exacerbating the erosion potential

Positive Factors:

(1) Low velocities around inner bend of spillway approach channel.
(2) A low consequence of erosion occurs at upper end.
(3) Controlled erosion of abutment may act as fuse plug – perhaps as part of original design.
(4) Training dike protects most of the left abutment.
(5) Spillway forms tailwater for dam abutment at upper end.

Potential Actions for Risk Reduction:

(1) Monitor emergency spillway performance during flood events.
(2) Monitor training dikes and abutments during flood events.
(3) Extend training dike beyond main section of the dam.

***Failure From Seepage Erosion Through An Earth Fissure(s) Causing A Transverse Crack Through The Dam Embankment In Association With A Major Flooding Event (Category II - Considered but not highlighted).***

Failure Mode Description: The presence of earth fissures nearby (within a few mile of Rittenhouse FRS) demonstrates the potential for an earth fissure to manifest itself at the dam embankment. An earth fissure would cause a structural failure of the embankment by opening a crack through the structure. A fissure could also potentially undermine the dam embankment by t abruptly causing separation of structure from the foundation with the dam bridging the fissure. If the fissure occurred in association with a large flood the structural failure from the fissure would provide a path for a seepage erosion breach.

Potentially there will not be a high level of water impounded during fissure expression. In such a case there would be a minimal loss of water.

Adverse Factors:

(1) Moderate to low-risk zone of subsidence at Rittenhouse.
(2) Settlement is continuing at the structure. But appears to be relatively uniform.
(3) Good documentation of fissures and subsidence in the area.
(4) Likely existence of Holocene soils under dam foundation.
(5) Localized groundwater pumping (deep high capacity wells).
(6) Recent (1984) and very recent (2000) evidence of earth fissures in vicinity of Powerline FRS.
(7) Fissure not likely not to be seen until flooding occurs.
(8) Potentially more groundwater wells from possible development especially at south end of structure.
(9) Development tends to mask potential fissures.
(10) Flood not frequency related.

Positive Factors:

(1) Awareness of existing fissures and fissure development. Can be monitored. District has large easements around structure.
(2) Requires flood to initiate fissuring / potential breach sequence. Small floods may cause fissure to express as surface manifestation without failure.
(3) Central Arizona Project canal helps to identify fissure on downstream side of dam. Can be used as a "warning" flag.
(4) Probability of fissure developing concurrent with a large impoundment is low.
(5) Effect of central filter – reduce impacts of water loss from impoundment.
(6) Fissure takes a lot of water but also could be plugged.
(7) Flow direction uncertain.
(8) Where dam is on resistant soil less potential for erosion.
(9) No evidence of strain accumulation.
(10) Farther away from bedrock highs. Less potential based on geology.
(11) Large land area in front of dam between dam and CAP canal. Area can be monitored for fissure potential.
(12) Low potential even if groundwater pumping in area of dam.
(13) Location of known fissures toward Powerline FRS. Five miles to located "new" fissure.

Potential Actions for Risk Reduction:

(1) Recommend a joint agency fissure/subsidence-monitoring program.
(2) Segment dam into smaller dams.
(3) Training O&M crews to know signs of fissure development.
(4) Inspections after rain events to include not only dam but impoundment area and downstream areas of dam.

(5) Adjust easement further out to control development.
(6) Recharge instead of pumping.
(7) Can be monitored if discovered.
(8) CAP canal is a good indicator of visualization of existing fissure.
(9) Small floods may provide warning.

***Failure From Seepage Erosion And Erosion Tunnel Developing Around Gated Irrigation Outlets (Two 24 Inch Gated Outlets) (Category II - Considered But Not Highlighted).***

Failure Mode Description: Infiltration of water into embankment material around pipe and soil is carried away due to seepage erosion along the culvert or piping leading to development of an erosion tunnel followed by caving and formation of a breach. Note that a transverse crack could form at the compaction boundary as discussed for the outlet pipe – such a failure process is part of transverse crack failure mode.

Adverse Factors:
(1) Cutoff collars difficult to compact embankment materials around RCP pipe.
(2) Loose materials beneath pipes.
(3) Possibility of differential settlement/collapsible soils.
(4) P.I.'s may be somewhat lower at Rittenhouse than at Vineyard.
(5) Shrinkage at the compaction boundary.
(6) No filter around outlet pipes.
(7) Presence of inlet and outlet structures.
(8) Smaller diameter pipes– 24 inch RCPs.
(9) Long time for drawdown than at Vineyard FRS - 30 days.
(10) Pipes have not been inspected.
(11) Can not detect sediment transport due to operation pattern.
(12) Gates are present on inlet structures.

Positive Factors:
(1) Short duration of head against the pipe
(2) Phreatic surface not developed.
(3) Not pressurized.
(4) Likely good compaction around pipe.
(5) No known open joints, little head to cause settlement.
(6) The irrigation outlets were tested several times (from stormwater impoundment) with good performance. Had full head in 1972 – no problems observed with these outlets.
(7) Reinforced concrete pipe material - good performance with RCP, meets regulations.
(8) RCP pipe and not CMP.

Potential Actions for Risk Reduction:
(1) Video interior of irrigation pipes.

(2) Monitor irrigation outlets and inspect during regular inspections and during flood events.
(3) Close the gates under normal operations. Open only when District decides to provide water to downstream (this would prevent development of precursor seepage paths developing from leakage out of the culvert flowing along the sides of the outlet).

***Failure From Potential Embankment Slope Instability (Category II - Considered But Not Highlighted).***

Failure Mode Description: Potential embankment slope failure due to internal pressure in the central filter.

Adverse Factors:
(1) Small slip occurrences in area of gated outlets.
(2) Lack of record stability analysis from previous studies.
(3) Presence of existing longitudinal and transverse cracks.
(4) No consideration of presence of central filter.
(5) Potential of hydrostatic head in longitudinal crack.
(6) Downstream slope has small slope ratio.
(7) Shallower upstream cutoff (than Vineyard Road FRS).

Positive Factors:
(1) No significant evidence of past slope instability – no evidence of slope slumping.
(2) Central filter could potentially allow long duration head and development of a phreatic surface (pore pressures) within the downstream slope, however unlike Vineyard and Powerline the presence of finger drains in Rittenhouse permits relief of any pressure buildup in central filter.
(3) Low driving head.
(4) Low internal pore pressure.
(5) Less longitudinal cracks than compared with Vineyard Road FRS.
(6) Some impoundment has occurred.
(7) Some settlement has taken place.
(8) FRS has an upstream cutoff trench

Potential Actions for Risk Reduction:
(1) Conduct stability analyses.
(2) Watch for slope distortion - integrate monument data.
(3) Confirm design slope by survey.

***Failure From Seepage Erosion Or Piping Due To A Flow Path Developing Through The Foundation (Category II - Considered but not highlighted).***

(Note: This potential failure mode has three potential flow paths. The Failure Mode associated with flow paths A and B were rated as Category I potential failure modes and

were presented above. The potential for the Failure Mode associated with flow path C was rated as a Category II failure mode and is presented as follows).
Failure Mode Description: The Failure Mode from pathway C is initiated through remnant upstream borrow areas that potentially have exposed coarse materials (e.g., sands and gravels) in contact with reservoir water which provides a seepage pathway through buried channels to Holocene materials in the dam foundation. Generally the potential failure mode mechanism is a loss of reservoir via the development of a seepage erosion or piping tunnel and a subsequent breach of the dam via downstream embankment slope collapse into the erosion tunnel.

Adverse Factors:
(1) Potential of buried stream channels – pathway from borrow area to downstream toe.
(2) Time for impoundment longer – 30 days.
(3) Have a relatively high degree of cracking.
(4) Some silt and fine sand lenses exist.
(5) Have seen longitudinal cracks on upstream slope and toe.
(6) More intense storm grouping may be more critical – may facilitate seepage erosion.

Positive Factors:
(1) Need significant storm event.
(2) Presence of erodible materials in foundation not verified as on Vineyard FRS.
(3) Erodible materials only in some locations – buried stream channels.
(4) Need a free exit on the downstream toe or downstream of dam for failure mechanism.
(5) No direct evidence of Holocene soils under Rittenhouse. However, generally shallower excavation. Drill hole logs similar with Vineyard.

Potential Actions for Risk Reduction:
(1) Need to evaluate existence of Holocene soils in foundation of dam.
(2) Need to prepare crack mapping.
(3) Remedial measure is to add drain to filter.
(4) Failure mode mainly impacts the lower half of the downstream toe (downstream portion only).

***Failure From Seepage Erosion Due To Flow/Seepage Erosion Through Animal Burrows (Category II - Considered but not highlighted).***

Failure Mode Description: Small animal burrows have been documented in previous inspections and field visits to the dam. The burrows are primarily due to ground squirrels and/or kangaroo rats. The potential failure mode is water impounded on the upstream face of the dam would infiltrate into the animal burrows. The infiltrated water would initiate seepage erosion ultimately leading to an expanded erosion tunnel, collapse of material from above the tunnel and breach on the dam.

Adverse Factors:

(1) Longitudinal and transverse cracks appear to be target zones for burrowing animals due to the open work.

(2) Length and depth of cracks potentially great enough to go through dam.

Positive Factors:

(1) Impounded water is available only for a short time at high elevations in reservoir.

(2) District has a rodent abatement and rodent repair program.

(3) Observation of animal burrows do not typically go through filter.

Potential Actions for Risk Reduction:

(1) Continue animal abatement and repair program.

## CATEGORY III– FAILURE MODES FOR WHICH INSUFFICIENT INFORMATION IS PRESENTLY AVAILABLE FOR MAKING ENGINEERING JUDGEMENTS

No Category III potential failure modes were identified by the FMCA team for Rittenhouse FRS at this time.

## CATEGORY IV – FAILURE MODES WHICH ARE NOT PHYSICALLY POSSIBLE OR WHICH ARE NOT CLEARLY CREDIBLE

***Failure From Seepage Erosion Around Principal Spillway (33 Inch Reinforced Concrete Pipe) (Category IV).***

(Note: This potential failure mode, as stated above, was evaluated as a Category IV failure mode. However the FMCA team recognized that with a slight modification of the above failure mode statement, namely "*Failure from seepage erosion through a transverse crack forming at the special compaction boundary adjacent to the Principal Spillway*" that this modification to this failure mode becomes a Category I for further evaluation due to the potential linkage of the failure mode with a transverse crack and no central filter around pipe. This mode can be treated as part of the Category I "Transverse Crack Failure Mode" described above. The comments below for the most part relate to the failure mode condition immediately adjacent to the outlet pipe.

Failure Mode Description: Infiltration of water into embankment material around pipe and is carried away due to seepage erosion along the pipe or piping leading to formation of an erosional tunnel, caving and formation of a breach.

Adverse Factors:

(1) Cutoff collars difficult to compact embankment materials around RCP pipe.

(2) Loose materials beneath pipes.
(3) Possibility of differential settlement/collapsible soils.
(4) P.I.'s relatively lower.
(5) Shrinkage/linkage at the compaction boundary.
(6) No filter around outlet pipe.
(7) Presence of inlet and outlet structures.
(8) Smaller diameter pipe – 33 inch RCP.
(9) Longer time for drawdown than at Vineyard– 30 days.

Positive Factors:
(1) The principal spillway pipe is located on concrete cradle.
(2) Collars interrupt potential shrinkage cracks.
(3) The outlet is ungated - less time for piping to develop.
(4) The outlet has been tested several times (from stormwater impoundment) with good performance. Had full head in 1972 – no issues observed / reported with outlet.
(5) Reinforced concrete pipe material - good performance with RCP, meets regulations.
(6) RCP pipe and not CMP.

Potential Actions for Risk Reduction:
(1) Video interior of spillway pipe.
(2) Monitor spillway outlet and inspect during regular inspections and during flood events.
(3) Upsize diameter of outlet pipe.
(4) Place filter material around pipe.

***Failure From Potential Erosion Of Emergency Spillway During Spillway Discharges (Category IV).***

Failure Mode Description: The emergency spillway is a limited service earth-lined spillway. Spillway discharges from extreme events could potentially damage the spillway floor and/or initiate headcut erosion into reservoir causing loss of reservoir. There would be little adverse consequences from this erosion– the main potential adverse consequence is relative to reduced flow capacity - this may increase overtopping potential - this seems extremely unlikely to be a significant factor.

Adverse Factors:
(1) The discharge velocity from the PMF event is estimated at 10 to 12 feet per second.
(2) No control sills to interrupt potential head cut.
(3) Damage to spillway basin and training walls at higher flows.
(4) No erosion protection for end of either left abutment.

Positive Factors:
(1) No drop at end of spillway to initiate headcut erosion.

(2) No principal outlet.
(3) This is not really relevant to the damage effect and the discharge is not really directed toward or away from dam.
(4) Relatively short duration of flows in spillway.
(5) Spillway ends with gentle sloping area all the way to the CAP canal.

Potential Actions for Risk Reduction:
(1) Monitor emergency spillway performance during flood events.
(2) Monitor training dikes and left abutment during flood events.
(3) Run SITES model by NRCS to evaluate spillway performance.

***Failure From Potential Seismic Activity Impacting Structure (Category IV).***

Failure Mode Description: A seismic event occurs that impacts structure by opening or expanding existing transverse cracks (this then leads to the earlier described transverse crack failure mode).

Adverse Factors:
(1) Natural variations amplify to cause localized higher acceleration values.
(2) Brittle nature of dam embankment.

Positive Factors:
(1) Theoretical analysis for small strain- no signs of effects.
(2) Dam is already segmented and broken up.
(3) Flood event with earthquake is very rare event for consideration.
(4) Opportunity to view in advance.
(5) No shallow groundwater – therefore no liquefaction potential.

Potential Actions for Risk Reduction:
(1) Examine and inspect dam and outlets and surrounding ground after earthquake (greater than a minimum of 0.05g) and high potential fissure areas.

## 4.0 SUMMARY AND CONCLUSIONS

Rittenhouse FRS was constructed pursuant to a relatively modern dam design. Construction appears to have been without any particular issues. Except for considerable cracking of the structure during and after its first major impoundment in 1972, which necessitated investigations and construction of a central filter repair the dam has performed normally and satisfactorily for 33 years. The structure is satisfactorily maintained and monitored.

However, it is prudent to recognize that there exist for all dams specific ways that failure could come about that warrant attention and diligent monitoring. The identification of a condition or process as a "potential failure mode" does not imply that the dam is about to fail or even necessarily that there is a dam safety deficiency at the site. Rather it

identifies physically possible conditions or processes (generally with a remote but still credible chance of occurrence) that persons associated with owning, inspecting, analyzing and operating the dam should be aware. Some of the potential failure modes are highlighted (or prioritized) for attention of the dam owners and operators. They are highlighted because the specific conditions at the dam and appurtenant structures are such that these failure modes are physically possible and are considered the most realistic and most credible potential failure modes definable at the site.

Three potential Category I failure modes were identified by the FMCA team. One of the Category I failure modes, overtopping due to a flood event approaching the PMF, will not be carried into the quantitative analysis portion of the risk assessments at this time as it is considered to be a very remote possibility. The Category I potential failure modes identified that were considered most significant and appropriate for consideration in the risk assessment were failure due to seepage erosion through transverse crack(s) causing a breach (including a transverse crack in association with the principal outlet), and failure from seepage erosion through Holocene soils left in the foundation. The likelihood of these failure modes, however, is dependent on the presence or rapid development of a seepage erosion path at the time of major flooding from a transverse crack through the central filter (or a flaw in the filter), or from a tunnel through the Holocene foundation deposits. The length of time for any of these failure mechanisms to develop is considered to be long compared to the time that water is impounded in the reservoir (less than 30 days). The length of time for these failure mechanisms to develop is considered long compared to the time that water is impounded in the reservoir during the 100-year event (a maximum of 30 days) but they are all physically possible.

Awareness, monitoring, understanding transverse crack development, and training (classroom training) of operation and maintenance teams of the possibility of the potential for the identified failure modes before, during, and after inspections/flood events are key risk reduction actions that appear warranted. There are not a large number of people and structures at risk in the flood path in the event of a spillway discharge or dam failure by breach. There could be a likely chance that casualties could occur due to a dambreak or spillway discharge. A very significant consequence of dam failure would be the adverse impact on the Central Arizona Project canal. Conversely, depending on the magnitude of the dam failure outflow, the canal and or the Queen Creek Wash could mitigate the adverse consequences of dam failure. This appears to be particularly true for any seepage erosion or piping type failure It is highly recommended that an individual emergency action plan be developed for Rittenhouse FRS.

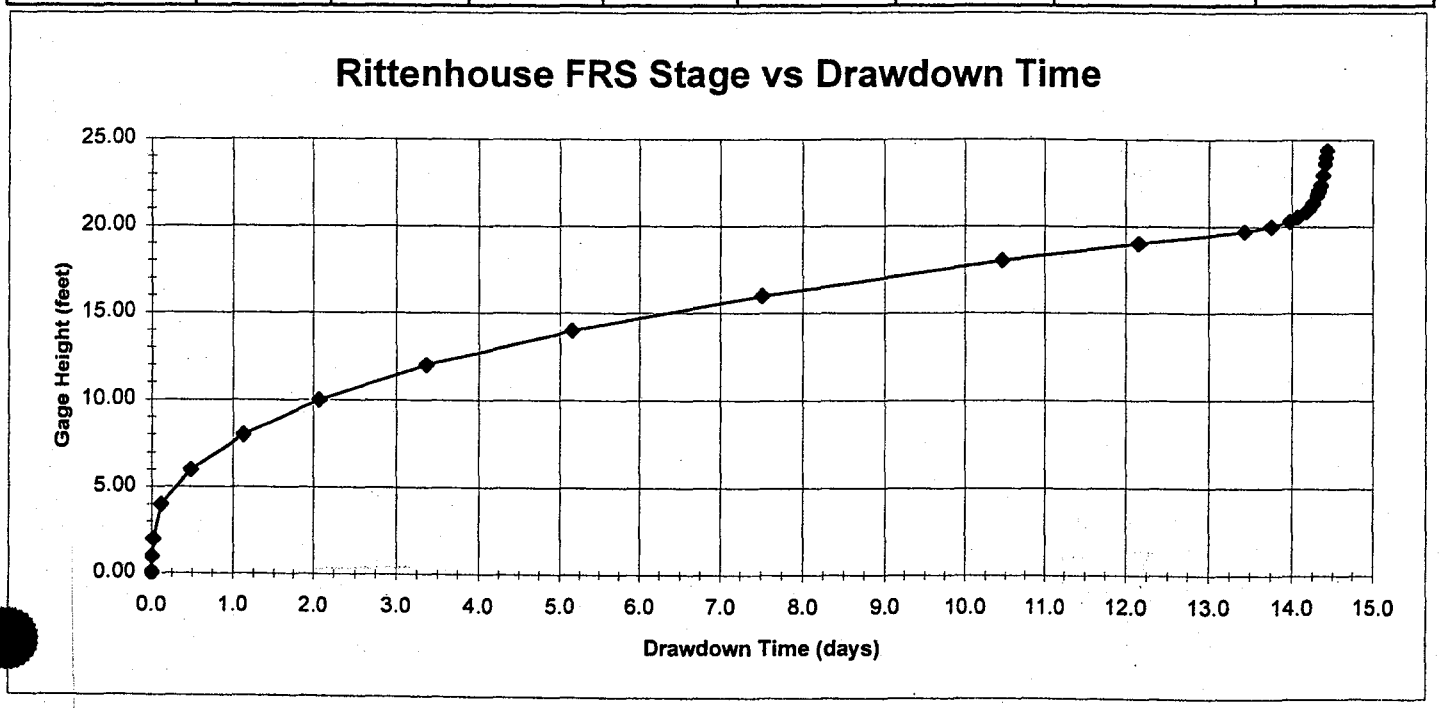
## RITTENHOUSE FRS APPENDIX - FMCA REFERENCE MATERIALS

Rittenhouse FRS south end looking north.
Photo date: October 2001 Photograph Copyright Kimley-Horn and Associates, Inc.

**Rittenhouse FRS Stage vs. Drawdown Curve**

**Curve assumes flow through ungated outlet only, inflow = 0, and no clogging of the outlet**

| Gage Height | Discharge | | | Volume | Incremental | Incremental | Cummulative | Cummulative |
|---|---|---|---|---|---|---|---|---|
| (ft) | (cfs) | (cu.ft/hr) | (ac-ft/hr) | (ac-ft) | Volume | Drawdown | Drawdown | Drawdown |
| | | | | | | (hours) | (hours) | (days) |
| 0.00 | 0 | 0 | 0.0 | 0 | 0.0 | 0.0 | 0 | 0.0 |
| 0.10 | 1 | 3600 | 0.1 | 0 | 0.0 | 0.0 | 0 | 0.0 |
| 1.00 | 6 | 21600 | 0.5 | 0 | 0.0 | 0.0 | 0 | 0.0 |
| 2.00 | 21 | 75600 | 1.7 | 1 | 1.0 | 0.6 | 1 | 0.0 |
| 4.00 | 55 | 198000 | 4.5 | 11 | 10.0 | 2.2 | 3 | 0.1 |
| 6.00 | 79 | 284400 | 6.5 | 69 | 58.0 | 8.9 | 12 | 0.5 |
| 8.00 | 93 | 334800 | 7.7 | 187 | 118.0 | 15.4 | 27 | 1.1 |
| 10.00 | 106 | 381600 | 8.8 | 387 | 200.0 | 22.8 | 50 | 2.1 |
| 12.00 | 117 | 421200 | 9.7 | 685 | 298.0 | 30.8 | 81 | 3.4 |
| 14.00 | 126 | 453600 | 10.4 | 1133 | 448.0 | 43.0 | 124 | 5.2 |
| 16.00 | 135 | 486000 | 11.2 | 1765 | 632.0 | 56.6 | 180 | 7.5 |
| 18.00 | 143 | 514800 | 11.8 | 2600 | 835.0 | 70.7 | 251 | 10.5 |
| 19.00 | 147 | 529200 | 12.1 | 3095 | 495.0 | 40.7 | 292 | 12.2 |
| 19.69 | 150 | 540000 | 12.4 | 3475 | 380.0 | 30.7 | 322 | 13.4 |
| 20.00 | 278 | 1000800 | 23.0 | 3651 | 176.0 | 7.7 | 330 | 13.8 |
| 20.30 | 402 | 1447200 | 33.2 | 3829 | 178.0 | 5.4 | 335 | 14.0 |
| 20.54 | 653 | 2350800 | 54.0 | 3952 | 123.0 | 2.3 | 338 | 14.1 |
| 20.88 | 1155 | 4158000 | 95.5 | 4198 | 246.0 | 2.6 | 340 | 14.2 |
| 21.00 | 1395 | 5022000 | 115.3 | 4266 | 68.0 | 0.6 | 341 | 14.2 |
| 21.38 | 2157 | 7765200 | 178.3 | 4527 | 261.0 | 1.5 | 342 | 14.3 |
| 21.77 | 3159 | 11372400 | 261.1 | 4794 | 267.0 | 1.0 | 343 | 14.3 |
| 22.00 | 3836 | 13809600 | 317.0 | 4934 | 140.0 | 0.4 | 344 | 14.3 |
| 22.11 | 4160 | 14976000 | 343.8 | 5000 | 66.0 | 0.2 | 344 | 14.3 |
| 22.41 | 5160 | 18576000 | 426.4 | 5216 | 216.0 | 0.5 | 344 | 14.4 |
| 23.00 | 7560 | 27216000 | 624.8 | 5657 | 441.0 | 0.7 | 345 | 14.4 |
| 23.64 | 10160 | 36576000 | 839.7 | 6117 | 460.0 | 0.5 | 346 | 14.4 |
| 24.00 | 11960 | 43056000 | 988.4 | 6436 | 319.0 | 0.3 | 346 | 14.4 |
| 24.40 | 13960 | 50256000 | 1153.7 | 6762 | 326.0 | 0.3 | 346 | 14.4 |




Prepared by Ted Lehman June 3, 1998

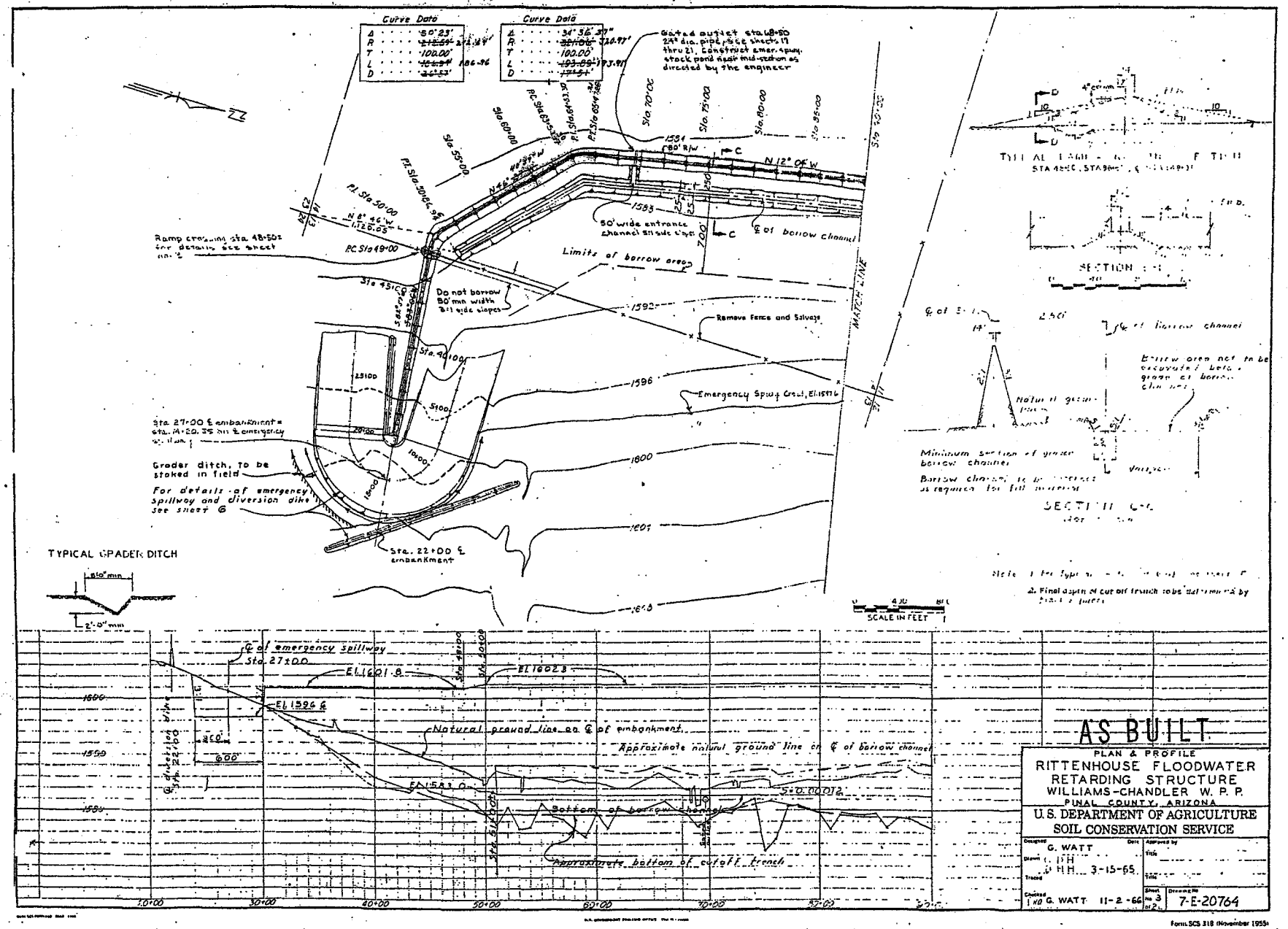



Figure 2-10 Rittenhouse FRS Embankment Plan and Centerline Profiles (Sheet 1)

Form SCS 318 (November 1955)

RITTENHOUSE FRS
DRAIN INSPECTION
PINAL COUNTY, ARIZONA

AS-BUILT GEOLOGIC REPORT

Submitted to:

U. S. Department of Agriculture
Soil Conservation Service - State Office
Room 3008
Federal Building
Phoenix, Arizona 85025

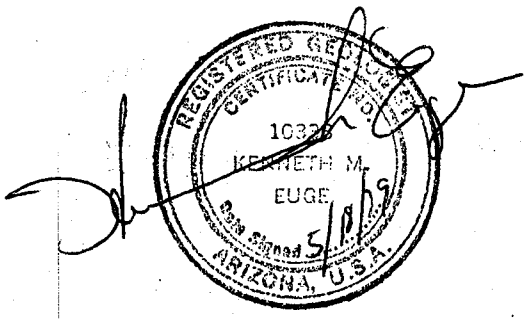
Submitted by:

Fugro, Inc., Consulting Engineers and Geologists
P. O. Box 68
Goodyear, Arizona 85338

SCS Contract No. 53-8A02-9-00047

Fugro Project No. 78-307

May 18, 1979



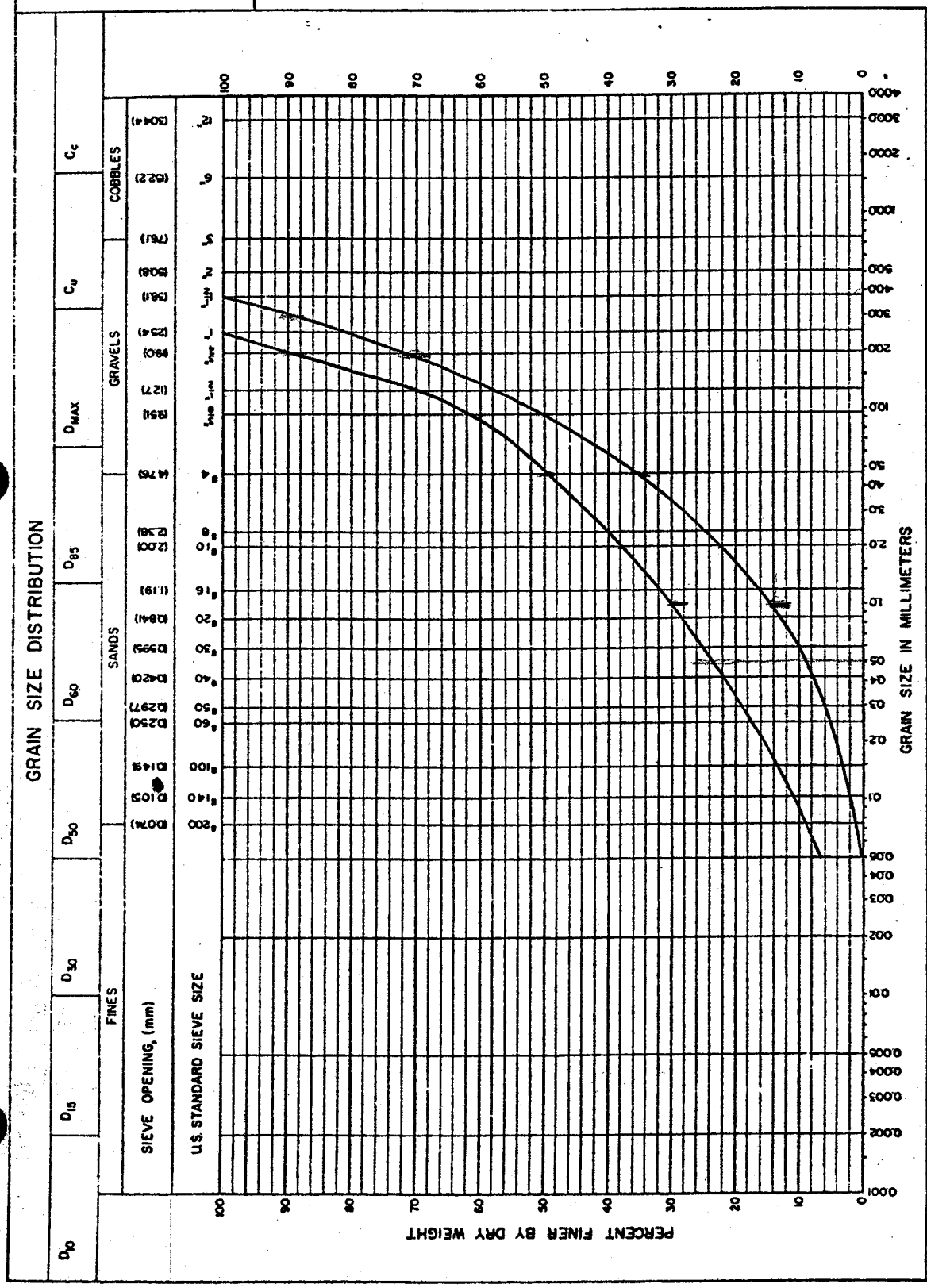

FUGRO

SCS-ENG-353A
Rev. 11-71

LABORATORY NO. _____

| MATERIALS TESTING REPORT | U. S. DEPARTMENT of AGRICULTURE SOIL CONSERVATION SERVICE | SOIL CLASSIFICATION |
|---|---|---|

| | | | |
|---|---|---|---|
| PROJECT and STATE: Rittenhouse FRS Drain, Arizona | | | SAMPLE LOCATION: Specification Limits |
| FIELD SAMPLE NO. | DEPTH | GEOLOGIC ORIGIN | |
| TYPE OF SAMPLE | TESTED AT | APPROVED BY | DATE |
| SYMBOL | DESCRIPTION: Drain Fill | | |




| UNDISTURBED CONDITION | | |
|---|---|---|
| MOISTURE | % | |
| DRY UNIT WEIGHT | g/cc | pcf |

| SHRINKAGE LIMIT | % |
|---|---|
| SOLUBLE SALTS | % |

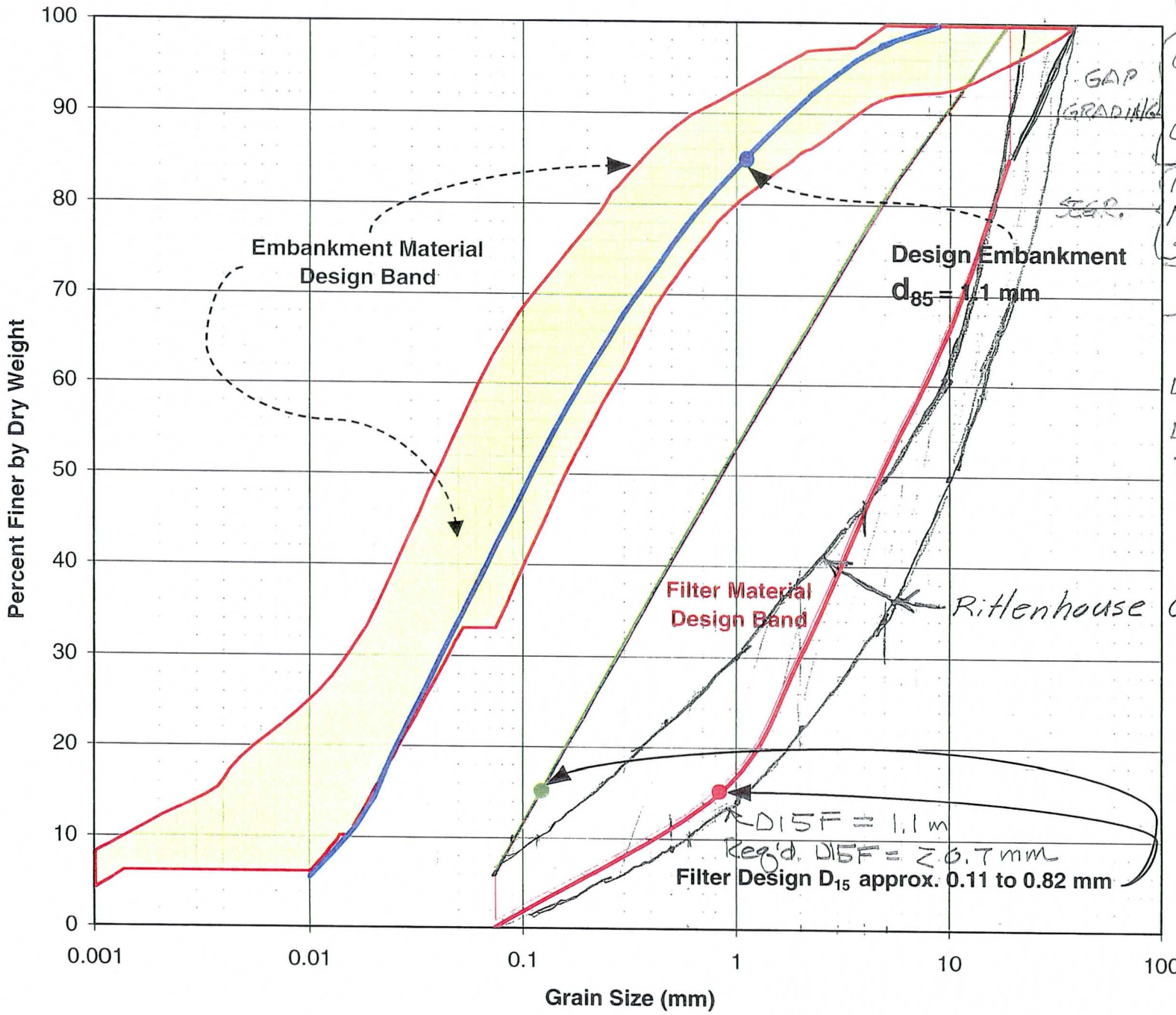
| ATTERBERG LIMITS | NATURAL MOISTURE | LL | PI | AIR DRY | LL | PI | OVEN DRY | LL | PI |
|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | | | |

| SPECIFIC GRAVITY ($G_s$) | (-) #4 | (+) #4 |
|---|---|---|
| | | |

REMARKS:

# FIGURE 1

Gradation of Embankment Soils & Central Drain Aggregate




RITTENHOUSE

$CU = \frac{9.5}{0.11} = 86.3$ v. 16.7

$CU = \frac{2.4}{0.6} = 40$ v 15.6

Min. D10F = 0.11

Max D90F = 32

Should be ≤ 20

Dc ≤

$\frac{D_F}{D_{10}} \frac{.6}{.11} = 5.4 = \frac{D_c}{D_F}$

D20 $D_c/D_F = \frac{1.8}{.33} = 5.45$

D60 $D_c/D = \frac{13}{10} = 1.3$

Requirement

$\frac{D_c}{D_F} = \leq 5$

Essentially OK

**FIGURE 3**

Percent Compaction -vs- Elevation, Rittenhouse FRS Embankment

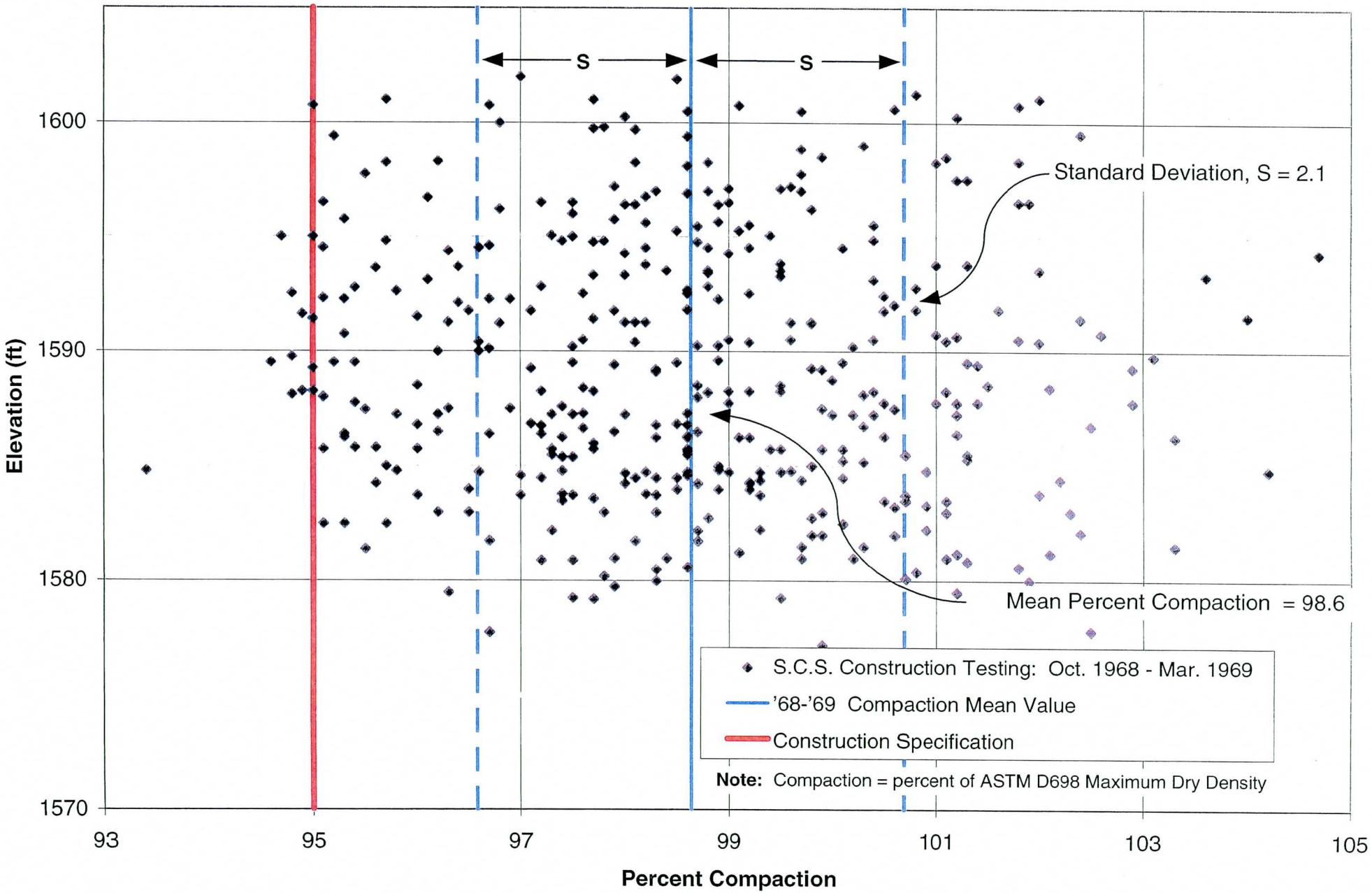


**FIGURE 4**

Variation of Field Moisture Content from Optimum Moisture Content, Rittenhouse FRS Embankment

