32.3-05-7-04/85 (Book 1)

# SIGNAL BUTTE F.R.S.

# Design Report

United States Department of Agriculture

Soil Conservation Service

201 E. Indianola Ave.
Suite 200
Phoenix, Arizona 85012

April 17, 1985



Dan Sagramoso
Chief Engineer and General Manager
Flood Control District of Maricopa County
3335 W. Virginia St.
Phoenix, Arizona 85009

RE: Signal Butte FRS/Pass Mountain Diversion & Outlet

Dear Dan:

With this letter we are transmitting five sets of final plans and specifications for the construction of Signal Butte FRS and Pass Mountain Diversion and Outlet.

Signal Butte FRS

Our response to your review comments for Signal Butte FRS was covered in our letter to you dated December 18, 1984. Additional changes from the plans you reviewed include:

1. The geomembrane has been raised to elevation 1720 feet and its manner of installation has been stated in the plans.

2. The anti-seep collars have been replaced with a zone of coarse material called a Filter Diaphragm (new TR-60 requirement).

3. Compaction required is 95% Standard Proctor Density at a moisture content not less than 1% below optimum.

4. The Bureau of Reclamation standard impact basin has been replaced by the Soil Conservation Service standard impact basin at the principal spillway outlet.

5. A drainage system has been added to the emergency spillway and the retaining walls have been redesigned.

Pass Mountain Diversion and Outlet

In response to the comments in the attached letter of May 17, 1984:

Plans

1a. The location of the project may now be located from section corner $\frac{2 \mid 1}{11 \mid 12}$.

Signal Butte FRS/Pass Mountain Diversion & Outlet

Dan Sagramoso

b. Sheet 6 now shows gap.

c. Horse trails are marked as ramp exits at these locations.

d. Acknowledged.

2a. O&M road has been provided.

b. Plans now show this area to be graded as requested.

c. The centerline can be located with respect to the section line and the clearing and grubbing limits can be located with respect to the centerline.

3. Differentiation is made by going to the sheet number shown for details.

4a. These are shown in design:

| | | | |
|---|---|---|---|
| Drop #1 : | 13.9 ac-ft | Retention time: | 1.7 days |
| Drop #2 : | 7.7 ac-ft | Retention time | 1.0 day |
| Drop #3 : | 15.0 ac-ft | Retention time | 2.7 days |

b. Entrance ramps have been provided between drop structures #2 & #3.

c. O&M roads have been made continuous.

d. Energy will be dissapated within the basin.

e. The tractive stresses have checked okay.

5a. Access O&M road has been provided.

b. This will be accomplished from Signal Butte FRS borrow.

Specifications

Item 13: Color has been added to grout.

If you have any questions, please contact Bill Payne at 241-5145.

Sincerely,

Verne M. Bathurst
State Conservationist

MAY 17 1984

Mr. Verne M. Bathurst, State Conservationist
Soil Conservation Service
230 North 1st Avenue
Phoenix, Arizona 85025

RE: Pass Mountain Diversion and Outlet
Buckhorn-Mesa Watershed Project

Dear Mr. Bathurst:

We have reviewed the preliminary construction plans dated February 1984 and the specifications for the referenced project. Please refer to the attached plans for the following comments:

Construction Plans

1. Sheet 2 of 17

   a. Show the location of the project with respect to section corners.
   b. The distance from the eastern edge of the spoil areas to the right-of-way line is about 660 feet while the cross section on Sheet 6 does not show any gap.
   c. The County Parks Department requests crossings or horse trails at the following locations:

      Sta 66+10.00
      Sta 96+20.00
      Sta 115+13.00
      Sta 122+60.00
   d. The spoil areas need to be restored and landscaped after completion of the dike and diversion as part of the revegetation program of the SCS.

2. Sheet 3 of 17

   a. Access to O and M road from Crismon Road need to be provided.

b. On typical cross section, the area between the top of channel bank and the upstream toe of dike need to be graded to drain into the channel.

c. Identify the clearing and grubbing limits with respect to the section line starting from this sheet to be used for relocation of cactus and other vegetation of value.

3. Sheet 4 of 17

Differentiate between vegetative outlets 3 and 5 from 1, 2 and 4..

4. Sheet 6 of 17

a. State the extent of water retention for drop structures.
b. The District requests entrance ramps installed between drop structures No. 2 and 3.
c. O and M roads be made continuous even around drop structures.
d. Changing grouted rock riprap at Sta 112+72 to loose rock riprap seems too abrupt. It is at the end of a curve and on a drop structure.
e. Please check the need for armoring the channel at the curve starting from Sta 110+00 to Sta 11+13.00.

5. Sheet 7 of 17

a. Access to O and M road from Signal Butte Road need to be provided.
b. Grade outlet starting at Sta 137+00 to drain.

Specifications

Item 13

a (5) We recommend color be added to the grout so the structure can blend with surrounding terrain.

We appreciate the opportunity of reviewing these plans and specifications. Please call this office if you have any questions.

Sincerely,

D. E. Sagramoso, P. E.

FERNANDEZ/DET

CO: ~~NPK~~
SLS 5/16

INFO:
RCP
~~RWS~~
RCB

File: SI.3.1

United States
Department of
Agriculture

Soil
Conservation
Service

201 E. Indianola Ave.
Suite 200
Phoenix, AZ 85012



December 18, 1984



Dan Sagramoso, P.E.
Chief Engineer
Flood Control Distict of Maricopa County
3335 West Durango St.
Phoenix, Arizona 85012

RE: Buckhorn-Mesa WPP, Signal Butte FRS, November 5, 1984 Review Comments

Dear Dan:

Thank you for your timely review of the final plans for Signal Butte FRS. Our response has been discussed with Cora Fernandez, of your staff, and the following documents the results:

1. Ramps shall be provided on the downstream side of the dam, as requested. The upstream ramp at station 208+00 will be eliminated. A crossing will be included at Emergency Spillway Station 11+00 for continuity.

2. No rework or seeding will be proposed at the downstream toe of the dam for a 15-foot wide strip. Use can be made of any downstream construction road. Any additional grading, if needed, for an access road could be done by FCD equipment. Access is, also, provided along the wider embankment crest.

3. This dam is four feet wider (18 ft.) than the other dams the SCS has designed in Maricopa County. We will move the guard posts closer to the upstream edge of the dam. Distance from the center of guard posts to edge of bank will read 4'0".

4. The galvanized steel pipe brace is encased in the 24"x24"x12" concrete anchor, as it has been on prior contracts.

5a. The existing channel is insufficient to handle a 100-year flood without the project. Emergency Spillway Hydrograph discharge will be similar to the pre-project condition for a 100-year event. You were provided maps and hydrology information April 3, 1984, completed by Harry Millsaps of my staff. If you need more information you may contact him at 241-2547.

5b. Any flooding at Meridian Road will be approximately the same condition as now exists until the Bulldog Floodway is completed. A new box culvert will replace the present culvert at that time. The centerline elevation of Meridian Road is 1718.5 feet. The elevation of the emergency spillway hydrograph peak is 1715.66 feet, therefore, the road is not expected to flood as a result of storage in the reservoir.

5c. The borrow area drains completely through the gated outlet, which the FCD will operate.

6. No relocation of downguys for Salt River Project is anticipated. That will be noted in the fencing specifications.

7. Cora Fernandez agreed to provide us with example drawings and specifications for the requested stage gage. We are checking whether this can be an SCS cost item at this time. If so, it could be added to the structure later.

Sincerely,

Verne M. Bathurst
State Conservationist

NOV 05 1984

SH 3.1

Mr. Bill Payne, Design Engineer
Soil Conservation Service
201 East Indianola
Phoenix, Arizona 85012

Re: Signal Butte FRS
Buckhorn-Mesa Watershed

Dear Mr. Payne:

We have reviewed the construction plans, specifications and design report for the referenced project. Please refer to the construction plans when reading the following comments:

1. Sheets 2, 3 & 36 (Ramp Location and Details). We request additional ramps on the downstream face at station numbers 202+00 and 208+00.

2. Sheets 2, 3, 4, 5 & 6 - A 15 ft. ± graded strip along the downstream toe of the structure is needed to accomodate vehicles used for the operation and maintenance of the project.

3. Sheet 32 (Reinforced Concrete Gate Stem Pedestal Details). The gate lift pedestal and guard posts need to be moved 1 ft. closer to the edge of the bank. Distance from eastern edge of pedestal to edge of bank to read 2'10". Distance from center of guard posts to edge of bank to read 4'6".

4. Sheet 38 (End or Corner Post Assembly Detail). The 24" x 24" x 12' concrete anchor block should be on top of the 1¼" dia. galvanized steel pipe brace.

5. We have concerns on the following:

   a. The impact of the emergency spillway with downstream properties. Please check if the right-of-way acquired is sufficient to allow discharge from the emergency spillway to revert back to conditions before the installation of the F.R.S.

   b. The flood runoff crossing Meridian Road.

   c. Ponding on the borrow area upstream of the dam can be a breeding ground for mosquitos.

Mr. Bill Payne, Design Engineer
Soil Conservation Service

Page Two

6. Transmission towers owned by the Salt River Project are located downstream of the dam. Some of these towers have downguys that may extend inside the right-of-way. Please adjust the strands of wire for the fencing to accomodate these downguys in order that relocation of these downguys will not be necessary.

7. We request a stage gage be included in the plans and telemetered into the Flood Control District's system. If this is agreeable, the plans should include provisions for installation of the gage. Please contact Tom LaMarche of the FCD's hydrology division for specifications.

Should you have any questions, please call this office.

Sincerely,

Cora Fernandez
Project Engineer

CWF/jnk

Coord: ~~NPK~~
~~RCP~~

Info: ~~REP~~
DRJ
~~RWS~~
SLS 12/14
~~ACE~~
~~JTR~~

File: SH.3.1

# DESIGN REPORT

# FINAL DESIGN REPORT

Job: Signal Butte FRS

Project: Buckhorn-Mesa Watershed

Location: Maricopa County, Arizona

Authority: WF-08

Phase: Final Design

## GENERAL

Signal Butte FRS is a "dry" flood control dam. It is one of a complex system of structures (dams and diversions) intended to divert floodwaters from north of the City of Apache Junction to the Salt River. It is located between Meridian Road (the Maricopa/Pinal county line ) on the east and Signal Butte Road on the west, north of the Salt River Project power lines and Brown Road (Section 12, Township 1N, Range 7E, Gila and Salt River Base and Meridian).

The final design is as anticipated in the work plan with regard to location, alignment and use of caliche borrow material. The dam is higher and has a substantially greater emergency spillway discharge due to the change from a dam to a diversion of the Pass Mountain structure. The end of the dam adjacent to Meridian Road will be designed and constructed with the Bulldog Floodway because of the complexity of natural drainage and the large road culvert to be relocated for Bulldog Floodway. The road needs to be reconstructed at a higher elevation. The location of the present drainage and culvert is not appropriate for the end of Bulldog Floodway and will not be needed after the floodway is constructed. Routing of the design hydrograph without discharge from Bulldog Wash confirms that the present abutment elevation is adequate for this interim condition.

There is no aquifer under this site, therefore, future subsidence and related fissures due to groundwater withdrawal are not anticipated. Not all crack causes are understood so it is possible that the Signal Butte FRS embankment will develop cracks over its 100-year design life.

A product, high density polyethylene (HDPE), has been used as an impermeable barrier by the Environmental Protection Agency, the Bureau of Reclamation, and the Army Corps of Engineers. An HDPE impermeable barrier in the form of 100-mil geomembrane buried in the embankment is proposed to stop flow through any potential crack in the embankment.

Other deviations from the Work Plan were discussed in the Preliminary Design Report dated February 1982.

## REFERENCES

1. National Engineering Handbook 3, Sedimentation

2. National Engineering Handbook 6, Structural Design

3. National Engineering Handbook 20, Specifications for Construction Contracts

4. Technical Release 5, Structural Design of Underground Conduits

5. Technical Release 20, Computer Program for Project Formulation-Hydrology

6. Technical Release 26, The Use of Soils Containing More than 5 Percent Rock Larger Than the No. 4 Sieve

7. Technical Release 46, Gated Outlet Appurtenances, Earth Dams

8. Technical Release 48, Computer Program for Project Formulation-Structure Site Analysis

9. Technical Release 50, Design of Rectangular Structural Channels

10. Technical Release 60, Earth Dams and Reservoirs

11. Soil Mechanics Note 1, Tentative Guide For Determining the Gradation of Filter Materials

12. Soil Mechanics Note 3, Soil Mechanics Considerations for Embankment Drains

13. ICES-LEASE II, Slope Stability Program

14. Manual of Steel Construction, A.I.S.C., 1982

15. Design of Small Canal Structures, USDI, Bureau of Reclamation, Aisenbrey, Hayes, Warren, Winsett and Young, 1978

16. Fundamentals of Geotechnical Analysis, Dunn, Anderson, Kiefer, Utah State University, 1976

17. Handbook of Hydraulics, 5th Ed., King and Brater, McGraw-Hill, 1963

18. "Hydraulic Design of Stilling Basins and Energy Dissipators", USBI, Bureau of Reclamation, Engineering Monograph No. 25, Peterka, 1978

19. "Installation of Flexible Membrane Lining in Mt. Elbert Forebay Reservoir", USDI Bureau of Reclamation REC-ERC-82-2, September, 1981

20. "Laboratory Testing of HDPE Proposed for Use as a Lining in the Mt. Elbert Forebay Reservoir", USDI Bureau of Reclamation Applied Sciences Referral Memorandum No. 79-1-28, R.K. Frobel, 1979

21. Simplified Design of Reinforced Concrete, 3rd Edition, Harry Parker, Wiley & Sons, 1968

22. Schlegel Lining Technology Handbook, and others e.g. Gundle, Staff and Lining Materials.

## LAYOUT AND DESCRIPTION OF JOB

The layout in plan view has not changed, except for dimensions, from the Preliminary design.

The total drainage area has not been changed, but the controlled drainage area is now 5.8 square miles and the uncontrolled area is 10.6 square miles. The structure is a 1.3-mile long earth embankment with a maximum height above original grade at centerline of 38.5 feet. One foot of that is for settlement in case any embankment consolidation takes place.

The principal spillway conduit is a 36-inch diameter reinforced concrete pipe with standard covered riser and Bureau of Reclamation standard impact basin. A gated 12-inch diameter conduit is provided through the dam for complete reservoir drainage.

## HYDRAULIC DESIGN

Spookhill FRS has already been constructed and Signal Butte Floodway is under construction at this time. Therefore, design principal spillway discharge is a given maximum and the increased discharge from Pass Mountain (160 cfs to 3623 cfs) cannot be transmitted to these structures that were not designed to handle it. Various combinations of embankment height and emergency spillway width were investigated (as in preliminary design) with selection of a dam crest elevation of 1721 feet and an emergency spillway width of 140 feet. The preliminary design crest was selected to avoid inclusion of Meridian Road in the construction contract.

The emergency spillway is a reinforced concrete baffled apron chute with a Fujimoto entrance. The freeboard storm discharge has increased from 4091 cfs to 11,126.5 cfs due to Pass Mountain Diversion. The expected 100-year, 24-hour storm discharge to the natural channel where the emergency spillway discharge would enter, without the project, would be 3232 cfs (TR-20). After the Bulldog Floodway is constructed, this channel will have almost no watershed upstream from the emergency spillway outlet and channel intersection. The design emergency spillway storm discharge is 2284 cfs.

In response to the preliminary design review comment that the recessed spillway inlet apron should be moved downstream to the crest of the baffled chute, manual backwater curves do not confirm this recommendation. Since this structure type was meant for a variety of applications including floodway channel and canal grade changes, design MAY require a reduction of flow velocity so that splash at the first set of blocks is minimized. Since our application is at the crest of the total reservoir, a purpose of the inlet is to efficiently INCREASE velocities so as to reduce the peak elevation of the floodwater against the total embankment. The designer's choice of inlet configuration has the required "critical velocity less five feet per second" at the entrance and also minimizes flow depth across the top of the dam. This is not meant to imply that there are not other equally effective inlet configurations that were not selected. (See Bureau of Reclamation design of Small Canal Structures pages 300 and 301 for discussion and example.)

## 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.

The compaction tests on non-cemented materials varied in 100% Std. Proctor density from 124.5 pcf to 126.5 pcf, remarkably consistent. When compacted to a density of 112 pcf these soils were strong and competent for this embankment.

Some caliche samples were tested to determine whether they gained strength over time. All samples gained stength (unconfined compression test) over the 28-day span of the test, but the time of gain varied. Sample 81W353 from the borrow area failed at 8 psi immediately after compaction and 12.7 psi after 28 days. This is a gain in strength of nearly 60%. This sample is 14% calcium carbonate by weight.

Most of the soil mechanics reports included in the final data are from the Pass Mountain site. Signal Butte FRS began as a structure with all borrow from its own reservoir. Then it was to be almost entirely constructed from excess materials from excavation of Pass Mountain Diversion and Outlet (500,000 ± cubic yards). Since this material will presumably come from the "outlet" immediately up-slope from the Signal Butte site, the only use for this laboratory information is for the reviewer to be able to see where some of the samples came from. The extra tests used samples from both sites.

Since the soil materials at the Signal Butte site are strong, even at less than 90% Standard Proctor Density, and a relatively impermeable material (HDPE geomembrane) will provide a positive barrier to any potential seepage, it does not appear necessary to make compaction control a major construction item. In an effort to simplify inspection requirements, a minimum density of 112 pounds per cubic foot has been specified. Any soil material that cannot be compacted to that density with Standard Proctor effort is probably not moist enough for a "workable mix" or is composed of a higher percentage of non-soil chemicals than any tested in the Laboratory. If the former, it is up to the contractor to add water. If the latter, the material should not be used in the fill because we have no information on its behavior (the chemicals may be soluble).

## WNTC-RECOMMENDED CRACK-PROTECTION ZONE

Selection of a granular fill material to protect against future cracks in this embankment is difficult. "Vertical Drains and Embankment Zones" by Clarence Dennis, often quoted in past Arizona design reports, recommended that the material in a zone to protect against cracks be coarser than the #4 sieve "so that water film interference will be negligible". He said this material "must have a maximum size large enough so that it will lodge in any conceivable crack...", that it "must be well graded" and "must have a width sufficient that the inverted filter will build well within the body of the drain material". His MINIMUM recommeded criteria for the protective zone was:

| | |
|---|---|
| Slope | 1:1 (desired vertical) |
| Cu | more than 5 |
| Cc | 1 to 3 |
| Minimum $D_{85}$ of 4 inches (prefers 6 inches) | |
| Minimum $D_5$ greater than or equal to the #200 sieve | |
| Minimum $D_{15}$ greater than or equal to the #100 (prefers greater than or equal to #4 sieve) | |
| Width | 0.2 x Height of dam with a 10-foot minimum |

According to the letter the WNTC sent to Arizona February 6, 1980, the crack protection zone must meet filter requirements for the embankment material. On the other hand, the minimum D-50 size of the filter must be equal to or greater than one-half the crack width and the minimum D-75 size of the filter must be equal to or greater than the crack width. Any (future) crack larger than protected by that criteria will be protected by an "outlet".

This is merely to point out that referenced criteria are conflicting and inadequate for design of a new structure.

This structure's borrow material is composed of varying quantities of calcium carbonate (a cementing agent). This material (and associated salts) has some mobility within the soil matrix. It is possible that, over the design life of this project, a coarse-grained zone could become partially cemented.

HIGH DENSITY POLYETHYLENE GEOMEMBRANE

Soils at the Signal Butte site (and Pass Mountain Outlet) are less cohesive than the soils that have apparently dominated other project dry dams in this part of Arizona. (Signal Butte tested soils averaged 27% fines and the fines were predominantly non-plastic.) It is possible that the dam will not crack as severely as those that have finer and more plastic soils in them. The Signal Butte site is high enough on the alluvial fan that it does not have a watertable to decline and cause deep subsidence. On the other hand, this lack of cohesive fines makes the formation of a "filter cake" or self-healing" of the cracks by expansion of the adjacent soil materials unlikely. The caliche may make the soils stronger and less erosive, but will probably make the structure more brittle, too.

Assuming that cracks are a possible hazard, a barrier, or crack curtain, made up of a 100-mil, high density polyethylene (HDPE) geomembrane will be installed in the middle of the dam.

The crack curtain is to be a continuous (welded) geomembrane from the bottom of the cutoff trench to elevation 1718 feet (MSL) at any given station. At that elevation the membrane is above the crest of the emergency spillway hydrograph and below the depth of any weather, temperature or rodent attack. The membrane will be a continuous (welded) geomembrane from one end of the dam to the other. Horizontal welds will probably be necessary to complete this installation. HDPE extrusion welds are stronger than the geomembrane itself since they are composed of the same material with a larger cross-sectional area.

This thickness of HDPE is very tough and stiff. Delivery and installation are the most hazardous conditions for damage to the material. A careful visual inspection of the geomembrane as it is installed/buried will be sufficient control to permit timely repair and assure that the final installation meets the design intent.

The dam is not very high, so the pressure of water against the membrane will not stress this material. Any small hole would discharge water at a "point" location within the mass of the embankment. The most water pressure possible is approximately 15 psi. It is not expected that this point discharge would damage the embankment. If the water seeps under the membrane, it will have to move up and some distance laterally to exit the downstream toe. In fact, since there is no water table and caliche is generally permeable, there is no barrier to deep seepage that would cause uplift pressures to develop under the dam.

## DESIGN OF STRUCTURES

The Principal Spillway inlet, conduit and outlet were (Preliminary Design) and are standard structures. We did not have a mylar of the inlet, so the layout sheet had been copied from NATIONAL ENGINEERING STANDARD DRAWINGS, Midwest RTSC, August, 1973.

The impact basin is a standard drawing from the Bureau of Reclamation that we have verbal WNTC approval to use. It is a simpler structure than the comparable SCS standard structure and requires less concrete. Neither the site or the application are particularly severe. No drainage is necessary, since 1) this site will be very dry except for short periods of time, 2) there is no watertable and 3) the Principal Spillway outlets the dam perpendicular to the natural slope of the alluvial fan they will be constructed on. We are, however, not satisfied with the clarity of the steel placement in the drawing. The one-sheet standard has been redrawn with one layout and dimension sheet and another sheet for steel details. The only other change is an increase in bar size to meet SCS T&S requirements.

## APPURTENANCES

The Gated Outlet structure was designed using TR-46. The standard inlet has been modified to fit the 2 1/2:1 upstream slope. The PWD Basin is standard and will be adequate for expected flows. It is assumed that the gate will NOT be open during a freeboard event. There are two purposes for a gated outlet: to provide a means of completely draining the sediment pool upstream and to provide water to native vegetation downstream. No additional water would be needed or wanted downstream by the time freeboard storm conditions existed. The structure will be maintained by the Flood Control District of Maricopa County. The FCDMC can be depended upon to operate the gate responsibly.

The 12-inch conduit is the same design as the ones in Saddleback FRS. TR-46, Figure E-2, calls for only #4 bars at 12 inches, we have called for #5's to meet SCS T&S requirements. Since the Gated Outlet is not really needed for the dam to function according to its intended purpose, it has been categorized a "water supply pipe" for the purposes of TR-60, and size is not a factor for design. This was discussed with WNTC shortly following the Preliminary Design Review with concurrence.

Anti-seep collars are INCLUDED on the Principal Spillway conduit, as required. (See request for deviation and response in correspondence.) The reason they were left on the Gated Outlet, but not the Principal Spillway was in response to their different attitudes to natural seepage direction and their much different elevations with respect to the bottom of the reservoir.

The crack protection was NOT shown enclosing the Principal Spillway and Outlet conduits and should have been. That has been corrected in the drawings. The Principal Spillway conduit trench has been shown, also.

MISCELLANEOUS

A diversion is to be excavated just north of the left abutment at Meridian Road to keep runoff from the road culvert away from the end of the dam.

There is a substantial quantity of vegetation in and near the major washes in the east half of the impoundment area. This vegetation includes mature trees and cacti and is NOT to be disturbed by construction. It provides screening and visual interest from Meridian Road and homes northeast of the dam. This vegetation is not expected to produce woody trash that might cause problems with performance of the principal spillway. Some additional maintenance may be necessary at the gated outlet. When the proposed A&E Landscape contract is performed, decisions can be made about any selective removal of vegetation.

Submitted: Suzanne Heckband Date: 7-31-84
Civil Engineer

Approved: William E. [illegible], Jr. Date: 8/2/84
State Design Engineer

# CORRESPONDENCE & REFERENCES

MEMO TO THE FILES: January 10, 1979

SIGNAL BUTTE FRS ALIGNMENT

Signal Butte Floodway has been relocated from its planned alignment. This relocation caused a change in invert elevation at the principal spillway for Signal Butte FRS and a new location for that principal spillway.

The Signal Butte FRS was designed as a flood control dam with some aerated sediment storage. There are no provisions in the work plan for draining the sediment pool. It is assumed that an outlet to drain the sediment pool was to be included as part of the principal spillway structure. The principal spillway outlet elevation is given as 1698.0. The new invert elevation for Signal Butte Floodway is 1687.0. The old invert elevation was 1682.7.

The following alternatives are available for design:

1. Leave the dam in its presently planned location with an inlet channel from the low areas. The planned principal spillway elevation would still be 1698.0, eleven feet above the floodway invert. Total dam would not drain.
2. Leave the dam in its presently planned location and provide drainage of the sediment pool through a vegetative outlet conduit. Some drainage channel upstream of the dam would still be required for adequate drainage. The principal spillway would outlet into the floodway independent of the vegetative outlet.
3. Move the dam upstream of its presently planned location so that it can be fully drained through the principal spillway to the floodway. This alternative will almost certainly increase the planned height of fill, change the emergency spillway location, and affect the location of the Bulldog Floodway.

The Flood Control District of Maricopa County has indicated a desire for a vegetative outlet from Signal Butte FRS. This would eliminate the problem of an aerated sediment pool. Alternate 3 seems to cause a number of potential changes from the original work plan. Therefore, alternate 2 is recommended.