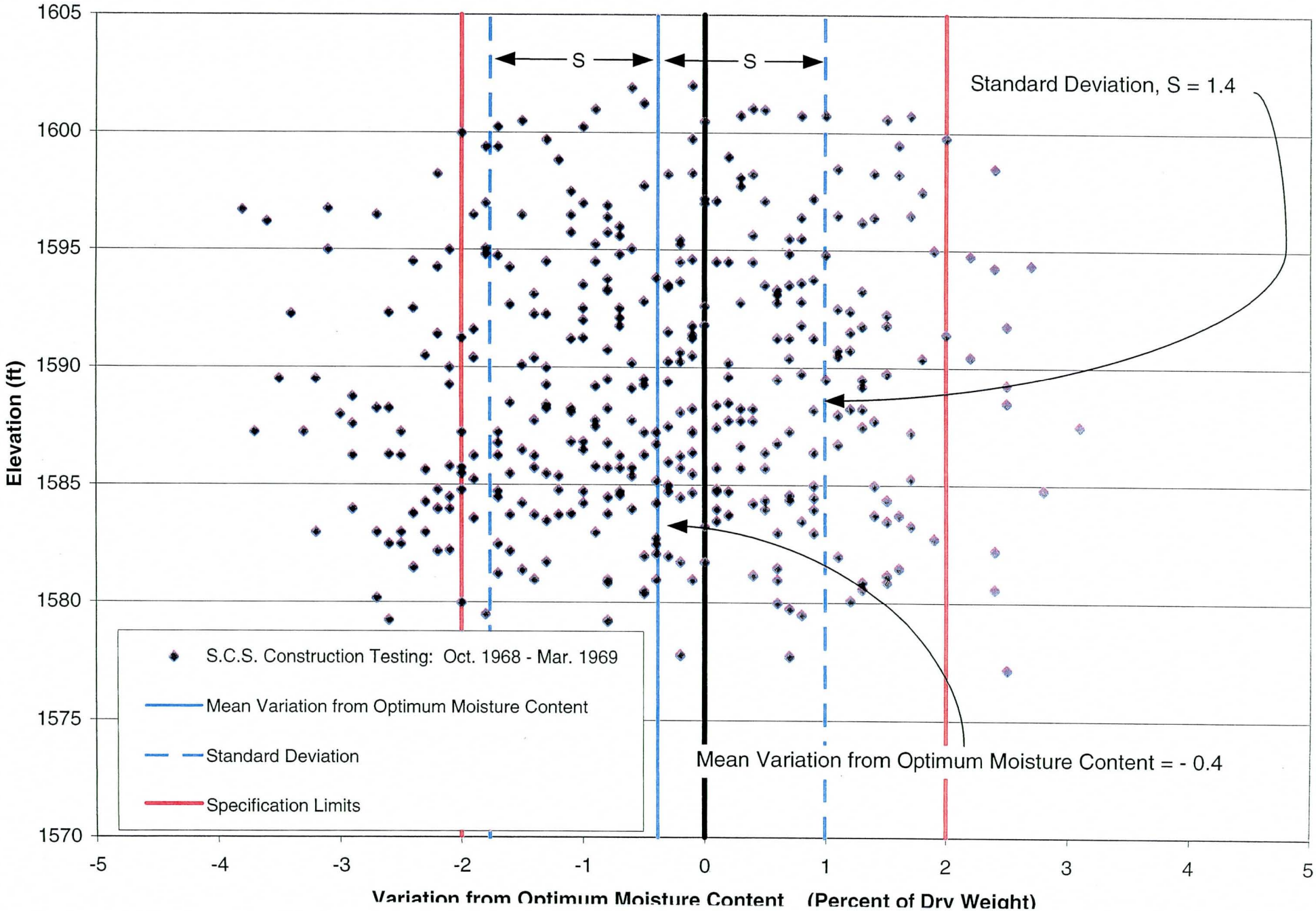


**FIGURE 5**

Degree of Saturation -vs- Depth, Rittenhouse FRS Embankment

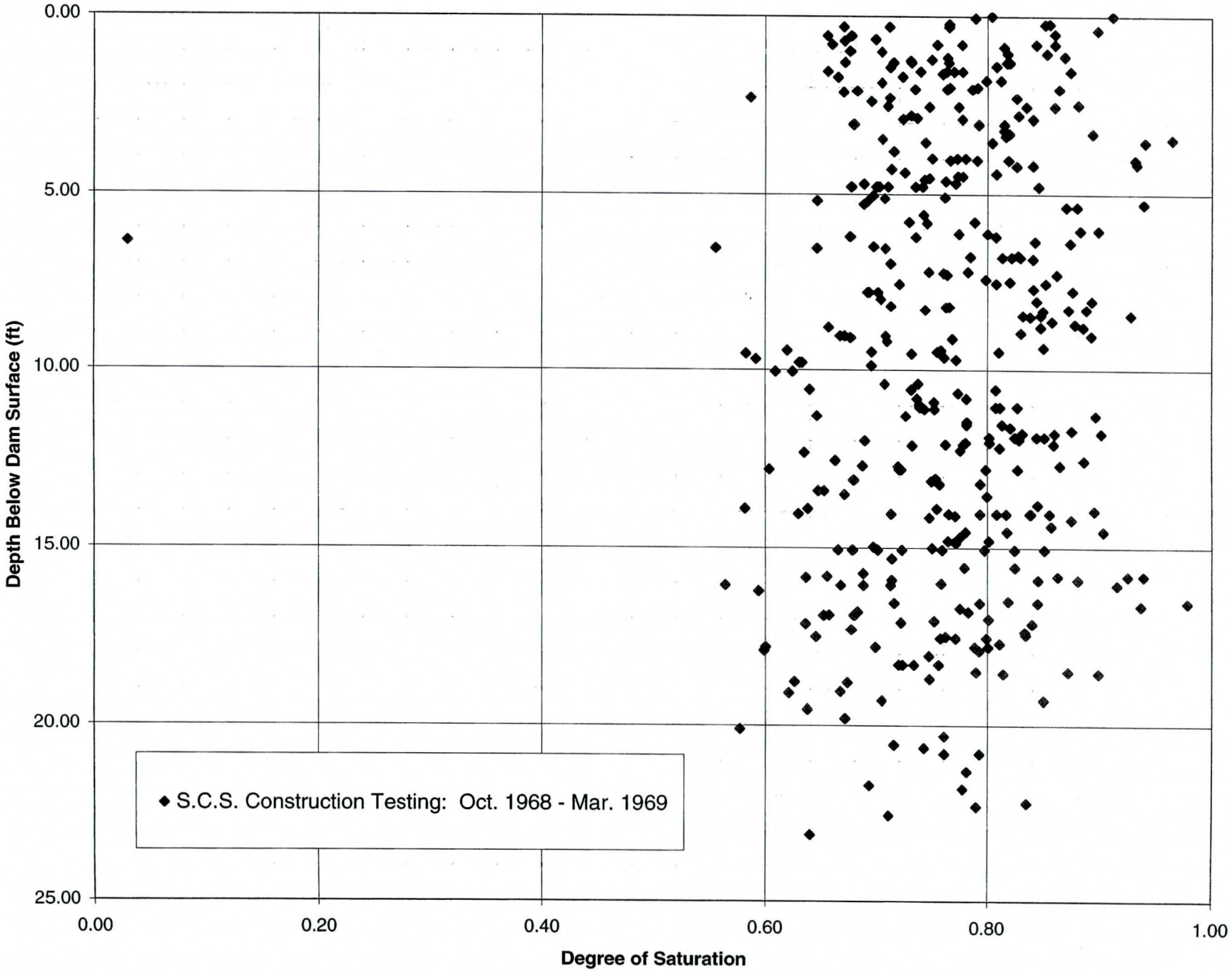
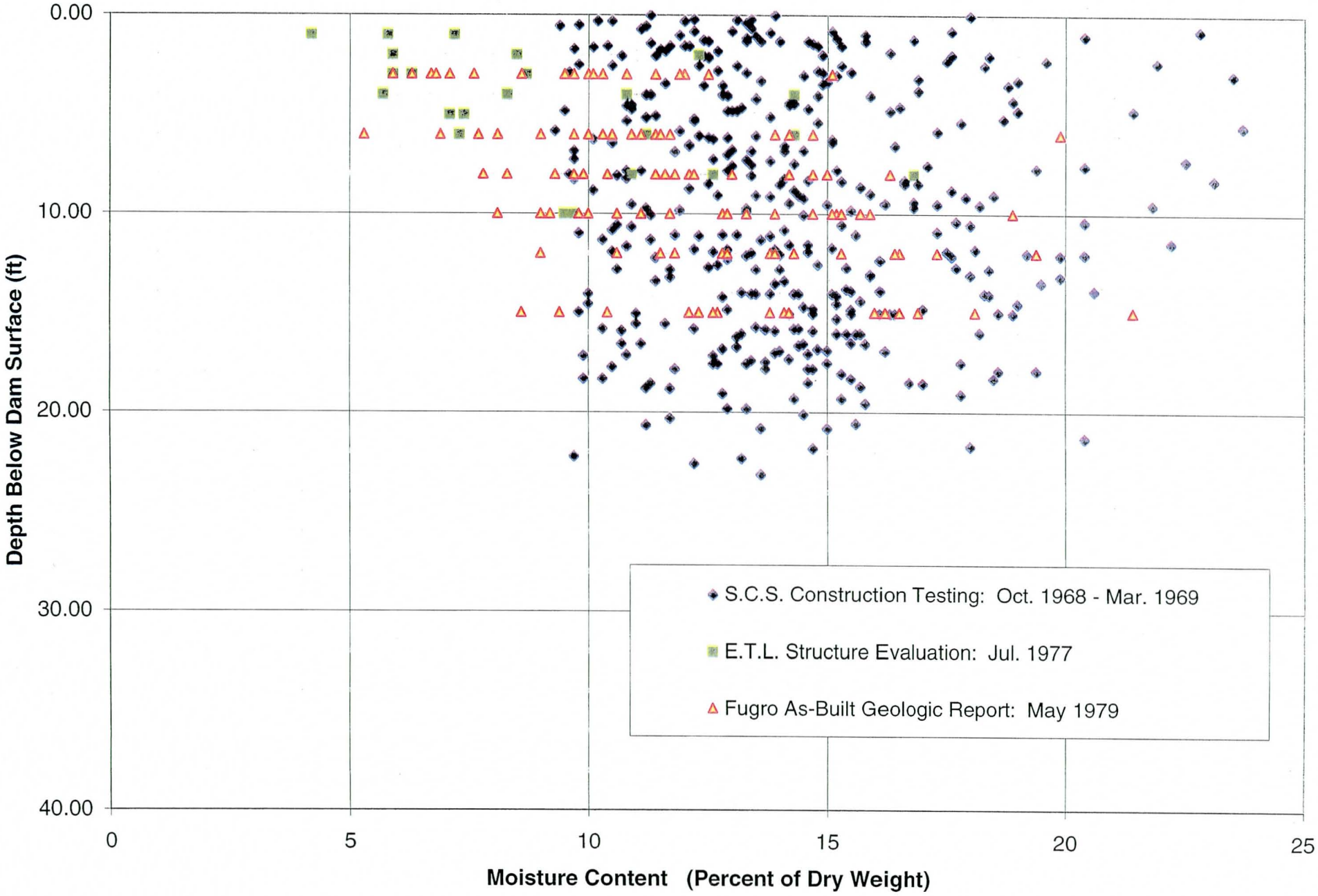
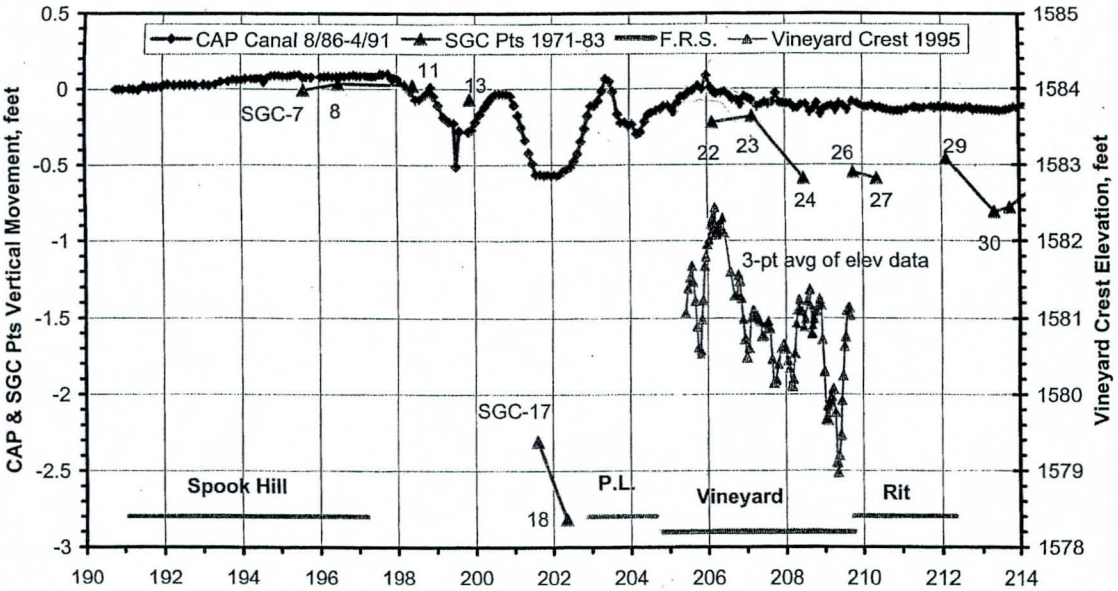


**FIGURE 6**

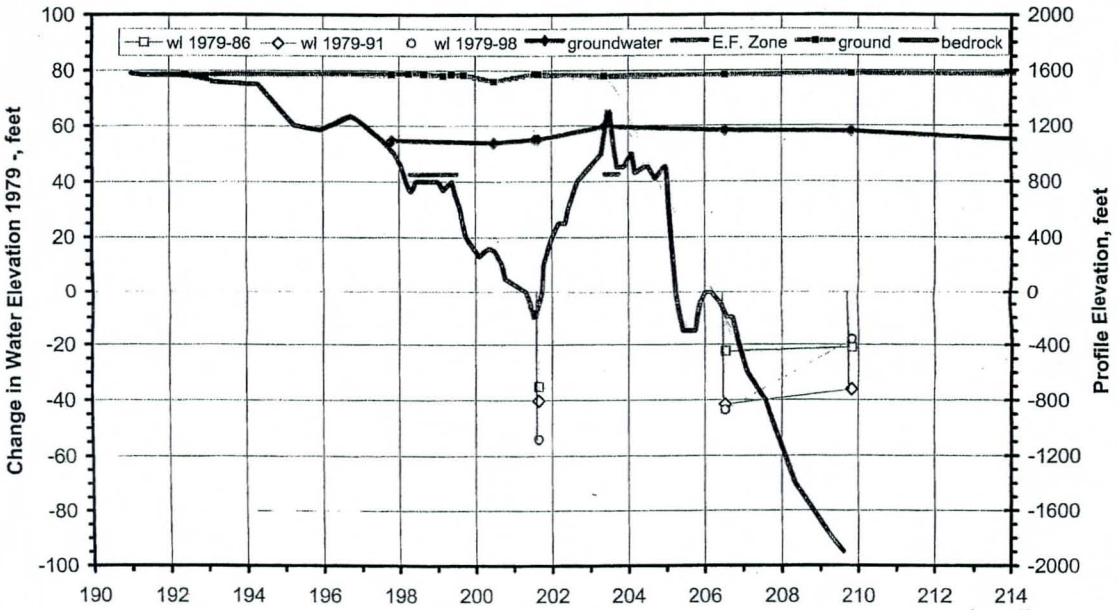
Moisture Content -vs- Depth, Rittenhouse FRS Embankment & Foundation



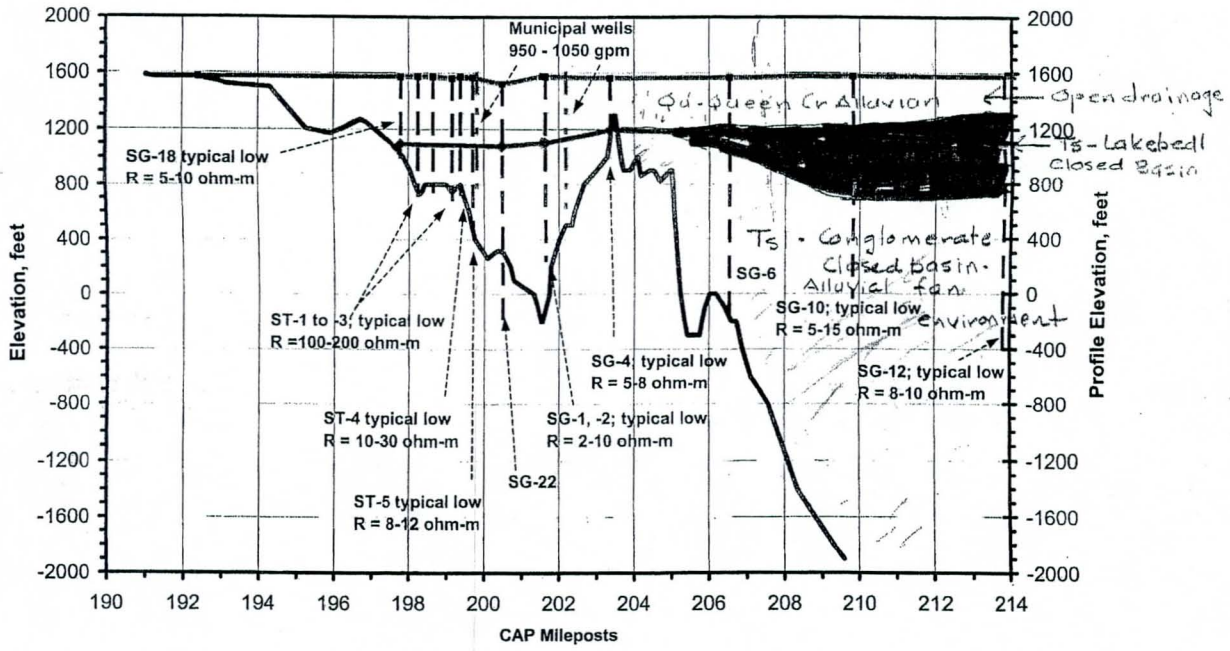
## Survey Data




## Hydrogeologic Profile & Water Level Change




## Deep Well Locations & Data Summary



## COMPARISON OF PROBABILISTIC AND DETERMINISTIC PEAK HORIZONTAL ACCELERATION

| Flood Retarding Structure (FRS) | USGS Acceleration Values for Exceedance Probabilities in 50 Years (g) | | | SEA 99 | Campbell | A & S |
|---|---|---|---|---|---|---|
| | 2% | 5% | 10% | | | |
| Spook Hill FRS | 0.11 | 0.07 | 0.05 | 0.04-0.05 | 0.12–0.15 | 0.10-0.12 |
| Signal Butte FRS | 0.12 | 0.07 | 0.05 | 0.04 | 0.14 | 0.11-0.12 |
| Apache Junction FRS | 0.12 | 0.07 | 0.05 | 0.04 | 0.14–0.15 | 0.11-0.12 |
| Powerline FRS | 0.11-0.12 | 0.07 | 0.05 | 0.03 | 0.10-0.11 | 0.09 |
| Vineyard FRS | 0.12 | 0.07 | 0.05 | 0.03 | 0.08–0.1 | 0.07–0.09 |
| Rittenhouse FRS | 0.12 | 0.07 | 0.05 | 0.02-0.03 | 0.07–0.08 | 0.07–0.08 |

Flood Control District of Maricopa County
Structures Assessment Phase II
Investigation of Seismic Hazard
FCD Contract No. 2000C006
Assignment 2 – Vineyard FRS
AMEC Job No. 0-117-001122
9 July 2001

## ANALYSIS RESULTS FOR DETERMINATION OF BACKGROUND MEAN PEAK HORIZONTAL ACCELERATION

| Activity Model | Magnitude with $10^{-4}$ Annual Frequency | Equivalent Radius (km) | Mean Peak Horizontal Acceleration (g) | |
|---|---|---|---|---|
| | | | A & S | A & S with Hanging-Wall Effect |
| AEIC + NOAA 150-Mile Boundary | 6.7 | 17.8 | 0.20 | 0.29 |
| AEIC + NOAA 150-Mile Boundary | 6.2 | 12.0 | 0.22 | 0.32 |
| AEIC + NOAA 150-Mile Boundary | 6.0 | 10.0 | 0.22 | 0.32 |
| AEIC + NOAA 150-Mile Boundary | 5.4 | 6.0 | 0.19 | 0.23 |
| AEIC + USGS 100-Mile Boundary | 5.1 | 17.8 | 0.05 | 0.07 |
| AEIC + USGS 100-Mile Boundary | 4.9 | 15.0 | 0.05 | 0.07 |

# FAILURE MODE AND CONSEQUENCE ANALYSIS
# For
# SPOOK HILL FLOOD RETARDING STRUCTURE

## PINAL COUNTY, ARIZONA
## NATDAMID AZ 00175

**July 19 2001**

## 1.0 INTRODUCTION

### General Description

Spook Hill FRS is located within the City of Mesa. The FRS parallels the Central Arizona Project canal and begins 1/2 mile north of McDowell Road (at Power Road) and ends at 1/4 mile south of Brown Road and 1/4 mile east of Hawes Road. The FRS is about 22 miles east of downtown Phoenix and approximately eight miles west of the City of Apache Junction. The project consists of the FRS structure, principal spillway, and an emergency spillway. The project is part of the Natural Resources Conservation Service Buckhorn Mesa Watershed Protection and Flood Prevention Project, which includes the Signal Butte and Apache Junction flood retarding structures

Spook Hill FRS is classified as a high hazard, small dam. The reservoir behind the FRS is 232 acres with a capacity of 900 acre-feet. Construction of the Spook Hill FRS was accomplished under contract to Mardian Construction Company. The completion date of the construction of the dam and landscape treatment was April 10, 1980.

The dam has performed satisfactorily to date.

### Dam Data

Dam type: Zoned earthfill
Dam height: 23.0-ft
Dam length: 21,712-ft
Dam crest: width 14-ft; elevation 1591.0-ft
Spillways: Principal - 7-ft 6-in x 7-ft RCB; inlet elev. 1574.5-ft; Top of Sediment pool also = 1574.5 Emergency spillway - 260-ft wide RC drop spillway; crest elev. 1582.0-ft
Freeboard: 6.3-ft
Reservoir Surface: 232 ac at spillway crest
Storage: 1,217-af at spillway crest
Hazard Classification: High

### Purpose and Scope

In general, the purpose of the Failure Mode and Consequence Analysis (FMCA) exercise was to:

- Identify potential site-specific failure modes for the dam.
- Discuss non-quantitatively the likelihood of the identified failure modes.

- Determine whether or not and how important failure mechanisms are being monitored
- Examine the consequences of failure and the adverse consequences of successful operation (e.g. – large spillway releases)
- Identify possible risk reduction actions that may be taken to reduce the likelihood of failure or to mitigate adverse consequences
- Determine what information, investigations or analyses may be needed to resolve uncertainties relative to potential failure modes.

(Note: In this phase, the FMCA team only examined the general nature of the "consequences" for the failure modes identified and where appropriate estimated how they may be different than previously anticipated. Greater detail on the estimate of the magnitude of the "consequences" for each significant failure mode will be addressed in the quantitative portion or risk analysis part of the risk assessment for the dam. At this time hopefully more detailed discussion on the emergency response plan for each structure will also be possible.)

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## 2.0 MAJOR FINDINGS AND UNDERSTANDINGS GAINED

The following is a summary of the major findings and understandings for Spook Hill Flood Retarding Structure as a result of the Failure Mode and Consequence Analysis (FMCA). The findings and major understandings are organized below in relation to the major topics considered in the FMCA discussions. First, as with Signal Butte Dam the greatest apparent risk at this dam relates to spillway discharges. The second topic of major interest is the design and construction of the dam related to the potential for development of failure modes (including the lack of a central filter as a crack stopper). Thirdly, are factors related to the impacts of the planned construction of the Red Mountain Freeway on / adjacent to the upstream slope of the dam. The consequences of adverse discharges from the dam and the actions that were identified to be taken for information gathering or risk reduction are the last two topics under which findings are given.

### Findings and Understandings Related to Flood/Spillway Discharges

- Geometry. As currently configured any appreciable emergency spillway discharge will pass directly over a modified section of CAP canal just downstream of the

emergency spillway. Both CAP canal banks (east bank and west bank), which form the freeboard for the canal, have been constructed as dip sections at the Spook Hill FRS emergency spillway channel in the modified section such that any spillway flow will pass directly over the canal and flood the residential area immediately downstream. The canal bank at this location would be immediate in jeopardy of failing by overtopping erosion. The converse of this statement is that any small emergency spillway discharges will outlet into the CAP canal and potentially avoid discharging into the downstream urbanized inundation areas.

- The Irregular Shape of the Dam Crest and the Existing Roadway Crossings Have In-Effect Sectioned the Dam. The existing roadway crossings (e.g. McKellips, McDowell Roads) have sectioned the dam (into several shorter length dams) with regard to potential consequences of failure, however, since the outflows from any dam failure would empty first into the CAP canal, the canal bank lowering directly across from the emergency spillway puts the impacts of any failure downstream of the spillway (downstream homes would be inundated). The exception to this would be if a dam failure at some other location along the dam would cause failure of the canal at that adjacent location. (Questions to be investigated include: How compartmentalized is the dam? What is the grade of McDowell Road and the other streets compared to the spillway and dam crest elevations? The degree of compartmentalization depends, in part , on the capacity of the culverts under the street embankments)
- Alternatives. Additional low level outlets could provide more capacity for discharge from the reservoir by directly passing flows into the CAP canal.
- Potential for use of CAP Canal to Accept Spillway or Breach Discharges. The CAP canal in this reach generally has approximately 9.0-ft of freeboard that could be used for accepting discharges and volume. However, this capacity is not currently available due to the lowering of the canal across from the Spook Hill Emergency Spillway. It was noted by the FMCA team that diversion of floodwaters into the CAP under large flood events is part of the CAP Reach 11 dikes design (see CAP Aqueduct plan sheet in Spook Hill FRS Appendix). Canal freeboard is consistently high over full structure length. Utilization of this capacity would change the threshold of potential damage downstream appreciably and could also serve to capture and control perhaps the entire outflow from any potential seepage erosion/piping related failure mode.
- Principal Spillway Trash Rack. The trash rack is a vertical corrugated metal pipe with orifice holes that appears to have a relatively high potential for plugging. However, when the reservoir stage reaches 11.5 ft. [El. 1577.5] it appears that flow will spill over into the 7 ft. by 14 ft. box structure. The current trash rack has a high potential for plugging and also would appear to restrict capacity. Risk reduction could be achieved by changing the trash rack design.
- Emergency Spillway. The emergency spillway has a zone of compacted earth on the upstream side of the sill crest; erosion damage is likely due to discharges. May consider grout or concrete lining to minimize maintenance at this location.
- No Overtopping. Hydrology studies indicate that the PMF does not overtop the dam (amount of estimated freeboard is 0.17 feet). Thus overtopping by the PMF is not a

credible failure mode. Based on available survey data the dam crest is at or near the design level.

- Increase Principal Spillway Capacity. Risk reduction by increasing the principal spillway outlet capacity has obvious benefits at low cost (large outlet capacity between elevations 1566-ft and 1574.5-ft is not currently being utilized). These benefits include lowering the threshold of damage flood and provision of greater freeboard under the PMF. Downstream impacts and requirements would need to be evaluated if the outlet capacity is increased.

**Findings Related to Design and Construction of the Dam and the Associated Potential for Failure Mode Development**

- No Central Filter. Spook Hill FRS was not designed or constructed with a central filter by the NRCS. The FMCA team notes that the character of the zoning and materials with which the dam was designed and constructed are markedly different than those for some of the other FRS dams in the region (more coarse materials, some caliche fill, different zones of material). Thus, use of a central filter to cutoff seepage from drying shrinkage cracks was not then and is not now an apparent need. The team's finding is that at the present there is no strong evidence based on the design of the dam and the performance of the dam that a central filter is needed at this time. For confirmation a limited amount of work is appropriate (e.g., shallow trenching, borings and a few SA's & PI's, moisture contents and calcium carbonate contents).
- Transverse and Longitudinal Cracks. Transverse and longitudinal cracks are more likely on upstream than downstream slope due to properties of the materials placed in these zones. The Spook Hill FRS apparently has a lower potential for shrinkage cracking than the Powerline FRS, Rittenhouse FRS and, Vineyard FRS. The limited available testing indicates that the predominately granular soils with nonplastic or low plasticity fines were used in construction. Thus it appears the embankment has a low potential for shrinkage cracking. Moreover, unlike the other five Buckhorn Mesa/Pinal County dams, the Spook Hill FRS was constructed primarily of strongly cemented calcareous soils (caliche) from the CAP canal excavation and the impoundment. Because of exchange of calcium ions and other factors, the cemented soils probably have a lower potential for drying shrinkage than similar uncemented soils embankment. Few transverse or longitudinal cracks of significance have been observed but due to the absence of a filter, special attention should be given to cracks in the dam constructions. As a risk reduction measure, shallow trenching should be performed at selected locations to better characterize the intensity and nature of diagonal and transverse cracks and their potential for seepage-induced erosion. Borings should also be made at selected locations and grain-size analysis, Atterberg limits, moisture content and calcium carbonate tests performed to better define the shrinkage properties of the embankment. Spook Hill has a lower potential for continuous transverse cracks across the section than for most of the other structures examined in this series of FMCA studies (Powerline, Rittenhouse, Vineyard) due to the nature and variability of materials used in the cross section of the embankment. (See Typical Embankment Section plan sheet in Spook Hill FRS Appendix). Risk reduction action relative to the possibility of potential failure due to transverse cracks

is to continue inspections to identify cracks if they appear and then investigate to verify if the crack(s) is/are significant.

- Foundation Cutoff Trench. Foundation cutoff trench depth in relation to the contact of strongly cemented soils is well documented by the "as-built" plans and ADWR inspection reports. The documentation indicates that the cutoff trench extends through the thin layer of collapsing Holocene soils to strongly cemented Late Pleistocene deposits (Caliche) throughout the dam. Thus, the available evidence indicates no potential for significant settlements due to seepage into the foundation and the development of differential settlement cracks. Foundation cutoff excavation was reviewed, approved and documented well by ADWR allowing good confidence by the FMCA team that loose, more erodible, more compressible Holocene soils were removed.
- Low Fissure And Subsidence Risk. The dam appears to be underlain by shallow bedrock throughout. Thus, there does not appear to be a risk of land subsidence due to groundwater withdrawal and the formation of earth fissures. Apparent low fissure and subsidence risk due to local geology.
- Coarse Material Was Placed Near The Center Of The Dam. Coarse Zone 5 material was raked out of the zone 4 toward the central section of the dam (below original ground grade according to the design drawings). Thus the potential exists for piping of the upstream materials (Zone 2) into coarse material placed near the centerline of the dam foundation. However, no clear free exit for the materials is evident (other than into the rock itself) if piping into the Zone 5 did occur. (NOTE: The zoning seemed strange to the FMCA team and far more complex than needed, but no internal erosion risk was perceived. Construction photos and tests indicate that for practical purposes, the embankment is homogeneous and of high quality.
- Hydraulic Gradient. The relatively low emergency spillway crest and 5-day drawdown time from crest of the emergency spillway to the principal spillway invert (top of sediment pool) makes for low hydraulic gradients across the section – greatly reducing the chance for a "static" reservoir type seepage erosion or piping type failure. The "active" pool – from emergency spillway crest to top of sediment pool is only 7.5 feet.
- Landscape Soil Layer. The presence of the landscape layer on the slopes of the dam tends to mask the location of any cracks that may develop and makes identification of cracks difficult.
- Landscape Layer Is A Sacrificial Zone With Regard To Erosion. Removal of this material is not a threat to stability of the dam. However, repair of the eroded area is still required as it may result in channeling flow and eroding actual embankment fill below it.
- Presence of Caliche in Foundation. The Holocene soils were removed from beneath the entire dam so that the caliche (strongly cemented calcareous soils) form the foundation. The caliche is a strong essentially incompressible material with few fractures. It sometimes has compaction problems due to breakdown. (Note: The construction photos showed the caliche was properly broken down on grade. There may not be evidence of a problem with improper breakdown, although the FMCA team discussed the subject at some length.)

- Dam Is Designed Inside Out And Outside In. As a general observation, as a part of the environmental design used at Spook Hill Dam, finer materials were placed on the outside slopes and coarser materials inside the dam. This is the reverse to the typical zoning of dams, which places more erosion resistant materials on the outside slopes to support the more impermeable and more erodible interior materials. However, the tests and photos indicate an essentially homogeneous embankment of high quality.
- Low Potential Failure Mode Associate with Principal Outlet. There is no significant risk of a seepage erosion failure along the principal spillway even though there is no filter diaphragm. The short duration of high reservoir stage and the four anti-seep collars reduce the probability of such a failure mode.
- Irrigation System on Dam. The irrigation system, which was placed on the dam for environmental reasons also, had a potential side benefit of keeping the dam moist and thus combat drying shrinkage. However when the system was taken out of service, it was determined by the NRCS that the irrigation was not needed for avoiding transverse cracks. Justification for this decision was documented by the NRCS (see memorandums in Spook Hill FRS Appendix). The FMCA team concurs with the NRCS judgment based on the material properties of the fill and the nature of the cross section as discussed above.
- Slope Stability. Slope failure is judged not to be a significant failure mode However, because no record of stability analysis was found, it is recommended that analysis be conducted for the record.
- Seismic Risk. Earthquakes are not a significant risk to dam safety. A Seismic Exposure Evaluation being conducted by AMEC indicates that the peak horizontal ground acceleration (PHA) that could potentially be created by the maximum credible earthquake (MCE) is about 0.13g. This is well below the threshold of significant deformation of the embankment. Seismic instability is not considered to be a failure mode.
- Wings Walls At Emergency Spillway. Development of a transverse crack at emergency spillway/embankment contact is not a potential failure mode because of the presence of extensive wing walls extending into the fill both upstream and downstream.
- Slope Erosion
    1. Identified potential failure mechanism is loss of crest.
    2. Treatable by repair guide and procedures.
    3. Low failure potential due to sequence of events/intervention.
- Low Potential Of Occurrence Of Failure Mode Associated With Piping Of Material Through The Lower Portion Of The Dam Where The Foundation Material Was Replaced With Coarse Fill Up To The Original Ground Line. This failure mode has a low potential due to time considerations – there is inadequate time for a flow path and piping to be established. A seepage-induce erosion failure along the foundation-embankment interface or within the foundation is not a significant risk. The highly erodible Holocene soils were removed from the foundation; the caliche foundation and embankment are both highly resistance to erosion. The foundation does not have fractures or other discontinuities that could provide pathways for seepage erosion.

## Findings Related to the Proposed Construction of Red Mountain Freeway

- Arizona Department of Transportation Actions. In the planning and design of the Red Mountain Freeway (Loop 202), ADOT should assure that the modified dam, as a minimum, provides the same level of flood protection as the existing dam and meets all of the present geotechnical and hydrological criteria including those incorporated into the "Rules and Regulations Pertaining to Dam Safety Procedures" promulgated by ADWR on June 12, 2000. This should include evaluation of the impacts of dam modifications on the inundation area of emergency spillway discharges and the reservoir pool inundation area.
- In the course of project planning and design, ADOT should perform risk analysis of the modified dam following the methodology of the present analysis. Reference letter from the District to ADOT dated July 2, 2001, which outlines these and other requirements. ADOT should perform a risk analysis and address potential risks relative to the impacts of the freeway construction on Spook Hill FRS, including reservoir impoundments, downstream consequences, etc.
- Red Mountain Freeway (Loop 202) Benefits. The freeway will effectively widen the embankment of the dam, lower global hydraulic gradients, increase the length of seepage paths and the factor of safety against seepage erosion and increase the margin of safety for slope failure.
- Characterization of Risk Issues Related to Proposed Red Mountain Highway Construction. If the Red Mountain Highway is constructed integral with Spook Hill FRS it is believed that the ADWR will require the dam to be raised to meet the PMF plus 3 feet of freeboard (Option A). To avoid this construction, one of the potential options is to eliminate the dam (make the dam non-jurisdictional). If the dam is eliminated, the required 100-year protection from flow off of the Spook Hill watershed would be provided through the outlet works to the Salt River (Option B). Option A and B are geared to meeting legal requirements and appear to bracket the exposure to risk situation applicable to downstream residents.

  Option A – provides maximum protection against potential overtopping. However, under Option A, other potential dam failure risks still exist and risks related to spillway discharges are equal to the current conditions. These include floods equal to, or exceeding the frequency flood that would result in spillway discharge with damage potential ($D_{SPC}$) up to the damage at PMF level discharge ($D_{SPPMF}$).

  Option B – Provides protection up to 100-year flood. Flow would be directed to one low point and then discharged. Discharges exceeding this would ostensibly pass through the current spillway location, over the canal, and into the neighborhood. Dam failure risks would be eliminated (or at dambreak locations at major crossroads and the effect (consequences) is approximately the same as for discharges from the emergency spillway – the flows would most likely result in inundation of the same downstream areas).

  From the FMCA, it was evident that the risk costs (damage cost times the likelihood of occurrence) from dam failure are small compared to the risk costs from emergency

spillway discharges. Thus, the optimum risk reduction alternative appears to be what is reasonable, possible, and cost effective with respect to containing and controlling spillway discharges, rather than either Option A or B, which do not really address discharges into the community. It was clear from the FMCA that the greatest opportunity for risk reduction is to control non-dam failure discharges and this is just as true if there is no dam (non-jurisdictional alternative/situation).

Several possibilities for risk reduction exist for both the dam and non-dam alternatives that may be able to reduce the risk to levels well below either Option A or B. These potential risk reduction measures combine all or part of the following actions:

- Use the excess capacity of the CAP canal to receive spillway discharges.
- Use an overflow floodway that would be constructed in association with the highway corridor (See Kimley-Horn and Associates report "Alternatives Analysis Report for Spook Hill FRS, Signal Butte FRS, and Apache Junction FRS – August 2001).
- Use an expanded principal outlet capacity and Spook Hill floodway capacity.
- Provide overtopping protection at various (one or more) locations along the CAP Canal to prevent canal breaching in the event flood flows discharged from Spook Hill into the Spook Hill floodway and CAP exceed those carrying capacities. By providing such features additional flood protection would be provided to the residential areas below the dam and canal and the flow depths in the event of any canal overtopping would be minimized.
- Direct discharge from contributory basins directly into canal by use of secondary principal outlets rather than crossing highway boundaries (See Kimley-Horn and Associates report "Alternatives Analysis Report for Spook Hill FRS, Signal Butte FRS, and Apache Junction FRS – August 2001).

The FMCA team discussions indicated that effective combinations of these options could be formulated to achieve lowest cost and lower risk than the "legal" Options A and B cited above. It is also possible (but not yet verified) that implementation of some of the options, alone or in combination could allow the existing dam to pass the PMF with the "required" freeboard because more flow would be passed earlier.

**Findings Related to Potential Consequences of Dam Failure or Spillway Discharges**

- Urbanization. Rapid urbanization continues upstream and downstream. This is expected to increase the consequences of emergency spillway discharges and dam breaches due to seepage-erosion failure modes and the importance of the upstream evacuation plan.
- Flood Volumes and Flows. Research record to determine if inflow hydrographs have been developed for various frequency flood events. Determine capability of CAP to accept flood flows from Spook Hill FRS (flow rate and volume). If these records cannot be located or have not been prepared, then initiate a study to calculate volumes and flows for the various floods and determine flow capacity in CAP canal (for

accepting spillway discharges or dam failure flows from the dam). (Note: Incipient flood frequency and threshold overtopping of CAP canal should be determined. Various design storms could be investigated including multiple winter storms)

- Upstream Easements. District has upstream easement to PMF line, which is only a short distance from the top of spillway line. Home construction appears to be abutting the PMF line over a considerable portion of the reservoir. The District should verify the easements.
- No Individual EAP. An individual Emergency Action Plan for Spook Hill FRS needs to be developed according to ADWR guidelines and FEMA 64 criteria.

**Actions To Be Taken With Respect To Risk Reduction Or Information Gathering**

- Buried Stream Channels under Dam. Relatively deep excavation of the Holocene soils was done at four buried stream channels to reach the strongly cemented deposits. These excavations were about 25 feet at Station 222+00, 15 feet at Station 242+00, 15 feet at Station 266+00 and 30 feet at Station 280+00. The SCS quality control testing from the original construction indicates that the fill was compacted to a mean of 98.7 percent of ASTM D698 maximum dry density and mean moisture content of 1.3 percent below the optimum moisture content. The fill appears to be of high quality and not susceptible to significant settlements and cracking when wetted. However special observation and review of monitoring data should be done in the areas of deep localized sub-excavation to confirm that that there are no significant differential settlements or cracks.
- Utility Crossings. The details and locations of utility crossings should be added to the as-built plans of the dam.
- Right-of-Way Permit Process. There is a need to improve right-of-way process to ensure the dam is not inappropriately "invaded". The FMCA team identified a need to improve right-of-way permit process pertaining to as-builts for modifications by others such as utilities.
- Clear Zones Upstream and Downstream Toes. The FMCA team identified a need for clear zones at upstream and downstream toes for emergency response and maintenance.
- Animal Burrows. It was concluded that animal burrows at this site is not a dam safety issue due to shallow depths of penetrations and primarily located on talus at toes and landscape soil layer. In addition the District O&M Division conducts an ongoing animal control program and animal borrow repair program.

## 3.0 POTENTIAL FAILURE MODES

Potential failure modes identified by the FMCA team are presented below. The failure modes were placed into one of four categories listed below. Except in special cases only those potential failure modes in Category I will be considered in the risk analysis phase of the risk assessment exercise.