Susanne Leckband

Susanne Leckband

Paul accepts recom. alt 2
clarify definition of aerated
sediment w/ planning (John Womack
Aubrey). SML 1-12-79

R. Arrington, STC, SCS, Phoen..

United States Department of Agriculture

Soil Conservation Service

West National Technical Center
511 NW Broadway, Room 510
Portland, Oregon 97209

Orig - Paxter File

Subject: ENG - Review of Geologic Report, Signal Butte FRS, Buckhorn-Mesa Watershed, Arizona

Date: March 29, 1982

To: Aubrey Sanders, Jr., State Geologist, SCS, Phoenix, Arizona

You and Ron have done a good job on your Signal Butte report. I have some comments that may also apply to future reports.

Page 1. Is pediment the right term here? If there is some confusion on the usage of pediment (or piedmont), maybe simply saying the site is on the lower end of a fan would be good enough.

Page 2. We are always glad to see lots of blow count data, which you have. I would suggest putting it beside the logs on the profiles for quick reference by the designers. For simplicity, we need plot only the blows for 1 foot. If the designer is interested in a specific area, he will look up the log to get details on blow count for each 6 inches and the other comments in the log.

In all the discussions of the caliche, no mention is made of the cementing agent--is it carbonate or silica? Any comments as to it's solubility?

Page 5. Borrow - You need 540,000 c.y. for the dam and you say about 500,000 c.y. is available, including a 20% shrinkage factor. I suggest you recommend an additional source, say about 30% more as a reserve. Ripping the caliche may be too expensive in comparison to going to another borrow area.

Can the borrow area be zoned? If so, a map would be needed. If not, we are assuming the dam will be made of mixed SM, SC, and ML (homogeneous).

If a drain is in the plans, do we have a source for drain material?

Logs. Logs that are plotted on your profiles sometimes lack the soil breaks shown on the log sheet. You may want to check this out. See hole 2-27 on sheet 2 of 5 as an example.

Figures. Plan and profile sheets. You can add considerably to the usefullness of these sheets if you were to highlight, darken, or color the caliche. It doesn't appear to be reasonable to try and correlate caliche contacts or individual soils from hole to hole. However, it appears the caliche will be a major consideration in the design of this dam, so highlighting it will help the designer when he considers stripping depths, foundation excavation, and cut-off depths. I would do this on all sheets where the logs are plotted, including the borrow area.

Recommendations. Some of the recommendations are design decisions and are best left to the designers. Number 1 is an example and Number 4 is another.

Keep up the good work.

C. E. Stearns

C. E. STEARNS
Engineering Geologist

cc:
Ralph Arrington, State Conservationist, SCS, Phoenix, Arizona

Memo to Files
Re: Subsidence potential at Signal Butte FRS

Groundwater which underlies the Signal Butte FRS is contained within fractures and cleavage planes of the granitic bedrock which underlies the site. There is no evidence that the water table in the area has ever been high enough to saturate the alluvium. Practically all of the local residents resort to hauling water for household use because they cannot obtain water in sufficient quantities from drilled wells. Lowering of the water table in the granitic bedrock should have no impact in regard to subsidence potential.

Anthony C Sanders Jr Geologist 8/17/82

# 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.

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 <u>in place</u> 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 <u>3 feet</u> from each other. Cracks in this dam <u>do</u> 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.

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 <u>settled</u> 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 1½ feet 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."

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.)

## 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


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    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

MAGMA DAM
REPAIR ALTERNATIVES
4-22-82

## Purpose

Magma Dam was to be repaired with the same method for crack repair that has been used before and is currently being installed on White Tanks No. 3 and No. 4. In this way, design schedules permitting a construction contract in fiscal year 1982 could be met.

This repair method consists of excavating a trench in the center of the dam to approximately three feet below the existing cracks and backfilling the trench with a granular material. Where cracks exceed a certain design width on the downstream side of the trench, an outlet trench filled with drainfill material is installed to safely convey any drainage to the downstream toe of the dam.

Investigation of Magma Dam and its environs has caused the designer to question the wisdom of proceeding with this repair method. The designer respectfully requests consideration of the following information:

## Magma Dam

Magma Retarding Dam was constructed in 1963 and 1964. It was completed in August of 1964. It is 28,623 feet (5.42 miles) long. The maximum height is less than 27 feet, which includes 0.5 feet for consolidation. (This consolidation has not occurred.) Approximately 2.3 miles of the dam are between 13 and 16.5 feet high and less than one mile is more than 16.5 feet high. The principal spillway is a 39-inch reinforced concrete pipe with a baffled outlet. The emergency spillway is a 150-foot wide earth channel at the left (southeast) end of the dam. The core trench was approximately 5 feet deep and located upstream of the centerline of dam.

A survey of the dam was just completed and tied to a Bureau of Reclamation, Central Arizona Project, subsidence monument that was surveyed December of 1981. The benchmark on the dam was supposed to be elevation 1605.1. SCS surveyors determined a present elevation of 1605.05 feet. The Bureau of Reclamation's monument, updated annually, has subsided an apparent 0.006 feet since 1971. Preliminary evaluation of the top of dam survey shows no appreciable difference in structure height from what was designed.

Storms in 1972 filled the reservoir nearly to the crest of the emergency spillway. This resulted in reduced damages of approximately $1,100,000 (1972). The dam and upper channel cost $419,924 in August of 1964. Runoff from numerous smaller storms has been controlled by Magma Dam. (1)

## Cracking

The first report of cracks in Magma Dam embankment was a diary entry by Turner (SCE) on July 30, 1965. He commented that transverse cracking was prevalent. W. R. Stanley (WTSC) made a construction inspection October 11-12, 1965, and commented in his trip report on the longitudinal cracking on Magma Dam and the absence of transverse cracking. 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. Dan. R. Lawrence and W. Bill 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.

Apparently, the first trenching to look at cracks was done in August of 1977 and reported by Dan Lawrence (AWC). The next trenching reported was by Ralph Arrington and Paul Pedone (SCS), June 1, 1978. At that time they dug two trenches to investigate a crack approximately forty feet upstream from the embankment. This crack was more than 14 and 16 feet deep respectively. It is the same crack that was investigated by Fugro ("Off-Structure Crack Investigation"). It is roughly parallel to the embankment near Station 127+39 and 72 feet long. The crack has no vertical offset, even where it passes through a calcium carbonate horizon at about 3.5 feet deep. The designer observed a crack located approximately 600 feet upstream from Station 45+00 in moist soil. Alternating crack and suspect subsidence features were noted for more than 50 feet. The alignment would intersect the dam just north of the principal spillway. The crack has not been investigated for depth or total length.

## Soils

The soil mechanics testing for Magma Dam was reported January 29, 1963 from the SCS lab at Portland. Thirty-eight samples were tested, most from the borrow area which parallels the dam 300 feet upstream. The borrow excavation opens a channel to drain runoff to the principal spillway.

Seven samples were undisturbed; the balance disturbed. The undisturbed samples were identified (mechanical analysis, Atterburgs, specific gravity, moisture content, unit weight, and remarks); then triaxial or quick consolidated shear tests, consolidation tests, and permeability tests were performed. Identification, compaction, permeability and shear tests were performed on most of the disturbed samples. Most of the samples reacted positively to hydrochloric acid. Two samples showed positive reaction to benzidine (montmorillonite clay). Both were on the principal spillway centerline. There is no indication as to why these samples were tested with benzidine and no comment as to whether other samples were similarly tested. Many samples flocculated.

Most of the soils tested were classified CL or CL-ML (Unified). ML's plotted just under the A-line (Atterburg limits) except for sample 103.3 which was non-plastic with 40% between 0.074 mm and 0.4 mm and 53% passing the #200 sieve. Sample 5.1 was an SM-SC with 49% fines. Sample 103.2 was a CL-ML with a dry density of 119.5 $\#/ft^3$ at 12% optimum moisture, liquid limit of 21, PI of 4, 60% passing the #200 sieve and 85% finer than 0.2 mm. Sample 104.3 was an SM-1 with 40% fines, a liquid limit of 40%, a PI of 8, a maximum dry density of 101#/ft$^3$ at an optimum moisture of 19.5%. Sample 107.2 was an SM-2 with 49% fines. At 50% fines it would have classified CL. Sample 107.3 was an SM-1 with 14% passing the #200 sieve. It was non-plastic with a maximum dry density of 108 #/ft$^3$ at 17% optimum moisture. Sample 111.2 was a CL-2 with a liquid limit of 49%, PI of 27, maximum dry density of 100 #/ft$^3$ at 21% moisture. Sample 114 A-2 was an SC with a liquid limit of 50%, PI of 34 and 47% fines, nearly a CH. Many of these samples had nearly or just over 50% fines. Atterburg limits usually plotted close to the A-line. Liquid limits seemed high.

A disturbed soil sample was taken just downstream from the principal spillway because of problems at the drop structure during the first storm discharge. The soil was classified CL-2 with 62% fines, liquid limit of 36%, PI of 16, shrinkage limit of 20.3% volumetric change of 35.3% and positive reaction to HCL.

The Eastern Maricopa and Northern Pinal Counties Area soil survey report overlaps a very small section of the dam, but generally stops downstream. The probable soil series to be found at the dam are Vecont (CH), Contine (CH and CL), and Mohall (ML or CL). Information about the Contine minerology is not available in this office. Vecont's clay minerology is made up of abundant mica fines, moderate amounts of montmorillonite fines and small to moderate amounts of Kaolinite fines. These were from two pedons. Clay fines (<.002 mm) were 48% in one and 42% in the other. The Mohall clay minerology has moderate to abundant montmorillonite to about five feet, then montmorillonite clay fines are dominant. Mica fines are small to abundant to just over two feet deep, then drop to small amounts. Kaolinite fines are a small to trace amount of the whole. Clay fines (<.002 mm) make up 20 to 36% of the total sample with the maximum between 2 and 3 feet deep. A soil surveyor from the Casa Grande field office tentatively identified Vecont in the reservoir upstream from the principal spillway (southeast end of dam) and Mohall in the reservoir along the central part of the reservoir. There is a very sandy soil at the northwest end of the dam.

Ten samples of soil materials were collected March 29, 1982, with a tile spade and soil survey power auger. Soils at the south end of the reservoir were very moist. Soils at the central part of the reservoir were damp to moist. It rained at the site within three days of sampling.

Erosion patterns, both in the reservoir and in the dam, are not typical of clays. The samples would be typical of materials used to construct the present dam and also borrow for any future earthfill used in repair of the dam. Identification tests, pinhole dispersion tests, and clay minerology were requested. It was requested that liquid limits be recorded even if soils were non-plastic.

Not all testing is complete at this time. Tests that have been completed (telephone conversation with Lorn Dunnigan) show that all samples were CL by the Unified Soil Classification System, though one sample was only 51% fines. Sample A was clearly dispersive in the double hydrometer dispersion test (81%). Sample C was questionable. Sample C has 79% fines, highest of the ten samples. Liquid limits ranged from 25 to 43 percent. Plastic indexes ranged from 8 to 25.

As far as construction of Magma Dam is concerned, the moisture control called for a workable mix. Review of the Weekly Summary of Density Determinations from April 6, 1964 to June 26, 1964 offered the following information:

1. Optimum moisture varied from 11.8 to 21.7%.

2. Standard Proctor compactions over 100% were not uncommon.

3. Actual compacted dry densities varied from 98.4 $\#/ft^3$ to 125.1 $\#/ft^3$.

4. The average compacted dry density for approximately 360 samples was 108.45 $\#/ft^3$.

5. Moisture contents tended to be lower as the job progressed into summer.

6. Of all the samples, 100 were above optimum moisture, 216 were below optimum moisture, and the balance were right at optimum moisture content. (This is in conflict with the opinion of various reviewers that cracking was caused or aggravated by soils being placed wet of optimum. This would be possible where soils were compacted to more than 100% Standard Proctor, but not true of most of the embankment.)

Bill Cutter, an SCS technician who was an inspector during the construction of Magma Dam, remembers some trouble with wet soils in the early part of the job, but later the soils were relatively dry. He also remembers the failure of backfill adjacent to two drop structures downstream from the dam on the outlet channel. These are the subject of the "Report of Drop Structures on Magma WPP" by J. J. Turner, State Conservation Engineer, August 20, 1965. Bill's recollection was that one of the structures had an area of very poor backfill compaction, but the other failure was not so easily understood. He recalls that backfill for these structures came from the Magma borrow area. Cohesive materials were considered desirable compared to the adjacent fine sands and silts, apparently. The nearest borrow area would have been in the vicinity of the dispersive clay sample mentioned above. The mode of failure, subsurface piping, would be consistent with a dispersive clay failure.

Fissures

Most authorities agree that fissures in Arizona are directly related to groundwater withdrawal and the subsurface topography of incompressible rock under the aquifers. This is not to imply that they agree on all the mechanisms involved beyond that. There appears to be a direct relationship between known fissures and subsurface mountains, ridges, and faults.

A fissure is a deep crack. Some of these cracks can be traced for miles on the ground surface. Cracks have been investigated up to 200 feet deep. They may or may not show vertical displacement. (Highway I-10 between Phoenix and Tucson north of Picacho Peak has been repaired numerous times due to cracking and vertical displacement of approximately 1.5 feet at this time.) Cracks that show up in residential areas and cropland are generally disced or graded over. Where water has access to a fissure crack in erosive soils, a "fissure gully" forms. This gully will follow the alignment of the fissure regardless of topography and may be 10 or 20 feet deep. Fissure gullies, according to Dick Raymond of the Bureau of Reclamation on assignment with the USGS, often begin with a subsurface void formed by piping of an erosive soil material into or along the fissure crack. He has seen voids as deep as 22 feet under the surface. Eventually these voids erode or collapse, leaving the surface feature identified as a fissure gully. Water added to the fissure near Apache Junction, was temporarily prevented from moving down with a gravel bed on the bottom of the gully. The water proceeded laterally into the cracks adjacent to the backhoe trench, eroded a hole progressing from the inlet, which collapsed to the surface. The gravel bed settled approximately two feet during this test and water eventually continued moving down into the crack around the side of the gravel until the pit was empty. (Discharge to the hole was approximately 500 gpm.)

Subsurface terrain has been mapped with gravity meters, well logs, and deep seismic refraction. The best method depends somewhat on what the subsurface geology consists of. Granite basement rock overlain by loose alluvial materials is mapped relatively well with gravity meters. Gravity meters do not differentiate well between conglomerates that do consolidate and volcanic flows that do not consolidate. These are better "seen" with seismic refraction. Well logs are used to confirm other studies. Depending on the experience and thoroughness of the well driller's logs, this is probably the most reliable subsurface information. Unfortunately, it is only accurate for one location. Correlation between wells even 1/4-mile apart is not always good.

The Soil Conservation Service is fortunate to have some seismic refraction information on Magma Dam. This was the result of Central Arizona Project work immediately downstream. Due to access difficulties (wet impoundment area, trees, and gullies) the DAM A-DAM B profile does not intersect the PIPE profile and the MUD profile is at an odd angle to the dam. The profile locations are as shown on the attached location map. As you can see, the south two miles of embankment were excluded and there are no verifying profiles perpendicular to the dam. Lee Pankratz, USGS geophysicist, sent partial results of this survey, copy attached, to Aubrey Sanders May 8, 1981. Lee was contacted to confirm his comment about potential fissuring along Magma Dam (telephone conversation April 9, 1982). He anticipates potential fissuring due to basement rock 1) in Sections 16 and 21 on the northwest third of the DAM A-DAM B profile and 2) at the southeast end of the DAM A-DAM B profile (at the approximate 45° bend in the dam).

Tentative contours of the basement geology have been drawn and are available <u>as a courtesy</u> to look at, at the USGS office in Phoenix, Arizona. Copies are not available, since the material has not passed through that agency's review procedures. Herb Schumann has made this information available to the designer and offered explanation of what was shown. He is, also, a Bureau of Reclamation employee on assignment to USGS to identify geologic hazards along the Central Arizona Project, specifically the Salt-Gila Aqueduct extending from Phoenix to Tucson. In conversation with him April 13, 1982, while viewing these tentative contours, he said: "the basin fill alluvial sediments appear to be thickening rapidly to the northeast, especially at the north end of the dam. The surface that these alluvial sediments rest on consists of relatively incompressible consolidated sediments and basaltic volcanics. At the north end of the dam, seismic data indicate that these materials form an irregular surface upon which the alluvium was deposited. Fissures have occurred in areas having similar geologic conditions when large-scale waterlevel declines occurred (more than 100 feet)." The nearest well log upstream from the dam, along Magma Railroad, showed a 250-foot decline in 1977. (He offered the quote and the well information.)

The USGS people this designer has talked to agreed that: 1) Magma's subsurface geology is complex; 2) more seismic refraction work is needed before the dam is relocated (It is Lee Pankratz and Bob Laney's expressed opinion that the north end of Magma Dam should be moved -- probably to the northeast); and 3) gravity meter analysis is not adequate at this complex site. If Magma Dam is, in fact, resting on top of a buried ridge of incompressible material, it would explain why the dam does not appear to have subsided at all in nearly twenty years even though there is at least 1.5 feet of subsidence measured within three miles downstream from the dam along the CAP survey line.

"Cracking of Dams in Arizona," the Report of the Crack Study Team, April 27, 1978, has a summary of subsidence information (pages 3-8). Unfortunately, new subsidence locations are still being classified. Paradise Valley, for instance. Page 6 of this report states that "Should the fissure occur within about 500 feet up or downstream, the dam would be unserviceable." The implication of this report is that the dam should be breached in that case.

Repair Methods

Several methods of repair of cracked sections of dams were discussed and summarized in ~~the above report.~~ None of these were proposed as solutions for fissure cracks. They included:

1. Sand and gravel filter (preferred)

2. Narrow, reworked compacted earth core (will probably crack again)

3. Narrow, soil cement core (too rigid)

4. Continuing program of cleaning and mud grouting (undependable, for more reasons than this report mentioned)

5.& 6. Cloth filter (unproven material, probably would tear and, in conjunction with granular material, be more expensive)

7.& 8. Lowering emergency spillways and installing floodgates (change scope of original project)

9. Segmenting the detention area with dikes (has merit in conjunction with other repair measures)

10. Combining repair with construction of the CAP (not viable because of unknown timing)

Methods of construction of dams to control cracking were also proposed and discussed. They included:

1. Granular filter zone within the embankment. A filter face will stop migration of fines and promote self-healing of the crack.

   1a. Compacting the embankment dry of optimum combined with a granular filter zone. Same as above with predicted smaller cracks from shrinkage due to drying.

2. Granular embankment shells. The 5 to 8-foot thick shell of granular materials will break capillary rise and insulate the dam from the heat.

3. Sand wick and irrigation system. Supplemental moisture (dry irrigation) will replace moisture lost by drying and keep the core of the dam at or above field capacity to prevent the buildup of high capillary stresses.

4. Install a vapor barrier over and around a core section of the embankment. The team believed this held promise in specific locations. The vapor barrier would be "12-mil , or thicker plastic, rubber sheeting, or other long-life watertight material". "Unless properly installed, the vapor barrier could be subject to tearing as the outer portions of the embankment crack. A thin sand section next to the barrier would provide protection from tearing." "Although the vapor barrier will also function as a diaphragm in the embankment, that is not its principal purpose."

Segmenting the detention area with dikes is a solution that has considerable merit with regard to fissure cracks. It does change the scope of the project, but still affords the sponsors some flood protection. A very similar solution would be a series of detention dikes similar to those the Bureau of Reclamation has constructed adjacent to their canal. These do not provide the same level of protection as a dam, but are not as great a hazard if they fail, either. Magma Dam could be replaced with a series of smaller dams in the upper sub-watersheds. This solution would require considerable study and land rights, but is feasible.

The option of doing nothing is not available. The Arizona Department of Water Resources is waiting for a repair proposal.

The obvious inadequacy of thin plastic membranes has apparently precluded their serious consideration. However, dams and public water supplies have been constructed of heavy membranes such as butyl rubber. A product that caught the attention of the designer several months ago is high-density polyethylene in 80 and 100-mil thicknesses. A similar polyethyelene chemistry material covers cables used by power companies that have been exposed to sunlight for 35 years.

Recent research done at the Lincoln Soil Mechanics Laboratory suggests that the filter gradations we have used in the past are too rigid and substantiates the formation of a filter cake on the upstream face of the filter zone when water is present. The details of this study are not yet available to this designer; therefore, specific evaluation for repair of Magma is not yet possible.

The designer's preferred method of repair is a combination of filter material and polyethylene diaphragm, as shown in Figure 1. The filter material at the upstream base of the diaphragm should seal any flow under the base if a crack should pass under the dam at the foundation. There should be enough material to stop or slow discharge even if a small void eroded under the diaphragm. Water moving in cracks upstream from the diaphragm would be stopped. The only way that water could circumvent this barrier would be if an erosion-prone horizon passed under the barrier without contact and upstream water had access to this horizon in sufficient volume to leave a void under the entire section of the dam. If this happened, the upstream and downstream embankment at that location would probably be destroyed. The diaphragm would be suspended above the exit location and would provide some defense to the adjacent embankment. If such a failure occurred, the dam would probably be breached at that location (assuming a fissure). The diaphragm would prevent a "sudden wall of water" but could not prevent a fissure from occurring. The 80-mil polyethylene will withstand pressure from water 103.6 feet deep before it will elongate more than 15%. Nowhere does the dam approach such a height.

Philosophy

While there is mounting evidence of the relationship between subsurface incompressible features and the location of fissures, the expense of reliable subsurface mapping precludes assurance that every potential problem area will be known in advance. Where potential areas are known, the exact location of the crack is still an unknown. Where one crack appears, within a few years a parallel crack is often located. Will there be a series of parallel cracks someday?

Even if the crack upstream from Magma (forty feet) is not a fissure, there is substantial evidence of the possibility for one in the future. This repair must, in the opinion of the designer, address the possibility.

Would the failure of Magma Dam result in loss of life? There are two farmsteads close enough to Magma Dam to be vulnerable. (These have been considered in the breach layout shown in Figure 2.)

At what point in the decision-making process for Magma Dam repair are the dam's owners consulted? The repairs could cost 1.5 to 2 million dollars. The repairs under discussion have only addressed the crack problem. The Corps of Engineers' Phase I Dam Safety Report also says the dam is unsafe due to insufficient emergency spillway capacity.

Conclusion

Magma Dam has 1) extreme potential for a fissure and 2) existing embankment cracks. The soils with which it was constructed show dispersive characteristics.

There appear to be three options available at this time.

1. Breach the dam.

2. Construct a test of the recommended repair method to verify its performance and accept the possibility of partial failure at some future time.

3. Repair the dam in the same manner that other dams without dispersive clays and fissures have been repaired.

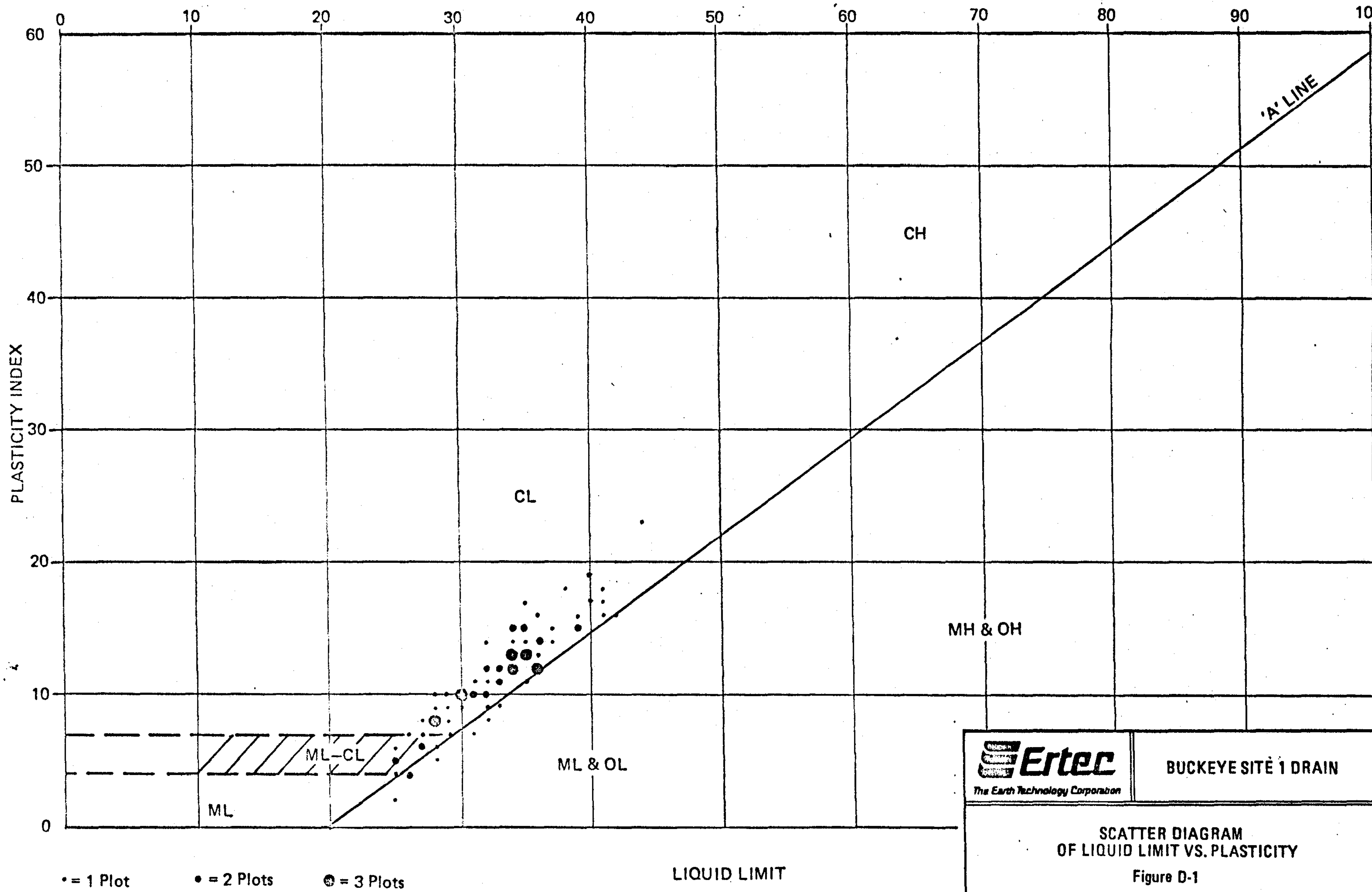
The designer recommends option No. 2.