- Category I – Failure modes of greatest significance.
- Category II – Failure modes of lesser significance (but not inconsequential).

- Category III – Failure modes for which insufficient information is presently available to allow the team to make a judgement on the significance of the failure mode. The development of additional data and information is warranted. Additional records research may be justified.
- Category IV – Failure modes which are not physically possible or which are clearly not credible.

For each of the potential failure modes identified, a Failure Mode Description is briefly described and the factors that make the failure mode more likely (adverse factors) or less likely (positive factors) to occur are listed following the Failure Mode Description. In addition, any identified potential actions for investigation or risk reduction for each potential failure are then provided.

## CATEGORY I - FAILURE MODES OF GREATEST SIGNIFICANCE

### *Adverse Consequences Resulting From Emergency Spillway Discharges During Major Rainfall Events (Category I)*

Failure Mode Description: The Spook Hill FRS emergency spillway is a 260 foot wide reinforced concrete drop spillway. Normal flood discharges from the spillway are directed into the CAP canal and directly into a subdivision within the City of Mesa. This potential "failure mode" does not "fail" the dam or emergency spillway. However any appreciable flows from the spillway would likely cause an overtopping breach of the CAP canal. This potential failure mode was rated as a Category I failure mode because normal "successful" operation of the emergency spillway can produce discharges that could have significant adverse consequences and the likelihood of occurrence of these adverse consequences appears to be much greater than any mode involving dam failure. Most of the floodwater will pass through the emergency spillway. From that point, the water will flow over the CAP canal where it will immediately flow into a large downstream housing development. The CAP canal would also be eroded at this same location adding to the discharge into the community.

Adverse Factors:

(1) Spillway flows are directed into the City of Mesa for flow greater than 100-year event in upstream watershed. Downstream populations at risk potentially around 10,000 people.
(2) No dedicated downstream easements from spillway.
(3) Time to peak for PMF is 4.3 hours.
(4) No individual emergency action plan.
(5) No regulation (development, density, land use) on downstream spillway discharge areas.
(6) The CAP canal bank is cut down at the emergency spillway (removing its normal freeboard) facilitating flows passing through canal to downstream housing.
(7) 72-hour or 24-hour PMF may be more critical.
(8) Proposed Red Mountain Freeway changes downstream consequences.

(9) Land use is changing in upstream watershed. More highly urbanized.

Positive Factors:

(1) Dam is instrumented as part of the District Alert System. Would provide warning.
(2) Spillway discharge exit is normal operation and straight forward.
(3) Short spillway discharge duration for PMF.
(4) The CAP canal is located downstream from dam – picks up a minor amount of spillway flow (but could pick up much more).
(5) Reinforced concrete drop block spillway – good erosion control.
(6) Canal bank is relatively high – providing potential capacity if low spot at spillway is restored to design level on most downstream bank.
(7) Likelihood of spillway flows maybe low.
(8) Precedent to use canal to receive discharges from the dam (CAP Reach 11 dikes).

Potential Actions for Risk Reduction:

(1) Prepare emergency action plan according to minimum standards in FEMA 64 and ADWR rules and regulations.
(2) Increase outlet capacity to Spook Hill floodway.
(3) Restore canal bank on opposite bank to full height.
(4) Add other outlets to discharge to canal.

***Failure Due To Seepage Erosion Along A Transverse Crack Through Embankment In Association With A Major Flooding Event (Category I).***

Failure Mode Description: Potential for impounded water to infiltrate and flow into a transverse crack(s) existing in the embankment prior to the major flooding event or a transverse crack that develops in association with the flooding. The transverse crack allows the entry of a great enough flow of water to initiate seepage erosion and breaching failure. Two modes were identified for the failure: Mode A - Failure through a major, large, wide transverse crack extending from upstream to downstream. Flow through the crack initiates seepage erosion which spreads lateral till a breach failure is formed; Mode B - failure through a number of upstream transverse cracks carrying water to a downstream transverse cracks initiating an erosional failure as described above.

Adverse Factors:

(1) Ability to recognize transverse cracks is difficult due to landscaping and deep erosion rills.
(2) Wind and traffic may mask transverse cracks.
(3) Three locations of deep excavation to remove SM sand in foundation - may cause differential settlement and thus form a potential location with a tendency for development of a transverse crack.
(4) No central filter/crack stopper.
(5) Minor transverse cracking documented since original construction.
(6) No significant impoundment history – dam not tested – (6.74 feet in 1993).

(7) Some longitudinal cracking documented.
(8) Abandoned irrigation system – potential pathway for seepage and erosion issues.

Positive Factors:
(1) Any crack to be a concern would show at crest.
(2) Can identify cracking by paving top of dam. Cracks promulgate through pavement.
(3) Only minor number of documented transverse cracks.
(4) Highly calcareous nature of soils - stiffer less erodible, less potential for drying shrinkage.
(5) No significant impoundment history.
(6) Short time of impoundment – less than 4.6 days – affects saturation and possibility of failure mode development.
(7) Variable materials in cross-section - less potential for cracks and greater chance of plugging.
(8) Low hydraulic gradient.
(9) Landscape topsoil protects embankment for drying.
(10) Irrigation system not really needed.
(11) ADWR records indicate foundation as "acceptable" (September 1, 1978 construction inspection report).
(12) Downstream dam section is essentially a crack stopper due to its gradation.
(13) Cracking more likely to occur on upstream side - borrow tests show low plasticity.
(14) Transverse cracking less likely compared to Pinal County dams.

Potential Actions for Risk Reduction:
(1) Monitor cracks and develop crack mapping program.
(2) Train O&M crews to on mechanism of transverse and longitudinal crack formation.
(3) Repair any significant cracks upon their development.
(4) Increase outlet capacity to reduce impoundment time even more.
(5) Evaluate need for central filter.

## CATEGORY II – FAILURE MODES OF LESSER (BUT NOT INCONSEQUENTIAL) SIGNIFICANCE

***Failure From Overtopping Of Spook Hill FRS (Category II – Considered but not highlighted).***

Failure Mode Description: New draft hydrologic studies conducted on behalf of ADOT for the Red Mountain Freeway for Spook Hill FRS indicate that there is a potential for Spook Hill FRS to be overtopped. Overtopping of Spook Hill FRS would occur at the low point of the dam crest (assuming that the crest is not level). Flow would overtop the dam at that point and as flows increased erosion of the crest and downstream slope potentially could occur leading to an eventual breach of the dam.

Adverse Factors:

(1) New draft hydrologic studies for Spook Hill FRS indicates overtopping to occur.
(2) A higher PMF is possible for this structure based on HMR 49.
(3) Land use changes and density of urbanization increased since construction of dam.
(4) Only 0.17 feet of freeboard under HMR-49 PMP.
(5) Multiple upstream reservoirs.

Positive Factors:

(1) Old hydrologic studies indicate no overtopping.
(2) The crest surveys shows a fairly level crest.
(3) Only 2 to 3 locations less than 1591.5 feet elevation.

Potential Actions for Risk Reduction:

(1) Confirm crest profile elevation.
(2) Verify capacity of Spook Hill FRS.
(3) Verify hydrology for PMF.
(4) Increase capacity of principal spillway outlet.

Consequences:

(1) Less buffer zone between dam and CAP canal but still exists over most of the dam.
(2) Large downstream population.
(3) 24-hour and/or multiple event may be more critical.
(4) Incremental damage assessment should be conducted.
(5) District has extensive easement upstream of dam to PMF- control of development.
(6) Upstream development will continue.

***Failure From Seepage Erosion Or Piping Around Principal Spillway Outlet (7.5 Ft By 7.0 Ft Reinforced Concrete Box) (Category II – Considered but not highlighted).***

Failure Mode Description: Infiltration of water into embankment material around outlet is carried away due to seepage erosion along the pipe or due to piping leading to formation of an erosional tunnel, caving and formation of a breach failure.

Adverse Factors:

(1) Seepage cutoff collars make it difficult to compact embankment materials around RCP box.
(2) Potential for loose materials beneath box.
(3) Possibility of differential settlement/collapsible soils.
(4) Shrinkage/linkage at the compaction boundary.
(5) No diaphragm filter around outlet box/pipe.
(6) Presence of inlet and outlet structures.

Positive Factors:

(1) Four concrete seepage collars help intersect any shrinkage cracks that would develop near the structure.
(2) Firm caliche foundation - built on caliche.
(3) Collars interrupt potential shrinkage cracks.
(4) The outlet has been tested (from minor storm water impoundments) with good performance.
(5) Reinforced concrete material - good performance with RCP, meets regulations.
(6) Short impoundment time - less time for development of piping.
(7) Box joints appear tight from visual inspection.
(8) No clear exit for potentially eroded material.

Potential Actions for Risk Reduction:

(1) Inspect for seepage outside of box area on downstream.
(2) Place elevation staff gage at inlet.

***Failure From Slope Erosion Leading To Dam Breach In Association With Major Flooding Event And Rain At The Dam (Category II – Considered but not highlighted).***

Failure Mode Description: Severe slope erosion gullies on downstream slope would progress to the dam crest level, then across the crest potentially leading to breach of dam during heavy rain at the dam and high impoundment level.

Adverse Factors:

(1) Existence of landscape soil layer materials on the slopes of the dam (Non-compacted topsoil).
(2) Fined grained materials were placed put on the outside zones of dam (behind the surface layer of landscape material) – this is adverse with respect to erosion potential.
(3) Over-steepened topsoil benches lead to and induce erosion.
(4) One-foot thick topsoil layer over slopes - loosely placed - initiates rills and gullies.
(5) Ten-foot width of Zone 1 material upstream slope, 10-foot width of Zone 2 on downstream slope.
(6) Landscape layer makes transverse crack identification difficult.

Positive Factors:

(1) Readily observable if crest was to be impacted.
(2) Vegetation stabilizes slopes.
(3) Dam is relatively level.
(4) Time: going to take time for the process. May take successive events.
(5) Upstream erosion may not lead to failure.
(6) Erosion zone is a sacrificial zone - repair after Zone 2 material is impacted or crest is reached.

Potential Actions for Risk Reduction:

(1) Set limits for repair of slopes - develop slope repair guidelines.
(2) Continue seeding of slopes.
(3) Keep Crest Level.
(4) Consider rock mulch at groins.
(5) As long as gully does not invade crest performance should be fine.

***Failure From Piping Along Base Of The Dam (Lower Portion Between Original Ground And Foundation) (Category II – Considered but not highlighted).***

Failure Mode Description: Flow path develops through Zone 2 into Zone 5 rock materials in the center of the dam. Removal of Zone 2 materials into voids of big rocks in cutoff trench. Caving occurs leading to a breach or a pipe is formed through downstream materials leading to caving and a breach.

Adverse Factors:

(1) Coarse material from rock raking of Zone (type) 4 materials into cutoff trench as documented in construction photographs. Coarse material most likely located in downstream edge of cutoff trench. Probably over full length of dam.
(2) Difficult to get compaction around large coarse materials in cutoff trench.
(3) Compaction problem for caliche - may have created layer of horizontal permeability.
(4) Likely that Zone 2 material can be piped into Zone 5 if a flow path develops.

Positive Factors:

(1) Large rock size likely to be part of total matrix - not isolated. (Refer to ADWR photo dated Sept 8 1978 from ADWR inspection report).
(2) Caliche may break down over time.
(3) Not enough time for the process to complete.
(4) No clear exit evident for eroded material.

Potential Actions for Risk Reduction:

(1) Monitor and observe downstream slope and toe during regular inspections and during flood events.

***Failure From Seepage Erosion Along Buried Utilities (Category II – Considered but not highlighted).***

Failure Mode Description: There are ten documented buried utility crossings of Spook Hill FRS. The general potential failure mechanism is water would seep along outside of the buried utility and erode embankment materials and/ or embankment/foundation materials. The erosion will cause a erosional tunnel to form leading to caving and a breach of the dam.

Adverse Factors:

(1) Buried utilities form a potential path for seepage.
(2) Do not have as-builts for all buried utilities.
(3) Pressure utilities could create a problem due to leakage.
(4) It is likely that none of the utilities have filter diaphragms.

Positive Factors:

(1) Documented buried utilities have been through approval process.
(2) Pressure losses show the problem of leakage.
(3) Could be identified as a problem in smaller storms.
(4) Resisitivity tests indicate low potential for corrosion.

Potential Actions for Risk Reduction:

(1) Identify and locate (including depth) of all buried utilities. Put on as-builts of dam.
(2) Improve right-of-way process.
(3) See KHA report "Policy and Program Report" and recommendations regarding utility crossings.

## CATEGORY III– FAILURE MODES FOR WHICH INSUFFICIENT INFORMATION IS PRESENTLY AVAILABLE FOR MAKING ENGINEERING JUDGEMENTS

No Category III potential failure modes were identified by the FMCA team for Spook Hill FRS at this time.

## CATEGORY IV – FAILURE MODES WHICH ARE NOT PHYSICALLY POSSIBLE OR WHICH ARE NOT CLEARLY CREDIBLE

**Other Considerations**: These issues were discussed but a potential failure mode was not identified for evaluation (and descriptions of adverse and positive factors were not developed).

**(1) Emergency Spillway**: The behavior of flows across the CAP canal is not known and should be investigated/evaluated especially for a raised CAP canal bank on the opposite (west) side. Inspect concrete chute/floor. Narrow point in dam crest occurs just beyond wing wall, then the dam section widens due to a road – this is a potential location for transverse crack to develop.

**(2) Seismic**: good foundation conditions, dry dam, no liquefaction concern, thirty miles from Sugarloaf fault. No failure mode identified.

**(3) Fissures and Differential Settlement**: Foundation is granitic pediment. Geology not conducive to fissuring and no evidence of fissures or existing fissures in area. No compressible sediments.

**(4) Animal/Vegetation**: Brought about by landscaped zone materials; loosely compacted materials. Depth observed not to be a problem at this time. District has animal abatement program. Considerable effort applied by District in vegetation control. Impoundment drawdown is rather quick and rapid. Not a dam

safety issue. Animal/Vegetation control is good dam safety practice. Remove trees from toe of slopes - maintain clear zone for emergency action measures.

(5) **Settlement/Erosion of Holocene Soils**: small thickness of material; excellent foundation inspection by ADWR. No failure mode identified.

(6) **Slope Stability**: No real documentation of stability analysis for dam embankment (Factor of safety for steady state 1.24). Without phreatic surface FS = 1.66. No shear strength data. Do record stability study.

(7) **ADOT Freeway**: Low level cyclic loading. Berm due to highway has no recognized adverse impacts. Successful routing of low flow to principal spillway and emergency spillway identified. Could result in lowering elevation/frequency relation.

(8) **Dam is sectioned**: with respect to consequences. Intersecting roadways (McKellips, McDowell) segment the dam into smaller dams.

## 4.0 POTENTIAL RISK REDUCTION MEASURES

A. Restore Canal Bank at Emergency Spillway.
   1. Discharge FRS flows into canal at designated location(s)
   2. Discharge flow from emergency spillway into downstream floodway in/with ADOT corridor.
   3. Canal could temporarily store floodwaters - approximately 9.2 feet freeboard.

B. Build an enhanced CAP section (e.g. RCC over D/S bank) to take overtopping flows without breaching the canal.

C. Direct flows at ends.

D. Increase outflow early by increasing the principal outlet capacity and expanding the floodway to the Salt River.

E. Add other principal outlets along dam to discharge to canal.

F. Run hydrology for 100-yr, 150-yr, 200-yr, and 250-yr events.
   1. Then add increased outlet capacity with and without added outlet.
   2. Use new models under ADMP.

G. Have ADOT determine impacts of freeway to downstream inundation area.

H. Have ADOT build a floodway to pass canal overtopping/emergency spillway discharge flows. Build floodway in ADOT corridor then branch to Salt River.

## 5.0 SUMMARY AND CONCLUSIONS

Spook Hill FRS was constructed pursuant to a modern dam design. Construction appears to have been without any particular issues. The dam has performed normally and satisfactorily for 21 years. The structure is satisfactorily maintained and monitored.

However, it is prudent to recognize that there exist for all dams specific ways that failure could come about that warrant attention and diligent monitoring. The identification of a condition or process as a "potential failure mode" does not imply that the dam is about to fail or even necessarily that there is a dam safety deficiency at the site. Rather it identifies physically possible conditions or processes (generally with a remote but still credible chance of occurrence) that persons associated with owning, inspecting, analyzing

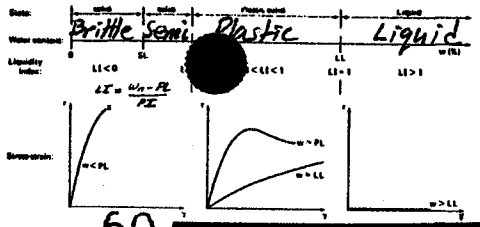
and operating the dam should be aware. Some of the potential failure modes are highlighted (or prioritized) for attention of the dam owners and operators. They are highlighted because the specific conditions at the dam and appurtenant structures are such that these failure modes are physically possible and are considered the most realistic and most credible potential failure modes definable at the site.

Two potential Category I failure modes were identified by the FMCA team. The first failure mode is not really a failure mode but rather adverse consequences associated with emergency spillway operations. The spillway discharges flows over the CAP canal and then into the City of Mesa and. There are a large number of people and structures at risk in the flood path in the event of a spillway discharge. There is a reasonable chance that casualties could occur due to spillway discharge. An immediate risk reduction measure is to develop an individual emergency action plan for the Spook Hill FRS.

The second Category I potential failure mode is seepage erosion along a transverse crack. The dam is over 20 years old and has very few documented transverse cracks associated with it. The landscape layer potentially masks the existence of any transverse cracks. A risk reduction measure is to repair transverse cracks and the slope erosion gullies on both the downstream and upstream slopes. Another measure is to remove impounded water as quickly and safely as possible by increasing the principal outlet capacity.

## SPOOK HILL FRS APPENDIX - FMCA REFERENCE MATERIALS

Spook Hill FRS north end looking south (emergency spillway and CAP canal).
Photo date: October 2001 Photograph Copyright Kimley-Horn and Associates, Inc.




| Characteristic | Soils at Equal LL with Increasing PI | Soils at Equal PI with Increasing LL |
|---|---|---|
| Dry strength | Increases | Decreases |
| Toughness near PL | Increases | Decreases |
| Permeability | Decreases | Increases |
| Compressibility | About the same | Increases |
| Rate of volume change | Decreases | — |

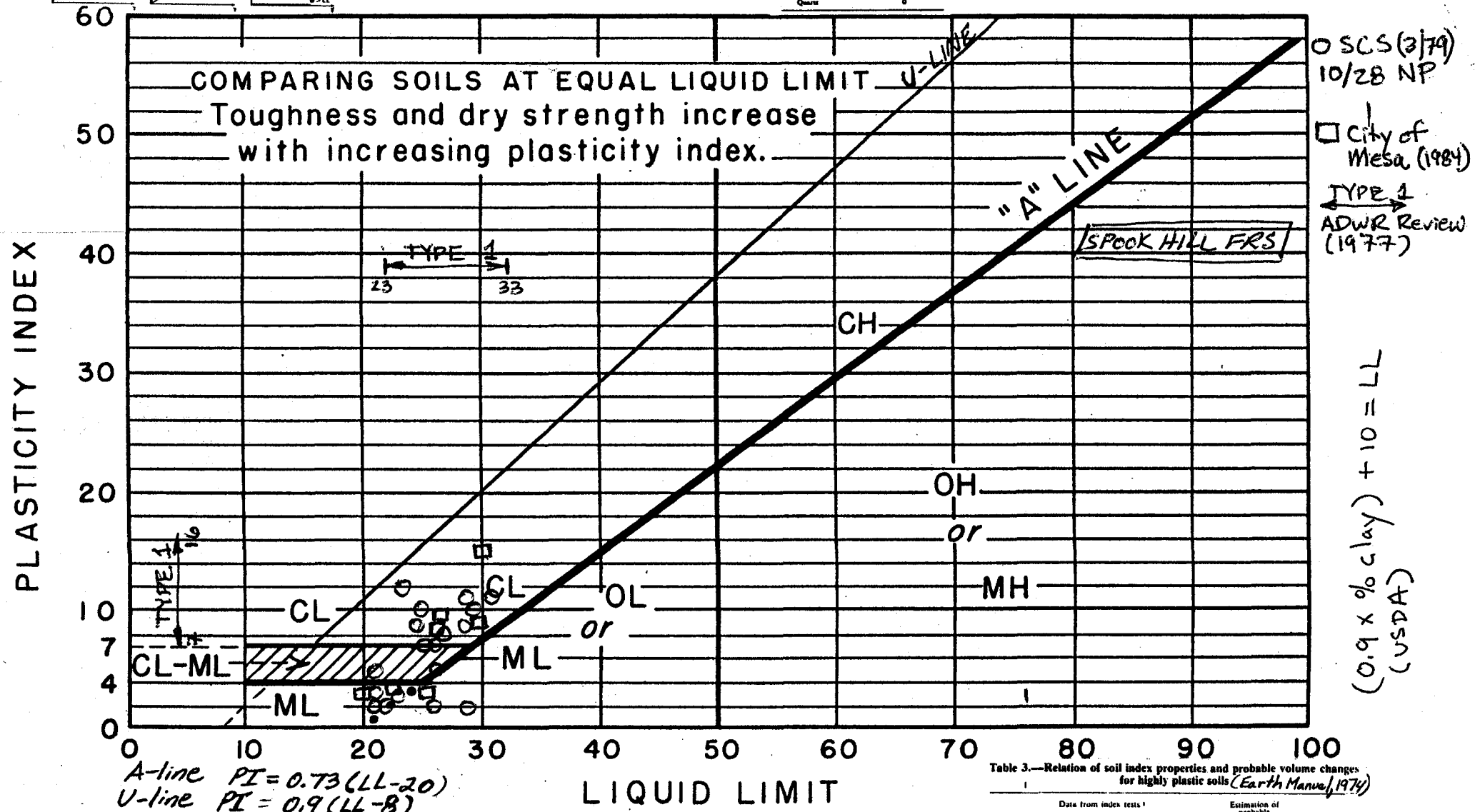
TABLE 4-1 Activities of Various Minerals*

| Mineral | Activity |
|---|---|
| Na-montmorillonite | 4-7 |
| Ca-montmorillonite | 1.5 |
| Illite | 0.5-1.3 |
| Kaolinite | 0.3-0.5 |
| Halloysite (dehydrated) | 0.5 |
| Halloysite (hydrated) | 0.1 |
| Attapulgite | 0.5-1.2 |
| Allophane | 0.5-1.2 |
| Mica (muscovite) | 0.2 |
| Calcite | 0.2 |
| Quartz | 0 |

$A = \frac{PI}{\% < 2\mu m}$

Table 4-1. WES Classification of Potential Swell

| Classification of potential swell | Potential swell $S_p$ percent | Liquid limit LL percent | Plasticity index PI percent | Natural soil suction $\tau_{Test}^{o}$ tsf |
|---|---|---|---|---|
| Low | <0.5 | <60 | <25 | <1.5 |
| Marginal | 0.5-1.5 | 50-60 | 25-35 | 1.5-4.0 |
| High | >1.5 | >60 | >35 | >4.0 |




A-line $PI = 0.73(LL-20)$
U-line $PI = 0.9(LL-8)$

# PLASTICITY CHART

Table 4-. Data for making estimates of probable volume changes for expansive soils (Chen 1975)

| Laboratory and field data | | | Probable expansion, percent total volume change | Swelling pressure, ksf | Degree of expansion |
|---|---|---|---|---|---|
| Percentage passing No. 200 sieve | Liquid limit, percent | Standard penetration resistance, blows/ft | | | |
| >35 | >60 | >30 | >10 | >20 | Very high |
| 60-95 | 40-60 | 20-30 | 3-10 | 5-20 | High |
| 30-60 | 30-40 | 10-20 | 1-5 | 3-5 | Medium |
| <30 | <30 | <30 | < 1 | 1 | low |

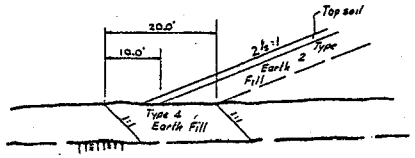
TABLE 3:3 / VOLUME CHANGE POTENTIAL
(Adapted from Holtz and Gibbs[116]) (1956)

| Volume Change | Shrinkage Limit | Plasticity Index |
|---|---|---|
| Probably low | 12 or more | 0-15 |
| Probably moderate | 10-12 | 15-30 |
| Probably high | 0-10 | 30 or more |

Table 3.—Relation of soil index properties and probable volume changes for highly plastic soils (Earth Manual, 1974)

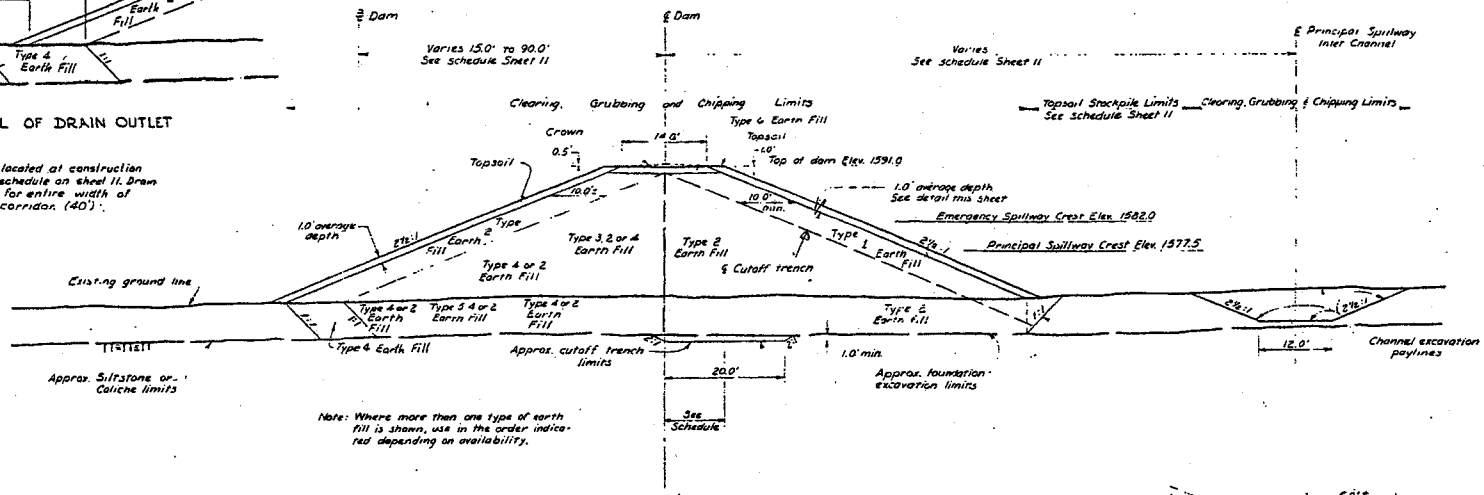
| Data from index tests[1] | | | Estimation of probable expansion,[2] percent total volume change (dry to saturated condition) | Degree of expansion |
|---|---|---|---|---|
| Colloid content (percent minus 0.001 mm.) | Plasticity index | Shrinkage limit, percent | | |
| >28 | >35 | <11 | >30 | Very high. |
| 20-31 | 25-41 | 7-12 | 20-30 | High. |
| 13-23 | 15-28 | 10-16 | 10-20 | Medium. |
| <15 | <18 | >15 | <10 | Low. |

[1] All three index tests should be considered in estimating expansive properties.
[2] Based on a vertical loading of 1.0 p.s.i. as for concrete canal lining. For higher loadings the amount of expansion is reduced, depending on the load and on the clay characteristics.



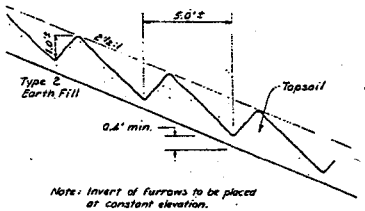

DETAIL OF DRAIN OUTLET

Note:
Drain outlet located at construction corridors as schedule on sheet 11. Drain outlet will be for entire width of construction corridor. (40').




TYPICAL CROSS SECTION

10 0 10
SCALE IN FEET




Note: Invert of furrows to be placed at constant elevation.

TOP SOIL PLACEMENT DETAIL
(Not to scale)

FOUNDATION AND CUTOFF TRENCH EXCAVATION SCHEDULE

| ℄ Station | Foundation Excavation | | Cutoff Trench Excavation | | |
|---|---|---|---|---|---|
| | Approx depth ft | Approx bottom width ft | Approx. depth ft. | Offset distance ℄ to ℄ ft. | Bottom width ft. |
| 87+00 | 0 | - | 0 | 0 | - |
| 87+10 | 4.0 | 12.5 | 9.5 | 0.4 | 20.0 |
| 98+30 | 4.0 | 92.5 | 9.5 | 10.0 | 20.0 |
| 104+60 | 5.5 | 108.5 | 10.0 | 10.0 | 20.0 |
| 118+60 | 2.0 | 92.0 | 5.0 | 10.0 | 20.0 |
| 122+00 | 4.0 | 90.0 | 8.5 | 10.0 | 20.0 |
| 131+45 | 2.5 | 104.0 | 8.5 | 10.0 | 20.0 |
| 140+35 | 2.0 | 112.0 | 5.5 | 10.0 | 20.0 |
| 143+25 | 5.0 | 111.5 | 7.0 | 10.0 | 20.0 |
| 158+25 | 3.0 | 103.5 | 6.0 | 10.0 | 20.0 |
| 165+40 | 3.0 | 111.0 | 5.0 | 10.0 | 20.0 |
| 172+30 | 5.0 | 109.0 | 7.0 | 10.0 | 20.0 |
| 179+30 | 2.0 | 102.0 | 9.0 | 10.0 | 20.0 |
| 186+35 | 3.0 | 101.0 | 4.0 | 10.0 | 20.0 |
| 190+00 | 6.0 | 106.5 | 6.0 | 10.0 | 20.0 |
| 192+25 | 1.0 | 115.5 | 5.5 | 10.0 | 20.0 |
| 201+95 | 2.5 | 114.0 | 3.5 | 10.0 | 20.0 |
| 207+70 | 2.5 | 111.5 | 3.5 | 10.0 | 20.0 |
| 217+85 | 2.0 | 122.0 | 6.5 | 10.0 | 20.0 |
| 220+40 | 3.5 | 133.0 | 4.5 | 10.0 | 20.0 |
| 222+85 | 2.6 | 131.2 | 25.5 | 10.0 | 20.0 |

FOUNDATION AND CUTOFF TRENCH EXCAVATION SCHEDULE

| ℄ Station | Foundation Excavation | | Cutoff Trench Excavation | | |
|---|---|---|---|---|---|
| | Approx. depth ft. | Approx. bottom width ft. | Approx depth ft | Offset distance ℄ to ℄ ft. | Bottom width ft. |
| 224+90 | 4.5 | 129.5 | 5.5 | 10.0 | 20.0 |
| 227+90 | 3.0 | 121.0 | 4.0 | 10.0 | 20.0 |
| 231+80 | 2.7 | 121.3 | 9.0 | 10.0 | 20.0 |
| 238+75 | 4.0 | 107.5 | 5.0 | 10.0 | 20.0 |
| 243+15 | 1.5 | 110.0 | 2.5 | 10.0 | 20.0 |
| 248+95 | 4.5 | 93.0 | 5.5 | 10.0 | 20.0 |
| 253+90 | 2.0 | 99.5 | 3.0 | 10.0 | 20.0 |
| 259+92 | 2.0 | 109.5 | 3.0 | 10.0 | 20.0 |
| 262+95 | 5.5 | 98.5 | 6.5 | 10.0 | 20.0 |
| 269+00 | 3.5 | 108.5 | 4.0 | 10.0 | 20.0 |
| 270+95 | 1.5 | 104.6 | 2.9 | 10.0 | 20.0 |
| 277+85 | 0 | 111.5 | 1.0 | 10.0 | 20.0 |
| 280+00 | 7.5 | 106.5 | 8.5 | 10.0 | 20.0 |
| 282+50 | 0 | 114.0 | 1.0 | 10.0 | 20.0 |
| 285+05 | 1.0 | 113.0 | 2.0 | 10.0 | 20.0 |
| 289+75 | 0 | 104.0 | 1.0 | 10.0 | 20.0 |
| 291+00 | 1.0 | 105.5 | 2.0 | 10.0 | 20.0 |
| 296+80 | 5.0 | 79.0 | 6.0 | 10.0 | 20.0 |
| 303+46± | 3.0 | 13.5 | 3.0 | 0 | 20.0 |
| 303+50± | 0 | - | 0 | 0 | - |

Completed: April 10, 1980

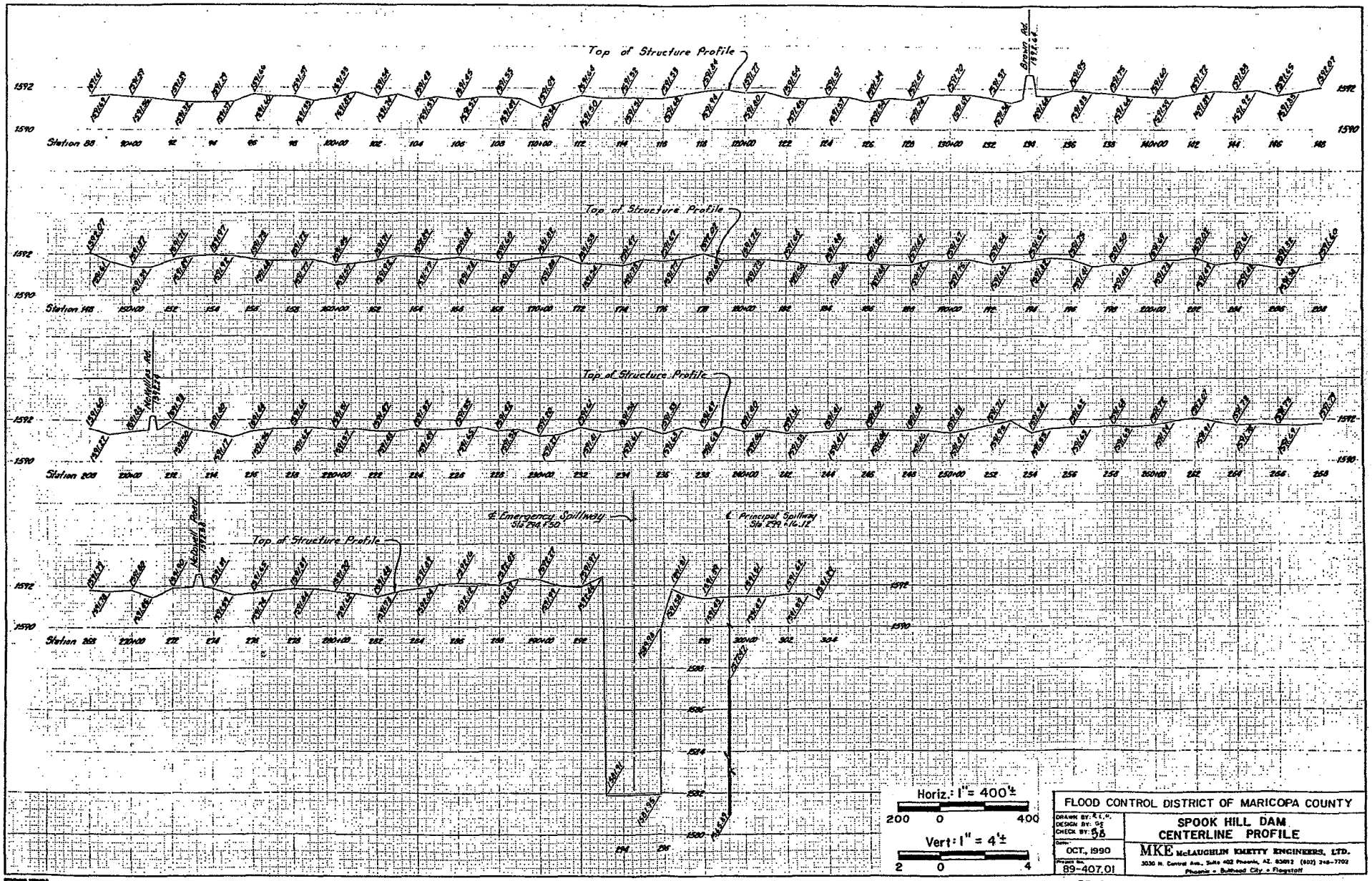
# AS-BUILT

CROSS SECTION & DAM
SPOOK HILL FRS.
BUCKHORN-MESA W.P.P.
MARICOPA & PINAL COUNTIES, ARIZONA

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

| | | Date | | |
|---|---|---|---|---|
| Designed | PJM | 7-76 | Approved by | |
| Drawn | | | Title | |
| Traced | ES | 8-76 | Sheet No. 15 of 45 | Drawing No. 7-E-23797 |
| Checked | PJM | 12-76 | | |

FORM SCS-211C




FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

SPOOK HILL DAM
CENTERLINE PROFILE

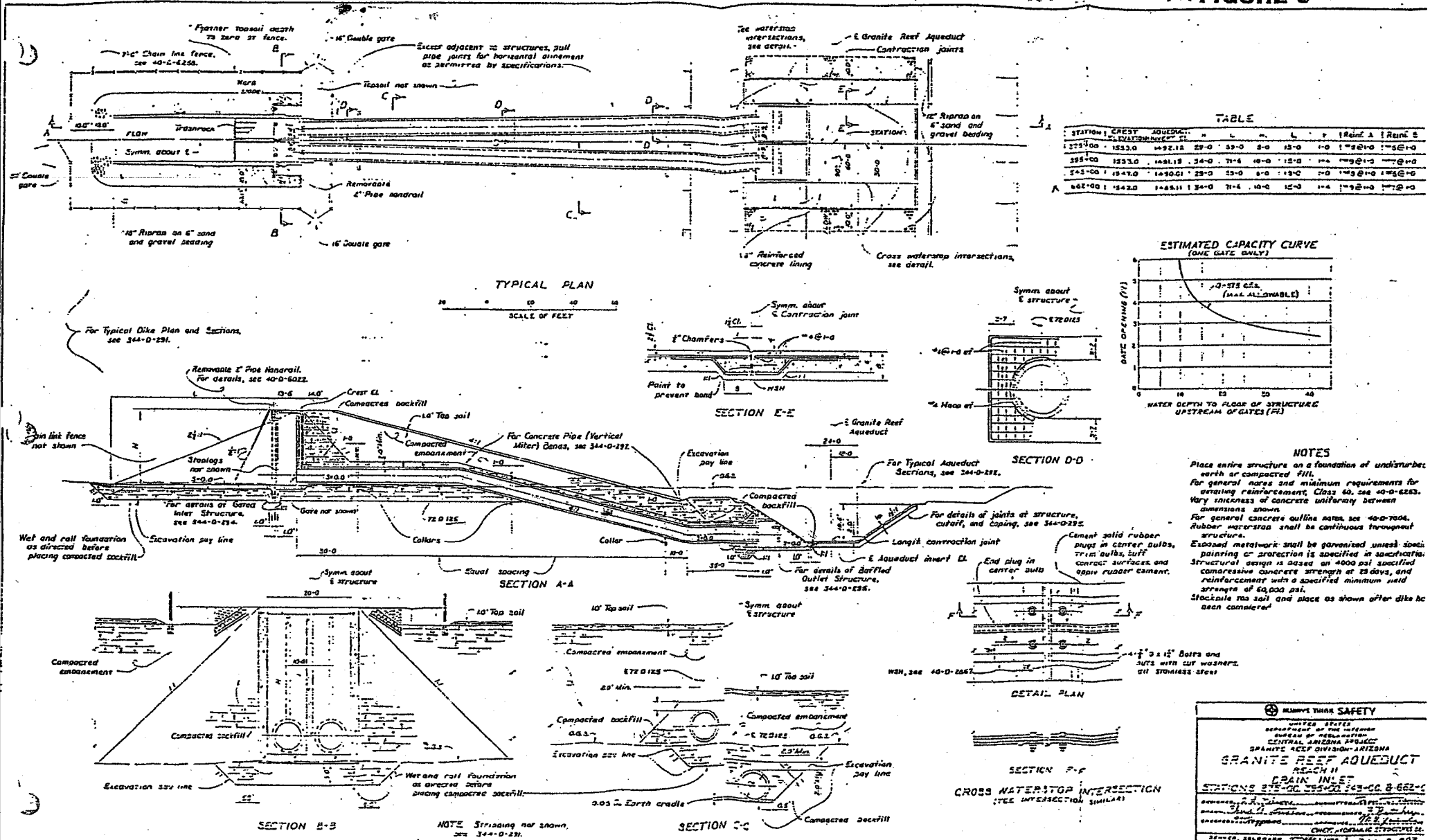
DRAWN BY: R.E.H.
DESIGN BY: G.S.
CHECK BY: B.S.
Date: OCT., 1990
Project No. 89-407.01

MKE McLAUGHLIN KMETTY ENGINEERS, LTD.
3030 N. Central Ave., Suite 402 Phoenix, AZ. 85012 (602) 248-7702
Phoenix • Bullhead City • Flagstaff



FIGURE II-3

# FIGURE 3




| STATION | CREST ELEVATION | AQUEDUCT INVERT ELEV. | H | L | m | L | r | Reinf. A | Reinf. B |
|---|---|---|---|---|---|---|---|---|---|
| 275+00 | 1533.0 | 1492.15 | 29-0 | 33-0 | 8-0 | 12-0 | 1-0 | [illegible]@1-0 | [illegible]@1-0 |
| 295+00 | 1533.0 | 1491.15 | 34-0 | 71-4 | 10-0 | 12-0 | 1-4 | [illegible]@1-0 | [illegible]@1-0 |
| 343+00 | 1547.0 | 1490.01 | 29-0 | 33-0 | 8-0 | 12-0 | 1-0 | [illegible]@1-0 | [illegible]@1-0 |
| 662+00 | 1542.0 | 1488.11 | 34-0 | 71-4 | 10-0 | 12-0 | 1-4 | [illegible]@1-0 | [illegible]@1-0 |

COMPUTATION SHEET
SCS-ENG-523 Rev. 8-69

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

| STATE Arizona | | PROJECT Buckhorn Mesa - Spook Hill | | |
|---|---|---|---|---|
| BY Burton | DATE 4/7/75 | CHECKED BY | DATE | JOB NO. 2005 |
| SUBJECT Stability Analysis - Basic Assumptions | | | | SHEET 2 OF 20 |

Use Test Hole #7 @ Station 223+00 ± as being critical.