Susanne M. Leckband

Susanne M. Leckband

April 22, 1982
Date

# SCATTER DIAGRAM OF LIQUID LIMIT VS. PLASTICITY INDEX
## BUCKEYE NO. 1 F.R.S.




Ertec
The Earth Technology Corporation

BUCKEYE SITE 1 DRAIN

SCATTER DIAGRAM
OF LIQUID LIMIT VS. PLASTICITY

Figure D-1

Ertec (formerly Fugro) Project Number 80-200
SCS Contract Number 53-8A02-0-00113

# VERTICAL DRAINS AND EMBANKMENT ZONES

Clarence E. Dennis
Lincoln EWP Unit
April 1971

## VERTICAL DRAINS AND EMBANKMENT ZONES

Vertical drains and embankment zones in addition to their normal water conducting functions may also be used to protect dams and their environs from a number of other extraordinary types of attack.

The following discussion assumes that the Soil Mechanics Notes on Tentative Guides for Determining the Gradation of Filler Materials and Soil Mechanics Considerations for Embankment Drains have been complied with and that some additional form of extraordinary attack must also be dealt with.

VERTICAL DRAINS (Chimney Drains, Foundation Trench Drains)

Vertical drains as used in the Soil Conservation Service serve two main functions:

I. A positive water cut-off (Interceptor Drain)

   A. Installed in an embankment to prevent the materials on its downstream side from saturating and thus to guarantee the strength and stability of those materials.

   B. Installed in a foundation to prevent the development of excess water pressures in the foundation materials on its downstream side.

II. A protective zone against seismic activity and against cracking and breaching.

   A. Installed both in an embankment and a foundation to intercept and block the spread of a shear crack and to control the release of water that develops because of the crack.

   B. That will remain a viable stable zone even under the stress of violent shaking and relatively large movements of adjacent materials in its immediate vicinity.

A positive water cut-off (Interceptor Drain) may have any stance as long as it physically cuts across the zones that carry water. It may be vertical, sloped, or multisloped.

It should be designed using the normal procedures for drains.

It is desirable that:

A. The drain material have:

| Normal Sections<br>Horizontal Dimension < 8' | Large Sections<br>Horizontal Dimension $\geq$ 8' |
|---|---|
| $D_{15} \geq$ #4 Sieve | $D_{15} \geq 1"$ |
| $D_{100} \leq 3"$ | $D_{100} \geq 12"$ |
| $Cu < 4$ | $Cu < 4$ |
| $1 < Cc < 3$ | $1 < Cc < 3$ |

MINIMUM WIDTH

W/O Filter = 2'

W/ Filter = 3'

B. The filter material have:

$Cu < 4$ ?

$1 < Cc < 3$

A protective zone against cracking must be vertical or very nearly vertical to be effective. It must have the quality of self-healing against shear stress and cracking and a material range that is capable of developing a dam of progressively smaller particles upstream against the water that flows through a discontinuity.

A. To be self-healing. The material:

1. Must be clean so that it will flow together when pulled apart by shear stress or movement.

2. Must have a vertical stance so that when material is separated it will flow together again without mixing with adjacent materials that could contaminate it.

3. Should be coarser than the #4 sieve so that water film interference will be negligible.

B. To be capable of developing a dam of progressively smaller particles. The material:

1. Must have a maximum size large enough so that it will lodge in any conceivable crack and start the process of inverted filter development.

2. Must be well graded to provide the range of sizes of material that will be capable of building an inverted filter against the water flow.

3. Must have a width sufficient that the inverted filter will build well within the body of the drain material.

4. Be graded so that it will act at once as its own filter and drain material.

C. To be capable of transporting all the water that a failure crack might provide.

1. Must have enough capacity to handle the computed flow from a large crack given the maximum potential head.

2. Must have enough separate outlets with a capacity of at least 50% of the large crack computed flow in no. 1 so that any crack location will outlet without overloading the capacity of the main drainage system.

D. To remain a viable working system after maximum shearing and movement have taken place.

1. Drain outlets should be coarse grained material without pipes. Pipes can bend and collapse in shear and movement zones and lose part or all of their water carrying capability.

E. The protective zone should be designed to meet the following criteria:

| | Minimum | Desirable |
|---|---|---|
| Slope | 1:1 | Vertical |
| Cu | > 5 | > 6 |
| Cc | 1 to 3 | 1 to 3 |
| $D_{85}$ | 4" | 6" |
| $D_5$ | $\geq$ #200 Sieve | $\geq$ #200 Sieve |
| $D_{15}$ | $\geq$ #100 Sieve | $\geq$ #4 Sieve |

| | | |
|---|---|---|
| Width | 0.2 x Height of Dam w/ 10' min. | 0.2 Height of Dam w/ 15' min. |
| Capacity (No Pipe) Between Outlets | 80% of computed large crack flow | 100% of computed large crack flow |
| Outlets (No Pipe) Capacity | 40% of computed large crack flow | 50% of computed large crack flow |

Material Placement in Drains

A. Susceptibility of drain materials to damaging segregation on placement is in direct proportion to their Cu value. Where Cu = 4 the sorting coefficient is less than 2 and the chance that segregation will be damaging to the function of the material is very low. Where Cu = 20 the chance that segregation could create a dangerous situation is very large.

The susceptibility to damaging segregation of drainage materials can be rated as shown in the following table:

| Very low | Moderate | High |
|---|---|---|
| $Cu \leq 4$ | $4 < Cu < 20$ | $Cu > 20$ |

B. When the susceptibility is very low no special measures need be taken in the placement of drainage materials.

C. When susceptibility is high the following precautions should be taken:

1. Wherever drain material is dumped or dropped the direction of fall should be vertical.

2. The height of unconfined drop should not exceed 5' in air and zero in water.

3. The best method of insuring a vertical drop is to pass the material through a short section of pipe. Baffle plates are not satisfactory.

4. When the drop in air exceeds 5' a kinked canvas drop chute or telescopic flexible hose tremie should be used. The bottom section should be vertical during placing.

5. When placing drain materials under water the pipe or tremie should be kept in full contact with the surface of the already placed material throughout the operation to keep any of the particles from free falling through water.

6. When chutes are used to place materials they should have a rounded cross section and a slope not flatter than 2 1/2:1.

7. When dropping materials into a trench they should not be allowed to strike the sides.

8. Material should be placed directly at its final location and not be allowed to flow laterally for more than 3 feet.

9. During material placing slopes steeper than 1:1 should not be allowed to develop.

10. After placement of each 36" deep layer of drain material it should be vibrated with an internal vibrator until the surface stops settling.

Fine sized ( < #4 sieve) filter material should not be placed where shear zone protection is desired. Without coarse material to start the dam of progressively smaller particles when a shear zone develops, the finer material can and has been washed through and out of the zone of protection.

EMBANKMENT ZONES

Generally transition zones correspond in function to large section positive water cut-offs and protective zones against cracking and earthquakes. As far as possible, desirable design features in one are also desirable in the other. Where transition zones have a much greater thickness (twice as thick or greater), some of the requirements listed under protective zones may be reduced.

Embankment transition zones must be wide enough to add significant stability and strength to the dam as well as to provide the same function as do vertical drains. This means that generally their base width should not be less than the height of the dam at that point along the centerline.

1. As long as the properly designed transition zone remains intact it will prevent the velocity and quantity of leakage from exceeding moderate values.

2. The coarseness of a transition zone provides:

   a. An inherent stability against washing or piping out of material through the interstices of the downstream shell.

   b. An ability to build dams of progressively smaller particles in the largest open cracks that may develop in the adjacent foundations and abutments, sealing these cracks and controlling the leakage to moderate values and preventing failure.

3. Wherever the effects of earthquakes are to be designed against, a single wide transition zone of a well graded sand-gravel mixture is superior to two or three adjacent zones (drains) of sand and gravel with gradually increasing coarseness from the core downstream. The wide single transition zone will withstand much more severe shocks and earth movements and still retain its integrity.

4. Where the effects of earthquakes are to be designed against it is very undesirable to have a single thick zone of sand (especially fine sand) located downstream from the core. Water breaking through a leak in the core could find an outlet through the foundation or abutment completely by passing the coarser filter zones downstream in the embankment. A minimum $D_{85}$ of a well graded material in such a filter zone should be 1".

5. CL and CH cores can be protected by a well graded sand and gravel mixture with a $D_{15}$ ranging from 2 mm (#10 sieve) for CL materials to 5 mm (#4 sieve) for CH materials.

TO: Ralph Arrington, State Cons. Engineer
SCS, Phoenix, Arizona

United States Department of Agriculture

Soil Conservation Service

West Technical Service Center
511 NW Broadway, Rm. 510
Portland, Oregon 97209

Subject: EN-40-13 - Buckeye Watershed, Buckeye Site 1, Drain, Arizona

Date: February 6, 1980

To: Thomas G. Rockenbaugh, State Conservationist
SCS, Phoenix, Arizona

The preliminary and final SCS Design Reports submitted for this job include a summary, description of the job, and the criteria used. In order for our reviewers to evaluate the technical feasibility of the proposed repair measure, a rundown of the design decisions and basis or rationale for making them is also prerequisite to review. See items 511.11(b)(3) through 511.11(b)(19) of the National Engineering Manual for specific guidance.

Our knowledge of flow conditions through cracks in embankments is not complete. However, evidence surfaced to date indicates that the following requirements are essential to assure safety of the dam:

1. Filter material must have "self-healing" qualities:

   a. It must not contain any plastic fines or other cementing agent.

   b. The content of fines must be less than 6%.

   c. Well-rounded, equidimensional particles are preferred.

   d. Filter requirements for the embankment material must be met.

2. To prevent filter material from washing into cracks downstream:

   a. The minimum $D_{50}$ size of the filter must be equal to or greater than one-half the crack width.

   b. The minimum $D_{75}$ size of the filter must be equal to or greater than the crack width.

   c. Separate outlets will be required where crack width exceeds criteria in "a." and "b." above.

3. Permeability of the filter should be less than 250 cubic feet per day to minimize potential for erosive velocities in cracks downstream from the filter.

This will require permeability tests on the filter materials.

REWRITE ... REPT ... SUPPLE. ... MENT)

CRITERIA FOR DESIGN REVISION

... CAMP & VERSE TO BE INCLUDED IN SUPPL.

RESEARCH ... STUDY & LITT REP TO DETERMINE DESIGN CRACK SELECT FILTER GRADATION

REWRITE SPECS

WRITE SPECS FOR ... DESIGN TEST ... OF ... MATERIALS

. REWRITE CONST. SPEC TO INCL. PERM.

A A coarse, highly-pervious section will be provided in the outlets to insure rapid removal of any water than can get into the outlets. Outlets should discharge upward beyond the toe of the slope. Refer to the attached figure.

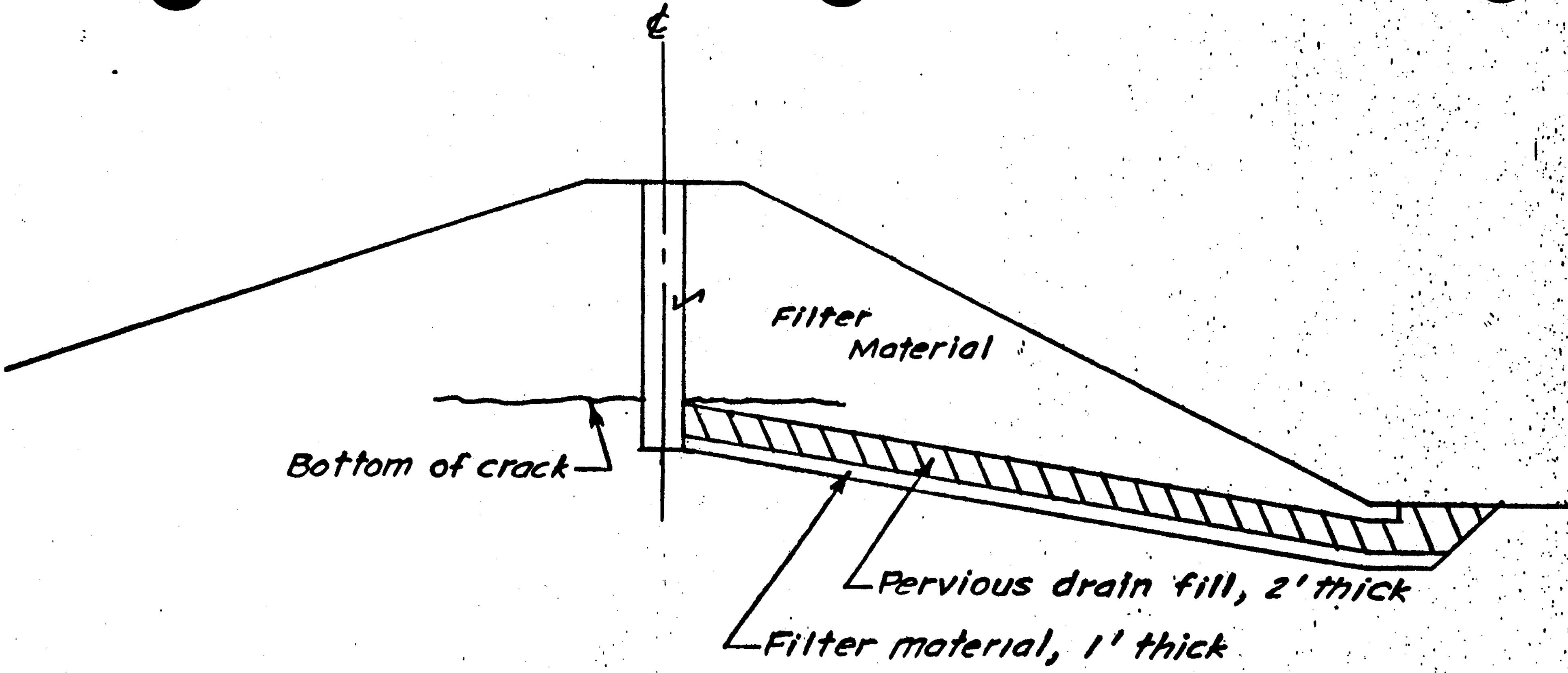
Conformance of the proposed filter and outlet design to the foregoing listed requirements will provide basis for our approval of the construction plans and specifications for the proposed work.