ML Material will be removed down to caliche layer.

A cutoff trench will extend to the sandstone.

As per Soil Mechanics report by Jim Talbot, March 6, 1975 — The stability can be checked using infinite slope procedures.

Use the following $\phi$ values:

SC : $\phi = 32°$
SM : $\phi = 33°$ > Use 33° for design

$\gamma_D = 120$ pcf

$C = 0$

$m = 10\%$

$\gamma_m = 120(1.10) = 132.0$ pcf

$m_s = 18\%$

$\gamma_s = 120(1.18) = 141.6$ pcf.

COMPUTATION SHEET
SCS-ENG-523 Rev. 8-69

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
* GPO: 1972 —440-199

| STATE Arizona | | PROJECT Buckhorn Mesa - Spook Hill Dam | | |
|---|---|---|---|---|
| BY D Riddle | DATE 4/10/75 | CHECKED BY | DATE | JOB NO. 2005 |
| SUBJECT Stability Analysis R=60 Downstream Slope | | | | SHEET 5 OF 20 |

| Slice | Depth | Sub Total Wt. of Slice $H_2O$ Equiv | Total Wt. of Slice $H_2O$ Equiv | T | N |
|---|---|---|---|---|---|
| 1 | 4.0 | 8.48 | 8.48 | 6.8 | 5.2 |
| 2 | 11.0 | 23.32 | 23.32 | 15.5 | 17.5 |
| 3 | 12.0<br>3.2 | 25.44<br>7.26 | 32.70 | 18.0 | 23.5 |
| 4 | 9.2<br>6.3 | 19.50<br>14.30 | 33.80 | 15.0 | 23.5 |
| 5 | 6.5<br>8.5 | 13.78<br>19.30 | 33.08 | 11.0 | 22.5 |
| 6 | 3.6<br>9.6 | 7.63<br>21.79 | 29.42 | 6.0 | 19.0 |
| 7 | 1.5<br>9.0 | 3.18<br>20.43 | 23.61 | 2.0 | 14.5 |
| 8 | 7.2 | 16.34 | 16.34 | -0.5 | 9.0 |
| 9 | 3.2 | 7.26 | 7.26 | -1.1 | 2.9 |
| 10 | 0.8 | 1.82 | 1.82 | -0.5 | 0.5 |
| | | | | 72.7 | 138.1 |

$\tan 33^\circ = 0.64941$

$$F.S. = \frac{(138.1)(0.64941)}{(72.7} = 1.23$$

COMPUTATION SHEET
SCS-ENG-523 Rev. 8-69

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
GPO: 1972 —460-109

| STATE Az | | PROJECT Buckhorn Mesa WPF - Spoon - FT I | | |
|---|---|---|---|---|
| BY A.C. | DATE 3-24-76 | CHECKED BY | DATE | JOB NO. |
| SUBJECT Linear Seepage - Slope Stability | | | | SHEET 1 OF 8 |

Permeability from Buckeye Site I, Sta 905+00 ℄ Dam, is typical

SM-SP, $K = .003\ ft^3/in^2/day = .43\ ft/day$ (156 ft/yr)

Permeability from Bureau of Reclamation Geology Laboratory Permeability test

$K \simeq 1$ to 100 ft/yr (with 600 ft/yr max.)
100 ft/yr = .27 ft/day (10⁻⁴ cm/sec, max)

K = .43 ft/day is certainly justifiable

Darcys Law

$V = ki$ the actual water velocity $V_a = V/n$, n = porosity
for fine grained material, n = .3 to .5 use .3 (gives higher V)

$V_a = ki/n$ $i = h/L$

The average gradient for i will be used since no flow net has been drawn. This is conservative for gradients at the upstream face of the dam. The difference in elevation of the phreatic surface at the upstream and downstream end of the dam is about 12' (Measured from phreatic line drawn on profile). The length, scaled, is about 76 feet.

$$V_a = \frac{.43}{.3}(12/76) = .227\ ft/day$$

After 17 days, the flood pool is below elev 1577.5,

the distance traveled by the water is .227×17 = 3.8 ft

This penetration is negligable when the Swedish circle method is used for Slope stability.

Assumptions Permeability value - appears reasonable
Porosity - guessed conservatively
Gradient - averaged conservatively
Pool level - change in gradient not considered from drawdown

Penetration distance for absolute worst case - (unrealistic)

$$V_a = \frac{1.64}{.3}(15/80) = 1.0\ ft/day$$

penetration length = (1.0)(20) = 20 ft This is unrealistic

| STATE | PROJECT | | |
|---|---|---|---|
| Az | Buckhorn Mesa WPP - Spook Hill FRS | | |
| BY A.C. | DATE 3-25-76 | CHECKED BY | DATE | JOB NO. |

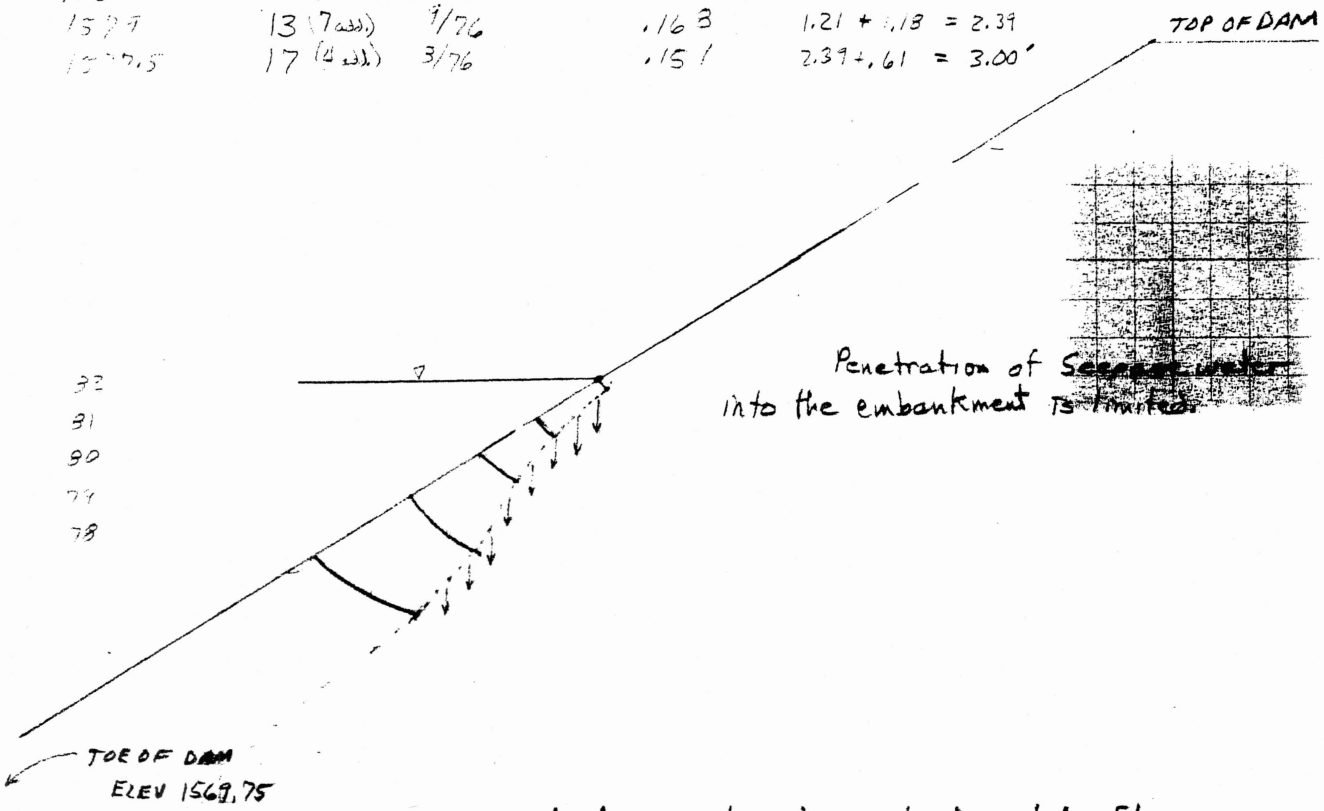
SUBJECT Penetration of water into embankment — SHEET 2 OF 8

Actual water penetration -> $K = .43$ ft/day and $n = .3$

$v_n = K/n$

$v_n = 1.43$

| ELEV | Duration (Total) | i | Seepage velocity (ft/day) | Actual penetration |
|---|---|---|---|---|
| 582 | 1 | 12/76 | .226 | .226 |
| 1581 | 3 (2 additional) | 11/76 | .207 | .226 + .414 = .640 |
| 1580 | 6 (3 additional) | 10/76 | .189 | .640 + .567 = 1.207 |
| 1579 | 13 (7 add.) | 9/76 | .168 | 1.21 + .18 = 2.39 |
| 1577.5 | 17 (4 add.) | 8/76 | .151 | 2.39 + .61 = 3.00' |




Penetration of Seepage water into the embankment is limited.

This penetration is based on steady state flow

1569 Seepage will progress much faster toward the toe of the dam

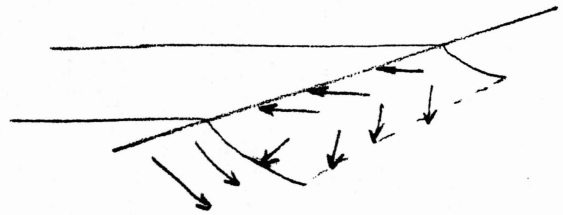
COMPUTATION SHEET
SCS–ENG–523 Rev. 8–69

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
GPO: 1972 —460-109

| STATE | | PROJECT | | |
|---|---|---|---|---|
| Az | | Buckhorn Mesa WPP Spook Hill FRS | | |
| BY | DATE | CHECKED BY | DATE | JOB NO. |
| A.C. | 3-25-76 | | | |

SUBJECT Drainage of embankment upon drawdown of reservior — SHEET 3 OF 8

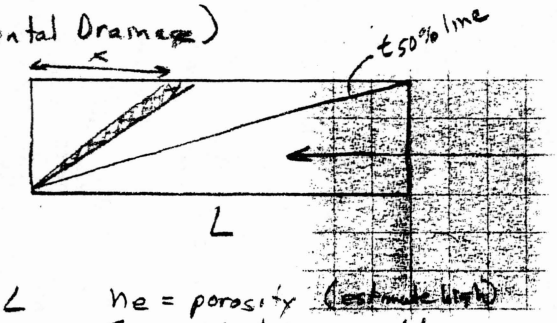
To investigate the drainage in the embankment material, a step drop is considered in the reservoir



Average drop is 6"/day

An Estimate of this situation will give at least a relative time scale for drainage

Example: Base Course Drainage (Horizontal Drainage)




from Casagrande Trans. ASCE 1952

$$t = \frac{n_e C x^2}{4KH} \quad \text{at } t = 50^\circ \quad x = L$$

$n_e$ = porosity (estimate high)
C = model study variable
= 2.4 if L/H > 5

$L = 3'$ $H = .5'$ $L/H = 6$

$$t_{50} = \frac{(.4)(2.4)(3)^2}{4(.43)(.5)} \text{ days} = 10 \text{ days}$$

– reservior drops this amount in one day

$$t = 1 \text{ day} = \frac{(.4)(2.4)(x)^2}{4(.43)(.5)} \qquad x = .94 \text{ ft}$$

about 1/6 the area is drained in the direction of the reservior

Vertical Drainage also takes place. Attempt is made to estimate lag betwee reservior level and drainage in soil.

[illegible] $K = .43$, $n = .4$

[illegible] particle permeability could be as low as .1 ft/day

$$V_a = \frac{(.1)(i)}{.4}$$

if $V_a$ = rate at which reservior drains ($V_a = .5$)

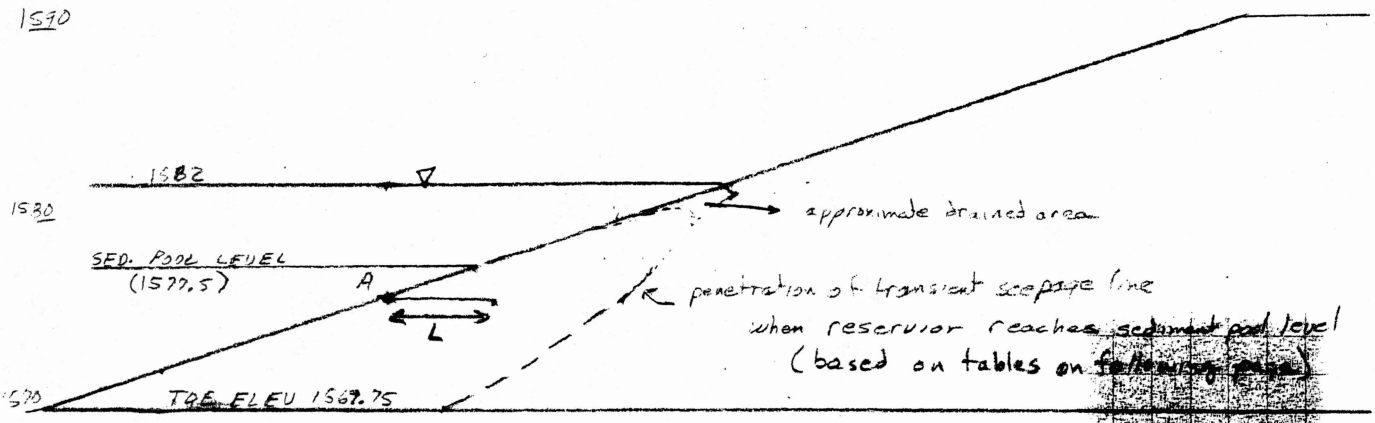
then $i = \frac{(.5)(.4)}{(.1)} = 2$ this will not develope in our case

It must be assumed that the embankment will not drain effectively in the drawdown period of the reservior. However, partial drainage will occur.

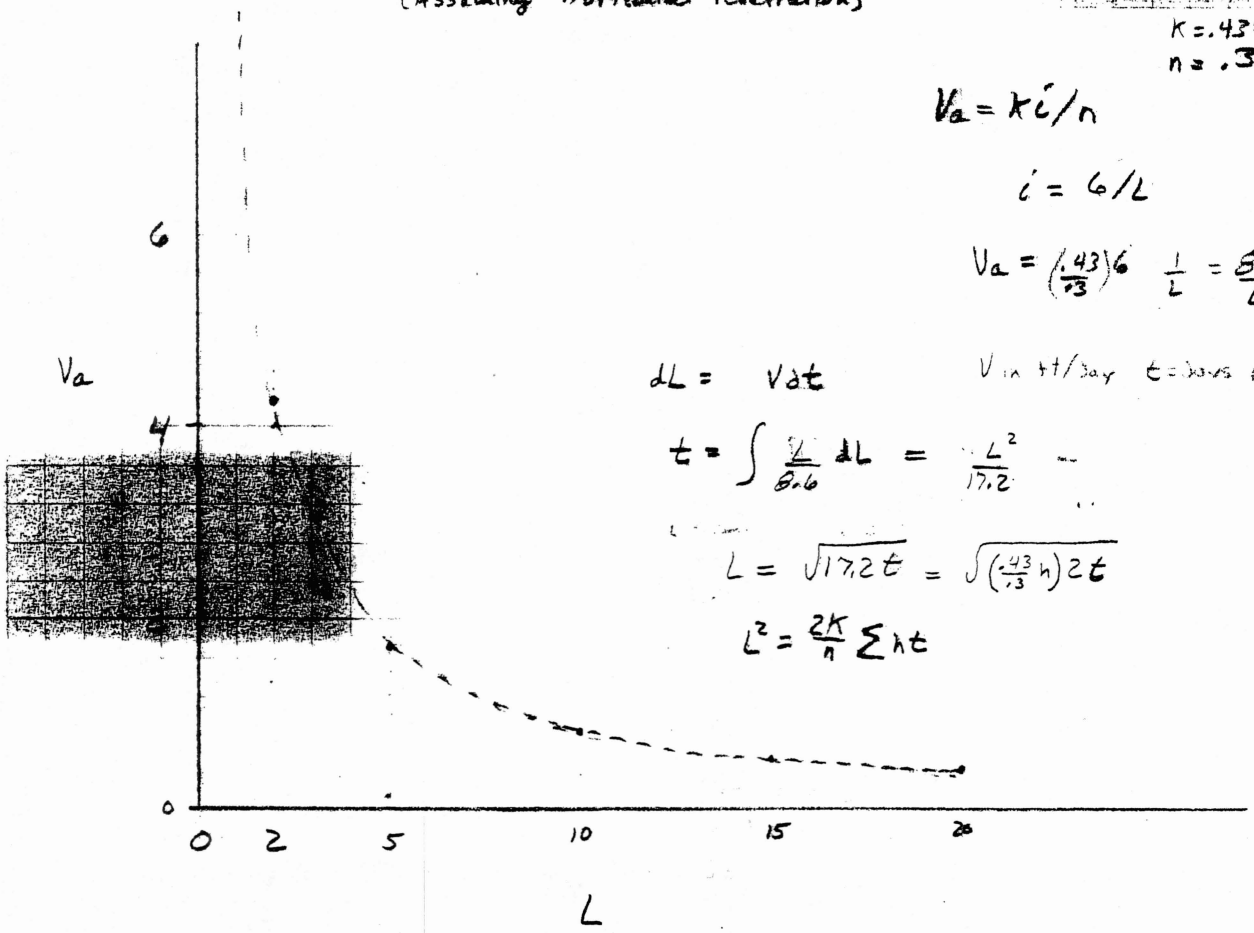
**COMPUTATION SHEET**
SCS–ENG–523 Rev. 8–69

**U. S. DEPARTMENT OF AGRICULTURE**
**SOIL CONSERVATION SERVICE**
GPO: 1972 —460-109

| STATE | PROJECT | | | |
|---|---|---|---|---|
| Az | Buckhorn Mesa WFP - Spook Hill FRS | | | |
| BY | DATE | CHECKED BY | DATE | JOB NO. |
| A.C. | 3-25-75 | | | |
| SUBJECT | | | | SHEET 4 OF 8 |




Point A is 6 ft below the maximum water surface.
(Assuming Horizontal Penetration)




$K = .43$ ft/day
$n = .3$

$V_a = Ki/n$

$i = 6/L$

$V_a = \left(\frac{.43}{.3}\right)6 \quad \frac{1}{L} = \frac{8.6}{L}$

$dL = V dt$ V in ft/day t = days L = ft

$t = \int \frac{L}{8.6} dL = \frac{L^2}{17.2}$

$L = \sqrt{17.2t} = \sqrt{\left(\frac{.43}{.3}n\right)2t}$

$L^2 = \frac{2K}{n} \sum ht$

COMPUTATION SHEET
SCS–ENG–523 Rev. 8–69

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
GPO: 1972—460-109

| STATE | | PROJECT | | |
|---|---|---|---|---|
| AZ | | Buckhorn Mesa WPI Spook Hill FRS | | |
| BY A.C. | DATE 3-25-75 | CHECKED BY | DATE | JOB NO. |
| SUBJECT | | | | SHEET 5 OF 8 |

| t | h | $L^2$ | L |
|---|---|---|---|
| 1 day | 6 ft | 17.2 | 4.15 |
| 3 days | 5 ft | $17.2(1)+\left(\frac{.43}{.3}\right)(2)(2)$ | 6.77 |
| 6 days | 4 ft | $17.2+28.6+\left(\frac{.43}{.3}\right)(4)(3)(2)$ | 8.76 |
| 13 days | 3 ft | $17.2+28.6+34.4+\left(\frac{.43}{.3}\right)(3)(7)(2)$ | 11.85 |
| 17 days | 1.5 ft | $17.2+28.6+34.4+60.2+\left(\frac{.43}{.3}\right)(1.5)(4)(2)$ | 12.55 |

Base of dam

| t | h | $L^2$ | L |
|---|---|---|---|
| 1 day | 12 ft | $\left(\frac{.43}{.3}\right)(12)(1)(2)$ | 5.87 |
| 3 days | 11 ft | $34.4+\left(\frac{.43}{.3}\right)(11)(2)(2)$ | 9.87 |
| 6 days | 10 ft | $34.4+63.1+\left(\frac{.43}{.3}\right)(10)(3)(2)$ | 13.55 |
| 13 days | 9 ft | $34.4+63.1+86.0+\left(\frac{.43}{.3}\right)(9)(7)(2)$ | 19.08 |
| 17 days | 7.5 ft | $34.4+63.1+86.0+80.6+\left(\frac{.43}{.3}\right)(7.5)(4)(2)$ | 20.70 |

The effect of this new seepage line on the stability analysis will have to be investigated. See diagram on previous page

Conclusions - A rapid drawdown condition need not be considered. The volume of submerged soil above the reservoir level is small compared to the volume within the failure plane.

The phreatic line will not reach the downstream toe of the dam unless the sediment pool is not drained. The phreatic line, if it developes, will form from the sediment pool elevation.

A steady seepage condition need not be considered since it applys only to the downstream slope. The seepage line should not effect the downstream slope stability. (SM-10, 17-5)

A slope stability analysis should be made with the water surface and phreatic line at the sediment pool elevation

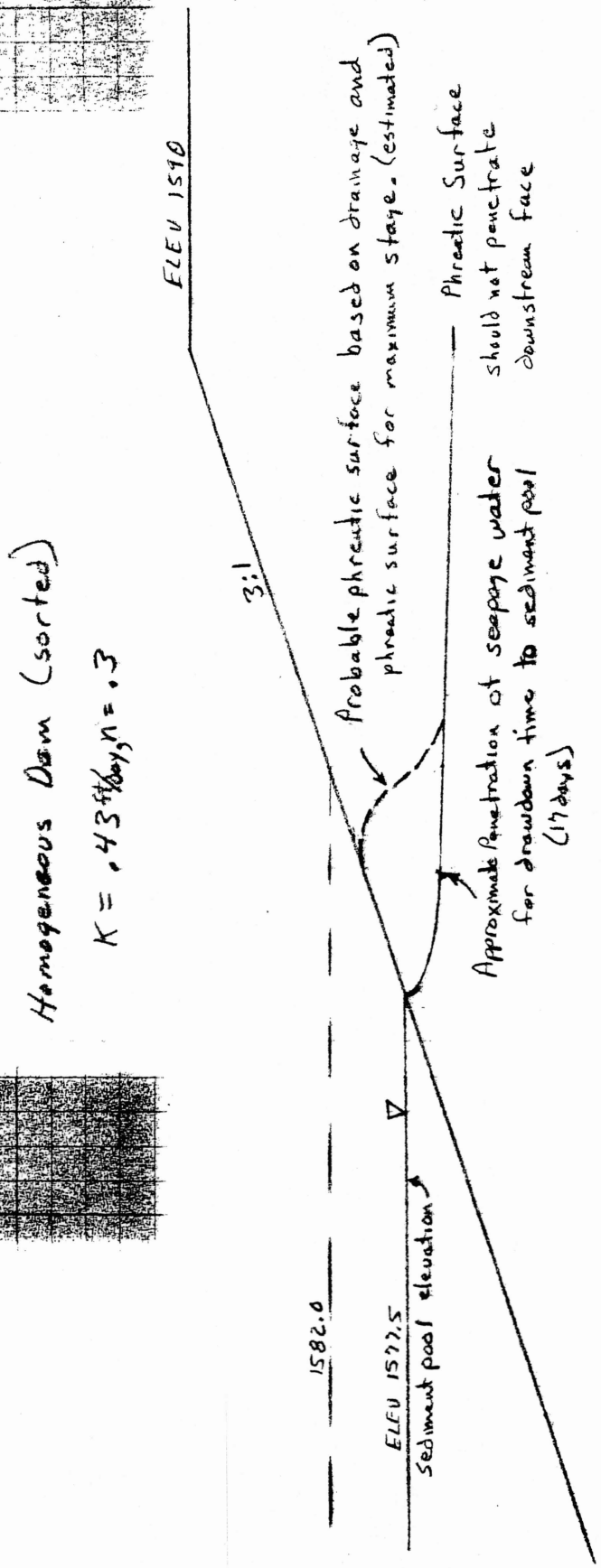
COMPUTATION SHEET
SCS-ENG-523 Rev. 8-69

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

138

GPO: 1972 —460-109

| STATE | | PROJECT | | |
|---|---|---|---|---|
| Az | | Buckhorn Mesa WPP Spook Hill F.R.S. | | |
| BY | DATE | CHECKED BY | DATE | JOB NO. |
| A.C. | 3-25-76 | | | |
| SUBJECT | | | | SHEET 6 OF 8 |
| Conditions for Upstream Slope Stability | | | | |



COMPUTATION SHEET
SCS-ENG-523 Rev. 8-69

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
GPO: 1972 —460-109

| STATE | | PROJECT | | |
|---|---|---|---|---|
| Az | | BUCKHORN MESA WPP - SPOOK HILL FRS | | |
| BY | DATE | CHECKED BY | DATE | JOB NO. |
| A.C. | 3-24-76 | | | |
| SUBJECT | | | | SHEET 7 OF 8 |
| UPSTREAM SIDE SLOPES | | | | |

One conclusion that can be drawn from this analysis is that the upstream stability conditions will be no worse than the down side slope stability conditions when the phreatic line has developed. This downstream condition was checked by the "Swedish Circle" Method 2:1 side slopes were acceptable with $F_s = 1.23$.

* 2½:1 side slopes are easily justifiable under these conditions on the upstream side of the dam and will be used.

Since the $\phi$ angle for the course material was used in

* this analysis, either a zoned dam or a homogeneous dam is acceptable (as far as slope stability).

COMPUTATION SHEET
SCS-ENG-523 Rev. 8-69

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

GPO: 1972 —460-109

| STATE | | PROJECT | | |
|---|---|---|---|---|
| Az | | Buckhorn Mesa WPP Spook Hill FRS | | |
| BY | DATE | CHECKED BY | DATE | JOB NO. |
| A.C. | 3-25-76 | | | |

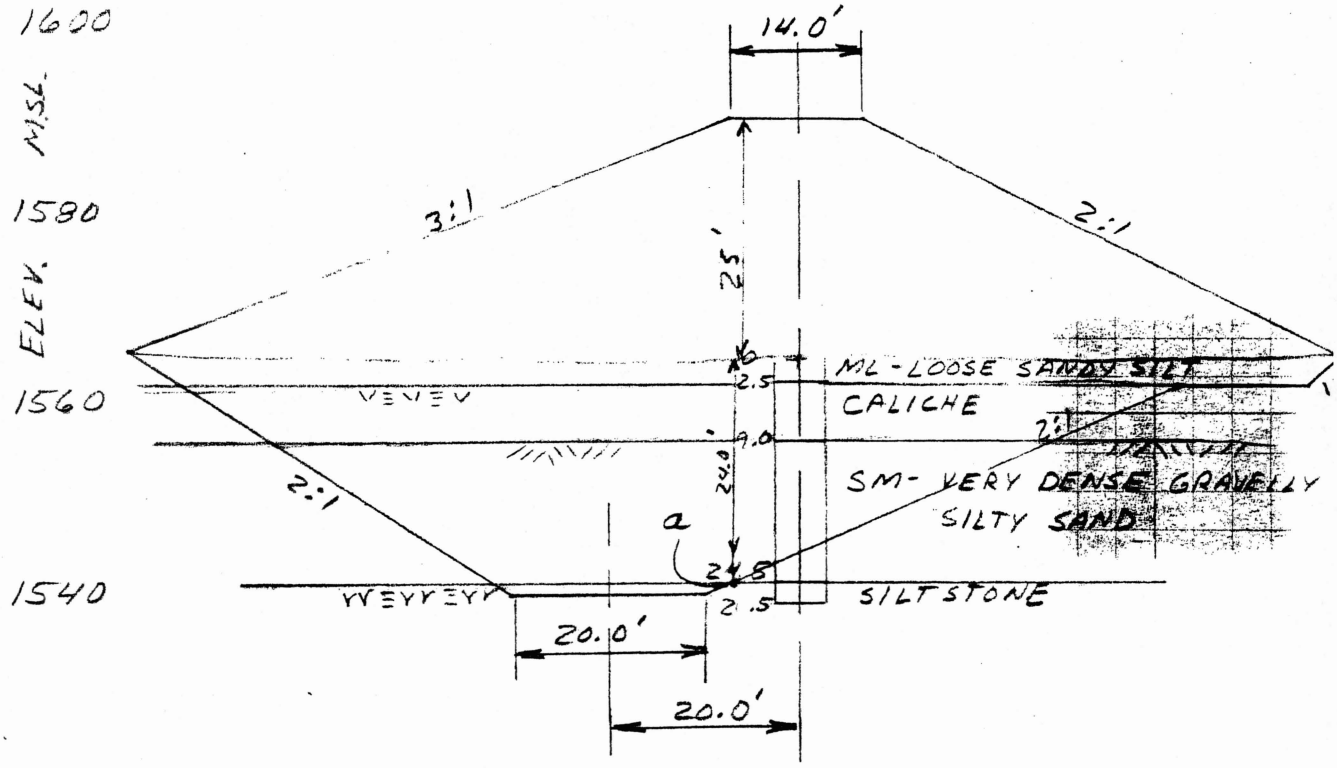
SUBJECT Use of Caliche as Embankment Material SHEET 8 OF 8

The soils in the Spook Hill area are very calcareous. They react to HCL. In many areas layers of Caliche have developed. these layers are poorly indurated. They are easily broken down to less than sand size particles. If this material is mixed with the other embankment material, it can be used. It has been suggested that this material be used on the downstream side of dam. The permeability of the soil is increased by the presence of the caliche. Tests have been made by the Bureau of Reclamation and they found these materials usable.

Based on discussions with Mr De Groot of the Bureau of Rec. and Jim Talbot of the SCS.

| STATE ARIZONA | | PROJECT BUCKHORN MESA WPP SPOOK HILL FRS | | |
|---|---|---|---|---|
| BY A.C. | DATE 7-31-75 | CHECKED BY | DATE | JOB NO. |
| SUBJECT EMBANKMENT AND FOUNDATION DESIGN | | | | SHEET 1 OF 2 |

**SETTLEMENT** - STATION 222+35




MAXIMUM SETTLEMENT WILL OCCUR @ U.S. EDGE OF DAM

BASIC SOIL MECHANICS SM(10) Vol. 2 FIG 14-3

$d = \frac{1566 - 1542}{2} = 12'$ $\quad b = (1591-1566)3 = 75.0'$

$d/b = 12/75 = 0.16$

$\Delta p = 0.89\,p$ @ pt a

$\gamma_m = .95(128.2) + (0.088)(62.4) = 127.3$ pcf

$p = 25'(127.3) = 3182.5$ psf

$\Delta p = 2832$ psf

FROM TABLE 14-2 - SM SOIL TYPE

| @ 2880 psf | @ 7200 psf |
|---|---|
| 1.2 % | 3.0 % |

COMPUTATION SHEET
SCS-ENG-523 Rev. 8-69

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
* GPO: 1972 —460-109

| STATE | PROJECT | | |
|---|---|---|---|
| ARIZONA | BUCKHORN MESA WPP SPOOK HILL FRS | | |
| BY A.C. / DATE 7-31-75 | CHECKED BY | DATE | JOB NO. |
| SUBJECT EMBANKMENT AND FOUNDATION DESIGN | | | SHEET 2 OF 2 |

SETTLEMENT OF FOUNDATION

@ 2832 psf ⟶ USE 1.2%

$\delta = 0.012(24') = 0.29$ FT

SETTLEMENT OF EMBANKMENT

CONSIDER THE SETTLEMENT ON THE LOWER HALF

$d = \frac{25'}{4}$ $b = \frac{3(75)}{2}$ $d/b = .08$ $\Delta p = .74p$

$p = 3187.5/2 = 1593.25$

$\Delta p = 1475.8$ USE 0.6% SETTLEMENT

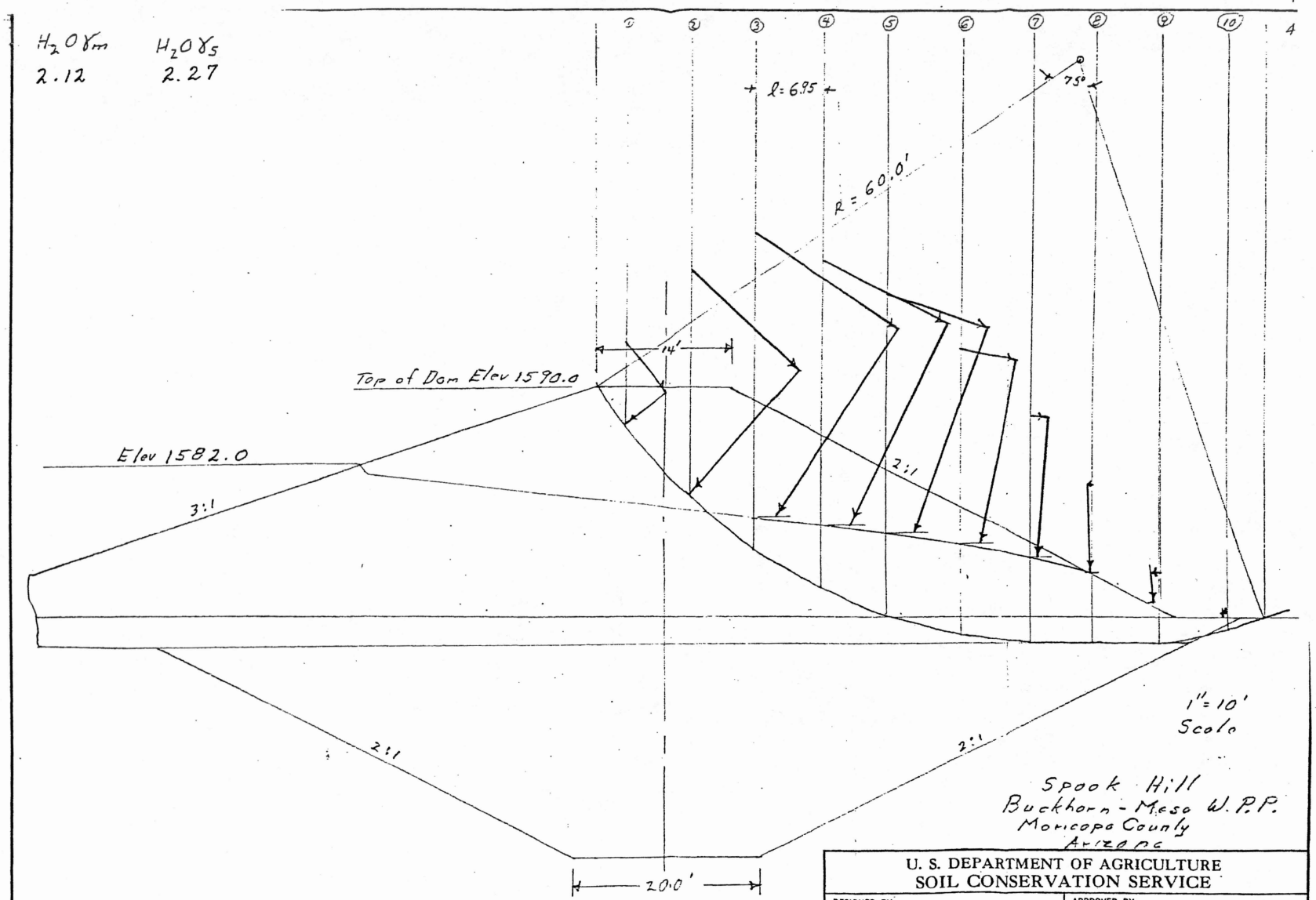
$\delta = 0.006(12.5) = 0.08$ FT

SETTLEMENT IN THE UPPER EMBANKMENT

WILL BE SMALL

0.29 FT + 0.08 FT = 0.37 FT USE 0.4 FT

TOP OF DAM ELEV = 1590.4 + 0.4
= 1590.8




Spook Hill
Buckhorn-Mesa W.P.P.
Maricopa County
Arizona

| U. S. DEPARTMENT OF AGRICULTURE SOIL CONSERVATION SERVICE | |
|---|---|
| DESIGNED BY D Riddle | APPROVED BY |
| CHECKED BY | DRAWING NO. |
| DATE: 4/19/75 | SHEET 6 OF 20 |

SCS-ENG-313A (Rev. 8-70)

U.S. GOVERNMENT PRINTING OFFICE : 1970 O—401-772

United States
Department of
Agriculture

Soil
Conservation
Service

201 E. Indianola Ave.
Suite 200
Phoenix, AZ 85012

RECEIVED
JAN 8 1988
ENGINEERING DIV.

January 6, 1987

Dan Sagramoso, P.E.
Chief Engineer and General Manager
Flood Control District of Maricopa County
3335 W. Durango Street
Phoenix, AZ 85009

07.50

Dear Dan:

We have evaluated your request dated 9-10-87 to discontinue the permanent irrigation system for the Spookhill FRS, Buckhorn-Mesa WPP. We agree with your assessment that the data compiled supports discontinuance of irrigation as an appropriate solution for the prevention of detrimental cracking at this site. A desirable desert landscape should persist without irrigation. Cactus and bursage will tend to increase while other plants such as saltbush and quailbush will tend to decrease.

The maintenance requirements for the irrigation system included as item 4 of the O&M agreement dated August 1, 1977 for the Spookhill structure are hereby cancelled and are to be deleted from the agreement.

Thank you for your understanding and patience in seeing this issue resolved.

Sincerely,

Verne M. Bathurst
State Conservationist

| | | | | | |
|---|---|---|---|---|---|
| DRL | | JRM | | WCJ | |
| NJN | | [illegible] | | [illegible] | |
| DMK | | DEC | | DEA | |
| AMB | | MTM | | GDC | |
| | | DL | | | |
| | | NMP | | | |
| | FILE | | | | |

cc: Stanley N. Hobson, Director, WNTC
W.R. Evans, Head, Engineering Staff, WNTC
Joe Knisley, AC, Tucson
Larry Martinez, DC, Chandler
Dan Lawrence, ADWR

# DEPARTMENT OF WATER RESOURCES

## ENGINEERING DIVISION

### SAFETY OF DAMS

## MEMORANDUM

**TO:** File

**FROM:** K.M. Hussain

**DATE:** August 6, 1986

**SUBJECT:** Spook Hill FRS Dam (07.50)
Abandoning of Sprinkling System

During the Guadalupe design review meeting today, Bob Payette and Nick Karan of the FCDMC discussed the present status of the testing and studies done to determine whether sprinkling system at the dam's slopes shall be abandoned.

According to the original design, the sprinkling system was installed to reguarly spray the slopes so that the internal moisture (in-place) within the embankment remains intact. This was thought to be the most effective way of deterring the cracks developing in the embankment. Cracking of earthern embankments in the arid climate is the typical and constant problem at the dams owned by the Flood Control District of Maricopa County. In addition to the constant operational difficulties of various components of the system, the FCDMC was finding the O&M and water costs to be excessive. They want to abandon the system. The SCS' position has been that since it will amount to deviation from the original design, the FCDMC should assume responsibility and the associated liabilities in case the dam develops cracks and/or fails. The FCD didn't like this condition. They started making field testing to compare moisture profiles of the sprinkled and unsprinkled portions of the slopes. The latest study done by WTI was recently completed. The FCD will send us the report in the near future.

Nick Karan stated that in his opinion the findings of the latest study by WTI are no different from their earlier conclusions. They are:

1. The present moisture content in the embankment is roughly 3 to 4 percent below in the sprinkled portion and 6 to 7 percent below in the unsprinkled portion

of the embankment as compared to the as-constructed moisture content.

2. In all the samples tested from the sprinkled and unsprinkled portions of the embankment, the present moisture content is below the shrinkage limit. (Hence no further shrinkage can occur).

3. Observations indicate that the sprinkled portion of the embankment has 30% rill erosion, whereas the unsprinkled portion has only 2%.

After the WTI report has been received and reviewed by the SCS and ADWR, Ralph suggested that we should perform a joint physical inspection of the dam and decide whether the sprinkling system shall be continued or abandoned. No date for the inspection was fixed.

ENGINEERING DIVISION
INCOMING CORRESPONDENCE

DATE RECEIVED: 8-6-86 REQUESTED ACTION:

TO: Lawrence__
Nicholson__
Fominko__
~~Jenkins~~
Bond__
Morris__

~~Hussain~~
Cox__
Alexander__

Jaynes__
Creighton__
Goff__
Harris__

Miller__
Lomeli__

Date:_____

Draft Response:
_____

Review & Comment:
_____

Other:_____
_____
_____

File: 07.50
ORIGINAL - NEED TO FILE AWAY

---

NOTES:

**WESTERN TECHNOLOGIES INC.**

3737 East Broadway Road
P.O. Box 21387
Phoenix, Arizona 85036
(602) 437-3737

# LETTER OF TRANSMITTAL

To Maricopa County Flood Control District
3335 West Durango Street
Phoenix, Arizona 85009

Date 8/1/86
Job No. 2125J296
Lab./Invoice No.
Ref. No.

Attn. Mr. Nick Karan, P.E.; Chief Engineering Division

Project/Subject Shrinkage Testing
Spookhill Flood Retention Structure
Maricopa County, Arizona

Please be informed that we are:

| | | |
|---|---|---|
| x | Enclosing | Engineering Reports |
| | Forwarding Separately | Laboratory Reports |
| | Per Your Request | Field Reports |
| 3 | No. of Copies | Proposals |
| x | Other | |

More fully described as follows: Shrinkage limits tests data sheets for remolded samples. On these sheets a negative volumetric change, % signifies an increase in volume.