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.

Stan Hobson

STANLEY N. HOBSON
Head, Engineering Staff

cc:
Ralph M. Arrington, State Conservation Engineer, SCS, Phoenix, Arizona
Neil F. Bogner, Director, Engineering Division, SCS, WDC




Outlet for filter trench

Buckeye, Site 1, Arizona
RN 2-80

RE: Vineyard FRS Repair File.

Room 3008 Federal Building
230 North First Avenue
Phoenix, Arizona 85025

November 21, 1983

Dick Raymond
Bureau of Reclamation
Suite 1880, Valley Center
210 N. Central Ave.
Phoenix, Arizona 85073

Dear Dick:

You have asked about the study the Soil Conservation Service (SCS) is doing at Vineyard Road Flood Retarding Structure (dam). The study is not complete, but I will be happy to provide you with a description of what has been done and what remains to be done. This information is necessarily informal to meet your deadline for review of proposed Bureau of Reclamation tests.

Introduction

The SCS does not do research. This study was initiated as a result of the unanticipated depth and number of cracks encountered during construction of Vineyard Road FRS Repair.

The repair design called for a trench in the center of the dam, three feet deeper than the existing cracks, backfilled with clean granular material. Every crack wider or deeper than specified dimensions would be eliminated on the downstream side by trenching and backfilling with granular material. The downstream trenches were modified out of the repair contract for the time being. The study was to determine behavior of the dam during a hypothetical flood event without elimination of the cracks downstream from the (already constructed) centerline granular material. (See the enclosed plans and specifications).

Cells

Four reservoirs, called cells, were constructed against the existing dam at sites where cracks were known to exist. One side of each cell was the existing dam, 100 to 200 feet long (see enclosed drawings). The cells were numbered during discussion sessions and the numbers never changed, therefore they are not in geographic order along the dam and there is no Cell 4.

Cell 3 was a control section of the dam left unrepaired and undisturbed by the repair contract. It is located between Stations 253+00 and 255+00.

Cell 1 was a repaired section, otherwise undisturbed. It is located between Stations 121+00 and 123+00.

Cell 2 was a repaired section of the dam that was re-excavated for the purpose of placing monitors on specific cracks and then backfilled with materials meeting the original repair contract specifications. Then a 100-foot section of the upstream face of the dam was scraped off (approximately one foot deep). This cell is located between Stations 103+50 and 105+00.

Cell 5 was a repaired section that was re-excavated for monitoring and installation of an impermeable barrier of 100-mil thick high-density polyethylene (HDPE). For the purposes of this test, the HDPE was installed against the downstream vertical wall of the trench. The granular material in the bottom one foot of the trench was not removed. new granular backfill was placed to approximately two feet up from the bottom of the HDPE. (The purpose of the granular backfill at the upstream bottom of the HDPE was to (a) provide a discontinuity at any crack that might form at the edge of the HDPE and (b) cause formation of a "filter cake".) The rest of the trench was backfilled with soil borrowed at the site. No compaction was attempted since this is not a permanent installation and the only purpose was to hold the HDPE in place. This is a proposed barrier against cracks in a future design. Cell 5 is located between Stations 109+00 and 110+00 (approximately). A 100-foot section of the face of the dam in this cell was scraped off, too. Our geologist, Aubrey Sanders, used an air compressor to expose and clean the cracks at the scraped off sections of Cells 2 and 5 to encourage water penetration.

Water

As you know, Vineyard Road FRS is immediately adjacent to Reach 2, Salt Gila Aqueduct, Central Arizona Project. Reach 2 was under construction during the repair contract and the study contracts. One of our greatest difficulties was locating a source of water at a high enough discharge rate to approximate a flood event. Most of the possible sources were already in use for construction prewet on the aqueduct.

We were trying to fill each cell within six hours. We finally settled for a minimum discharge rate of 500 gallons per minute. The final arrangement consisted of an irrigation well that discharged into an existing concrete lined ditch which conveyed the water to a sump. A portable pump at the sump pumped water through portable irrigation pipe to each cell. There were two flow meters in the line, one adjacent to Cell 2 and one adjacent to Cell 3. Cells 2, 5 and 1 are fairly close together. Cell 3 is 2.5 miles from Cell 1. The sump was several miles from the dam.

Cell 3, the largest, was filled in 44.5 hours. This was the first cell to be filled and there were a few problems with the pump, materials in the pipe jamming the flow meters and lack of communications between the cell and the pump sites. The smallest cell (2) was filled in just less than 22 hours. Considerably more water was required to fill Cell 3 than the actual storage volume.

All four cells were filled by August 5, 1983 and maintained full until at least September 1, 1983. Depth in each cell was about fourteen feet.

Depth was maintained within a range of one foot measured on a staff gauge in each cell. Cell 5 was maintained full until October 11, 1983 and has not been drawn down.

The original idea was just to keep the cells full about fifteen days to simulate a worst-case condition. The flood control dam is designed to drain in a maximum of ten days. Since none of the monitors gave any indication of water movement through the structure after fifteen days, the test time was increased.

One of the questions we hoped to resolve was: Does a filter cake form at the contact between a crack in a dam and the granular zone when water flows through the crack? Laboratory tests indicate the rapid formation of a positive seal, filter cake, when water is introduced to a crack in soil adjacent to granular material with a minimum D15 between 0.4 and 0.7 mm (varies with specific soil). The concern revolves around the low height of dams in this location and the part pressure and velocity may play in formation of that seal.

Unfortunately, we do not have any samples of runoff water from a typical storm to compare with the water used in the study. Three samples of water used in the study were tested for cations, anions and pH. (See enclosed test results.)

## Excavation

Water in Cell 2 was drawn down five feet to permit excavation at the crack sites from the upstream face of the dam. Three of these cracks were investigated August 25, 1983. Trenches were approximately eight feet deep and ended four feet from the upstream side of the filled centerline trench. The cracks were visible on the slope prior to excavation with the backhoe. The trenches were left open and the cell refilled as a further attempt to get water through the cracks to the repair work. During this excavation, one of the electric crack monitors indicated water. It was later discovered that the activities on the dam caused that monitor wire to be damaged.

Cells 2 and 1 were evacuated and Cell 3 was drawn down the first week in September. A team of geologists and engineers supervised and participated in investigative excavations on these cells the week of September 6 - 9, 1983. They ran out of time and did not participate in excavations on Cell 5. Phil Jones (NM State Soil Mechanics Engineer) and I investigated Cell 5 for flow in cracks and through the embankment September 12, and I supervised the investigation October 11, 1983. The investigation proceeded with a full reservoir to make location of cracks and seepage simpler and unquestionable.

## Monitors

No monitor other than a staff guage was used at Cell 3. The dam at that site was totally undisturbed.

Cell 1 had a staff gauge and several PVC piezometers driven into the granualar material at known crack locations. The piezometers were protected by a steel pipe sleeve, for the top three feet, with a threaded cap. The pipe extended one foot above grade. The caps were to keep rain and dirt out, but if left on all the time moisture condensed in the pipe and caused the electric sensor to show water. No water apparently reached the centerline of this part of the dam.

Cells 2 and 5 were monitored the same way. Piezometers were installed in the same relative locations as in Cell 1. Pairs of piezometers were located at selected cracks. One six inches from the upstream trench wall and one six inches from the downstream trench wall (or at the wall behind the HDPE in Cell ). Two additional piezometers were installed in the center of the trench approximately 25 feet away from the north and south crack monitors, respectively, in each cell. The trench was twenty feet deep and the piezometers were set into the bottom of the trench about six inches to maintain their location during backfilling operations. Water entered through slots in the pipe.

A minimum of three cracks were monitored at each cell (2 and 5). Electric sensors were stapled into the cracks with plumbers tape at the apparent bottom of the crack and at the widest part if the crack on the upstream wall of the trench. Two more sensors were set below the cracks on the upstream and downstream bottom of the trench, respectively. Wires from these monitors were run to a construction trailer between the two cells. Clocks and buzzers were attached to alert us of any water that came through and the exact time and location. None of the electric monitors ever actually registered water. The damaged wire on Cell 2 produced a false indication.

There was an attempt to keep evaporation and rainfall data. These are questionable, but may show some trends.

Staff gauges were read often. They indicated relatively slow loss of water due to seepage.

Photography

Much of the study is on video tape and/or slides. These have not been edited and spliced, so they are very time-consuming to watch. In fact, I have not had an opportunity to look at the last film (October 11).

Some locations and results were hard to photograph due to darkness in the trenches and similarity of color and texture between wet soils and dry soils.

Soils

Ten soil samples from Cell 2 are at the Soil Mechanics Lab at this time. Nine are CL's and one is SC. These were taken adjacent to cracks.

More samples of embankment materials are planned along the length of the dam to aid in projecting the results of the study throughout this structure and, potentially, to other structures.

Dams constructed with different soils may behave in an altogether different manner than this study would indicate.

## Observations

The fully saturated "phreatic line", after more than 30 days of storage, appeared to parallel the upstream slope of the dam roughly six inches deep. The surface material under water appeared to behave in a structureless single-grained manner except at the scraped off sections.

The overall moisture content of the embankment materials appeared to increase with time and proximity to the upstream face of the reservoir. A memo in the files concerning the first excavation (August 25) at Cell 2 says that "The water seemed to be following...seams of horizontal layers..." (Ralph Arrington, SCE). In Cell 5 both September 12 and October 11 this was very evident. The source of all of the moisture in the middle of the dam at Cell 5 appeared to be a horizontal layer of saturated material at approximately eight vertical feet from the top of the dam and another layer about three feet below that.

No water passed all the way through the dam at any of the four sites. Seepage reached the center of the dam at Cell 5 and the downstream side of the center of the dam at Cell 3.

Excavation of the dam from the downstream side while water was still in the reservoir made location of flow paths positive. Cracks were otherwise difficult to locate when the embankment soils became uniformly more moist. The appearance (color) of saturated soils did not vary appreciably from the appearance of merely moist soils.

## Conclusions

No conclusions can be made before final data have been collected and analyzed. In the meantime, you are welcome to look at the data we have. It is not reduced and organized at this time. As mentioned before, the results are only applicable to the sites tested until soil correlations are made. Also, we only looked at relatively short-term storage.

Susanne Leckband, P.E.
State Soil Mechanics Engineer

Enclosures

cc: Ralph Arrington
Clifton Deal
Jim Talbot
Aubrey Sanders

FILE CODE ENG-22

# MATERIALS TESTING REPORT

U. S. DEPARTMENT of AGRICULTURE
SOIL CONSERVATION SERVICE

# SOIL CLASSIFICATION

LABORATORY NO.