For your:

| | | |
|---|---|---|
| x | Use | Information |
| | Approval | Action |
| | Files | Other |

Material forwarded by:

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WATER RESOURCES

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| ✓ | FINANCE | 1 | CGF |

REMARKS

By Frank Costello

Frank Costello

SHRINKAGE LIMITS

Sample ID. STA 265+00 (6.6-7.6) Job No. 21251296

| | | | |
|---|---|---|---|
| Placement Density,pcf | 125 | Undisturbed | _____ |
| Placement Moisture,% | 7.4 | Remolded | * |

| | | | |
|---|---|---|---|
| Present Density,gm/cc | 1.802 | Initial Measurements,in. | |
| Present Moisture,% | 7.4 | Height | Width |
| Soil Sp.Gr.,gm/cc | 2.666 | 4.642 | 3.985 |
| | | 4.601 | 3.971 |
| Max. Dry Density,gm/cc | 1.743 | 4.612 | 3.975 |
| Minimum Volume,cu.ft. | 1.1496 | 4.632 | 3.989 |
| Wt. of Solids,lbs. | 4.02 | | 3.979 |
| | | | 3.984 |
| Initial Volume,cu.ft. | 0.0333 | | |
| Int. Dry Density,gm/cc | 1.802 | 4.622 | 3.981 |
| | | Avg. | Avg. |
| Final Volume,cu.ft. | 0.0342 | | |
| Final Dry Density,gm/cc | 1.743 | Final Measurements,in. | |
| | | Height | Width |
| Shrinkage Limit,% | 19.88 | 4.691 | 4.011 |
| Volumetric Shrinkage,% | -2.7 | 4.694 | 4.009 |
| Linear Shrinkage,% | -------- | 4.655 | 4.010 |
| | | 4.653 | 4.013 |
| Volumetric Change,% | -2.7 | | 4.021 |
| Wt. After Drying,Lbs. | 3.72 | | 4.016 |
| | | 4.673 | 4.013 |
| | | Avg. | Avg. |

SHRINKAGE LIMITS

Sample ID. STA 265+00 (9.2-10.2) Job No. 2125J296

| | | | |
|---|---|---|---|
| Placement Density,pcf | 122.7 | | Undisturbed _____ |
| Placement Moisture,% | 8.5 | | Remolded * |
| Present Density,gm/cc | 1.926 | Initial Measurements.in. | |
| Present Moisture,% | 8.5 | Height | Width |
| Soil Sp.Gr.,gm/cc | 2.693 | 4.496 | 3.936 |
| | | 4.476 | 3.935 |
| Max. Dry Density,gm/cc | 1.931 | 4.500 | 3.927 |
| Minimum Volume,cu.ft. | 1.0185 | 4.506 | 3.935 |
| Wt. of Solids,lbs. | 4.12 | | 3.926 |
| | | | 3.936 |
| Initial Volume,cu.ft. | 0.0316 | | |
| Int. Dry Density,gm/cc | 1.926 | 4.495 | 3.933 |
| | | Avg. | Avg. |
| Final Volume,cu.ft. | 0.0320 | | |
| Final Dry Density,gm/cc | 1.931 | Final Measurements.in. | |
| | | Height | Width |
| Shrinkage Limit,% | 14.66 | 4.458 | 3.977 |
| Volumetric Shrinkage,% | -1.4 | 4.459 | 3.974 |
| Linear Shrinkage,% | _____ | 4.454 | 3.979 |
| | | 4.452 | 3.978 |
| Volumetric Change,% | -1.4 | | 3.981 |
| Wt. After Drving,Lbs. | 3.86 | | 3.976 |
| | | 4.456 | 3.978 |
| | | Avg. | Avg. |

station 265+00

# SHRINKAGE LIMITS

Sample ID. 16-2-18.2

Job No. 2125J296

| | | | |
|---|---|---|---|
| Placement Density,pcf | 121.9 | | Undisturbed |
| Placement Moisture,% | 8.0 | | Remolded * |
| Present Density,gm/cc | 1.938 | Initial Measurements,in. | |
| Present Moisture,% | 8.0 | Height | Width |
| Soil Sp.Gr.,gm/cc | 2.710 | 4.469 | 3.983 |
| | | 4.466 | 3.982 |
| Max. Dry Density,gm/cc | 1.956 | 4.471 | 3.977 |
| Minimum Volume,cu.ft. | 0.9990 | 4.466 | 3.982 |
| Wt. of Solids,lbs. | 4.2 | | 3.975 |
| | | | 3.979 |
| Initial Volume,cu.ft. | 0.0322 | | |
| Int. Dry Density,gm/cc | 1.938 | 4.468 | 3.980 |
| | | Avg. | Avg. |
| Final Volume,cu.ft. | 0.0320 | | |
| Final Dry Density,gm/cc | 1.956 | Final Measurements,in. | |
| | | Height | Width |
| Shrinkage Limit,% | 14.24 | 4.412 | 3.984 |
| Volumetric Shrinkage,% | 0.4 | 4.475 | 3.982 |
| Linear Shrinkage,% | -------- | 4.414 | 3.987 |
| | | 4.470 | 3.982 |
| Volumetric Change,% | 0.4 | | 3.980 |
| Wt. After Drying,Lbs. | 3.91 | | 3.986 |
| | | 4.443 | 3.984 |
| | | Avg. | Avg. |

SHRINKAGE LIMITS

Sample ID. STA 265+00 (18.3-20.3) Job No. 2125J296

| | | | |
|---|---|---|---|
| Placement Density,pcf | 128.9 | | Undisturbed _____ |
| Placement Moisture,% | 9.6 | | Remolded * |
| Present Density,gm/cc | 1.853 | Initial Measurements,in. | |
| Present Moisture,% | 9.6 | Height | Width |
| Soil Sp.Gr.,gm/cc | 2.679 | 4.608 | 3.977 |
| | | 4.607 | 3.981 |
| Max. Dry Density,gm/cc | 1.859 | 4.592 | 3.979 |
| Minimum Volume,cu.ft. | 1.1112 | 4.584 | 3.966 |
| Wt. of Solids,lbs. | 4.19 | | 3.975 |
| | | | 3.989 |
| Initial Volume,cu.ft. | 0.0331 | | |
| Int. Dry Density,gm/cc | 1.853 | 4.598 | 3.978 |
| | | Avg. | Avg. |
| Final Volume,cu.ft. | 0.0329 | | |
| Final Dry Density,gm/cc | 1.859 | Final Measurements,in. | |
| | | Height | Width |
| Shrinkage Limit,% | 16.47 | 4.603 | 3.962 |
| Volumetric Shrinkage,% | 0.4 | 4.600 | 3.967 |
| Linear Shrinkage,% | --------- | 4.571 | 3.976 |
| | | 4.602 | 3.971 |
| Volumetric Change,% | 0.4 | | 3.975 |
| Wt. After Drying,Lbs. | 3.82 | | 3.977 |
| | | 4.594 | 3.971 |
| | | Avg. | Avg. |

# SHRINKAGE LIMITS

Sample ID. STA 265+00 (20.2-22.2) Job No. 2125J296

| | | | |
|---|---|---|---|
| Placement Density,pcf | 122.8 | Undisturbed | _____ |
| Placement Moisture,% | 8.4 | Remolded | * |

| | | Initial Measurements,in. | |
|---|---|---|---|
| Present Density,gm/cc | 1.995 | Height | Width |
| Present Moisture,% | 8.4 | 4.2667 | 3.957 |
| Soil Sp.Gr.,gm/cc | 2.716 | 4.259 | 3.960 |
| | | 4.277 | 3.965 |
| Max. Dry Density,gm/cc | 1.988 | 4.273 | 3.953 |
| Minimum Volume,cu.ft. | 0.9897 | | 3.965 |
| Wt. of Solids,lbs. | 4.11 | | 3.973 |
| Initial Volume,cu.ft. | 0.0305 | | |
| Int. Dry Density,gm/cc | 1.995 | 4.269 | 3.962 |
| | | Avg. | Avg. |
| Final Volume,cu.ft. | 0.0306 | | |
| Final Dry Density,gm/cc | 1.988 | Final Measurements,in. | |
| | | Height | Width |
| Shrinkage Limit,% | 13.47 | 4.266 | 3.975 |
| Volumetric Shrinkage,% | -0.5 | 4.241 | 3.969 |
| Linear Shrinkage,% | _____ | 4.252 | 3.983 |
| | | 4.287 | 3.977 |
| Volumetric Change,% | -0.5 | | 3.974 |
| Wt. After Drying,Lbs. | 3.8 | | 3.980 |
| | | 4.262 | 3.976 |
| | | Avg. | Avg. |

# SHRINKAGE LIMITS

Sample ID. STA 280+00 (5.5-6.5)

Job No. 2125J296

| | |
|---|---|
| Placement Density,pcf | 123.8 |
| Placement Moisture.% | 8.6 |
| Present Density.gm/cc | 1.894 |
| Present Moisture,% | 8.5 |
| Soil Sp.Gr.,gm/cc | 2.695 |
| Max. Dry Density,gm/cc | 1.898 |
| Minimum Volume,cu.ft. | 1.0452 |
| Wt. of Solids,lbs. | 3.99 |
| Initial Volume,cu.ft. | 0.0311 |
| Int. Dry Density,gm/cc | 1.894 |
| Final Volume.cu.ft. | 0.0321 |
| Final Dry Density,gm/cc | 1.898 |
| Shrinkage Limit,% | 15.58 |
| Volumetric Shrinkage,% | -3.0 |
| Linear Shrinkage,% | --------- |
| Volumetric Change,% | -3.0 |
| Wt. After Drying,Lbs. | 3.8 |

Undisturbed _____

Remolded *

Initial Measurements,in.

| Height | Width |
|---|---|
| 4.437 | 3.934 |
| 4.43 | 3.935 |
| 4.426 | 3.927 |
| 4.426 | 3.925 |
| | 3.927 |
| | 3.937 |
| 4.430 | 3.931 |
| Avg. | Avg. |

Final Measurements,in.

| Height | Width |
|---|---|
| 4.442 | 3.981 |
| 4.471 | 3.982 |
| 4.443 | 3.981 |
| 4.447 | 3.981 |
| | 3.990 |
| | 3.979 |
| 4.451 | 3.982 |
| Avg. | Avg. |

SHRINKAGE LIMITS

Sample ID. STA 280+00 (12.0-13.0) Job No. 2125J296

| | | | |
|---|---|---|---|
| Placement Density,pcf | 118.3 | Undisturbed | _____ |
| Placement Moisture,% | 10.1 | Remolded | * |

| | | | |
|---|---|---|---|
| Present Density,gm/cc | 2.007 | Initial Measurements,in. | |
| Present Moisture,% | 10.1 | Height | Width |
| Soil Sp.Gr.,gm/cc | 2.643 | 4.108 | 3.932 |
| | | 4.111 | 3.94 |
| Max. Dry Density,gm/cc | 1.997 | 4.206 | 3.936 |
| Minimum Volume,cu.ft. | 0.9496 | 4.217 | 3.926 |
| Wt. of Solids,lbs. | 4.03 | | 3.929 |
| | | | 3.926 |
| Initial Volume,cu.ft. | 0.0292 | | |
| Int. Dry Density,gm/cc | 2.007 | 4.161 | 3.932 |
| | | Avg. | Avg. |
| Final Volume,cu.ft. | 0.0294 | | |
| Final Dry Density,gm/cc | 1.997 | Final Measurements,in. | |
| | | Height | Width |
| Shrinkage Limit,% | 12.25 | 4.154 | 3.926 |
| Volumetric Shrinkage,% | -0.5 | 4.146 | 3.924 |
| Linear Shrinkage,% | _____ | 4.231 | 3.928 |
| | | 4.233 | 3.921 |
| Volumetric Change,% | -0.5 | | 3.931 |
| Wt. After Drying,Lbs. | 3.66 | | 3.933 |
| | | 4.191 | 3.927 |
| | | Avg. | Avg. |

SHRINKAGE LIMITS

Sample ID. STA 280+00 (16.0-18.0) Job No. 21251296

| | | | |
|---|---|---|---|
| Placement Density,pcf | 120.5 | Undisturbed | _____ |
| Placement Moisture,% | 9.7 | Remolded | * |
| Present Density,gm/cc | 1.921 | Initial Measurements,in. | |
| Present Moisture,% | 9.7 | Height | Width |
| Soil Sp.Gr.,gm/cc | 2.717 | 4.582 | 3.985 |
| | | 4.618 | 3.982 |
| Max. Dry Density,gm/cc | 1.924 | 4.622 | 3.973 |
| Minimum Volume,cu.ft. | 1.0036 | 4.600 | 3.982 |
| Wt. of Solids,lbs. | 4.36 | | 3.972 |
| | | | 3.987 |
| Initial Volume,cu.ft. | 0.0332 | | |
| Int. Dry Density,gm/cc | 1.921 | 4.606 | 3.980 |
| | | Avg. | Avg. |
| Final Volume,cu.ft. | 0.0332 | | |
| Final Dry Density,gm/cc | 1.924 | Final Measurements,in. | |
| | | Height | Width |
| Shrinkage Limit,% | 15.17 | 4.612 | 3.981 |
| Volumetric Shrinkage,% | -0.2 | 4.619 | 3.981 |
| Linear Shrinkage,% | _____ | 4.607 | 3.980 |
| | | 4.617 | 3.983 |
| Volumetric Change,% | -0.2 | | 3.980 |
| Wt. After Drying,Lbs. | 3.99 | | 3.980 |
| | | 4.614 | 3.981 |
| | | Avg. | Avg. |

SHRINKAGE LIMITS

Sample ID. STA 280+00 (18.0-20.0) Job No. 21251296

| | | | |
|---|---|---|---|
| Placement Density,pcf | 120.7 | Undisturbed | _____ |
| Placement Moisture,% | 11.0 | Remolded | * |
| Present Density,gm/cc | 1.911 | Initial Measurements,in. | |
| Present Moisture,% | 11.1 | Height | Width |
| Soil Sp.Gr.,gm/cc | 2.695 | 4.545 | 3.979 |
| | | 4.542 | 3.979 |
| Max. Dry Density,gm/cc | 1.924 | 4.547 | 3.978 |
| Minimum Volume,cu.ft. | 1.0051 | 4.500 | 3.972 |
| Wt. of Solids,lbs. | 4.32 | | 3.980 |
| | | | 3.982 |
| Initial Volume,cu.ft. | 0.0326 | | |
| Int. Dry Density,gm/cc | 1.911 | 4.534 | 3.978 |
| | | Avg. | Avg. |
| Final Volume,cu.ft. | 0.0327 | | |
| Final Dry Density,gm/cc | 1.924 | Final Measurements,in. | |
| | | Height | Width |
| Shrinkage Limit,% | 14.86 | 4.571 | 3.980 |
| Volumetric Shrinkage,% | -0.4 | 4.572 | 3.982 |
| Linear Shrinkage,% | _____ | 4.548 | 3.978 |
| | | 4.501 | 3.978 |
| Volumetric Change,% | -0.4 | | 3.981 |
| Wt. After Drying,Lbs. | 3.93 | | 3.975 |
| | | 4.548 | 3.979 |
| | | Avg. | Avg. |

SHRINKAGE LIMITS

Sample ID. STA 280+00 (20.5-22.5) Job No. 2125J296

| | | | |
|---|---|---|---|
| Placement Density,pcf | 120.7 | Undisturbed | ---------- |
| Placement Moisture,% | 11.0 | Remolded | * |
| Present Density,gm/cc | 1.980 | Initial Measurements,in. | |
| Present Moisture,% | 11.0 | Height | Width |
| Soil Sp.Gr.,gm/cc | 2.690 | 4.213 | 3.976 |
| | | 4.282 | 3.976 |
| Max. Dry Density,gm/cc | 1.964 | 4.226 | 3.972 |
| Minimum Volume,cu.ft. | 0.9850 | 4.214 | 3.966 |
| Wt. of Solids,lbs. | 4.16 | | 3.965 |
| | | | 3.969 |
| Initial Volume,cu.ft. | 0.0303 | | |
| Int. Dry Density,gm/cc | 1.980 | 4.234 | 3.971 |
| | | Avg. | Avg. |
| Final Volume,cu.ft. | 0.0305 | | |
| Final Dry Density,gm/cc | 1.964 | Final Measurements,in. | |
| | | Height | Width |
| Shrinkage Limit,% | 13.75 | 4.206 | 3.974 |
| Volumetric Shrinkage,% | -0.6 | 4.23 | 3.976 |
| Linear Shrinkage,% | ---------- | 4.25 | 3.976 |
| | | 4.289 | 3.98 |
| Volumetric Change,% | -0.6 | | 3.980 |
| Wt. After Drying,Lbs. | 3.74 | | 3.981 |
| | | 4.244 | 3.978 |
| | | Avg. | Avg. |

Trip Notes Arizona, September 6-9, 1977, S. Schauman

1. Spook Hill FRS

The final design included a permanent irrigation system to be used temporarily to establish plant material. They have now decided to use this system to prevent desiccation. Obviously the amount of water needed to prevent desiccation is not the same as the water needs for desert shrubs. Nor is the sustained need for desiccation comparable with that to temporarily establish desert vegetation.

Recommendation # 1

The engineering and landscape architecture situation be carefully monitored to determine the problems and benefits occuring through use of irrigation to prevent desiccation. The monitoring data should include:

1. Check of the saquaro cacti root growth and general survivability.

2. Identification of the shrubs that result from the sown seed and check of their root growth.

3. Identification of voluntary plants and check of their root growth.

Recommendation # 2

The desiccation and planting design problems on all future structures be solved concurrently to achieve the best overall engineering/landscape architecture solution. (See recommendation 3 & 4 below.)

2. Queen Creek FRS

The present planning of this structure calls for a vapor barrier of GP and SP blanket, at 2 1/2:1 slope ranging between 3' and 9' deep. The planners (State agency) have been advised by SCS that vegetation cannot be installed on this blanket layer. It is difficult for me to accept this judgment without field investigation supporting evidence. The site is rural and will not have high public visibility but some planting should be attempted.

Recommendation # 3

All feasible planting techniques (e.g. hydroseeding, fiber coated with seeds, change in gradation of the blanket particles, others) should be investigated and some technique to establish indigenous shrubs should be developed.

3. Option 3 - Use of a temporary (portable or water hauled) irrigation system for plant establishment.

   a. Same as 2-a above.

We believe the irrigation, plantings and cracking problems may be interrelated. Since we have no clear answers, we recommend that test situations of the different conditions be installed and monitored on this structure. We recommend that these be considered pilot tests and monitored yearly for at least the next five years. The plantings and irrigation could either reduce or aggrevate the cracking conditions.

A yearly report could give us the needed insight to the ongoing condition so that we can develop technical guidance on future structures. We would be pleased to advise your staff on the factors that should be monitored. We are returning the bid schedule but are keeping the plans.

NEIL F. BOGNER
Director
Engineering Division

Attachment

cc:
Kenneth L Williams, Director, TSC, SCS, Portland, Oregon
Paul O. Tilker, Head, EWP Unit, SCS, Portland, Oregon
Robert MacLauchlan, Plant Materials Specialist, SCS, WO

# FLOOD CONTROL DISTRICT
of
Maricopa County

3335 West Durango Street • Phoenix, Arizona 85009
Telephone (602) 262-1501



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PR

PAUL

40-5-0-

July 2, 1979

Mr. Thomas G. Rockenbaugh
State Conservationist
Soil Conservation Service
230 North 1st Avenue
Phoenix, Arizona 85025

Re: Spook Hill Project, Subsidence Monuments

Dear Mr. Rockenbaugh:

We have reviewed your proposed plans for the installation of subsidence monuments on the Spook Hill F.R.S. as submitted by letter to our office on June 18, 1979. The District agrees with the plans with one exception. We feel the monument spacing of one half mile center to center will not provide us with sufficient information and control. For this reason the District prefers the spacing be 1000 feet center to center. Perhaps our staff members can discuss this further and establish criteria that can be a standard for all flood retarding structures.

Should further discussion be required, please contact Sid Brase of my staff.

Sincerely,

Herbert P. Donald

Herbert P. Donald, P. E.

UNITED STATES DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

~~PAUL~~ c/c - Rocky + [illegible]

West Technical Service Center, 511 N.W. Broadway, Rm. 510, Portland, OR 97209

SUBJECT: ENG - Design - Buckhorn-Mesa WPP, Arizona - Spook Hill FRS

DATE: September 30, 1977

TO: Thomas G. Rockenbaugh, State Conservationist
SCS, Phoenix, Arizona

The proposed instrumentation for the Spook Hill Dam has been reviewed. Our comments are given in the order of your request.

1. Instrumentation is always desirable in an earth embankment. However, instrumentation should be selected and designed to assess or monitor the likely physical conditions that will exist during the service life of the structure. In our opinion, the two major potential sources of problems for the Spook Hill Dam are: (1) cracking with desiccation, and (2) cracking associated with subsidence caused by ground water withdrawal. It is our understanding that geologic analysis has fairly well established that this structure is close enough to the bedrock control that the likelihood of adverse effects of ground water removal caused subsidence is remote. We also understand that the irrigation system will be used regularly to maintain the designed vegetation. This irrigation, if properly done, will maintain the soil moisture level sufficiently to assure that desiccation cracking is unlikely.

With these thoughts in mind, we believe it would take a very special type of instrumentation to be of any value.

2. The strain gages proposed in the attachments to your letter, are not applicable to the special needs of this project. Their principal capability is to register differential movements and would not likely pick up the subtile movements associated with desiccation cracking. Their value mainly lies in monitoring movements in areas of predictable differential stress.

3. Other types of differential movement gages have the same basic limitations as do the proposed gages.

4. We feel that meaningful data could be obtained from a series of moisture tension gages to monitor capillary stresses and assess the need for irrigation. With water applied, cracking should not occur. However, if readings are ignored, the dam will crack.

We agree that monuments should be established on the completed embankment. These monuments should be established with highly accurate horizontal and vertical control.

Richard M. Matthews

Richard M. Matthews
Acting Head, E&WP Unit

STATE OF ARIZONA
OFFICE OF STATE WATER ENGINEER
SUPERVISION OF DAM SAFETY

## INSPECTION OF DAM CONSTRUCTION

PROJECT Spook Hill Dam (7-50) SHEET 1 OF 1

CONTACTS Jim Mittleton and Don Riddle - SCS

IMPORTANT ACTIONS TAKEN:

Mittleton informed me that he had been instructed to permit the placement of Type 3 material in the entire downstream section of the dam except for the 10 feet of Type 2 at the downstream face. He said this had been agreed to at the meeting yesterday between SCS, Mardian, Flood Control District, and AWC. This is a change in the specifications as shown on Sheet 13 of the drawings and should be documented in writing by SCS.

DESCRIPTION OF WORK:

Contractor excavating foundation between Sta. 265-270. Buried channel at approximate Sta. 269 appears to have been excavated down to firm material and is about 20 ft. deep, but loose material has not been cleaned off. Most of the unconsolidated material has been cleaned off of the foundation between Sta. 265-268 except downstream of the centerline. Contractor requested and SCS agreed to stake cutoff trench before the rest of the foundation is cleaned off so that the excavated material could be placed directly in the downstream section of the embankment. Contractor was not quite ready to place this fill so work on the foundation between Sta. 265-270 was suspended for today.

Contractor working fill between Sta. 275-290 but mostly between Sta. 275-280 where Types 2, 4, & 5 was being placed.

Culvert pipe for floodway under McDowell Road did not meet specification and rejected by SCS.

INSPECTED BY G. D. Cox GDC ON 6-23-78 PHOTOS No

GDC

STATE OF ARIZONA
OFFICE OF STATE WATER ENGINEER
SUPERVISION OF DAM SAFETY

## INSPECTION OF DAM CONSTRUCTION

PROJECT Spook Hill Dam (7-50) SHEET 1 OF 1

CONTACTS Tom Jayo, Don Riddle, Jim Mittleton - SCS

IMPORTANT ACTIONS TAKEN:

Made preliminary inspection of foundation Sta. 249-260 after contractor finished excavating cutoff trench and noted several discrepancies. However, I was not able to discuss these with Jayo today and plan on having a joint inspection with SCS tomorrow morning.

DESCRIPTION OF WORK:

Contractor excavated cutoff trench Sta. 249-256. Placed Type 2, 4, and 5 materials below grade Sta. 260-265. Working and placing fill from approximate Sta. 265-290.

Most of fill being placed today is coming from pre-wetted borrow areas and very little watering is being done on the embankment. Some of the pre-wetted borrow material is too wet and has to be worked quite a bit.

Jim Mittleton informed me that he and the other SCS inspector, Albert Rutledge, no longer permit caliche in any form, cemented or uncemented (powder /silt/sand) to be placed in the Type 1 material zone. They do this on the basis that the caliche will not pass the 5% PI requirement. He also said that they do not now permit uncemented caliche to be placed in the Type 2 material below existing ground level upstream of the centerline of the dam. This is another example of the changing, and largely undocumented, use of caliche in this dam.

New pipe for the floodway under McDowell Road has been delivered to the job and is being placed in the trench today. This pipe now apparently meets specifications but fitting the pipe together is proving to be difficult.

No work is being done on the principal or emergency spillways.

INSPECTED BY G. D. Cox GDC ON July 18, 1978 PHOTOS No

BJS

STATE OF ARIZONA
OFFICE OF STATE WATER ENGINEER
SUPERVISION OF DAM SAFETY

INSPECTION OF DAM CONSTRUCTION

PROJECT Spook Hill Dam (7-50) SHEET 1 OF 1

CONTACTS Tom Jayo, Aubrey Sanders - SCS

IMPORTANT ACTIONS TAKEN:

Tom Jayo, Aubrey Sanders (SCS geologist) and I looked at foundation between approximate Sta. 236-249, and specifically at the excavation in the area of Sta. 244 where a buried channel filled with uncemented, sandy material crosses the foundation. This channel has sort of an elongated "S" shape with the long part of the "S" parallel to, and about the same position as, the cutoff trench. All of the sand had not been excavated and after some discussion it was decided that the best way to proceed would be to excavate the cutoff trench to caliche and then excavate the adjacent foundation as required.

We also looked at the rest of the foundation briefly, and although it looked pretty good except for a couple of questionable sandy areas, there was still a lot of loose material on the foundation. I told Jayo the loose material should be cleaned up some and the foundation rechecked before the cutoff trench was staked and excavated.

Jayo requested that someone from the AWC be at he site first thing tomorrow morning to inspect the foundation as the sandy material from the channel was excavated, and also to recheck the rest of the foundation prior to staking the cutoff trench.

GDC

INSPECTED BY G. D. Cox ON August 1, 1978 PHOTOS No

STATE OF ARIZONA
OFFICE OF STATE WATER ENGINEER
SUPERVISION OF DAM SAFETY

## INSPECTION OF DAM CONSTRUCTION

PROJECT Spook Hill (7-50) SHEET 1 OF 1

CONTACTS Jayo, Riddle - SCS

IMPORTANT ACTIONS TAKEN:

With Jayo, made preliminary inspection of foundation from approximately Sta. 200-210. Most of foundation to acceptable materials (i.e. "siltstone"; dirty, dense sands; small amount of caliche) but two channels of loose sand were encountered at Sta. 202+30 and 208. Dozer excavations showed sand depth of not more than 2 ft. additional. Jayo agreed that the sand should be removed.

DESCRIPTION OF WORK:

Placing Types 1, 2, 3, and 4 fill between Sta. 212-228. Type 2 material being borrowed from the foundation excavation just south of McKellips Road.

Placing fill in the emergency spillway outlet channel. Finish grading of McDowell Road ramp subgrade in progress.

Jayo believes that more unacceptable Type 2 material will have to be removed from Sta. 228-248. No work in this area today.

Surveyors will stake key trench from Sta. 200-210 today. This section should be ready for final approval Tuesday, September 5.

Quality control tests (moisture/density determinations in particular) are normally scheduled at approximately 3000 c.y. intervals, and are used primarily for documentation. However, as this job has progressed, SCS has had to do more and more tests to confirm opinions of its inspectors. Almost all recent tests have been made to resolve controversies between SCS and Mardian.

Jayo reported that additional inspectors could not be recruited from neighboring states because none were available. SCS intends to hire temporary employees as needed, and as available.

WCJ

INSPECTED BY BGS/WCJ ON September 1, 1978 PHOTOS Yes

BGS

STATE OF ARIZONA
ARIZONA WATER COMMISSION
SUPERVISION OF DAM SAFETY

## FLOOD ESTIMATE AND SPILLWAY ANALYSIS

Name of Dam Spook Hill Type of Dam Zoned earthfill (routed homogenous) Dam No. 7-5

County Maricopa Hydrologic Class III - High hazard potential, medi size.

Located on _____ Tributary to Salt River

_____ - ¼ Sec. 6, 8, 16 Twp. 1 north Range 7 east B &

USGS Quadrangle Buckhorn (7½'), 15' (Circle One)

| I. DRAINAGE BASIN | | II. DAM AND RESERVOIR | |
|---|---|---|---|
| 1. Drainage Area - Sq. Mi. | 16.38 | 1. Reservoir Area @S/W. Ac. | 305 |
| 2. Channel Reach Length, Ft. | Varies - sub-watersheds | 2. Res. Capac. to S/W-A.F. | 992 |
| 3. Maximum Elevation | 2673 | 3. Surcharge Storage-A.F. | 2570 |
| 4. Minimum Elevation | 1567 | 4. Spillway Crest Elevation | 1582.0 ft. |
| 5. Average Watershed Slope, % | - | 5. Dam Crest Elevation | 1591.0 ft. |
| 6. Cover Density, % | - | 6. Total Freeboard | 9.0 ft. |
| 7. Cover Type, | Desert brush | 7. Max. Storage Level | 1589.55 |
| 8. Soil Group | - | 8. Gated or Ungated | ungated |
| 9. Runoff Curve Number | Varies 75 - 79 | 9. Max. Spillway Q,-cfs @ Max. W.S. | 20040 |
| 10. Impaired? | No | | |

Sketch of Spillway Attached, as per Sheets 8 & 10 of Drawing 7-E-23797 Date 12-76

Remarks, Special Conditions, Etc. Outlet (principal spillway) flow at maximum water level is 1060 cfs.

Calculated by W.C.J. from SCS TR-20 Data Date April 1977

Checked by _____ Date _____

## FLOOD HYDROLOGY

| | | | | | | |
|---|---|---|---|---|---|---|
| 1. | Flood Type | SCS Class C FBH | | | | |
| 2. | Storm Precip-In. | 13.0 | | | | |
| 3. | Precip. Dur. -Hr. | 6 | | | | |
| 4. | Peak Intensity -In/Hr. | - | | | | |
| 5. | Time of Concentration -Hr. | Varies subwatersheds | | | | |
| 6. | Peak Inflow -cfs | 48500 | | | | |
| 7. | Peak Inflow -csm | - | | | | |
| 8. | Runoff -A. F. | 6887 | | | | |
| 9. | Runoff -In. | 9.5 | | | | |
| 10. | Runoff Coeff. | Varies 75 - 79 | | | | |
| 11. | Routed? | Yes | | | | |
| 12. | Peak Outflow -cfs | 22100 | | | | |
| 13. | Peak Outflow -csm | - | | | | |
| 14. | Max. Water Surface Elev. | 1589.55 | | | | |
| 15. | Residual Freeboard | 1.45 | | | | |
| 16. | Diverted Inflow, cfs | 2200 | | | | |
| 17. | Check Adopted Flood | ✓ | | | | |

Remarks:

M E M O R A N D U M

REVIEW OF GEOLOGIC CONDITIONS

SPOOK HILL FLOODWATER RETARDING STRUCTURE (7-50)

By: G. D. Cox GDC
April 29, 1977

Application for approval to construct the Spook Hill Floodwater Retarding Structure has been filed with the Arizona Water Commission. A review of geologic conditions and construction materials has been made, and this memorandum summarizes the results and conclusions reached.

The site of the proposed dam is about 10 miles northeast of Mesa in Section 31, T2N, R7E, and Sections 6, 7, 8, 16 and 17, T1N, R7E. The maximum height of the dam is about 23 feet and it is 21,575 feet long. The dam is a zoned earthfill structure that will function as a runoff retarding dike; there will be no permanent storage of water. The proposed Central Arizona Project (CAP) canal alignment parallels the toe of the dike and will be protected by it.

The geologic review consisted of a review of available geologic data and plans and specifications for the dam, as well as a site inspection. One geologic report was submitted with the application:

> Report of Geologic Investigation, Buckhorn-Mesa Watershed, Spook Hill Damsite; U.S. Soil Conservation Service; undated.

REGIONAL GEOLOGY

The proposed dam site is located on the edge of an alluvial basin within the Basin and Range physiographic province. The ranges are fault-block mountains composed mostly of intrusive igneous and metamorphic rocks together with some volcanic and sedimentary rocks. The basins are broad alluvial plains formed by erosion of the ranges and are often several thousand feet thick.

DAM SITE GEOLOGY

The site is located on a pediment along the southwest side of the Usery Mountains. The sedimentary veneer covering the granite bedrock is composed of a mixture of silt, sand, gravel and some clay. This thin mantle of alluvial deposits is unconsolidated or is weakly to strongly cemented with calcareous cement forming widespread caliche beds. Streams have cut channels in the bedrock and

the cemented deposits and have subsequently backfilled them with relatively loose alluvial material. Scattered hills of weathered, coarsely crystaline granite protrude through the alluvium. Faults occur in the Usery Mountains, but any that might be in the vicinity of the dam site are covered by alluvium. Ground water is not known to occur in the thin alluvium at the site. Ground water is present in the underlying granite bedrock but depth to water is 400 to 600 feet. The alluvial deposits thicken considerably about one mile west of the dam site and ground water occurs there at a depth of 500 to 600 feet.

Ground subsidence and earth fissures occur 1-2 miles southwest of the dam site. West and south of the site the alluvium thickens rapidly and the ground water in the alluvium has declined over 300 feet since 1923. Thick alluvium and declining ground water levels are necessary for the occurrence of subsidence and earth fissures, and it is expected that subsidence will continue and additional earth fissures may occur within 1-2 miles south and west of the dam site. However, at the dam site the alluvium is thin and ground water is not known to occur in the alluvium. Surveys by the USBR along the CAP canal alignment between the Salt River and the Apache Trail highway have shown no evidence of subsidence. Ground subsidence and earth fissures are not expected to affect the Spook Hill Dam site.

## SEISMICITY

None of the known faults in the immediate vicinity of the site are known to be active and there have been no recorded earthquakes in the immediate vicinity during the recorded seismic history of Arizona. However, 12 earthquakes have occurred in historic time within a radius of 100 miles of the site; the largest of these was a 5.1 magnitude event near Globe in 1969 and a 5.0 magnitude quake in Chino Valley in 1976. The closest historic earthquakes occurred at New River in 1974 with magnitudes of 2.5 and 3.0.

Based on geology and seismic history of the area, the dam site could experience an expected maximum probable intensity of VI-VII on the Modified Mercalli Scale. Presently, the closest known active fault is about 40 miles from the site; assuming an earthquake of about 6 or less in magnitude on this fault, a seismic coefficient of 0.05g would be acceptable and 0.1g should be conservative for stability analysis.

## ENGINEERING GEOLOGY

### Foundation Exploration

Sixteen holes were drilled and 22 backhoe pits were dug along the centerline of the dam to investigate foundation conditions. Disturbed samples were taken for classification and dissolved solids testing, but no undisturbed samples were obtained.

Eight holes were also drilled for standard penetration testing in order to evaluate soil density and potential consolidation.

Foundation Conditions

The dam site is underlain entirely by alluvial deposits. Surficial soils consist mostly of gravelly silty sand with some clayey sand from the southern end of the dike (Sta. 88+00) north to about Sta. 200+00. From about Sta. 200+00 to the northern end of the dike (Sta. 304+75) the surficial soils are gravelly sandy silt and gravelly silty sand. Depth of the surficial soils range from 0-5.5 feet but averages about 3 feet in thickness. These soils have a low density and are considered by SCS to have potential for consolidation.

From the south end of the dam north to about Sta. 230+00 the surficial soils are underlain by fine-to-coarse grained gravelly silty sand and clayey sand. These soils are more dense than the overlying soils as indicated by blow counts. Beneath these soils there is weakly cemented gravelly siltstone from Sta. 88+00 to about Sta. 165+00. The siltstone interfingers with caliche and some sand between approximate Sta. 165+00 to Sta. 230+00. From Sta. 230+00 to the north end of the dam only variably cemented gravelly caliche is present beneath the compressible surficial soils.

Bearing capacity of the foundation materials beneath the surficial soils should be adequate for the structure as designed.

Foundation Seepage

Five open-hole infiltration tests were made in three drill holes along the present dam alignment. From these tests, the estimated permeability was 7.6 and 87.8 ft/day in the surficial soils, 62.1 ft/day in the upper zone of the siltstone, and 9.3 and 3.3 ft/day in the caliche. These estimates seem quite high, especially for the caliche. It is possible that the casing was not sealed at the bottom of the hole allowing water to get up and around the outside of the casing, thus resulting in a higher water loss and estimated permeabilities than expected.

The dam is designed with a cut-off trench that will extend through the more permeable surficial soils. It is unlikely that the actual permeability of the siltstone and caliche will be high enough to pose a problem to a retarding dam.

Foundation Stripping

The SCS geology report recommended stripping material considered to have consolidation potential from the foundation; this recommendation has been followed in the specification drawings except at Sta. 280+00 where silt overlying the caliche should be removed. The geology report also recommends that the cutoff trench be excavated about one foot

into indurated caliche or siltstone. This will require about a 25-feet excavation in the vicinity of Sta. 233+00 where a buried stream channel was found. Another buried stream channel may occur in the vicinity of Sta. 180+00, and others may be discovered during foundation excavation.

Outlet Structure (Principal Spillway)
The proposed outlet structure is located at the north end of the dam and discharges into a floodway leading to the Salt River. No exploration holes or pits were dug along the axis of the outlet at its final design location. However, several rotary holes in the area indicate that a thin (1-4 feet) mantle of soil overlies firm caliche. Standard penetration blow counts were high in the caliche and no consolidation problems with the outlet structure are expected.

Spillway (Emergency Spillway)
The proposed spillway is located very near the north end of the dam. It is designed to be constructed through the embankment and will discharge into the CAP canal. Over a dozen holes were drilled to investigate a spillway site around the north end of the dam, but only four holes were drilled and one backhoe pit dug in the area of the final design spillway location. These indicated that a mantle of unconsolidated gravelly sandy silt 2-5 feet thick overlying earthy to firm gravelly caliche will be encountered in the excavation for the spillway.

The upper 1.5 feet of caliche was somewhat soft but below this upper zone standard penetration blow counts were relatively high, less than 0.5 ft. per 50 blows. The caliche below the upper soft zone should have sufficient bearing capacity for the concrete control structure. The spillway channel will be unlined except for a concrete spillway outlet channel protection apron at the downstream end of the channel. This should provide sufficient erosion protection for the infrequent spills expected.

Reservoir Area
The reservoir area is underlain by alluvial deposits similar to those in the dam foundation. No particular difficulty is expected with the reservoir; however, siltation of the principal spillway inlet channel could be a maintenance problem.

Construction Materials

The proposed principal borrow area is along the CAP canal alignment parallel to the downstream toe of the dam. Additional borrow will be available from required excavation and from the reservoir area. Forty-seven backhoe pits, 3-15.5 feet deep, were dug for borrow exploration; 22 of these pits were located along the CAP canal alignment.

Embankment: Type 1 - Type 1 materials will be placed as an upstream blanket-zone about four feet in thickness. The material consists of a mixture of silt, sand, gravel and clay, with gravelly silty sand and clayey sand being the probable dominant soil types. Seven tests were run on the low plastic clayey sand; the fine content ranged from 14.9 to 43.5%, the liquid limit from 23 to 33, the plasticity index from 7 to 16, compacted density from 122.2 to 130.6 pcf, and optimum moisture from 8.9 to 10.9%. Two direct shear tests on the clayey sand gave Ø angles of 32 and 32.25 degrees; no permeabilities were obtained. The specifications require only that Type 1 materials have at least 15% fines and a maximum size of 6 inches. A general borrow location for Type 1 materials is given in the specifications, and a sufficient quantity of material should be available to meet requirements.

Embankment: Type 2 - Type 2 materials will comprise the upstream half of the embankment along with Type 1, and will also be used as a downstream blanket-zone. These materials are similar to Type 1 soils except that Type 2 soils are dominantly nonplastic. Six tests were run on nonplastic soils; the fine content ranged from 22.8 to 43.1%, compacted density from 119.6 to 129.2 pcf, and optimum moisture from 8.5 to 13.0%. Two direct shear tests on silty sand gave Ø angles of 33.5 and 35 degrees. As with Type 1, Type 2 materials must have at least 15% fines and a maximum size of no more than 6 inches. A sufficient quantity of material should be available from the designated borrow areas to meet requirements.

Embankment: Type 3 - Type 3 materials will be placed in the center of the dam downstream of the centerline. These materials consist of excavated siltstone and caliche which reportedly will break down to a mix of silt, sand and gravel. One sample of caliche was tested which broke down to a silty sandy gravel. The fine content was 14.1%, compacted density was 109.5 pcf, and optimum moisture was 17.9%. There are no specifications for this material except the maximum size can be no larger than 6 inches. A sufficient quantity of this material will be available to meet requirements.

Embankment: Type 4 - Type 4 material will be placed in the downstream part of the embankment between Types 2 and 3, and will replace excavated surficial soils downstream of the centerline of the dam. The material will consist of sand, gravel, and excavated weathered, fractured granite. No testing was done on this material. The specifications require only that Type 4 material have a maximum of 15% fines and a maximum

size of no larger than 6 inches. Materials classification testing indicate most of the designated borrow soils have more than 15% fines so excavated granite will probably have to form a large part of the Type 4 material.

Embankment: Type 5 _ Type 5 is an optional material that can be used in place of Type 4 as backfill for excavated surficial soils. Type 5 material will consist of oversize Type 4; no other requirements are given in the specifications.

Drain - A small amount of drain material will be required for the emergency spillway structure. No source for the drain material was given in the specifications but should be available from nearby commercial plants.

Riprap - A small amount of riprap is required for the emergency and principal spillway structures. The specifications propose to use sound granite blocks, 3 to 18 inches in size, excavated from the floodway channel north of the dam. However, granite from the proposed borrow area will probably be badly weathered; if sufficient sound granite is not available from this source suitable rock should be available from nearby granite outcrops.

CONCLUSIONS

1. The site is suitable for the proposed floodwater retarding structure.

2. No testing was done on undisturbed samples of foundation materials. However, based upon descriptions of the foundation materials and standard penetration blow counts, the bearing capacity of the foundation materials beneath the surficial soils should be adequate for the embankment as designed.

3. Permeability tests conducted in the foundation materials are of doubtful validity. However, the cut-off trench will extend through the more permeable surficial soils, and it is reasonable to assume that the in-place permeability of the underlying siltstone and caliche will not be high enough to be a problem.

4. A buried stream channel is known to exist in the dam foundation. Others may be discovered during foundation excavation.

5. Ground subsidence and earth fissures occur 1-2 miles south and west of the dam site, however, the dam site should not be effected by these phenomena.

6. The dam site could experience an expected maximum probable intensity of VI-VII on the Modified Mercalli Scale.

For stability analysis, a seismic coefficient of 0.05g would be acceptable and 0.1g should be conservative.

7. Both the principal and emergency spillways are underlain by caliche at shallow depth. The upper 1.5 feet or so of caliche is somewhat soft but caliche below that zone should be competent enough to support the structures without appreciable consolidation.

8. A suitable quantity of borrow material is available at or near the dam site. However, Type 4 material must have no more than 15% fines and it is unlikely that there is very much soil in the designated borrow area that can meet that requirement. Therefore, much of the Type 4 material may have to consist of excavated weathered granite obtained from floodway borrow areas north of the dam site.

9. Only a small amount of soil testing was done on borrow materials, however, the test results are probably fairly representative of the various soils available.

Memorandum of Design Review
Spook Hill Dam (7-50)
Damsite Inspection
By D. R. Lawrence

An inspection of the proposed damsite was conducted on March 8, 1977. John Sullivan of the S.C.S. accompanied me. During the month of March, Messrs. B. G. Scott, W. C. Jenkins, and I drove through the drainage area during the course of other inspections.

Location

The site is located immediately upstream of the proposed Salt-Gila Aqueduct in eastern Maricopa County, approximately 10 miles east-northeast of Mesa, Arizona. It is in Sections 6, 8, and 16 of T1N, R7E of the Gila and Salt River Base and Meridian. The north end of the site is in Section 31, T2N, R7E G&SRB & M. This area is shown on the Buckhorn, Arizona 7.5 USGS Quadrangle map. The dam will act as a flood retardation structure for the normally dry washes of the drainage area.

Proposed Dam

The proposed homogeneous earthfill dam will be a maximum of 15 feet high and approximately 4 miles long. The freeboard will be 9 feet. The borrow will come from excavation of the floodway, the principal spillway inlet channel, and the Salt-Gila Aqueduct. In an attempt to take greatest advantage of those borrow materials, the design calls for routing the borrow as follows: (1) Silt and clayey sand and silts to the upstream half of the embankment, (2) caliches downstream of the centerline and (3) sands, decomposed granite and granites to the downstream toe of the embankment. Both the upstream and downstream slopes will be at 2.5:1. This routing may present contractor-engineer conflicts.

The emergency spillway will be a reinforced concrete drop structure, 260 feet wide and 12 feet high, located near the right abutment. It will be founded on caliche and granite.

The outlet or principal spillway is located just north of the emergency spillway at the right abutment and will consist of a 7 foot by 7.5 foot concrete box conduit with a drop inlet and an SAF outlet. The outlet will be founded on caliche.

Drainage Area

The 16.38 square mile drainage area is gradually sloping desert land. Approximately 2 square miles of the area have been developed for residential purposes. The upper end of the drainage is in the south end of the Usery Mountains. The USGS 7½' Orthophoto Quad., Buckhorn, Arizona has been beneficial in our study of the drainage area. The upper elevation of the watershed is 2673 feet in the Usery Mountains. The elevation at the damsite is approximately 1567. A study of the Orthophoto Quad. indicates that flow from the emergency spillway would be directed into two washes which flow to the Salt River.

Site

The site is located within the Desert Region of the Basin and Range Physiographic Province. It lies primarily upon unconsolidated and semi-consolidated regolith composed primarily of alluvial fan deposition derived from the Usery Mountains and associated igneous activity. The regolith overlies granitic rock.

The S.C.S. Report of Geologic Investigation of the damsite (summary attached) provides study of the site and the construction materials. Mr. G. D. Cox will make a review of this report and of the damsite including seismicity and proximity to ground subsidence due to ground water withdrawal.

Local utility pipelines presently cross the axis of the dam. Some will be abandoned. Those remaining will be encased in concrete under the cutoff trench. Plans are now being finalized.

Construction Materials

As mentioned above, borrow will come from the floodway excavation, principal spillway inlet channel excavation and the excavation for the Salt-Gila Aqueduct. The soil is silty sand (SM) and sandy silt (ML). The sand is fine to course grained. The fines are nonplastic silts. These soils are susceptible to cracking when compacted dry of optimum and upon dessication. Similar desert soils have been used to construct other flood control structures within Maricopa County. Nearly everyone of these structures exhibit problems of transverse and longitudinal cracking. Clayey sand (SC) soils are available between Stations 88 and 200. It has been suggested that the foundation extend to a medium density clayey sand in this area. The specifications call for, 12.a (2) (a) "stripping the foundation to a depth of approximately 1.0 foot of topsoil, roots . . . (b) removal of the remaining unconsolidated materials to depths varying from zero (0) to six (6) feet below existing ground. Approximate depths are shown on the drawings. Final depths will be determined by the Engineer after

examination of the materials encountered." We should receive a statement from the engineer regarding the criteria which he will use to determine final depths of excavation.

A buried stream channel filled with highly permeable silty sand (SM) crosses the dam centerline in the vicinity of Station 223. Other such channels could be revealed by foundation preparation. This would require additional foundation excavation.

## Conclusions and Recommendations

1. The site appears to be acceptable for a dam of this type.

2. The silty sand and sandy silt soils are the soil types most susceptible to erosion and to cracking upon dessication.

3. One buried stream channel (Sta. 223) has been detected. Others may be found as foundation excavation progresses.

4. The emergency spillway will be founded on caliche and granite.

5. Flows from the emergency spillway (from storms in excess of 1/100 frequency) will be into the Salt-Gila Aqueduct and, should it be overtopped, into channels which drain to the Salt River.

6. The outlet will be founded on caliche and will flow into a floodway designed to carry flows to the Salt River.

## References

1. U.S.D.A., Soil Conservation Service, Preliminary Drawings, Spook Hill F.R.S., Jan. 11, 1977.

2. Draft Specifications, Buckhorn Mesa W.P.P., AZ - Spook Hill F.R.S., 12/76.

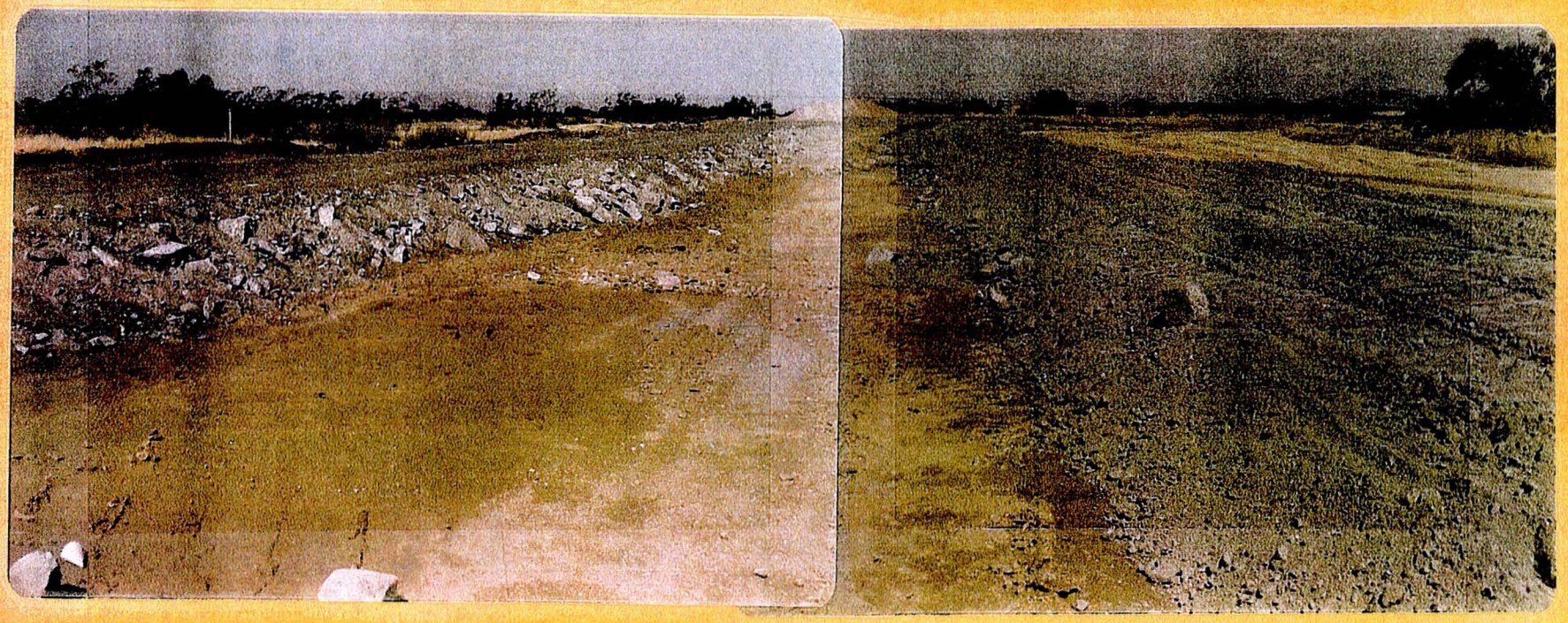
3. Geologic Investigation Report, Buckhorn-Mesa Watershed, Spook Hill Damsite, undated.

4. Design Report, Paul Monville, December 22, 1976.

BJS

SPOOK HILL DAM (7-50)

September 8, 1978 WCD



Type 4 fill at both sides of photo. Oversize rock pushed to center where Type 5 fill is acceptable. Photo from Sta. 205 looking up the project (upstream to right).

SPOOK HILL DAM (7-50)
August 28, 1978 WD

Type 2 fill placement over deep cuts at Sta. 223-224.

Photo taken at approx. Sta. 228 looking up the project. Shows excavation of unacceptable Type 2 material which had been placed from Sta. 228-248.

# FAILURE MODE AND CONSEQUENCE ANALYSIS
# For
# SIGNAL BUTTE FLOOD RETARDING STRUCTURE

## PINAL COUNTY, ARIZONA
## NATDAMID AZ 200205

**July 20 2001**

## 1.0 INTRODUCTION

### General Description

Signal Butte FRS is located within the City of Mesa. The FRS begins west of Meridian Road and north of Brown Road. The FRS is about 28 miles east of downtown Phoenix and approximately 3 miles west of the town of Apache Junction. The project consists of the FRS structure, principal spillway and an emergency spillway. The project is part of the Buckhorn Mesa Watershed Protection and Flood Prevention Project, which includes the Spook Hill and Apache Junction flood retarding structures. The Flood Prevention Project was prepared, designed, and constructed by the U.S. Department of Agriculture, Natural Resources Conservation Service (NRCS).

Signal Butte FRS is classified as a high hazard, small dam. The reservoir behind the FRS is 140 acres with a capacity of 1540 acre-feet. Construction of the Signal Butte FRS was accomplished under contract to Pulice Construction Company. The completion date of the construction of the dam and landscape treatment is January 1987.

The dam has performed satisfactorily to date.

### Dam Data

Dam type: Rolled earthfill
Dam height: 38.5-ft
Dam length: 7,022-ft
Dam crest: width 18-ft; elevation 1721.0-ft
Spillways: Principal - 36-in RCP; inlet elevation 1701-ft; 1701 = top of sediment pool
Emergency spillway - 140-ft wide RC baffle block spillway; crest elevation 1712.4-ft
Freeboard: 4.8-ft
Reservoir Surface: 140 ac at spillway crest
Storage: 1,540-af at spillway crest
Hazard Classification: High

**Purpose and Scope**

In general, the purpose of the Failure Mode and Consequence Analysis (FMCA) exercise was to:

- Identify potential site-specific failure modes for the dam.
- Discuss non-quantitatively the likelihood of the identified failure modes.
- Determine whether or not and how important failure mechanisms are being monitored
- Examine the consequences of failure and the adverse consequences of successful operation (e.g. – large spillway releases)
- Identify possible risk reduction actions that may be taken to reduce the likelihood of failure or to mitigate adverse consequences
- Determine what information, investigations or analyses may be needed to resolve uncertainties relative to potential failure modes.

(Note: In this phase, the FMCA team only examined the general nature of the "consequences" for the failure modes identified and where appropriate estimated how they may be different than previously anticipated. Greater detail on the estimate of the magnitude of the "consequences" for each significant failure mode will be addressed in the quantitative portion or risk analysis part of the risk assessment for the dam. At this time hopefully more detailed discussion on the emergency response plan for each structure will also be possible.)

**Team Members**

Larry Von Thun, Engineering Consultant and FMCA facilitator
Bob Eichinger, Kimley-Horn and Associates, Inc., Project Manager
Jim Scott, URS Corporation, Principal
Ken Euge, Geological Consultants, Principal
Tom Renckly, Flood Control District of Maricopa County, Project Manager,
Larry Lambert, Flood Control District of Maricopa County, Dam Safety Engineer
George Beckwith, Flood Control District of Maricopa County, Geotechnical Engineer
Justin Beeler, Kimley-Horn and Associates, Inc, Session Recorder
Tim Murphy, Flood Control District of Maricopa County, Civil Engineer (consequences)

## 2.0 MAJOR FINDINGS AND UNDERSTANDINGS

The following is a summary of the major findings and understandings for Signal Butte Flood Retarding Structure as a result of the Failure Mode and Consequence Analysis (FMCA). Only one major adverse consequence was highlighted and that was discharges from the emergency spillway due to major flooding up to the PMF. However, the other potential failure mode, considered to great extent on this and the other related structures in the region was a seepage erosion failure due to cracking from drying shrinkage or other causes. In the case of Signal Butte, the presence of the HDPE central liner was a key aspect of the discussions. Thus these major findings and understandings are organized first according to these two issues and then according to other failure modes considered, consequence aspects (not already mentioned) and risk reduction or investigative or preparatory actions to be taken.

## Flood Related Findings and Understandings

- The PMF Does Not Overtop The Dam. The FMCA team considered that because there are no major factors identified in the region which are changing current flood estimations (e.g., land use, density) that there is no strong need to reevaluate the PMF at this time. The available surveys for the dam show that the dam crest is relatively level except the dam is 1-foot higher in the vicinity of the principal outlet. The stage of the PMF pool is 1720-feet and the top of dam crest is 1722-feet (freeboard of 2-feet).
- Only One Highlighted "Failure Mode". Emergency spillway discharges resulting from large rainfall events is the only highlighted failure mode (resulting in adverse consequence). It appears that by far the greatest risk (product of likelihood of occurrence with magnitude of consequence) lies in "Normal (no dam failure) Spillway Discharge Operations".
- Possibility of Expanding Outlet Capacity. By opening the T-riser at its base the outlet capacity can be increased and the drawdown can begin sooner during a flood event to the reservoir. This could be done inexpensively and is feasible. There would be a need to consider downstream impacts and requirements would need to be evaluated if the outlet capacity is increased. By creating an opening at the base of the outlet a gain of approximately 11-ft in head is achieved. The reservoir impoundment could be evacuated in shorter duration that existing conditions. (The invert of the outlet pipe is at elevation 1690 feet while the current inlet to this pipe is at elevation 1701 feet.)
- Contributing Flow to the Signal Butte Reservoir is from Two Basins. The PMF design flood comes from 2 different contributing watershed basins.
- Sediment Pool Not Used In Routings. The available storage in the sediment pool was apparently not used in any hydrologic routings. By accounting for the sediment pool and by increasing the outlet capacity as noted above the routings should indicate that the threshold of damage flood has a greater frequency and that the PMF has greater freeboard. Note that this is true of Spook Hill FRS and the other dams as well.
- Upstream Easements To PMF – The District has easements upstream of the dam that includes the impoundment area for the full PMF facilitating management of the risks associated with filling of the reservoir. The upstream easements should be verified. It is also anticipated that development will occur slowly above the reservoir. Future land use upstream of dam will likely be the same as existing development because of the regional park.
- The Critical Flooding Events For These Detention Dams Are More Likely To Be A Result Of Multiple Winter Storms Than A Single Very Large Flood (i.e., PMF). However, the reservoir at this site can be emptied in less than 5 days making the multiple storm less critical for Signal Butte FRS than for other dam sites (e.g., Rittenhouse FRS is designed for a 30 day drawdown). The length of time of significant reservoir impoundment is an important factor in evaluating the probability of most failure modes. Multiple winter storms may result in longer reservoir loading times. A series of winter storms in close sequence would probably result in impoundment of water in the reservoir for a longer duration than a single storm approaching the PMF. Thus, a scenario of multiple storms is an important

consideration in evaluation whether seepage erosion processes could advance to the point of initiation of a dam breach.

**Findings Related to Drying Shrinkage Crack Induced Failure Mode and HDPE**

- Favorable Embankment Properties. With regard to shrinkage and cracking the soil tests on the materials in the Signal Butte FRS borrow showed it to be more granular, with significantly less fines and less plasticity than the previous Flood Retention Structures reviewed in this series of FMCAs. Thus this soil should be much less likely to experience significant drying shrinkage cracks. However, the pervious nature of the granular fill allows infiltration from surface moisture and thus could likely undergo cementation. Available tests indicate that the dam is constructed primarily with sands with non-plastic to low plasticity fines. The embankment appears to have a low potential for drying shrinkage and is likely cemented with calcium carbonate. The field density tests performed by the SCS for quality control during the original construction indicate a mean compaction of the embankment of 101.1 percent of ASTM D698 maximum dry density and a mean compaction moisture content of 0.02 percent below the optimum moisture content. Statistical analysis indicated standard deviations of 3.6 for percent compaction and 1.6 for variation from the optimum moisture content. This is indicative of a relatively high quality embankment.
- Transverse and Longitudinal Cracking. No significant cracking has been observed in the Phase I Structures Assessment or documented in any of the inspection reports or documents. However, subsequent to this report, a hairline longitudinal crack near the centerline of the dam crest was found between Stations 255+39 to 256+24 during an inspection on December 3, 2001.
- There Are Really Two Lines Of Defense Against Desiccation Cracking Related Failure At Signal Butte FRS. 1) Owing to the low shrinkage potential of the embankment soils and the essentially incompressible foundation, the risk of significant cracking of the embankment appears to be low. 2) The HDPE hydraulic barrier provides a defensive design against flow through cracks and seepage-erosion. Given these two lines of defense, the probability of a seepage erosion failure through the embankment is very low.
- HDPE To Full Depth Of Dam/Cutoff. A High-Density Polyethylene (HDPE) geomembrane extends from below the crest of the dam to the full depth of the cutoff trench and is connected to a concrete anchor block, which rests on the strongly cemented calcareous soil (caliche). Brainstorming of potential or expected performance characteristics of the HDPE led to the following points:
  1. The technical literature indicates that HDPE will not significantly deteriorate for 200 -300 years in most environments. This should be verified.
  2. The need to do periodic testing of HDPE to confirm that no deterioration has occurred should be evaluated.
  3. HDPE has a high degree of integrity and capacity to withstand large deformations.
  4. HDPE behaves elastically to a yield point at about 10 percent tensile strain. Special tests done for the McMicken Dam Restoration and the Combustion Waste Disposal Ponds at the Apache Generating Station at Cochise, Arizona indicated

that HDPE will safely bridge cracks up to about 3 inches for height of dam involved.

5. The thickness of the HDPE (100 mil) is conservative for the height of the dam, relatively difficult to construct a hydraulic barrier with this stiff material, and adds no support to dam, but does reduce chance for a puncture or a tear in the material.
6. If leakage develops, remediation could be a toe drain (drain at downstream toe would act as third line of defense).
7. No viable failure mode was identified for the dam involving malfunction of the HDPE material as a cutoff for flow through potential cracks in the dam.
8. Extremely low probability of a transverse crack lining up with a defect in the HDPE hydraulic barrier and creating significant seepage.

- Presence of HDPE Membrane. Despite concerns about the use of geomembrane material as internal hydraulic barriers in dams, it is judged that the Signal Butte FRS has the lowest risk of failure of any of the seven dams under assessment. This is due to the HDPE geomembrane extending to the caliche foundation. If the other six dams had this feature, it would have greatly reduced the estimated risks of failure.

**Findings related to all other Potential Failure Modes Considered**

- Excavation of Cutoff. The cutoff excavation extended through the collapsing Holocene surface layer to the strongly cemented soils. This is well documented by the as-built plans, ADWR inspection reports and construction photos. ADWR performed inspection and approval of each portion of the foundation.
- Dam Geometry. Dam crest appears to be at design grade and top width is greater than NRCS standard design. Therefore an alternative to reduce downstream risks associated with emergency spillway discharges by raising the emergency spillway crest elevation and raising the dam crest elevation (by addition of material to the crest of the exiting dam) may be feasible.
- Piping along outside of Principal Outlet is a Low Potential Failure Mode. Principal outlet has filter diaphragm therefore, a seepage erosion failure along the outlet pipe is not a credible failure mode.
- Seismic Issues Not A Concern. Earthquakes are not a significant risk. A Seismic Exposure Evaluation by AMEC, which is in progress, indicates that the peak horizontal acceleration (PHA) created by the MCE is only about 0.12g
- Failure Mode Potential along Foundation Reduced. Removal of the highly erodible surface layer of Holocene soils beneath the embankment effectively eliminates the risk of a seepage erosion failure along embankment foundation interface. Cutoff to the strongly cemented soils was approved and documented by ADWR
- Emergency Spillway Erosion is a Low Potential Failure Mode. Caliche (strongly cemented calcareous soils) are exposed on the sides and bottom of the emergency spillway. Therefore, erosion of the spillway was judged not to be a credible. The spillway is a baffle block dissipator.
- Utilities. No documented utilities through the structure. No utilities through the dam. The FMCA team recommends that a District policy be established that does not allow utilities to be constructed through the dam.

- Lack of Evidence of Subsidence and Fissures. The dam appears to be underlain by shallow bedrock throughout. Thus, the weight of evidence indicates that there is not a significant risk of land subsidence and earth fissuring and is not considered a failure mode.
- Reservoir Drawdown Time is Short. The time water is within the pool is small. The drawdown times are 4.8 days from emergency spillway crest to principal spillway inlet and just a few hours from maximum water level to the emergency spillway crest. Thus there is an extremely small likelihood of the reservoir being higher than spillway crest for any "static or seepage through the dam" related failure mode to develop (reduces that probability of seepage erosion failure modes). Also the reservoir will be emptied quickly in the event of a follow up storm, thus reducing the risk from multiple storm events.
- Transverse Cracking at the Contact between the Dam and Spillway. Available evidence indicates that the embankment is composed of sand and fine gravel with non-plastic or slightly plastic fines and rests on essentially incompressible strongly cemented calcareous soils (caliche). The risk of subsidence and earth fissuring is very low. Thus, the potential for transverse cracks in the dam and foundation is judged low. Transverse cracks developing at spillway wall is not a failure mode due to the extensive wing wall protection that extends 27-feet into the embankment fill.

**Actions to be taken for Risk Reduction or Investigation for Risk Assessment**

- No Individual EAP. An individual Emergency Action Plan needs to be developed according to ADWR and FEMA 64 criteria.
- As-Builts Needed For Cutoff Wall Below Emergency Spillway. Documentation states that during construction a cutoff wall was constructed below the emergency spillway. This documentation is in the ADWR construction inspection reports. However, the cutoff wall is not reflected on the As-built Plans.
- Action: Need construction Quality Assurance test results on HDPE lining.
- Crack Monitoring. Monitor for development of longitudinal cracks and be aware of future cracks. (Note: the as-built drawings show that the Holocene soils were removed from beneath the cutoff trench over the entire length of the dam. However, longitudinal cracks could develop due to collapsing Holocene soils beneath the upstream slope. Cracking is not mentioned in any of the Annual Inspection Reports in the Data Book which date back to 1990)
- Slope Stability. Slope stability analysis needs to be conducted for record conditions (slope stability not identified as a failure mode).
- Field Inspections. Locations for special inspection for settlement cracks should be identified. These are the areas of localized deep sub-excavation of the Holocene soils to the contact of the strongly cemented calcareous soils.
- Staff Gage. It was observed by the FMCA team that the principal outlet does not have a staff gage, but the gated outlet does.

**Findings related to Potential Consequences**

- Emergency Spillway Discharge Downstream Inundation Area. Downstream inundation area is highly developed with a seasonally variable population. The high rate of urban development and variable population downstream because of influx of seasonal residents in the late fall, winter and spring effect the consequences of downstream inundation.
- Central Arizona Project Canal. CAP canal and overchutes have the capacity to handle approximately up to 4500-cfs of discharge from a dam breach. It was noted by the FMCA that diversion of floodwaters into the CAP under large flood events is part of the CAP Reach 11 dikes design.
- The existence of a higher dam at Signal Butte results in a significantly greater estimated discharge due to potential dam failure than computed for the other flood retention dams in this series. Interestingly, no highlighted failure mode was identified so the likelihood of a dam failure appears less.

## 3.0 POTENTIAL FAILURE MODES

Potential failure modes identified by the FMCA team are presented below. The failure modes were placed into one of four categories listed below. Except under special circumstances only those potential failure modes in Category I will be considered in the risk analysis phase of the risk assessment exercise.

- Category I – Failure modes of greatest significance.
- Category II – Failure modes of lesser significance (but not inconsequential).
- Category III – Failure modes for which insufficient information is presently available to allow the team to make a judgement on the significance of the failure mode. The development of additional data and information is warranted. Additional records research from sponsoring agencies may be justified.
- Category IV – Failure modes which are not physically possible or which are clearly not credible.

For each of the potential failure modes identified, a Failure Mode Description is briefly described and the factors that make the failure mode more likely (adverse factors) or less likely (positive factors) to occur at listed following the Failure Mode. In addition, any identified actions for risk reduction or investigation for the potential failure modes are also provided.

## CATEGORY I - FAILURE MODES OF GREATEST SIGNIFICANCE

***Adverse Consequences Resulting From Emergency Spillway Discharges During Major Rainfall Events (Category I).***

Failure Mode Description: The Signal Butte FRS emergency spillway is a 140 foot wide baffle-block reinforced concrete spillway. Flood discharges from the spillway are directed into the City of Apache Junction. This potential "failure mode" does not "fail" the dam or emergency spillway.

This potential failure mode was rated as a Category I failure mode because normal "successful" operations of the emergency spillway can produce discharges that could have significant adverse consequences and the likelihood of occurrence of these adverse consequences appears to be much greater than any mode involving dam failure. Most of the floodwater will pass through the emergency spillway. From that point, the water will travel through developed neighborhoods until it reaches the CAP canal where it will either pond against, flow into and be carried by the canal or overtop the canal into another housing development.

Adverse Factors:

(1) Spillway flows are directed into the City of Apache Junction. Downstream population at risk potentially around 10,000 people.
(2) No dedicated downstream easements from spillway.
(3) Time to peak for PMF is 11.4 hours (see Signal Butte FRS Appendix).
(4) No individual emergency action plan.
(5) No regulation on downstream spillway discharge areas.
(6) Emergency spillway channel is not lined.

Positive Factors:

(1) Dam is instrumented as part of the District Alert System. Would provide warning.
(2) Spillway discharge exit is normal operation and straight forward.
(3) Short spillway discharge duration for PMF.
(4) The CAP canal is located well downstream from dam – picks up approximately 4,500 cfs of flow (see Signal Butte FRS Appendix – "Effect of CAP canal on Emergency Spillway and Breach Hydrograph).
(5) Reinforced concrete baffle block spillway – good erosion control.
(6) Caliche near surface in spillway channel.
(7) Lower flow velocities (velocity/depth product appears lower).

Potential Actions for Risk Reduction

(1) Prepare emergency action plan according to minimum standards in FEMA 64 and ADWR rules and regulations.
(2) Use lower level outlets at Signal Butte.
(3) Increase outlet capacity to Spook Hill FRS.

## CATEGORY II – FAILURE MODES OF LESSER SIGNIFICANCE (BUT NOT INCONSEQUENTIAL)

***Failure Due To Seepage Erosion Along A Transverse Crack Through Embankment Which Ruptures Or Encounters A Defect In The HDPE Liner Leading To Failure) (Category II – Considered but not highlighted).***

(Note – Although the team classified this failure mode as Category II it will be included in the Quantitative Risk Analysis if time allows to allow comparison with the other dams that do not have an HDPE liner.)

Failure Mode Description: Potential for impounded water to infiltrate and flow into a relatively wide transverse crack(s) in embankment causing seepage erosion and breach development through lateral erosion of the side walls. (Note this is the same process as breach widening after initial breach development in an overtopping type failure). Two modes were identified for failure: Mode A - The flow path for seepage erosion is through a major large wide transverse crack carrying water to the upstream face of the high density polyethylene (HDPE) membrane and then passing through the HDPE via a defect in the HDPE membrane (caused by rupture or other defect) and continuing seepage erosion in the downstream portion of the crack. If the downstream crack is not directly aligned then water would be required to flow longitudinally until water finds a downstream transverse crack initiating seepage erosion and a breach failure; Mode B –As for Mode A except that the upstream water source is through intense multiple transverse cracks. The flow path continues via a defect in the HDPE carrying water downstream to downstream transverse cracks initiating seepage erosion along the crack and a breach failure. Generally, the mechanics of the potential failure need to be linearized such that there is an upstream transverse crack(s) lined-up with a defect in the HDPE membrane, which in turn is lined-up with a downstream transverse crack(s).

Adverse Factors:

(1) Potential problems with installation of HDPE membrane – compaction on either side of the membrane and the construction technique of flipping the membrane as it was built vertically.
(2) No major inundation – There is approximately 11 feet of head between the emergency spillway crest ant the invert of the principal outlet (which is also the top of the sediment pool).
(3) No detailed control of welding of seams (documentation not evident).
(4) Finding and repairing HDPE tears or defects difficult.
(5) Penetrations due to vegetation, animals, utilities possible but not likely.

Positive Factors:

(1) No cracks of significance documented.
(2) HDPE properties very elastic (8 –10%) – tests show success with 3-inch crack.
(3) Studies show 200 to 300 year lifespan.
(4) Wider dam crest – foundation conditions are better.
(5) No significant impoundment history.
(6) Short time of impoundment – affects saturation and failure mode development.
(7) Thickness of HDPE is very conservative (100mm).
(8) Sandy soils is first line of defense, HDPE is second.
(9) Crack(s) would have to line up with rupture or tear in HDPE.
(10) Soils in Signal Butte FRS borrow area less susceptible to shrinkage cracking.
(11) Fewer fines, low P.I.'s, only 22% SC and CL and the inspection records show that the contractor and design engineer were aware of the need to mix. The more clayey materials with the more granular if they were encountered so as to avoid a concentrated location of materials susceptible to drying shrinkage in the fill.
(12) Seepage path is very long.

(13) Dry climate and environment.
(14) Better foundation conditions than Powerline, Vineyard Road, and Rittenhouse FRSs.

Potential Actions for Risk Reduction:
(1) Monitor cracks and develop crack mapping program.
(2) Train O&M crews to on mechanism of transverse and longitudinal crack formation.
(3) Repair cracks at dam.
(4) Locate seam welding QC results.
(5) Could protect any penetrations into the dam with filter material downstream of the penetration.
(6) A similar type of repair may be acceptable for repairs at other similar dams.

## CATEGORY III– FAILURE MODES FOR WHICH INSUFFICIENT INFORMATION IS PRESENTLY AVAILABLE FOR MAKING ENGINEERING JUDGEMENTS

No Category III potential failure modes were identified by the FMCA team for Rittenhouse FRS at this time.

## CATEGORY IV – FAILURE MODES WHICH ARE NOT PHYSICALLY POSSIBLE OR WHICH ARE NOT CLEARLY CREDIBLE

***Failure From Overtopping The Dam From Extreme Flood Events (Category IV).***

(Note: This failure mode was not considered because the PMF does not overtop the dam. The calculated freeboard is 2-feet. The spillway and low level outlets pass flows through the dam before a significant amount of water can built up to overtop the dam. The crest of the dam is fairly wide which would allow the height of the dam to be raised if needed.)
Failure Mode Description: Dam failure due to flood overtopping the dam crest.

Adverse Factors:
(1) The abutments taper to elevation 1721.0-feet. The top of dam crest from Station 211+00 to Station 248+00 is 1722.0-feet.

Positive Factors:
(1) The PMF does not overtop the dam.
(2) 24-hour PMF used in analysis.
(3) Contributing drainage area is from two watersheds.
(4) Sedimentation pool is assumed full of sediment, which it is not.
(5) Time for draining pool is very short so multiple [back to back] storms are not as critical as for some of the other structures.

Potential Actions for Risk Reduction/Investigation:

(1) Lower outlet opening.
(2) Sediment pool assumed to be full.
(3) Maintenance activities are good.
(4) Evaluate routing with sediment pool available.

**Other Category IV Considerations:** These issues were discussed by the FMCA team but a potential failure mode was not identified for evaluation (and descriptions of adverse and positive factors were not developed).

- **Upstream impacts**: (potential for development minimal due to national forest and local park) and District easements to PMF. Little potential for changing hydrology.
- **Slope stability**: Existing studies show a Factor of Safety of 1.5 but the cases run were not ideal, therefore, it is recommended to run basic stability studies as has been recommended for the other dams in this series of FMCAs. Downstream slope is 2:1, good foundation conditions, no problems observed or expected)
- **Settlement, subsidence, and fissuring**: Not considered a problem at this site (yet still needs to be documented and verified). Based on geology (strain data) and no observations of problems.
- **Excavation for Holocene soils**: At Station 212+00 – 200 feet wide and at Station 229+00 for 100 foot wide. Potential for longitudinal cracks due to settlement after saturation. No failure mode identified.
- **Potential erosion along foundation contact**: Holocene soils cut off by cutoff trench. Requires transverse crack in foundation material to be a significant failure mode due to short loading time. Therefore, no failure mode identified. However the location of the Holocene soils should be noted so that inspections can examine for any evidence of a developing condition.
- **Piping around outlet pipes**: not identified as a failure mode due to filter diaphragm. Principal outlet cradled. The vegetation outlet is encased in concrete. Therefore, no failure mode was identified.
- **Emergency spillway**: Transverse crack at emergency spillway not an issue due to wing walls. Structural separation on wall – repair if structural integrity is questionable. Erosion is minimal due to caliche and cutoff wall.
- **Seismicity**: No unique concerns at this site (same as Apache Junction FRS).
- **Piping Dam Breach**: Estimated at 24,000 cfs, however, HDPE liner would have a retarding effect. Also loss of reservoir with time of breach development would reduce peak outflow.
- **Pass Mountain Diversion Dike**: Evaluate effect of failure of dike. Currently it is a 100-year detention facility – same as floodway.

## 4.0 SUMMARY AND CONCLUSIONS

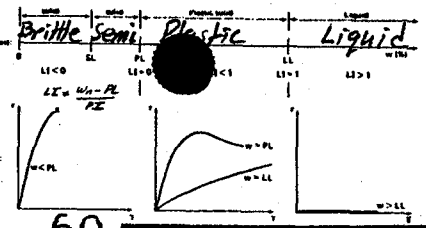
Signal Butte FRS was constructed pursuant to a modern dam design. Construction appears to have been without any particular issues. The dam has performed normally and satisfactorily for 14 years. The structure is satisfactorily maintained and monitored.

However, it is prudent to recognize that there exist for all dams specific ways that failure could come about that warrant attention and diligent monitoring. The identification of a condition or process as a "potential failure mode" does not imply that the dam is about to fail or even necessarily that there is a dam safety deficiency at the site. Rather it identifies physically possible conditions or processes (generally with a remote but still credible chance of occurrence) that persons associated with owning, inspecting, analyzing and operating the dam should be aware. Some of the potential failure modes are highlighted (or prioritized) for attention of the dam owners and operators. They are highlighted because the specific conditions at the dam and appurtenant structures are such that these failure modes are physically possible and are considered the most realistic and most credible potential failure modes definable at the site.

Only one potential Category I failure mode was identified by the FMCA team. This failure mode is associated with normal emergency spillway operations. The spillway is directed to discharge flows into the City of Apache Junction. There is a large number of people and structures at risk in the flood path in the event of a spillway discharge or dam failure by breach. There could be a likely chance that casualties could occur due to a dambreak or spillway discharge. An immediate risk reduction measure is to develop an individual emergency action plan for the Signal Butte FRS.

## SIGNAL BUTTE FRS APPENDIX - FMCA REFERENCE MATERIALS

Signal Butte FRS and emergency spillway.
Photo date: October 2001 Photograph Copyright Kimley-Horn and Associates, Inc.




| Characteristic | Soils at Equal LL with Increasing PI | Soils at Equal PI with Increasing LL |
|---|---|---|
| Dry strength | Increases | Decreases |
| Toughness near PL | Increases | Decreases |
| Permeability | Decreases | Increases |
| Compressibility | About the same | Increases |
| Rate of volume change | Decreases | — |

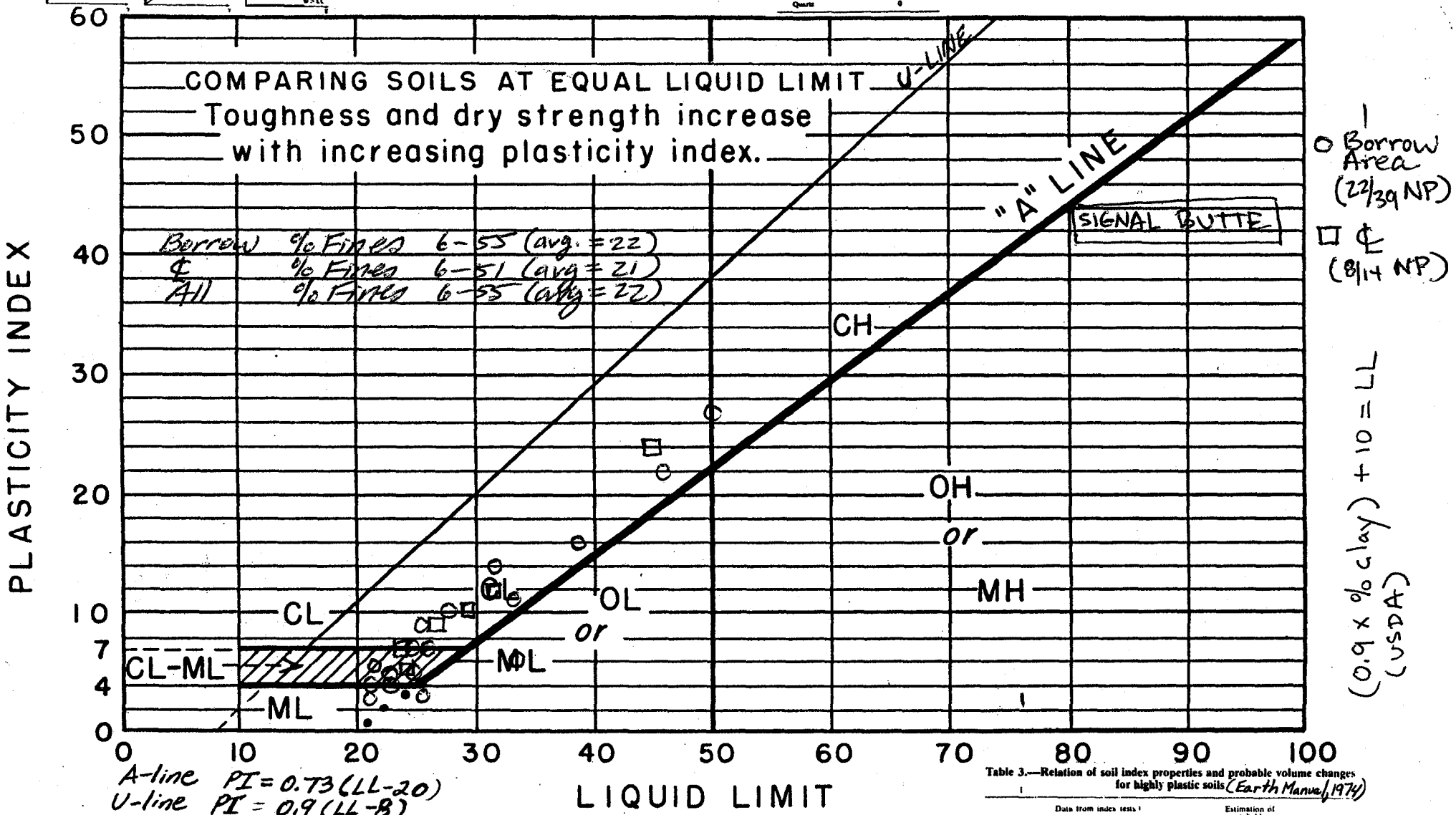
TABLE 4-1 Activities of Various Minerals*

| Mineral | Activity |
|---|---|
| Na-montmorillonite | 4–7 |
| Ca-montmorillonite | 1.5 |
| Illite | 0.5–1.3 |
| Kaolinite | 0.3–0.5 |
| Halloysite (dehydrated) | 0.5 |
| Halloysite (hydrated) | 0.1 |
| Attapulgite | 0.5–1.2 |
| Allophane | 0.5–1.2 |
| Mica (muscovite) | 0.2 |
| Calcite | 0.2 |
| Quartz | 0 |

$A = \frac{PI}{\% < 2\mu m}$

Table 4-1. WES Classification of Potential Swell

| Classification of potential swell | Potential swell $S_p$ percent | Liquid limit LL percent | Plasticity index PI percent | Natural soil suction $\tau_{nat}$ taf |
|---|---|---|---|---|
| Low | <0.5 | <50 | <25 | <1.5 |
| Marginal | 0.5–1.5 | 50–60 | 25–35 | 1.5–4.0 |
| High | >1.5 | >60 | >35 | >4.0 |




# PLASTICITY CHART

Table 4–. Data for making estimates of probable volume changes for expansive soils (Chen 1975)

| Laboratory and field data | | | Probable expansion, percent total volume change | Swelling pressure, ksf | Degree of expansion |
|---|---|---|---|---|---|
| Percentage passing No. 200 sieve | Liquid limit, percent | Standard penetration resistance, blows/ft | | | |
| >35 | >60 | >30 | >10 | >20 | Very high |
| 60-95 | 40-60 | 20-30 | 3-10 | 5-20 | High |
| 30-60 | 30-40 | 10-20 | 1-5 | 3-5 | Medium |
| <30 | <30 | <30 | < 1 | 1 | low |

TABLE 3:3 / VOLUME CHANGE POTENTIAL
(Adapted from Holtz and Gibbs[3,10]) (1956)

| Volume Change | Shrinkage Limit | Plasticity Index |
|---|---|---|
| Probably low | 12 or more | 0–15 |
| Probably moderate | 10–12 | 15–30 |
| Probably high | 0–10 | 30 or more |

Table 3.—Relation of soil index properties and probable volume changes for highly plastic soils (Earth Manual, 1974)

| Data from index tests [1] | | | Estimation of probable expansion,[2] percent (total volume change (dry to saturated condition) | Degree of expansion |
|---|---|---|---|---|
| Colloid content (percent minus 0.001 mm.) | Plasticity index | Shrinkage limit, percent | | |
| >28 | >35 | <11 | >30 | Very high. |
| 20–31 | 25–41 | 7–12 | 20–30 | High. |
| 13–23 | 15–28 | 10–16 | 10–20 | Medium. |
| <15 | <18 | >15 | <10 | Low. |

[1] All three index tests should be considered in estimating expansive properties.
[2] Based on a vertical loading of 1.0 p.s.i. as for concrete canal lining. For higher loadings the amount of expansion is reduced, depending on the load and on the clay characteristics.