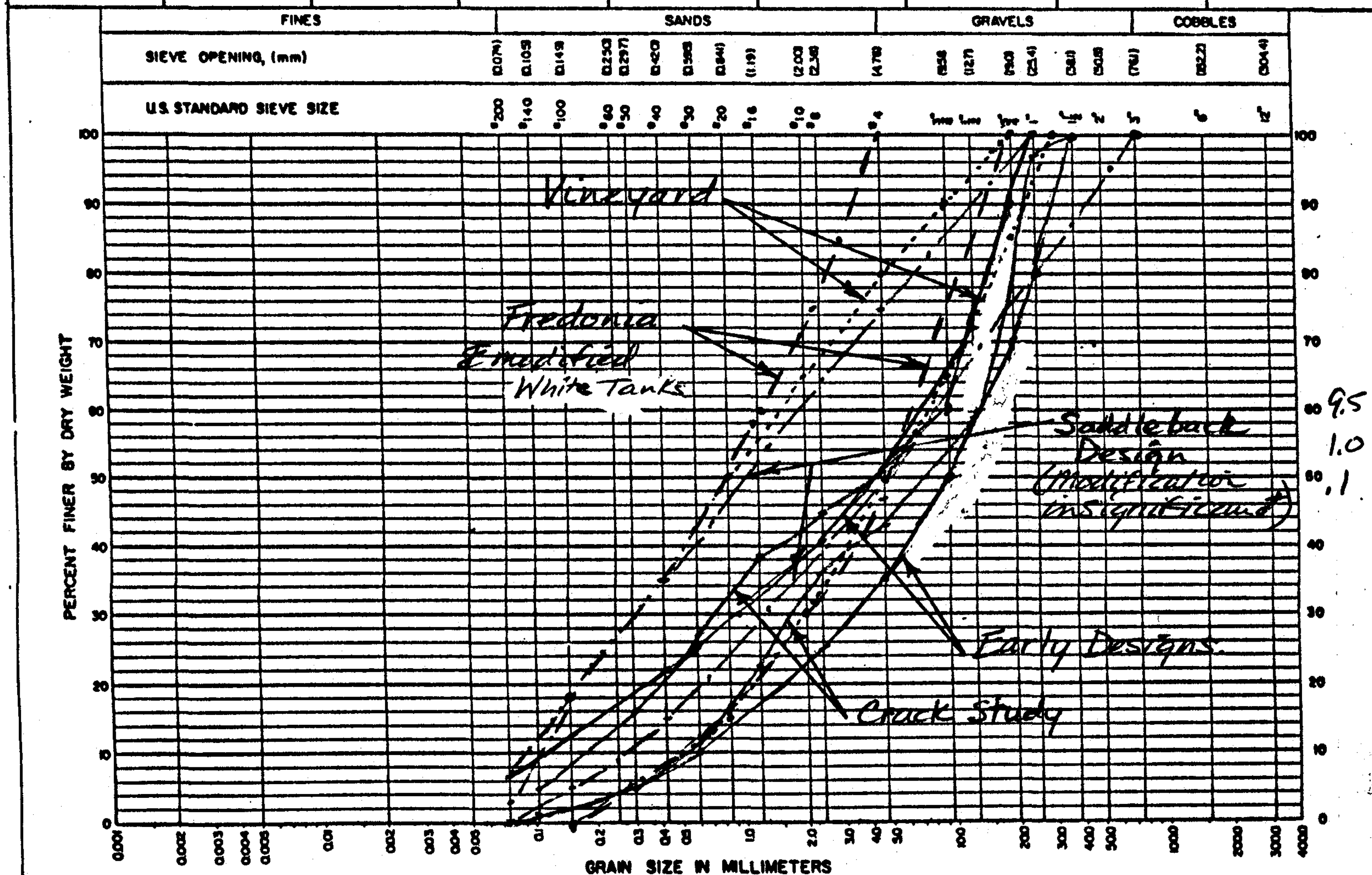
| | |
|---|---|
| PROJECT and STATE | Comparison of Drainfill Specifications |
| FIELD SAMPLE NO. | |
| DEPTH | |
| GEOLOGIC ORIGIN | |
| SAMPLE LOCATION | |
| TYPE OF SAMPLE | |
| TESTED AT | |
| APPROVED BY | S. Leckband |
| DATE | 5-30-84 |

SYMBOL

DESCRIPTION

## GRAIN SIZE DISTRIBUTION

| $D_{10}$ | $D_{15}$ | $D_{30}$ | $D_{50}$ | $D_{60}$ | $D_{85}$ | $D_{MAX.}$ | $C_u$ | $C_c$ |
|---|---|---|---|---|---|---|---|---|
| | | | | | | | | |




| SPECIFIC GRAVITY ($G_s$) | | ATTERBERG LIMITS | | | | | | SOLUBLE SALTS | SHRINKAGE LIMIT | UNDISTURBED CONDITION | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | NATURAL MOISTURE | | AIR DRY | | OVEN DRY | | | | MOISTURE | DRY UNIT WEIGHT |
| (-) #4 | (+) #4 | LL | PI | LL | PI | LL | PI | % | % | % | g/cc pcf |

REMARKS:

2.1 mm
3 mm

1.25
0.3
0.1

$C_c = .72$
$C_u = 12.5$

*The following article on Geomembrane Products is the second of a series of two articles. The first article, entitled Geotextile Products, by J. P. Giroud and R. G. Carroll, Jr, was published in the first issue of the "Geotechnical Fabrics Report" (Summer 1983). These two articles are intended to provide manufacturers, designers, and users with clear and practical classifications of geotextile and geomembrane products. Such classifications are necessary because of the increasing variety of products available on the market.*

*Geotextiles and geomembranes refer to textiles (fabrics) and membranes used in geotechnical engineering. Geotechnical engineering, according to the American Society for Testing and Materials (ASTM), embraces the fields of soil mechanics, rock mechanics, and many of the engineering aspects of geology, geophysics, hydrology, and related sciences.*

*It is important not to confuse geotextiles with geomembranes. Geotextiles are permeable by construction, and geomembranes are designed to have a permeability as low as possible. In other words, geotextiles allow or conduct fluid flow, while geomembranes restrict fluid flow. Although the mechanisms by which fluids pass through soils, geotextiles and geomembranes are different, for comparison purposes permeabilities of these materials can be evaluated using the same methods. A convenient method consists of using the hydraulic conductivity, also called coefficient of permeability. Typical values of hydraulic conductivity are: $10^{-5}$ to 1m/s ($10^{-3}$ to 100 cm/s) for geotextiles (or even more in the case of some products such as open nets or grids) and $10^{-13}$ m/s ($10^{-11}$ cm/s) or less for geomembranes. The hydraulic conductivity of geotextiles is of the same order of magnitude as the hydraulic conductivity of highly permeable soils such as sand and gravel. The hydraulic conductivity of geomembranes is much smaller than the hydraulic conductivity of clay, which is the least permeable soil.*

# Geomembrane Products

By
**J. P. Giroud and R. K. Frobel**

Geomembranes are very low permeability membrane liners and barriers used with any geotechnical engineering related material so as to control fluid migrations in a man-made project, structure or system. The term liner applies when a geomembrane is used as an interface or a surface revetment. The term barrier is usually reserved for the cases where the geomembrane is used inside an earth mass. Geomembrane is a generic term which has been proposed to replace many terms such as: synthetic membranes, polymeric membranes, plastic liners, flexible membrane liners, impermeable membranes and impervious sheets. These terms are not appropriate because: (i) synthetic, polymeric and plastic are too restrictive; (ii) geomembranes are not always used as liners; (iii) flexible membrane is redundant; and (iv) no material is absolutely impermeable or impervious. In addition, many users of these materials habitually designate them with trade names, which adds to the terminology confusion. Geomembranes should not be confused with other similar membranes used for such applications as single-ply roofing, floating covers and air supported roofs. Also, geomembranes should not be confused with geotextiles as explained in the foreword.

The types of geomembranes that adhere to the above definition include those composed of polymeric or asphaltic materials, non-reinforced or reinforced with a fabric, made in a factory or applied in situ (ie, at the construction site). Compacted earth linings, incorporating various types of manufactured or natural additives, and hard surface linings such as steel, concrete, gunite, asphaltic concrete and soil cement are not considered as geomembranes.

Geomembranes are used in the construction of potable water reservoirs, distribution canals, municipal and hazardous solid waste landfills, liquid waste lagoons (also called liquid impoundments or surface impoundments), cutoff walls, dam facings, final closure landfill cover, spill containment systems, etc. In these structures, geomembranes serve the primary function of controlling the migration of fluids.

This paper will discuss composition, production, classification and identification of geomembranes.

## Composition of Geomembranes

Geomembranes are composed of a very low permeability material, reinforced or not with a fabric.

Very low permeability materials are materials having a very low hydraulic conductivity (also called coefficient of permeability), typically $10^{-14}$ to $10^{-13}$ m/s ($10^{-12}$ to $10^{-11}$ cm/s). (Although, as explained in the foreword, the mechanism by which fluids pass through soils, it is convenient to evaluate permeability of geomembranes using coefficients originally defined for soils.) Among materials having a very low permeability are compounds of which the base product is asphalt and/or a polymer.

Asphalt is obtained either from natural deposits or as a by-product of oil distillation. Blown asphalt, often used to make geomembranes, has been hardened by blowing air through the molten asphalt to raise its softening temperature and decrease its tendency to flow.

Polymers are chemical compounds of high molecular weight. Only synthetic polymers are used to make geomembranes. The most common types of polymers presently used as base products in the manufacture of geomembranes can be classified as follows (symbols in parenthesis are adopted from symbols used by the National Sanitation Foundation (NSF) Joint Committee on Flexible Membrane Liners (FML)):

1. *Thermoplastics:* Polyvinyl Chloride (PVC); Oil Resistant PVC (PVC-OR); Thermoplastic Nitrile-PVC (TN-PVC); Ethylene Interpolymer Alloy (EIA);
2. *Cristalline Thermoplastics:* Low Density Polyethylene (LDPE); High Density Polyethylene (HDPE); High Density Polyethylene-Alloy (HDPE-A); Polypropylene; Elasticized Polyolefin;
3. *Thermoplastic Elastomers:* Chlorinated Polyethylene (CPE); Chlorinated Polyethylene-Alloy (CPE-A); Chlorosulfonated Polyethylene (CSPE), also commonly referred to as "Hypalon"; Thermoplastic Ethylene-Propylene Diene Monomer (T-EPDM);
4. *Elastomers:* Isoprene—Isobutylene Rubber (IIR), also commonly referred to as Butyl Rubber; Ethylene-Propylene Diene Monomer (EPDM); Polychloroprene (CR), also commonly referred to as "Neoprene"; Epichlorohydrin Rubber (CO).

In addition to the base product, compounds used in geomembranes generally include various additives.

Additives typically compounded with asphalt are fillers, fibers, and elastomers. Fillers are small mineral particles (typically 1 to 200 microns) used to reduce the cost of the asphaltic compound and increase its stiffness, without altering its very low permeability. Examples of particles used as fillers are: limestone, ground calcium carbonate, slate flour, kaolin clay, talc, mica, fly ash, barite, graphite. The weight ratio filler/(filler + asphalt) is usually between 0 and 60%, typically 30%. Fibers, such as asbestos or glass fibers, are sometimes added to asphalt to reinforce it. Elastomers, such as thermoplastic butadiene-styrene-butadiene copolymer, or reclaimed rubber from tires, are sometimes included in asphaltic compounds to improve their mechanical behavior and their resistance to weathering. Typical proportion of elastomer added is between 5 and 15%.

Additives typically compounded with polymers are fillers, fibers, processing aids, plasticizers, carbon black, stabilizers, antioxidants and fungicides. Fillers used with polymers are mineral particles (such as the fillers used with asphalt discussed above), metallic oxides (such as alumina, magnesia, zinc oxide, antimony oxide), ground polymers, saw dust, etc. The weight ratio filler/(filler + polymeric compound) is usually between 0 and 20% for thermoplastics and cristalline thermoplastics, and between 10% and 50% for elastomers and thermoplastic elastomers. Fibers (typically chopped glass, polyester or nylon fibers) are sometimes included in the compound. Inclusion of chopped fibers is delicate and may trigger the formation of pinholes in the geomembrane. Processing aids are used to reinforce or soften the compound during the manufacturing process. Plasticizers are used to impart flexibility to the compound to produce membranes from otherwise stiff compounds such as PVC. Plasticizers may also facilitate the manufacturing process. The weight ratio plasticizer/base product typically varies from 0 to 2% in elastomeric compounds (mostly to facilitate the manufacturing process) to 55% in PVC compounds. Carbon black (typically 1 to 2% of the base product in the case of thermoplastics and cristalline thermoplastics, and 10% to 45% in the case of elastomers and thermoplastic elastomers) imparts a black color to the compound which retards aging by ultraviolet light from the sun and increases the stiffness of elastomeric compounds. In hot climates, light color geomembranes with a low carbon black content are sometimes used to decrease the risk of degradation of the geomembranes by sun generated heat. Light color geomembranes made with some polymers such as Hypalon may be protected from ultraviolet light by addition of titanium dioxide. Various stabilizers and antioxidants reduce the effect of outdoor aging (by ultraviolet light, ozone, etc) as well as provide compound stability during the manufacturing process. Fungicides prevent fungi and bacteria from attacking the polymer.

Fabric reinforcement is used for one or several of the following reasons: (i) to impart stability to the compound (eg, asphalt, Hypalon) during the manufacturing process; (ii) to provide dimensional stability to compounds that would excessively shrink or expand as a result of change in physical conditions such as temperature; (iii) to increase the strength (tensile, tear, burst, puncture) of the geomembrane to prevent it from being damaged during handling and installation, and to allow it to withstand the design stresses; and (iv) to increase the modulus of the geomembrane in order to decrease its elongation when subjected to stresses. Fabric reinforcement can be of various types depending on the manufacturing process of the geomembrane as discussed below.

In the recent years, knitted fabrics have been introduced to reinforce geomembranes, especially the geomembranes made in a factory by spread coating. However, the most widely used reinforcement fabrics are the nonwovens and the wovens, especially the scrims, as discussed below.

Nonwoven fabrics are used to reinforce geomembranes made in situ and some geomembranes made in a factory by spread coating. Nonwoven fabrics can also be bonded to geomembranes by the calendering method. The nonwoven fabrics used to manufacture geomembranes are usually needlepunched, with a mass per unit area typically ranging between 200 and 600 g/m$^2$ (6 to 18 oz./sq. yd.) (see the article entitled "Geotextile Products", by J. P. Giroud and R. G. Carroll, Jr, published in the first issue of the Geotechnical Fabrics Report, Summer 1983).