## FOUNDATION TREATMENT

Discussions with Paul Pedone and Aubrey Sanders, SCS Geologists, familiar with the Signal Butte FRS foundation, confirm that the "loose material" in the foundation is made up of coarse soil materials and is shallow, except for one buried remnant of a channel. They agree that normal construction operations would be expected to consolidate these materials. In-place densities recorded in the geology report are 103.5 and 108.2 pounds per cubic foot in SM materials at 2-foot and 1-foot depth, respectively. These appear to be the equivalent of, respectively, 83% and 87% Standard Proctor at those sites and depths. The design minimum density for the embankment is 112 pounds per cubic foot, roughly 90% Standard Proctor for most SM materials. These soils average 27% fines and 13% gravel, judging from the samples classified in the SML at Lincoln, NE. These facts indicate that materials under the embankment are not collapse-prone and will be consolidated by the construction operations. Removal of the foundation beyond the cutoff trench excavation is unnecessary.

## EMBANKMENT DESIGN

One of the difficulties anticipated with borrow materials that have variable amounts of carbonates and other salts in them (caliche) is that moisture content test results may be misleading. There are a lot of variables involved in the workability of caliche as far as moisture content is concerned. Some of the materials hydrate. Therefore, the time between addition of water to the borrow materials and actual placement in the embankment can be more important to workability than the actual quantity of moisture in the soil. Because of this characateristic of caliche material, the specifications call for a workable mix, i.e. any moisture that allows for blending of the embankment soils into a "homogenous mass without laminations".

Seventy-eight percent of the soils on the Signal Butte FRS site were classified SM or SW-SM and fifteen percent SC or SC-SM by the Lincoln Soil Mechanics Laboratory (SML). The borrow is part of an alluvial fan that has been cemented by calcium carbonate. In fact, caliche is actually at the surface of a section of the borrow area. Any cohesive materials are expected to be mixed by normal operations prior to placement on the fill or during the compaction procedures because they are very limited in volume and found in discontinuous lenses.

Pass Mountain Outlet will contribute approximately 100,000 cubic yards of excavated material to be used in the dam. All of the samples from that site were classified by the SML as coarse-grained, and more than 70% were non-plastic (21% were SC-SM, the most plastic samples tested). Again, there is no source of plastic materials that would not be mixed with coarser or less plastic materials before placement in the embankment.

All the caliche tested was classified SM in the laboratory. The compacted dry density of this material varied from 121.5 pcf (sample 276.1, no longer part of Signal Butte site) to 108.5 pcf (sample 141.1, borrow area) for 100% S.P.D. (The Modified Standard Proctor density for sample 141.1 was 116.0 pcf.)

It is interesting to note that these soils are well-graded and the "clean" samples would make excellent "drainfill" material.

201 E. Indianola Ave.
Suite 200
Phoenix, AZ 85012

File

ENG-Signal Butte FRS, Buckhorn Mesa WPP

November 14, 1984

Jack Stevenson, Head Engineering Staff, WNTC

210 - 13

We are submitting our response to the issues raised in your memo of October 19th, 1984 for the Signal Butte FRS. Susanne Leckband has prepared the requested cost analysis and description of method of installation. We have delayed the drafting of the method of installation on the construction drawings until your review is completed.

In our opinion the high density polyethylene geomembrane will provide the required crack control and is the best alternative because of the following:

1. A continuous impermeable barrier prevents seepage through the earth embankment.

2. HDPE has the highest extensibility (800%) of any geomembrane we know. It will, therefore, conform to any differential movement of the dam or its foundation.

3. The installation in the Vineyard Road Crack Study gave us an opportunity to observe the material and we are confident of its installed properties.

4. Contractors interviewed about installation procedures indicated that it could be successfully installed, but would require additional effort.

5. HDPE material is chemically inert, so deterioration or chemical reaction to water-bearing substances will not be a factor.

6. Simple weld procedures allow easy repair of punctures, tears, etc., should any occur during installation.

7. We have shown a favorable cost comparision with past installed crack control measures.

We can respond to the need for a table top review in Portland should one be favored.

Please call if you have further questions.

Ralph M. Arrington
State Conservation Engineer

cc: Wayne Killgore
Joe Knisley w/o attachments
Steve Revie w/o attachments

RMA:nn

United States Department of Agriculture

Soil Conservation Service

West National Technical Center
511 N. W. Broadway, Room 514
Portland, Oregon 97209-3489

Subject: ENG - Signal Butte FRS, Buckhorn-Mesa Watershed, Arizona

Date: October 19, 1984

To: Ralph Arrington, State Conservation Engineer, SCS, Phoenix, Arizona

File Code: 210-13-22

Review of the subject job is in progress. As discussed by phone with Bill Payne and noted below, we have identified several items that require clarification or further input from your staff. An early response will help in meeting the scheduled review completion date.

1. The documentation includes very limited information on geomembranes other than the designer's thoughts. Several papers from the "International Conference on Geomembranes" (reference Susanne Leckband's letter to you dated June 26, 1984) appear of interest. We have asked Don Paulus to send copies and anticipate receiving them this week. done

2. The design concept of using high density polyethylene (HDPE) for crack control has merit. However, a case for use of HDPE over other conventional methods is not presented in the supporting documentation. As a minimum, we recommend a cost analysis comparing the proposed design with conventional internal drainage systems. The cost analysis for HDPE must be complete and account for such items as increases or decreases in fill, excavation, hauling distances, equipment stand by time, changes in compaction and maximum rock size allowed in fill materials next to the liner and a support or handling system for the membrane during construction.

3. The drawings show vertical placement of the HDPE membrane. We are concerned with construction problems that may develop. Also, to provide the cost data in item 2 above a method of construction will need to be developed. We recommend a proposed construction method be included in the drawings and specifications with allowance for other methods acceptable to the engineer. Value engineering using the incentive approach might also be a consideration since it is an allowable option per FAR clause 52.248-3 and can be included if considered beneficial as determined by the contracting officer.

4. Page 7 of the Design Report includes a paragraph regarding antiseep collars with reference to a request for deviation. We could not find a copy of this request or response in our files or in the supporting documentation as indicated. Also TR-60 has been revised (TR Notice 60c, April 6, 1984) changing the requirement for seepage control along conduits from antiseep

if HDPE OK, no need for either.

collars to drain filters. A narrative statement regarding the timing of design and criteria used would be appropriate in the design report.

If there are any questions on the above, please call Leland Saele.

H H Middlecamp
ACTG JACK C. STEVENSON
Head, Engineering Staff

cc:
Verne Bathurst, State Conservationist,
SCS, Phoenix, Arizona
Donald E. Wallin, Head, Design Unit, WNTC

② RAC.
9/2000

Hydrologic Investigation
Emergency Action Plan
Signal Butte FRS
Buckhorn-Mesa W/S

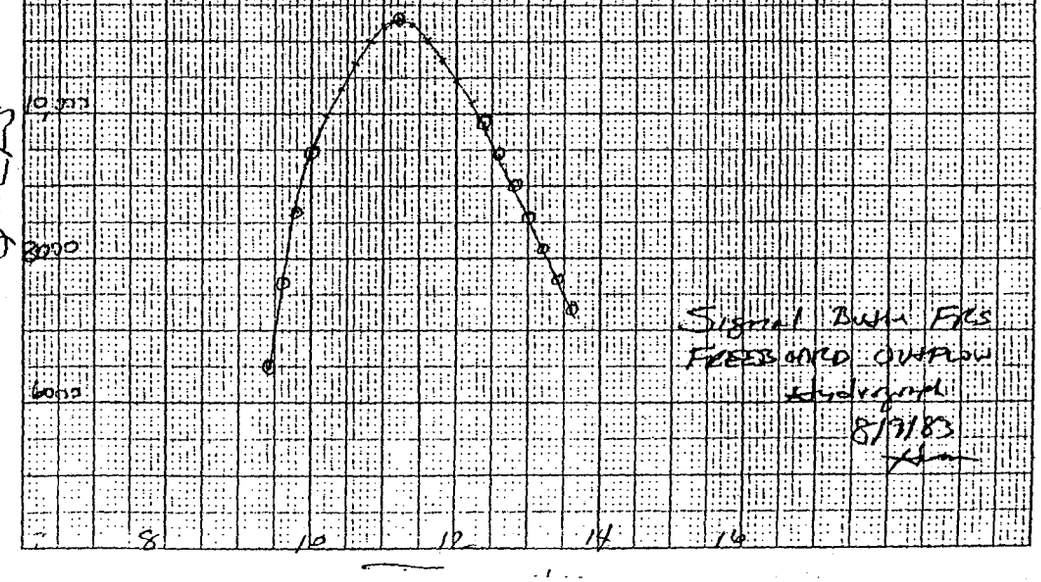
| | | | | | | |
|---|---|---|---|---|---|---|
| 3.40 | 11. | [illegible] | [illegible] | 0 | 0. | |
| 3.60 | 31.5 | 14.2 | 1700.8 | 249.5 | 51.8 | . |
| 3.80 | 73.2 | 14.6 | 1700.8 | 255.4 | 52.8 | . |
| 4.00 | 141.4 | 15.4 | 1700.8 | 256.0 | 53.0 | . |
| 4.20 | 225.2 | 16.8 | 1700.9 | 257.5 | 53.2 | . |
| 4.40 | 325.9 | 19.1 | 1701.0 | 260.3 | 53.7 | . |
| 4.60 | 440.7 | 22.2 | 1701.1 | 264.7 | 54.4 | .I |
| 4.80 | 600.2 | 26.5 | 1701.2 | 270.7 | 55.1 | .I |
| 5.00 | 789.4 | 32.3 | 1701.4 | 278.9 | 56.0 | .I |
| 5.20 | 1002.9 | 39.7 | 1701.6 | 289.9 | 57.3 | . I |
| 5.40 | 1243.4 | 49.0 | 1701.9 | 304.1 | 58.9 | . I |
| 5.60 | 1400.8 | 60.3 | 1702.3 | 321.9 | 60.8 | . I |
| 5.80 | 1731.2 | 73.6 | 1702.7 | 343.5 | 63.2 | . I |
| 6.00 | 1975.9 | 85.2 | 1703.2 | 369.0 | 66.1 | . I |
| 6.20 | 2351.7 | 93.5 | 1703.7 | 398.3 | 69.3 | . I |
| 6.40 | 2891.1 | 103.7 | 1704.3 | 432.6 | 73.2 | . I |
| 6.60 | 3439.4 | 116.0 | 1704.9 | 474.3 | 77.3 | . I |
| 6.80 | 4052.1 | 127.6 | 1705.5 | 524.5 | 81.8 | . I |
| 7.00 | 4618.4 | 130.9 | 1706.2 | 584.7 | 87.1 | . I |
| 7.20 | 5299.1 | 134.5 | 1707.1 | 654.2 | 93.2 | . I |
| 7.40 | 6114.0 | 138.3 | 1708.1 | 734.0 | 100.3 | . I |
| 7.60 | 6897.8 | 141.9 | 1709.0 | 826.1 | 108.2 | . I |
| 7.80 | 7892.6 | 146.0 | 1710.0 | 931.3 | 114.8 | . I |
| 8.00 | 8836.2 | 149.7 | 1711.1 | 1051.1 | 122.4 | . I |
| 8.20 | 9874.1 | 153.8 | 1712.2 | 1186.9 | 130.0 | . I |
| 8.40 | ****** | 409.6 | 1713.3 | 1339.1 | 138.3 | . I |
| 8.60 | ****** | 1225.0 | 1714.5 | 1506.3 | 146.3 | .E |
| 8.80 | ****** | 2464.2 | 1715.7 | 1692.3 | 154.6 | . E |
| 9.00 | ****** | 3856.9 | 1716.7 | 1859.8 | 163.1 | . E |
| 9.20 | ****** | 5257.5 | 1717.6 | 2032.6 | 170.1 | . E |
| 9.40 | ****** | 6488.4 | 1718.4 | 2192.1 | 176.1 | . E |
| 9.60 | ****** | 7660.3 | 1719.0 | 2331.9 | 181.4 | . E |
| 9.80 | ****** | 8641.2 | 1719.6 | 2451.1 | 185.9 | . E |
| 10.00 | ****** | 9454.3 | 1720.0 | 2550.9 | 189.7 | . E |
| 10.20 | ********** | | 1720.4 | 2633.5 | 192.8 | . E |
| 10.40 | ********** | | 1720.6 | 2700.1 | 195.3 | . E |
| 10.60 | ********** | | 1720.8 | 2749.4 | 197.2 | . E |
| 10.80 | ********** | | 1720.9 | 2783.3 | 198.4 | . E |
| 11.00 | ********** | | 1720.9 | 2805.2 | 199.2 | . E |
| 11.20 | ********** | | 1720.9 | 2817.8 | 199.7 | . E |
| 11.40 | ********** | | 1720.9 | 2822.1 | 199.9 | . E |
| 11.60 | ********** | | 1720.9 | 2817.1 | 199.7 | . E |
| 11.80 | 928 | 8.6 ****** | 1720.9 | 2803.3 | 199.2 | . E |
| 12.00 | 8606 | 3 ****** | 1720.9 | 2781.2 | 198.3 | . E |
| 12.20 | 7938 | 0 ****** | 1720.6 | 2751.2 | 197.2 | . E |
| 12.40 | 7278 | 69861.3 | 1720.4 | 2715.5 | 195.9 | . E |
| 12.60 | 6762 | 69434.3 | 1720.2 | 2674.9 | 194.4 | . E |
| 12.80 | 6273 | 19995.9 | 1720.0 | 2631.5 | 192.7 | . E |
| 13.00 | 5888 | 68557.7 | 1719.8 | 2586.9 | 191.1 | . E |
| 13.20 | 5486 | 19126.3 | 1719.5 | 2542.4 | 189.4 | . E |
| 13.40 | 5055 | 97697.1 | 1719.3 | 2498.5 | 187.7 | . E |
| 13.60 | 4712 | 67274.3 | 1719.0 | 2454.8 | 186.1 | . E |
| | | | 1718.8 | 2411.8 | 184.4 | . E |

24hr PMF

DAMS2 OUTPUT
Freeboard Hydrograph (PMF)
11,300 cfs @ 11.5 hours

46 1240

K+E 20 X 20 TO THE INCH • 7 X 10 INCHES KEUFFEL & ESSER CO. MADE IN U.S.A.




Signal Butte Res
Freeboard Outflow
Hydrograph
8/31/83

Effect of CAP Canal on Emergency Spillway

and Breached Hydrographs

Signal Butte FRS, Buckhorn Mesa Watershed

It is ~~somewhat~~ difficult to determine the exact effect of the Central Arizona Project Canal on the emergency spillway and breach hydrographs of the Signal Butte FRS. The analysis has to be based on certain basic assumptions. For instance, will the CAP embankments fail (i.e. breach) when and if they are overtopped? At what location is it most likely to fail? What is the capacity of the failed section? How much flood storage is available in the CAP Canal? etc?

In this analysis, the assumptions varied based on the magnitude of the flood analized. For the ESH, it was assumed that the overchutes at Station 427+15, 429+20 and 456+50 would function at an average rate of about 1000 cfs (Note, total design capacity of these structures is 1400 cfs) 1/ for about 12 hours based on the computed hydrograph at X-Section No. 4. This gives a total volume of: (12 hrs)(1000 cfs)(.08264) = 992 ac-ft. The available storage capacity in the Canal was estimated at 982 ac-ft. This latter volume was computed assuming a freeboard depth of 10 feet 2/ and an effective canal length of about nine miles, i.e., the distance between the control gate located at mile post 197.999 (near University) and the one to the southeast at mile post 207.214. The total controlled volume then is 1974 acre feet and

1/ See attached Table 7-3
2/ See attached Exhibit 7-4

exceeds the total volume (1850 ac. ft.) of the ESH. Thus it is theorized that with the ESH, the CAP would control the total discharge from the FRS and limit flows passing downstream of the CAP to overchutes at Stations 427+15, 429+20, 456+50, with a maximum discharge of 1400 cfs. Ponding, of course, would occur upstream of the overchutes causing overtopping of the upstream Canal embankment in the vicinity of Stations 427+15 and 429+20. This analysis assumes "non-storm" conditions downstream of the dam.

For the FBH, the conditions would be very similar to that described above except the total volume (6514 ac-ft) could neither be stored in the CAP nor pass through the overchute structures. The flow estimated to pass through the ovechutes was calculated at about 3250 ac-ft based on the capacities of the overchutes located at Stations 427+15, 429+20, 456+50 and 471+03. The total capacity of these structures is about 2885 cfs (Table 7-3), with a flow duration of 10 to 12 hours at maximum capacity and continuing several more hours at a lesser rate. Thus, the CAP would regluate about 4232 acre-feet, but nearly 2300 acre-feet would overtop the Canal on its south side and continue downstream. Exactly where the overtopping will occur is hard to perdict, but the most logical point would be near Station 456+50 since there is a major bend in the Canal at this point and ponding will tend to occur upstream of Canal at this location due to its alignment. This is also the location where water exited the CAP during the historical flood of July 17-18, 1984. Ponded depths near Station 456+50 could be as much as 10 to 12 feet. The area inundated downstream of the Canal would not be much different than that projected ignoring the effects of the CAP, although the total peak might be attenduated to some extent due to the available storage in the CAP.

It should be recognized, however, should the CAP Canal breach at Station 456+50 due to the overtopping, a large volume (at least 858 acre-feet, which

is the storage below normal depth between the two designated control gates) could be added to the downstream hydrograph. A breach would also tend to concentrate the flow rather the than it being spread out over a wide area.

With reference to the FRS breach hydrographs, the effect of the CAP on flows downstream of the Canal is dependent to some extent on the total volume assumed to be under in the hydrograph; that is conditions assuming flow or no inflow into the structure. Under the "without" inflow conditions, the total volume to the top of dam is only about 2822 ac-ft. About one-third of this volume could be stored in the Canal. This could attenduate the peak discharge to some extent if it was assumed that the downstream canal embankment did not fail. With a 19000 cfs peak discharge at X-Section No. 4, however, I believe it is more logical to assume that the Canal will be overtopped and breached and flow will continue downstream almost unimpeded. The exact location of the breach is again hard to predict, but Station 456+50 is a logical location. The breach would cause the volume to increase under the hydrograph, but its effect I think would be minimal with reference to depth or area inundated due to the unconfined conditions of the flow. Thus, under this condition (i.e. with flow through the overchutes and the projected overtopping) the area inundated downstream of the Canal would be about the same as that delineated ignoring the effect of the Canal. Depths immediately downstream of the CAP breach might be somewhat greater, but these would soon be disipated as the water spreads out over a wide area. The area of major depths and damages would be similar to that experienced below the CAP during the July 17-18, 1984 flood.

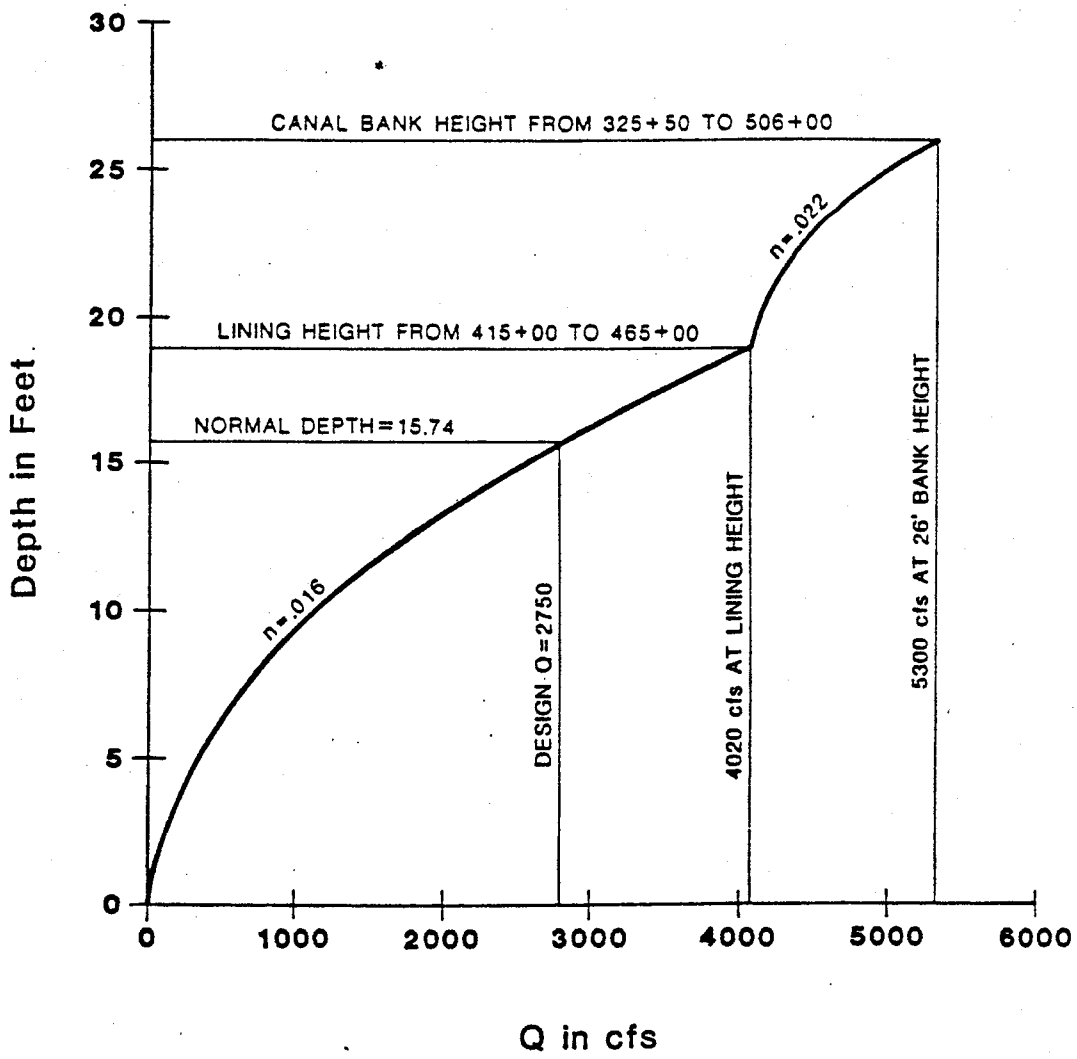
Should inflow to the FRS be assumed with the breach, the only difference in the above discription would be to remove any doubt about overtopping and breaching of the CAP Canal. With the larger volume of water, breaching of the

CAP would almost be a certainity. Again the area inundated downstream of the Canal would be about the same as presently delineated, although it could be enlarged due to the spreading effect of the CAP. Water could be transported by the CAP to the southeast and overtop the Canal at a point further downstream. However, to predict a location is almost impossible, therefore, I suggest leaving the inundation map as originally delineated.

In conclusion, the CAP Canal definitely will have some effect on any emergency spillway or breach hydrograph discharge occurring from the Signal Butte FRS. The effect on the ESH will be to limit discharges downstream of the CAP to those which will pass through the overchutes structures located at Stations 427+15, 429+20 and 456+50. The total design capacity of these structures is 1400 cfs. Although this is a decrease from the 2369 cfs estimated at X-Section No. 4, the area inundated downstream of the CAP should remain about the same, although the depths will be decreased to some extent. Since the maximum capacity at Station 456+50 is only 270 cfs, this flow should be contain within the channel downstream of the CAP, and not inundate any additional land. Some minor ponding also will occur upstream of the CAP in the vicinity of the designated structures under this flow condition.

For the FBH and breach hydrographs, it is assumed that the CAP embankments will overtop and breach near Station 456+50 and at other possible locations. Thus, with flow through the overchutes, the area inundated downstream of the CAP will be almost the same as estimated ignoring the effect of the Canal. Ponding will occur upstream of the CAP to the height of the maintence road embankment with average depths of about 4 feet, but with possible depths as high as 10 to 12 feet in the vicinity of Station 456+50.

# AQUEDUCT RATING CURVE




Source: U.S. Bureau of Reclamation, "Assessment of Flood Damages, Salt-Gila Aqueduct Reach 1B, July 17-18, 1985", by International Engineering Co. Inc.

EXHIBIT 7-4

TABLE 7-3

CASE III

DISCHARGE HYDROGRAPHS

| Structure | | Case III | | | | Bureau[1] Original Design | | |
|---|---|---|---|---|---|---|---|---|
| | | Drainage Area * | | Discharge | | Drainage Area | Discharge | |
| No. | Station | Designation | Area (sq mi) | Peak (cfs) | Volume (ac-ft) | (sq mi) | Peak (cfs) | Volume (ac-ft) |
| 1 | 345+30 | 1A | 0.40 | 436 | 31 | 0.40 | 460 | 27 |
| 2 | 374+00 | 2A | 0.87 | 887 | 67 | 0.82 | 870 | 60 |
| 3 | 396+40 | 3A | 0.76 | 762 | 58 | 0.76 | 800 | 51 |
| 4 | 427+15 | 4A+4B+4D | 1.88 | 530* | 72* | 1.73 | 565* | 59* |
| 5 | 429+20 | | | | | | | |
| 6 | 456+50 | 10A | 0.19 | 286 | 15 | 0.19 | 270 | 13 |
| 7 | 471+03 | 5A | 1.67 | 889 | 128 | 1.67 | 910 | 113 |
| 8 | 479+00 | 6A | 2.28 | 543* | 87* | 2.26 | 575* | 52* |
| 9 | 504+25 | | | | | Total | 3460 cfs | |
| 10 | 529+50 | 7A | 3.17 | 803* | 121* | 3.10 | 840* | 105* |
| 11 | 542+50 | | | | | | | |
| 12 | 552+50 | 8A | 4.06 | 778* | 154* | 3.99 | 760* | 135* |
| 13 | 563+50 | | | | | | | |
| 14 | 574+50 | 9A | 0.66 | 401 | 50 | 0.62 | 400 | 42 |

Breach outflows

Source: U.S. Bureau of Reclamation, "Assessment of Flood Damages, Salt-Gila Aqueduct Reach 1B, July 17-18, 1984," by International Engineering Co. Inc.

(1) Reference 7

* per each structure

Ariz Buckhorn Mesa W/S
Hem 4/30/85
Sierra Bette Floodway Breach Analysis
Available Storage Volume of CAP Aqueduct.

1. Esti Vol. of Available Freeboard Storage Bet Control Gate @ Mile Post 197.999 and Mile 207.214 (i.e. From Stw. Univ. University Dr. Southeast 9.215 miles)

Given:
CAP BW = 24'
S:S = 1½:1
Nor Depth = 15.74' (use 16')
Total Depth = 26' (To top of Main. Rd)

Find Vol to D = 16'

A = 768.0 □'
TW = 72'

$$Vol = \frac{(768\ \square')(9.215\ miles)(5280\ ft/mi)}{43560\ ft/Ac\ ft} = 858\ Ac\ ft$$

Find Vol to D = 26'

A = 1638 □'
TW = 102'

$$Vol = \frac{(1638)(9.215)(5280)}{43,560} = 1830\ Ac\ ft$$

Therefore Available Storage:

Vol = 1830 − 858 = 982 Ac ft

# FAILURE MODE AND CONSEQUENCE ANALYSIS
# For
# APACHE JUNCTION FLOOD RETARDING STRUCTURE

## PINAL COUNTY, ARIZONA
## NATDAMID AZ 00211

**July 17 2001**

## 1.0 INTRODUCTION

### General Description

Apache Junction FRS is located within the City of Apache Junction. The FRS begins 1,200 ft west of Apache Trail and ends at 1/4 mile south of McKellips Road and 1/4 mile west of Idaho Road. The FRS is about 30 miles east of downtown Phoenix and approximately one mile north of the City of Apache Junction. The project consists of the FRS structure and an emergency spillway. The project is part of the Buckhorn Mesa Watershed Protection and Flood Prevention Project, which includes the Signal Butte and Spook Hill flood retarding structures. The Flood Prevention Project was prepared, designed, and constructed by the U.S. Department of Agriculture, Natural Resources Conservation Service.

Apache Junction FRS is classified as a high hazard, small dam. The reservoir behind the FRS is 98 acres with a capacity of 635 acre-feet. Construction of the Apache Junction FRS was accomplished under contract to Ashton Construction Company. The completion date of the construction of the dam and landscape treatment is December 1988.

The dam has performed satisfactorily to date.

### Dam Data

Dam type: Rolled earthfill
Dam height: 22.0-ft
Dam length: 8,764-ft
Dam crest: width 14-ft; elevation 1810.0-ft
Spillways: Principal - 30-in RCP; inlet elevation 1793.5-ft; Emergency spillway - 100-ft wide RC baffle block spillway; crest elevation 1799.77-ft
Freeboard: 10.0-ft
Reservoir Surface: 98 ac at spillway crest
Storage: 635-af at spillway crest
Hazard Classification: High

**Purpose and Scope**

In general, the purpose of the Failure Mode and Consequence Analysis (FMCA) exercise was to:

- Identify potential site-specific failure modes for the dam.
- Discuss non-quantitatively the likelihood of the identified failure modes.
- Determine whether or not and how important failure mechanisms are being monitored
- Examine the consequences of failure and the adverse consequences of successful operation (e.g. – large spillway releases)
- Identify possible risk reduction actions that may be taken to reduce the likelihood of failure or to mitigate adverse consequences
- Determine what information, investigations or analyses may be needed to resolve uncertainties relative to potential failure modes.

(Note: In this phase, the FMCA team only examined the general nature of the "consequences" for the failure modes identified and where appropriate estimated how they may be different than previously anticipated. Greater detail on the estimate of the magnitude of the "consequences" for each significant failure mode will be addressed in the quantitative portion or risk analysis part of the risk assessment for the dam. At this time hopefully more detailed discussion on the emergency response plan for each structure will also be possible.)

**Team Members**

Larry Von Thun, Engineering Consultant and FMCA facilitator
Bob Eichinger, Kimley-Horn and Associates, Inc., Project Manager
Jim Scott, URS Corporation, Principal
Ken Euge, Geological Consultants, Principal
Tom Renckly, Flood Control District of Maricopa County, Project Manager,
Larry Lambert, Flood Control District of Maricopa County, Dam Safety Engineer
George Beckwith, Flood Control District of Maricopa County, Geotechnical Engineer
Justin Beeler, Kimley-Horn and Associates, Inc, Session Recorder
Tim Murphy, Flood Control District of Maricopa County, Civil Engineer (consequences)

## 2.0 MAJOR FINDINGS AND UNDERSTANDINGS GAINED

The following is a summary of the major findings and understandings for Apache Junction Flood Retarding Structure as a result of the Failure Mode and Consequence Analysis (FMCA). The findings are organized according to:

1. Findings related to factors affecting potential failure mode development.
2. Findings related to consequences of dam failure or spillway discharges from flood events.
3. Action items related to risk reduction or investigations.

## Findings Related To Factors Affecting Potential Failure Mode Development

- A Central Filter Drain Was Part Of The Original Design Of The Dam And It Was Constructed To The Full Depth Of The Foundation Excavation: The central filter extended locally to below the design foundation depth as deeper excavation was made to improve conditions due to the presence of looser soils at depth. The central filter extends through the entire embankment to the bottom of the cutoff trench, but does not fully penetrate the collapsing soil layer between about Stations 44+25 and 52+50.
- The Central Filter Is Potentially Susceptible To Supporting A Crack - An internal SCS report on the general subject of cracking of dams in Arizona (Leckband, 1984; "A Barrier to Cracks in Dry Dams") indicates that the central filter of the Vineyard Road FRS stood vertically in temporary excavations that extended for the full height of the dam. This suggests that the filter may be cemented and thereby susceptible to cracking in response to shrinkage or settlement. This possibility leads directly to the postulation of a seepage erosion potential failure mode even where a central filter has been constructed.
- Low Potential for Fissures to Develop. Based on a preliminary review of the geology (owing to the presence of shallow bedrock) land subsidence history and strain potential the likelihood of land subsidence and earth fissures developing at this site is judged to be low. A final evaluation of the information is necessary to verify this relatively confident preliminary assessment.
- Holocene Soils Are Present In The Dam Foundation. Relatively thick collapsing Holocene soils are present under a portion of the dam cutoff trench and central filter (900-ft of Holocene soils along foundation which is approximately 10 percent of dam length). Holocene soils potentially create/contribute to two failure modes – Seepage erosion along cracking due to differential settlement and internal erosion of the foundation material itself.
- A Defense Against The Presence Of The Silty Sand/Sandy Silt Layer In The Foundation (Holocene Deposit) Was Not Fully Addressed in Design. A continuous relatively thick deposit of collapsing silty sand and sandy silt is present beneath the cutoff trench and bottom of central filter between Stations 52+50 and 44+25. This feature has the potential for creating large differential settlements and transverse cracking through the embankment and upper part of the foundation. This could lead to seepage erosion modes of failure passing along the embankment –foundation interface or through embankment cracks and a defect in the central filter (perhaps associated with cementation of the filter aggregate).
- Potential for Shrinkage Cracking: The available geotechnical tests indicate a relatively low potential for shrinkage cracking, but the test data is limited and inconclusive. A program of test borings and laboratory classification tests is recommended to better characterize the drying shrinkage potential and extent of collapsing soil beneath the embankment. The dam is only 13 years old, so the drying shrinkage process may not be complete. Soil properties based on the amount of fines and the clay content of borrow area materials indicate the potential for shrinkage cracking (generally similar to the Vineyard Road, Rittenhouse and Powerline soils). How severe the cracking may become is not known. To date little cracking has been experienced – however,

the dam is only 13 years old (little time for drying out) and has not experienced cycles of significant reservoir filling and emptying.

- Low Hydraulic Gradient Across the Structure . Because the emergency spillway is 10.23 feet below the dam crest and the maximum height of dam is only 22 feet, low global hydraulic gradients driving seepage-erosion processes prevail during storage of water in the reservoir. This and the time of release of less than ten days reduce the risk of seepage erosion modes of failure. The amount of head/hydraulic gradient available to create seepage related or static loading type failure is low for this dam (only 6.25-ft of head from the emergency spillway crest to the principal spillway invert (also the top of the sediment pool)). This is a small reservoir loading relative to dam height.
- The Dam Design Is Modern, Reflects Lessons Learned At Vineyard, Rittenhouse And Powerline, And Is Well Documented. The design of Apache Junction FRS reflects lessons learned by observation and evaluation of the performance of the older SCS dams. This is reflected by the provision of a central filter throughout the embankment, a diaphragm filter around the principal spillway conduit rather than anti-seep collars, and over-excavation of most of collapsible soils from beneath the cutoff trench.
- Short Impoundment/Storage Time. The very short time of storage between top of dam and emergency spillway (approximately 10 hours) and the short time of storage between emergency spillway crest and principal spillway (approximately 4.6 days) reduces the risk of seepage erosion failure modes advancing to the point of initiation of a dam breach. The top of the emergency spillway is the appropriate reservoir elevation for consideration of a seepage erosion related failure mode and dam breach.
- No Drain Outlet. There are no drain outlets for the central filter. If work were ever done on this structure (e.g. – exploration of the extent of a transverse crack) consideration should be given to installing a central filter drain outlet. However, at this time the situation does not warrant the installation of drains. The hydraulic conductivity of the filter zone material is not high enough to allow effective drainage for any reasonable spacing of finger drains. The major problem is the potential for a seepage erosion failure in the highly erodible Holocene soils at the embankment-foundation interface
- Transverse and Longitudinal Cracks. Only one transverse crack has been observed across the crest. Longitudinal cracking does exist at a few locations on the crest and it appears to be directly associated with the location of the filter drain. Such a finding is not really unexpected since the construction procedure (excavation of a trench through the crest shortly after completion of the fill) created the potential for this crack to form as the dam materials consolidated and spread laterally. The excavation would have removed natural bonding of the more clayey materials along the centerline of the dam.
- The Filter On The Dam Crest Is Exposed To Infiltration Of Surface Water. Potential wetting/filling of the central filter due to rainwater infiltration is possible and the potential development of cementation of the filter due to the water infiltration and downward seepage is probable. There is a possibility of cementation of the central filter and propagation of shrinkage or settlement cracks through the filter.
- Embankment Bulges. Two documented bulges have been identified on the downstream face of the embankment. These are likely to be construction related since

the dam has not undergone any significant loading. This possibility can only be assumed but it can be easily tested via monitoring by surveys to see if any change/expansion takes place. The inspection logs indicate that the bulges may be due to overbuilding in the curved section of the dam and/or erosion of the lower part of the slope

- Potential Failure Mode at Emergency Spillway and Embankment. The likelihood of a failure mode involving a transverse crack and seepage erosion at the concrete emergency spillway and underlying dam embankment is low. The extensive wing walls at emergency spillway appear to eliminate this failure mode. The spillway was constructed with extensive (27-foot) wing walls into the embankment fill, which essentially eliminates the potential failure mode and any concern for a crack along the side wall/embankment contact.
- No Identified Failure Modes For Principal Spillway And Emergency Spillway.
- Seismic Risk. No credible seismic-related failure modes were identified although minor cracking is possible.
- History of Filling. The Dam has not been "tested" with a significant impoundment. Thus, the potential for seepage-induced settlement of the collapsible soils, embankment/foundation cracking and related failure modes is not well defined.
- Elevation 1793.5-Ft Top of Sediment Pool. – This relatively high elevation is not a reality – the pool will actually be filled with flood inflows and this should be appropriately accounted for in determining PMF freeboard and threshold frequency for potentially damaging spillway discharges.
- No overtopping for PMF. Hydrologic analyses conducted for Apache Junction FRS indicates no overtopping for the probable maximum flood. The estimated freeboard under the PMF is 5-feet (maximum stage 1805-feet; top of dam crest 1810-feet).

**Findings Related To Consequences Of Dam Failure Or Flood Discharges**

- Short Time to Peak. Time to peak for PMF is approximately 2.5 hours. This is a very short warning time for alerting residents about emergency spillway discharges. (Note: This probably refers to the 6-hr. summer PMF. The situation would be different for the 72-hr. winter PMF. In either case, it seems likely that satellite imagery will provide considerable warning time for major storms).
- No Individual Emergency Action Plan has been developed. There is a need to develop an Apache Junction individual EAP meeting minimum requirements of ADWR and FEMA 64.
- Emergency Spillway Discharges Flows Directly into City of Apache Junction. The highest risks apparent at Apache Junction dam are related to design discharge operations of the spillway. Flood events into Apache Junction FRS in excess of 100-year flood result in discharges through the emergency spillway. Emergency spillway discharges have the potential to inundate developed and inhabited properties within the City of Apache Junction.
- Complexity of Emergency Response. The location of the Apache Junction FRS at the boundary of two counties and within a city creates a situation of combined/variable emergency response from the counties and city involved (Maricopa and Pinal County,

City of Apache Junction). The District has no significant downstream easements for regulatory control of emergency spillway discharge inundation areas.

- Breach Estimation. The FMCA Team estimated a peak outflow of a dam breach hydrograph from a piping failure of approximately 3600-cfs. The FMCA team recommends that further dam breach analysis and downstream inundation mapping be conducted. Special attention should be given to quantification of the erosivity of the stiff unsaturated embankment soils.

**Action Items Related To Risk Reduction Or Investigations**

- Potential Risk Reduction Measures:
  1. Repair of transverse cracks on case-by-case basis.
  2. Repair of longitudinal crack (repair/seal).
  3. Inspection procedures for event related inspections (during and post-event).
  4. Extend central filter through the collapsing Holocene soils to the underlying cemented deposits between Stations 44+25 and 52+50.
- Stability Analyses. The designer made some slope stability calculations but those studies are not comprehensive and are not really adequate for record study. Slope failure is not considered to be a significant risk, but the existing stability calculations do not provide adequate documentation. It is recommend that further stability analyses be conducted for the record.
- Utility Crossings. There does not appear to be a potential failure mode associated with utility crossings, but proper documentation of utility crossings is needed. No as-built plan drawings were found for the 4-inch abandoned waterline line located at Station 85+24.
- O&M Plan. The Operations and Maintenance Plan should be updated.

## 3.0 POTENTIAL FAILURE MODES

Potential failure modes identified by the FMCA team are presented below. The failure modes were placed into one of four categories listed below. Except under special circumstances only those potential failure modes in Category I will be considered in the risk analysis phase of the risk assessment exercise.

- Category I – Failure modes of greatest significance.
- Category II – Failure modes of lesser significance (but not inconsequential).
- Category III – Failure modes for which insufficient information is presently available to allow the team to make a judgement on the significance of the failure mode.. The development of additional data and information is warranted. Additional records research may be justified.
- Category IV – Failure modes which are not physically possible or which are clearly not credible.

For each of the potential failure modes identified, a Failure Mode Description is briefly described and the factors that make the failure mode more likely (adverse factors) or less likely (positive factors) to occur are listed following the Failure Mode Description. In

addition, any identified actions for risk reduction or investigation for the potential failure modes are also provided.

## CATEGORY I - FAILURE MODES OF GREATEST SIGNIFICANCE

***Failure From Normal Emergency Spillway Discharges (Category I).***

Failure Mode Description: The Apache Junction FRS emergency spillway is a 100 foot wide baffle-block reinforced concrete spillway. Normal flood discharges from the spillway are directed into the City of Apache Junction. This potential "failure mode" does not fail the dam or emergency spillway. This potential failure mode was rated as a Category I failure mode because normal "successful" operations of the emergency spillway can produce discharges that could have significant adverse consequences and the likelihood of occurrence of these adverse consequences appears to be much greater than any mode involving dam failure. .

Adverse Factors:

(1) Spillway flows are directed into the City of Apache Junction. Downstream population at risk potentially around 10,000 people.
(2) No dedicated downstream easements from spillway.
(3) Time to peak for PMF is 2.5 hours.
(4) No individual emergency action plan.
(5) No regulation on downstream spillway discharge areas.

Positive Factors:

(1) Dam is instrumented as part of the District Alert System. Would provide early warning.
(2) Spillway discharge exit is normal operation and straight forward.
(3) Short spillway discharge duration for PMF.

Potential Actions for Risk Reduction:

(1) Prepare emergency action plan according to minimum standards in FEMA 64 and ADWR rules and regulations.

## CATEGORY II – FAILURE MODES OF LESSER SIGNIFICANCE (BUT NOT INCONSEQUENTIAL)

***Failure Due To Seepage Erosion Through A Transverse Crack In The Embankment During A Major Flooding Event (Category II- Considered but not highlighted).***

(Note: This potential failure mode was initially judged by the team as a Category I failure mode but during final discussions was rated as Category II because of the minor amount of cracking observed to date. The team noted that this judgment could change with time because relatively young age of the dam - constructed in 1988, and lack of significant impoundment.)

Failure Mode Description: Potential for impounded water to infiltrate and flow into a transverse crack(s) existing in embankment prior to the major flooding event or a transverse crack that develops in association with the flooding. The transverse crack allows the entry of a great enough flow of water to initiate seepage erosion and breaching failure.. Two modes were identified for failure: Mode A - Failure through a major, large, wide transverse crack carrying water to the upstream face of the central filter and passing through the central filter via a defect in the filter (caused by cementation of the filter allowing it to support a crack or) causes water to flow longitudinally in filter until water finds a downstream transverse crack initiating a piping failure; Mode B - failure through intense, multiple transverse and longitudinal cracks water then flows through a defect (caused by segregation or internal instability) in the central filter and begins seepage erosion (and loss of central filter material) through a downstream transverse crack. Generally, the mechanics of the potential failure need to be such that there is an upstream transverse crack or intense cracking zone lined-up with or near a defect in the central filter, which in turn is lined-up with a downstream transverse crack(s).

Adverse Factors:

(1) Areas where a silt layer was encountered (Sta 52+50 and 44+25) existing below cutoff trench – may produce differential settlement.
(2) One transverse crack found at Station 36+00 on downstream and upstream slope.
(3) Potential for cracking based on soil material properties (see appendix for material properties).
(4) Longitudinal cracks on crest over filter – exposes filter to rainwater and potentially initiates localized cementation of filter material.
(5) No significant impoundment history – dam not tested – (4.8 feet in 1993).
(6) Multiple storms may be a more critical flooding condition.

Positive Factors:

(1) Trench for filter constructed deeper locally.
(2) Trench to full depth and width.
(3) Only one documented transverse crack.
(4) Very minor cracking to date.
(5) No significant impoundment.
(6) Short time of impoundment – less than 4.6 days – affects saturation and failure mode development.

Potential Actions for Risk Reduction:

(1) Monitor cracks and develop crack mapping program.
(2) Train O&M crews to on mechanism of transverse and longitudinal crack formation.
(3) Repair cracks.

***Failure From Seepage Erosion Due To Seepage Erosion Of Foundation Material Along Contact (Category II- Considered but not highlighted).***

Failure Mode Description: Water reaches foundation via large settlement crack or a system of cracks and then moves along the dam/foundation contact causing Holocene foundation material left in place to erode, developing a seepage erosion tunnel, followed by caving and a breach.

Adverse Factors:
(1) Holocene materials exist along over a significant portion of the dam.
(2) Soils may become wetted with time (ultimately potentially become saturated).
(3) Relatively thick deposit of silts at two locations (Stations 52+50 and 44+25).

Positive Factors:
(1) Higher horizontal permeability – trend of water to flow directly through on these horizontal paths without causing development of an erosional tunnel.
(2) Seepage path would take a long time to develop.
(3) Relatively long path.
(4) Very short impoundment time – 4.8 days.
(5) Low average head.
(6) Central filter can act as a collector/sink for subsurface flow and thus could break up a direct upstream/downstream connection.
(7) Cutoff trench breaks up potentially erodible deposits.

Potential Actions for Risk Reduction:
(1) Train operation and maintenance personnel and inspectors to know where and what to look for.
(2) Inspect and monitor during event and post-event.

## CATEGORY III– FAILURE MODES FOR WHICH INSUFFICIENT INFORMATION IS PRESENTLY AVAILABLE FOR MAKING ENGINEERING JUDGEMENTS

No Category III potential failure modes were identified by the FMCA team for Apache Junction FRS at this time.

## CATEGORY IV – FAILURE MODES WHICH ARE NOT PHYSICALLY POSSIBLE OR WHICH ARE NOT CLEARLY CREDIBLE

***Failure From Seepage Erosion Due To A Transverse Crack Adjacent To Emergency Spillway (At Embankment/Spillway Contact) (Category IV).***

Failure Mode Description: Potential seepage erosion failure due to seepage erosion through a transverse crack that would form at the contact of the embankment and the emergency spillway. Water flows into transverse crack carrying eroded materials in crack

and manifests on downstream face releasing water at downstream toe or on the downstream slope.

Adverse Factors:
(1) Discontinuities – openings commonly occur.
(2) A 1989 ADWR report states a separation of concrete wall with soil mass.
(3) Concrete wall will deflect upon loading as well as foundation carrying load.

Positive Factors:
(1) Can monitor closely.
(2) Can be treated and repaired interactively.
(3) Wing walls on emergency spillway structure extend 27 feet into fill cutting off flow path from reservoir.
(4) Only one side would erode.
(5) Only 10-hours of duration of spillway flow at PMF.
(6) Weep holes indicate potential for flow.
(7) Wing walls.

Potential Actions for Risk Reduction:
(1) Repair cracks.
(2) Monitor spillway discharges and adjacent fill up against emergency spillway.

***Failure from Potential Embankment Slope Instability (Category IV).***

Failure Mode Description: Potential embankment slope failure due to internal pressure in the central filter.

Adverse Factors:
(1) Documented evidence of longitudinal cracks at dam crest centerline.
(2) Documented evidence of bulges on downstream slope (may be construction manifestation).
(3) No drain outlet for central filter (filter may fill with water).

Positive Factors:
(1) Slope stability analysis conducted by NRCS (EBASCO).
(2) Standard slopes – do not expect problem.
(3) No pore pressures.
(4) Upstream slope is 2.5:1.
(5) Low seismic acceleration.
(6) High strength of embankment soils.
(7) Low hydraulic head.
(8) Short time for impoundment.

Potential Actions for Risk Reduction:
(1) Update stability analyses to actual cases.
(2) Watch for slope distortion - integrate monument data.

(3) Confirm design slope by survey.
(4) Survey crest at bulge stations.
(5) Install drains to central filter.

**Other Considerations**: The following issues were discussed by the FMCA team but a failure mode was not identified. The reason(s) for no failure mode being postulated is given for each.
**Fissure Developing Below Dam Embankment**: not likely and not considered for this dam.
**Seepage Erosion due Piping Around ATT Cable**: buried in foundation, well constructed, documented filter around cable.
**Seepage Erosion due to Piping at Abandoned 4-inch waterline**: constructed under foundation. No actual verification of grouting. No as-builts to confirm.
**Seismic Activity in Conjunction with Impoundment** : Dry conditions - No liquefaction; Well compacted embankment; No Pore Pressure in Dam; Less potential for amplification; no incipient condition to be triggered by earthquake; closed fault is Sugarloaf 15 miles from dam .

No failure modes were found to be associated with the following structures at this dam.

- Principal spillway
- Emergency spillway

## 4.0 SUMMARY AND CONCLUSIONS

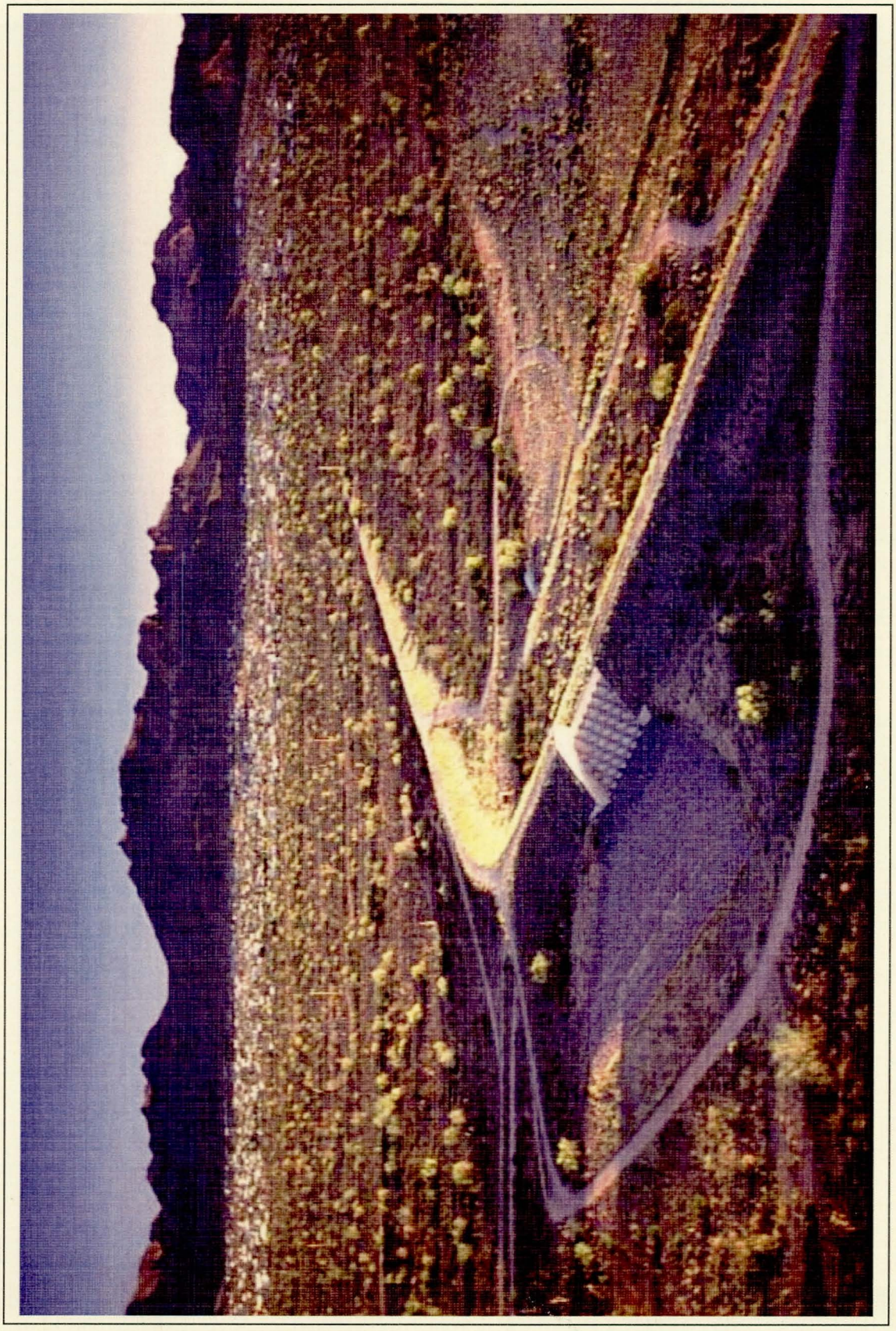
Apache Junction FRS was constructed pursuant to a modern dam design. Construction appears to have been without any particular issues. The dam has performed normally and satisfactorily for 13 years. The structure is satisfactorily maintained and monitored.

However, it is prudent to recognize that there exist for all dams specific ways that failure could come about that warrant attention and diligent monitoring. The identification of a condition or process as a "potential failure mode" does not imply that the dam is about to fail or even necessarily that there is a dam safety deficiency at the site. Rather it identifies physically possible conditions or processes (generally with a remote but still credible chance of occurrence) that persons associated with owning, inspecting, analyzing and operating the dam should be aware. Some of the potential failure modes are highlighted (or prioritized) for attention of the dam owners and operators. They are highlighted because the specific conditions at the dam and appurtenant structures are such that these failure modes are physically possible and are considered the most realistic and most credible potential failure modes definable at the site.

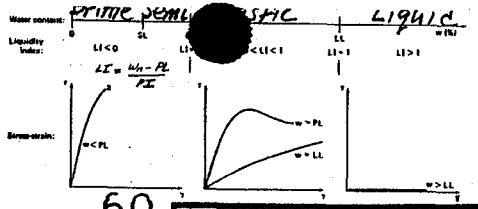
Only one potential Category I failure mode was identified by the FMCA team. This was not really a failure mode but rather adverse consequences associated with normal emergency spillway operations. The spillway discharges flows directly into the City of Apache Junction. There are a large number of people and structures at risk in the flood path in the event of a spillway discharge (or dam failure by breach). It is possible that casualties could occur due to large spillway discharges as the advance warning time for

the PMF is not large (several hours). An immediate risk reduction measure is to develop an individual emergency action plan for Apache Junction FRS.

## APACHE JUNCTION FRS APPENDIX - FMCA REFERENCE MATERIALS



Apache Junction FRS and emergency spillway.
Photo date: October 2001 Photograph Copyright Kimley-Horn and Associates, Inc.




| Characteristic | Soils at Equal LL with Increasing PI | Soils at Equal PI with Increasing LL |
|---|---|---|
| Dry strength | Increases | Decreases |
| Toughness near PL | Increases | Decreases |
| Permeability | Decreases | Increases |
| Compressibility | About the same | Increases |
| Rate of volume change | Decreases | — |

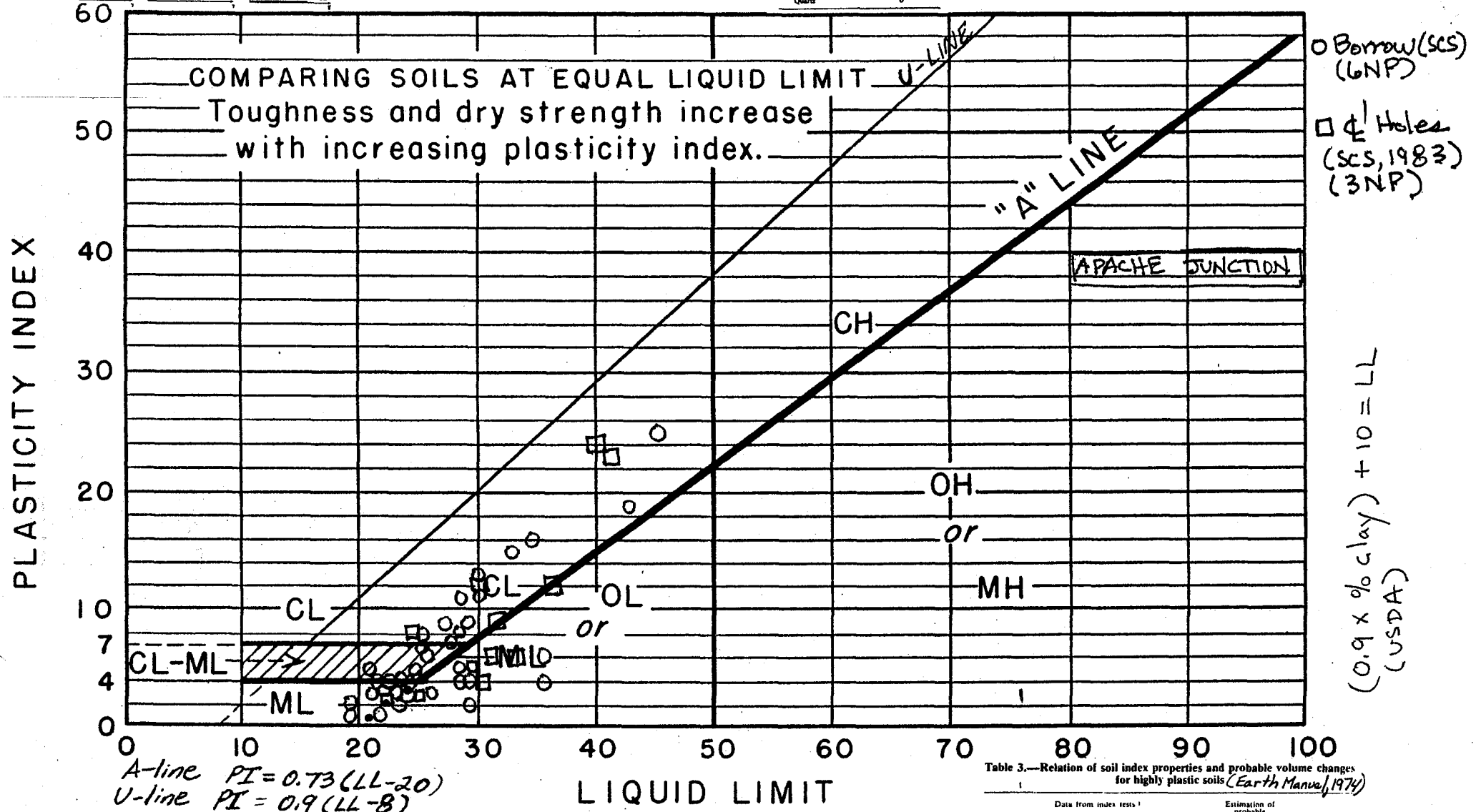
**TABLE 4-1 Activities of Various Minerals***

| Mineral | Activity |
|---|---|
| Na-montmorillonite | 4–7 |
| Ca-montmorillonite | 1.5 |
| Illite | 0.5–1.3 |
| Kaolinite | 0.3–0.5 |
| Halloysite (dehydrated) | 0.5 |
| Halloysite (hydrated) | 0.1 |
| Attapulgite | 0.5–1.2 |
| Allophane | 0.5–1.2 |
| Mica (muscovite) | 0.2 |
| Calcite | 0.2 |
| Quartz | 0 |

% < 2 μm

*Table 4-1. WES Classification of Potential Swell*

| Classification of potential swell | Potential swell $S_p$ percent | Liquid limit LL percent | Plasticity index PI percent | Natural soil suction $\tau_{nat}$ taf |
|---|---|---|---|---|
| Low | <0.5 | <50 | <25 | <1.5 |
| Marginal | 0.5–1.5 | 50–60 | 25–35 | 1.5–4.0 |
| High | >1.5 | >60 | >35 | >4.0 |




A-line $PI = 0.73(LL-20)$
U-line $PI = 0.9(LL-8)$

# PLASTICITY CHART

**Table 3.—Relation of soil index properties and probable volume changes for highly plastic soils** (Earth Manual, 1974)

| Data from index tests [1] | | | Estimation of probable expansion,[2] percent total volume change (dry to saturated condition) | Degree of expansion |
|---|---|---|---|---|
| Colloid content (percent minus 0.001 mm.) | Plasticity index | Shrinkage limit, percent | | |
| >28 | >35 | <11 | >30 | Very high. |
| 20–31 | 25–41 | 7–12 | 20–30 | High. |
| 13–23 | 15–28 | 10–16 | 10–20 | Medium. |
| <15 | <18 | >15 | <10 | Low. |

[1] All three index tests should be considered in estimating expansive properties.
[2] Based on a vertical loading of 1.0 p.s.i. as for concrete canal lining. For higher loadings the amount of expansion is reduced, depending on the load and on the clay characteristics.

Table 4-. *Data for making estimates of probable volume changes for expansive soils* (Chen 1975)

| Laboratory and field data | | | Probable expansion, percent total volume change | Swelling pressure, ksf | Degree of expansion |
|---|---|---|---|---|---|
| Percentage passing No. 200 sieve | Liquid limit, percent | Standard penetration resistance, blows/ft | | | |
| >35 | >60 | >30 | >10 | >20 | Very high |
| 60-95 | 40-60 | 20-30 | 3-10 | 5-20 | High |
| 30-60 | 30-40 | 10-20 | 1-5 | 3-5 | Medium |
| <30 | <30 | <30 | < 1 | 1 | low |

**TABLE 3:3 / VOLUME CHANGE POTENTIAL**
**(Adapted from Holtz and Gibbs[116])** (1956)

| Volume Change | Shrinkage Limit | Plasticity Index |
|---|---|---|
| Probably low | 12 or more | 0–15 |
| Probably moderate | 10–12 | 15–30 |
| Probably high | 0–10 | 30 or more |

(24-5)

9. ITEMS OF WORK AND CONSTRUCTION DETAILS - Bulldog Floodway/Apache Junction Outlet

Items of work to be performed in conformance with this specification and the construction details are:

(a) Bid Item 8, Transition Fill

(1) This item consists of furnishing and installing the transition fill material necessary to construct the transition fill portion of the structure drain along the reinforced concrete channel and side channel inlets.

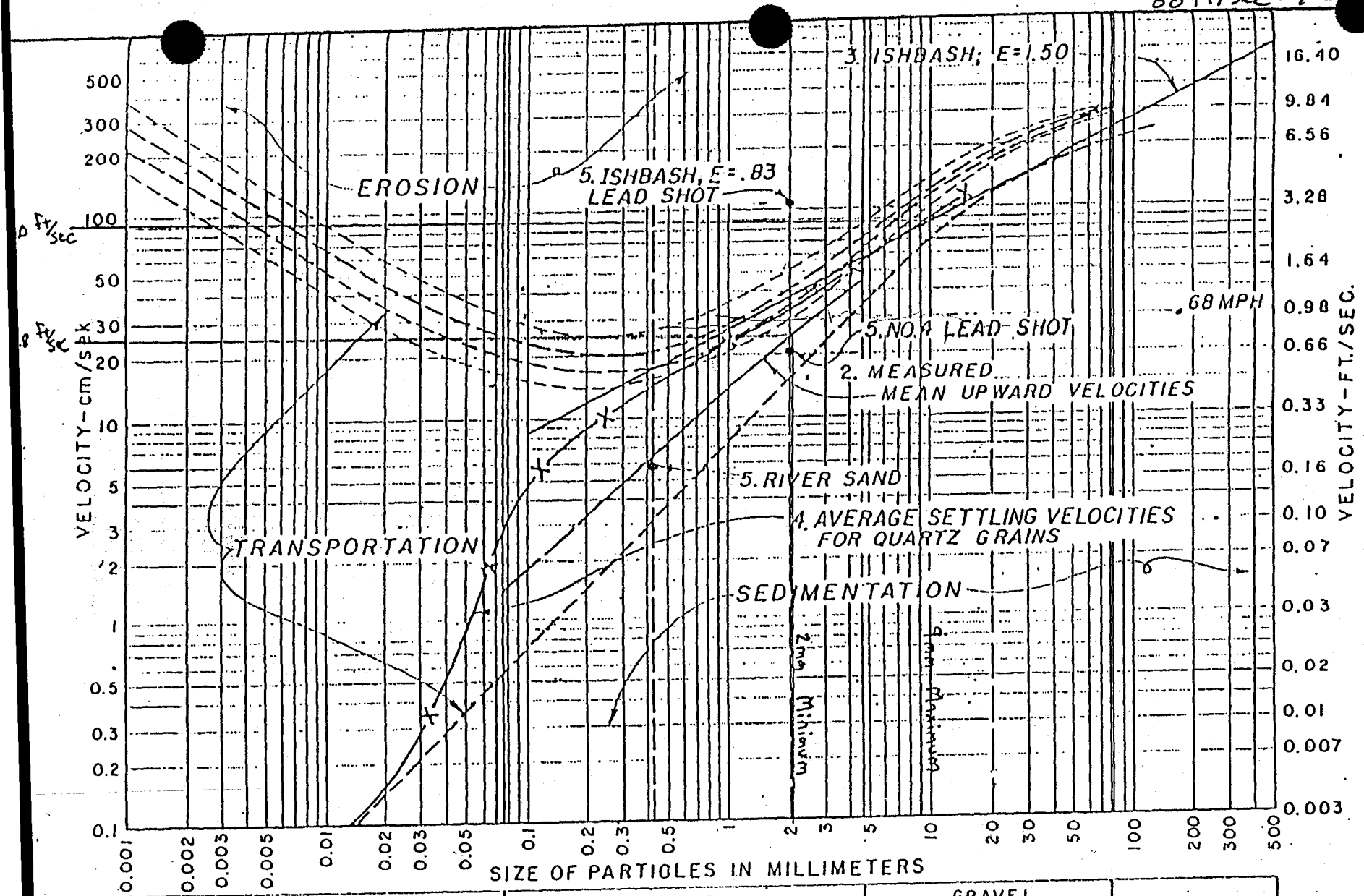
(2) The Transition Fill material shall be well graded within the following limits of gradation:

| Sieve Size | Percent Passing |
|---|---|
| 2" | 100 |
| 3/4" | 90-100 |
| #4 | 60-100 |
| #10 | 40-100 |
| #20 | 20-75 |
| #40 | 0-55 |
| #60 | 0-40 |
| #100 | 0-25 |
| #200 | 0-5 |

(3) The Transition Fill material shall contain sufficient moisture to permit placing with minimum segregation.

(4) Compaction shall be Class III.

88 ft/sec => 60 mph

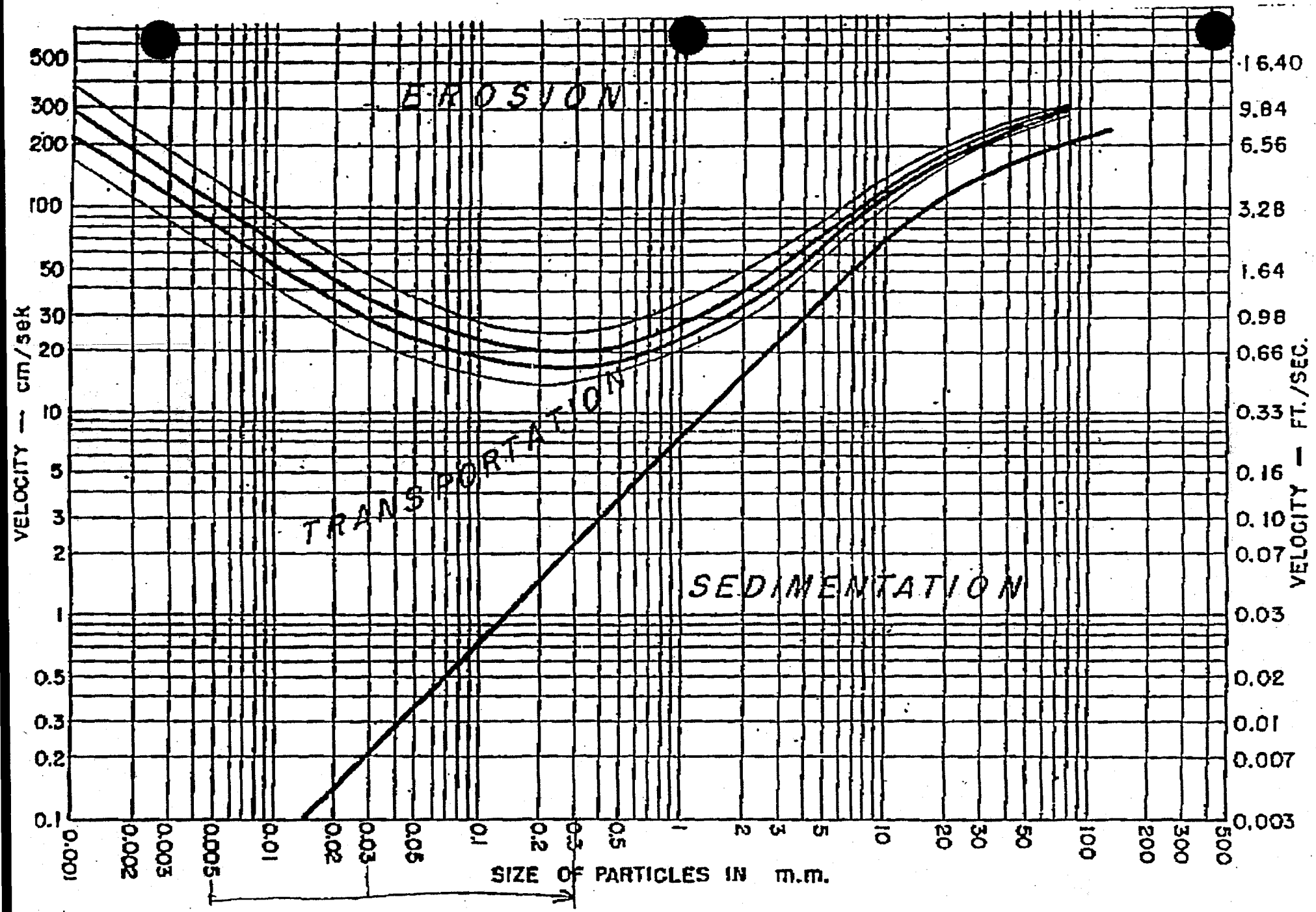



Adapted from: F. Hjulström; Bulletin of Geol. Inst. of Upsala, Vol. XXX and 2. M.D. Campbell and J.H. Lehr; Water Well Technology; 3. S.V. Ishbash; 2nd Conf. on Large Dams, Vol. V, 1936; 4. W.W. Rubey, Amer. Jour. of Sci; Vol. 25, 1933. 5. C.A. Mockmore & J.W.; Dougherty; Vol. 100, Trans. ASCE.

VELOCITY CURVES FOR PIPING, EROSION, AND DEPOSITION OF SOILS

Source: (16)

FIGURE 13.




THE CURVES FOR EROSION AND DEPOSITION OF A UNIFORM MATERIAL

# Foundation Engineering:

## CURRENT PRINCIPLES AND PRACTICES

### Volume 1

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LEGEND

**A.** Alluvial fans at base of mountains (Holocene).
**Z.** Generally thin slopewash or apron fan materials (Holocene).
**I.** Infill fans (Holocene).
**S.** Coalescing alluvial fans along the borders of rivers and major streams (Holocene).
**F.** Channel and floodplain deposits of rivers and intermittent streams (Holocene).
**T.** River terrace deposits (Pliocene).
**B.** Older Quarternary and Tertiary basin fill deposits. Often alluvial fan and lakebed deposits formed in closed basins.
**M.** Mountains.

Note: Depiction and designation of landforms modified from Peterson (47).

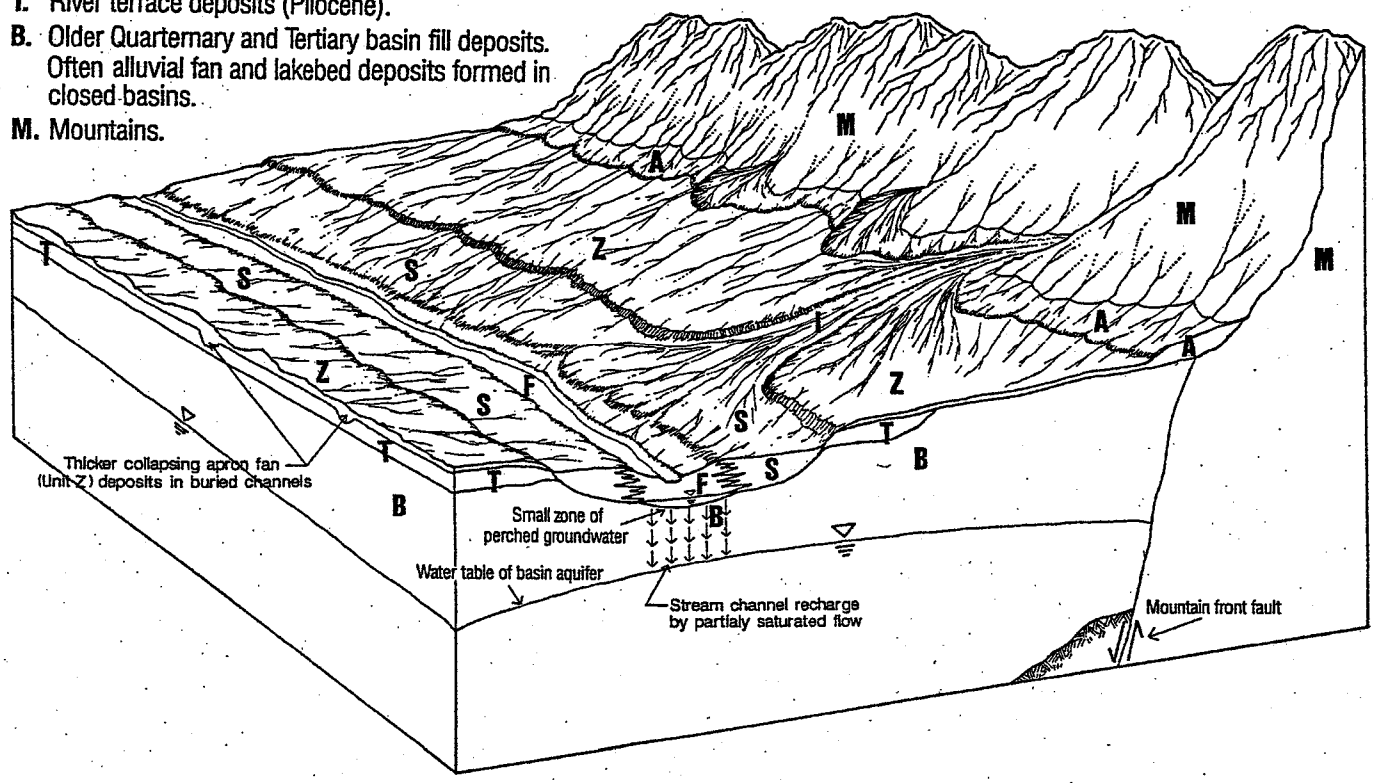


Figure 2. Geologic Setting of Collapsing Alluvial Soil Deposits

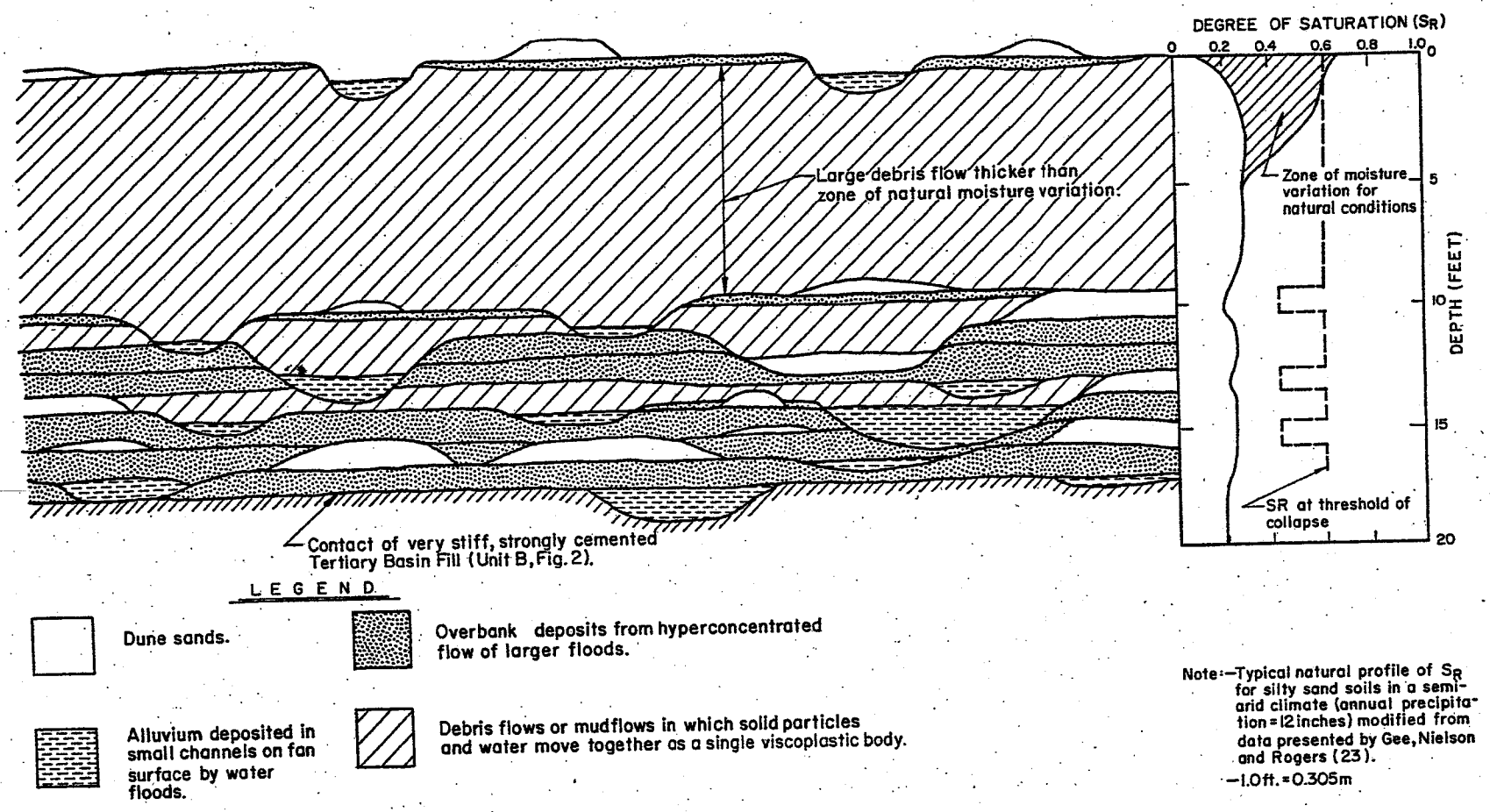


Figure 3. Conceptual Depiction of Features of Collapsing Alluvial Fan - Mudflow - Debris Flow Deposits and Seasonal Variations of Degree of Saturation in a Non-Infiltrating Geologic Profile

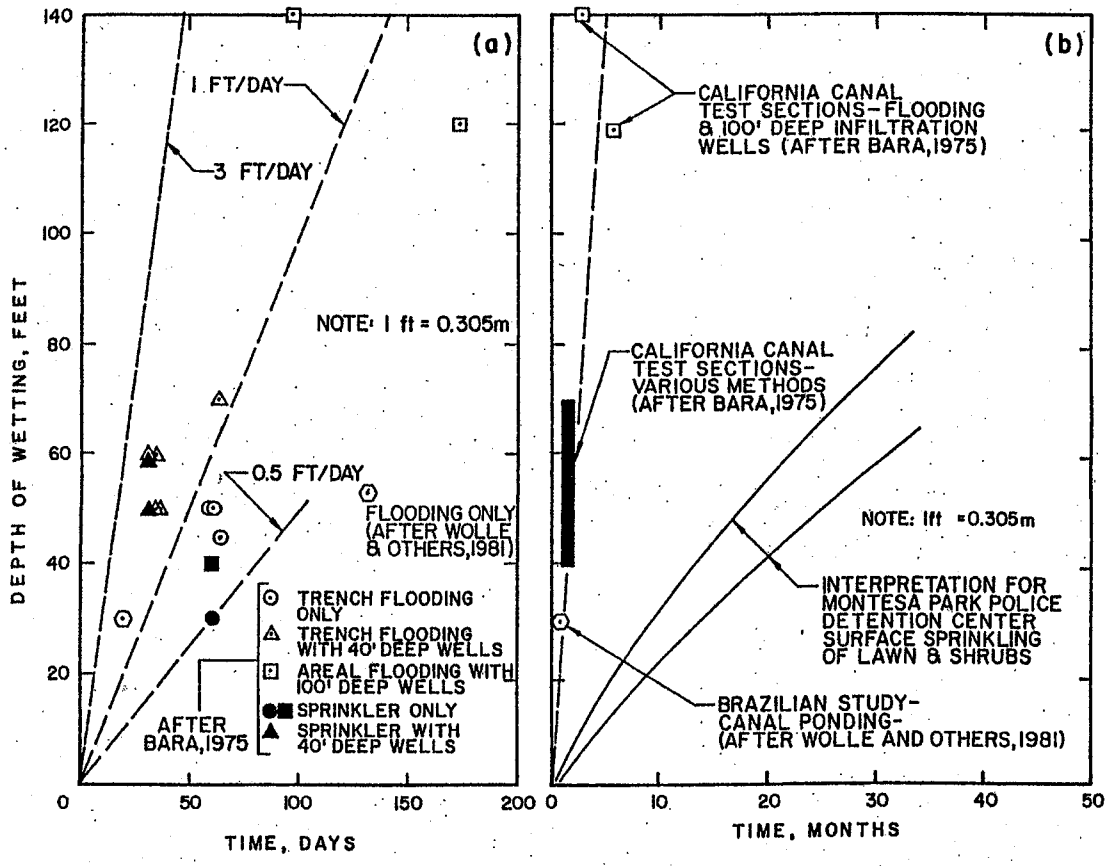


Figure 14. Depth of Wetting Related to Time in Days and Months for Several Prewetting Techniques and for One Site Structure