Woven fabrics are used to reinforce some spread coated and some calendered geomembranes. The type of woven fabric generally used to reinforce calendered geomembranes is a scrim. A scrim is a type of open weave fabric with a low mass per unit area (ie, a "lightweight" fabric). A plain weave scrim is one in which each filling (cross-machine direction) yarn passes successively over and under each warp (machine direction) yarn, alternating each row. A leno weave scrim is one in which warp yarns are arranged in pairs and twisted around each other between picks of filling yarn (each warp yarn passing successively over and under each filling yarn). This type of weave imparts strength and prevents slippage in an open weave fabric. In some scrims, one half of the filling yarns are over the warp yarns, the other half being under the warp yarns (ie, yarns in one direction do not pass successively over and under yarns in the other direction). These scrims have no stability. They must be dipped into a liquid that bonds yarns together. Sometimes scrims are made thinner by calendering them prior to calendering the compound. A scrim is characterized by its count and the linear density of its yarns. The count is the number of yarns per unit width (in meter, centimeter, or inch) in each direction (warp and filling). The linear density of a yarn is its mass per unit length. Units for linear density are kg/m or, more conveniently, tex which is $10^{-6}$

press before rolling

*Continued on page 40*

**Geomembrane Products**
*Continued from page 39*

kg/m (ie, g/km or mg/m). The traditional unit for linear density is the denier (one tex = 9 deniers, ie 1000 deniers = 111 tex). Examples of scrims available in the United States are:

- 630 × 315/m (16 × 8/inch), 14 tex warp/28 tex filling (125 deniers warp/250 deniers filling), leno weave, often referred to as 8 × 8, 250;
- 480 × 240/m (12 × 6/inch), 55 tex warp/111 tex filling (500 deniers warp/1000 deniers filling), leno weave, often referred to as 6 × 6, 1000;
- 4 × 4/cm (10 × 10/inch), 111 tex (1000 deniers), plain weave, often referred to as 10 × 10, 1000.

Examples of scrims available in Europe are:

- 4 × 4/cm (10 × 10/inch), 28 tex (250 deniers);
- 2 × 2/cm (5 × 5/inch), 111 tex (1000 deniers);
- 3 × 3/cm (7.5 × 7.5/inch), 111 tex (1000 deniers).

The mass per unit area of a scrim is derived by multiplying the count by the linear density in both directions and adding. Example: the mass per unit area of a 480 × 240/m, 55 tex warp/111 tex filling is $480 \times 55 \times 10^{-6} + 240 \times 111 \times 10^{-6} = 0.053$ kg/m$^2$ = 53 g/m$^2$ (1.6 oz./sq. yd.)

Although all reinforced geomembranes presently available are, to the best of our knowledge, reinforced with fabrics, it is possible that, in the future, other forms of reinforcement will be available.

## Production of Geomembranes

Most geomembranes are made in a plant using one of the following manufacturing processes: (i) extrusion, (ii) spread coating, or (iii) calendering.

Extrusion process is a method whereby a molten polymer, usually of the polyolefin family (such as polyethylene, polypropylene), is extruded into a non-reinforced sheet. Immediately after extrusion, when the sheet is still warm, it can be laminated with a fabric, through light calendering; the geomembrane thus produced is reinforced.

Spread coating process usually consists in coating a fabric (woven, nonwoven, knit) by spreading a polymer or asphalt compound on it. The geomembranes thus produced are therefore reinforced. Non-reinforced geomembranes can be made by spreading a polymer on a sheet of paper which is removed and discarded at the end of the manufacturing process.

Calendering is the most frequently used manufacturing process. A calendered non-reinforced geomembrane is usually a single sheet of compound made by passing a heated polymeric compound through a series of heated rollers (calender). Some calendered non-reinforced geomembranes are produced by simultaneously running two sheets of compound through heated rollers. The purpose of this process is to minimize the risk of having a pinhole through the entire thickness of the geomembrane. Pinholes are small holes that can exist in a sheet of compound as a result of grit or other cause during the manufacturing process. Calendered reinforced geomembranes are produced by simultaneously running sheets of compound and scrims through heated rollers. A three-ply calendered reinforced geomembrane is made of the following layers: compound/scrim/compound. A five-ply calendered reinforced geomembrane is made of the following layers: compound/scrim/compound/scrim/compound. The polymeric compound, when heated and pressed by the rollers, tends to flow through the openings of the scrim, thus providing adhesion between the sheets of compound located on both sides of the scrim. This adhesive mechanism is commonly known as "strike-through".

Geomembranes manufactured by the above processes are produced in rolls approximately 1.5 m to 10 m (5 to 33 ft.) in width. Geomembranes that are produced in wide rolls, typically 5 to 10 m (16 to 33 ft), and heavy geomembranes such as asphaltic geomembranes are commonly transported to the field site where they are seamed together. Geomembranes that are produced in narrow, lighter rolls are first transported to a fabrication factory where they are seamed into large blankets. Blankets can be fabricated to any designed shape and are limited only by handling weight and dimension. They are commonly less than 2000 m$^2$ (20,000 ft$^2$). Blankets are packaged and transported to the construction site where they are seamed together. Small facilities can often be lined with a single blanket, thereby eliminating the need for field seaming.

Seaming methods depend upon the composition of the geomembrane. Some geomembranes can be seamed by several different methods. The most common seaming methods for polymeric geomembranes are: (i) methods involving heat only, such as electronic (dielectric) bonding, hot air bonding, hot wedge (or knife) bonding; (ii) methods involving supply of hot base product, such as extrusion (or fusion) welding; (iii) methods involving solvents and/or cements, such as solvent bonding, bodied solvent adhesive, solvent cements, contact cements; and (iv) methods involving vulcanizing tapes or adhesives. Methods involving heat are applicable only to geomembranes made with base products sensitive to heat, ie, thermoplastics, cristalline thermoplastics and thermoplastic elastomers. All seaming methods can be used in a plant or in the field, except the dielectric method which is not used in the field because it is sensitive to dust and humidity and the equipment is cumbersome. Extrusion welding is used only for high density polyethylene. Asphaltic geomembranes are typically seamed using flame or hot wedge, with or without supply of hot liquid asphalt.

Geomembranes made in situ (ie, at the construction site) are usually continuous (ie, with no seams) and are made by spraying or otherwise placing a hot or cold viscous material onto a substrate. The geomembranes made by spraying are commonly referred to as "spray-applied geomembranes" or "spray-on geomembranes". The base product of the sprayed material is commonly asphalt, an asphalt-elastomer compound (eg, asphalt-latex, asphalt-butadiene-styrene, etc), or a polymer such as polyurethane. The sprayed material forms a continuous flexible film with little or no tack after curing. If the material is applied onto an existing surface (ie, earth or concrete) the spray-applied geomembrane is non-reinforced. If the material is applied onto a fabric or geotextile, the resulting membrane is reinforced (however, at fabric overlaps, the reinforcement is continuous only if the reinforcing fabric is sewn). The sprayed material must penetrate the fabric and thus adhere to it after curing to provide a consistent reinforced spray-applied geomembrane.

## Classification of Geomembranes

Based on the information discussed above, geomembranes can be classified according to production process and reinforcement:

1. **Made in situ, non-reinforced geomembranes** are made by spraying or otherwise placing a hot or cold viscous material directly onto the surface to be lined (earth, concrete, etc). The non-reinforced geomembranes made by spraying are called "sprayed-on (or spray-applied, or sprayed in situ) non-reinforced geomembranes". Typical materials used are based on asphalt, asphalt-elastomer compound, or polymers such as polyurethane. Due to the spray application, the final thickness of such geomembranes is not easy to control and may vary significantly from one location to another. Typically, required thicknesses range between 3 and 7.5 mm (120 and 300 mils).

2. **Made in situ, reinforced geomembranes** are made by spraying or otherwise placing a hot or cold viscous material onto a fabric. The reinforced geomembranes made by spraying are called "sprayed-on (or spray-applied, or sprayed in situ) reinforced geomembranes". Typical materials used are the same as for the made in situ non-reinforced geomembranes described above. Typical fabrics used are the needle-punched nonwoven geotextiles because they can absorb viscous materials. As discussed above, the final thickness of such geomembranes is not easy to control. Typically, required thicknesses range between 3 and 7.5 mm (120 and 300 mils).

3. **Manufactured, non-reinforced geomembranes** are made in a plant by extrusion or calendering of a polymeric compound, without any fabric reinforcement, or by spreading a polymer on a sheet of paper removed at the end of the manufacturing process. Typical thicknesses range from 0.25 to 4 mm (10 to 160 mils) for geomembranes made by extrusion and 0.25 to 2 mm (10 to 80 mils) for geomembranes made by calendering. Typical roll width for geomembranes made by extrusion is 5 to 10 m (16 to 33 ft), although some are narrower. Typical roll width for geomembranes made by calendering is 1.5 m (5 ft), with some manufacturers producing 1.8 to 2.4 m (6 to 8 ft) wide rolls.

4. **Manufactured, reinforced geomembranes** are made in a plant, usually by spread coating or calendering. In spread-coated geomembranes, the reinforcing fabric (woven or nonwoven) is impregnated and coated on one or both sides with the compound, either polymeric or asphaltic. In calendered reinforced geomembranes, the reinforcing fabric is usually a scrim. Calendered geomembranes are always made with polymeric compounds and are usually made up of three plies: compound/scrim/compound. Sometimes they are made of five plies: compound/scrim/compound/srim/compound. Geomembranes with additional plies can be made on a custom basis. Typical thicknesses of asphaltic spread-coated geomembranes are 3 to 10 mm (⅛ to ⅜ inch). Typical thicknesses for polymeric spread-coated and three-ply calendered geomembranes are 0.75 to 1.5 mm (30 to 60 mils). Typical thicknesses for five-ply calendered geomembranes are 1 to 1.5 mm (40 to 60 mils).

5. **Manufactured, reinforced geomembranes laminated with a fabric** are made by calendering a manufactured geomembrane (usually a non-reinforced geomembrane previously made by calendering or extrusion) with a fabric (usually a nonwoven) which remains apparent on one face of the final product.

## Identification of Geomembranes

An abbreviated system for identifying geomembranes consists of providing the generic name (or initials) of the base product, followed by the letter R if the geomembrane is reinforced. Examples: a PVC geomembrane; a CPE-R geomembrane. (Note: the term "supported", sometimes used for "reinforced", is not recommended because it may create a confusion with the "supporting" soil or geotextile on which the geomembrane is resting.)

A comprehensive system for identifying geomembranes consists of listing: (i) the production process, only if the geomembrane is made in situ (if the geomembrane is made in a plant, this does not need to be mentioned); (ii) generic name of the base product (ie, asphalt or type of polymer); (iii) thickness (since the significant thickness is the thickness of the low permeability compound, the thickness to be indicated is the total thickness of geomembranes types 1 through 4 (except the case of fabrics coated one side only), while it is the thickness excluding the associated fabric in the case of geomembranes type 5); (iv) reinforcing fabric, if any (the type of fabric and the type of polymer, such as polyester or nylon, should be given; if the fabric is a woven (including scrim) or a knit, count and linear density of yarns, and the type of weave or knit should be given; if the fabric is a nonwoven, the mass per unit area should be given). (Note: for fabric types see the article entitled "Geotextile Products", by J. P. Giroud and R. G. Carroll, Jr, published in the first issue of the Geotechnical Fabrics Report, Summer 1983.)

Examples of geomembrane identification are as follows:

- sprayed in situ, asphalt-neoprene compound, 3 mm (120 mils) in thickness, non-reinforced;
- sprayed in situ, asphalt, 2.5 mm (100 mils) in thickness, reinforced with a polyproplene spunbonded needlepunched nonwoven, 370 g/m² (11 oz./sq. yd.);
- polyvinyl chloride (PVC), 0.75 mm (30 mils) in thickness, non-reinforced;
- asphalt-elastomer compound, 3.5 mm (140 mils) in thickness, reinforced with a polyester staple fiber needlepunched nonwoven, 230 g/m² (7 oz./sq. yd.);
- chlorinated polyethylene (CPE), 0.75 mm (30 mils) in thickness, reinforced with 4 × 4/cm (10 × 10/inch), 111 tex (1000 deniers) plain weave polyester scrim;
- chlorosulfonated polyethylene (CSPE), 0.9 mm (36 mils) in thickness, reinforced with 480 × 240/m (12 × 6/inch), 55 tex warp/111 tex filling (500 deniers warp/1000 deniers filling) leno weave polyester scrim

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(Note: The scrim can more simply be described by: 240 × 240/m (6 × 6/inch), 111 tex (1000 deniers) leno weave);

- chlorosulfonated polyethylene (CSPE), 1.15 mm (45 mils) in thickness, five-ply, reinforced with two plies of 315 × 315/m (8 × 8/inch), 28 tex (250 deniers) leno weave polyester scrims;
- butyl rubber, 1.5 mm (60 mils) in thickness, laminated with a polypropylene spunbonded heatbonded nonwoven fabric, 270 g/m² (8 oz./sq. yd.).

The above identifying characteristics are only minimum descriptors. Detailed information on physical and mechanical properties as well as chemical resistance and compatibility with hot, cold or wet environments is needed before a geomembrane is determined to be suitable for a specified application. All pertinent characteristics and properties must be considered before final selection of a geomembrane is made.

## Conclusion

The information presented herein should provide the reader with a basic knowledge of the types of geomembranes in use today. This article has purposely avoided such subjects as geomembrane polymer technology, physical and mechanical test procedures, design methodology, and installation technology. These are separate topics that are addressed in numerous technical journal articles and books available through libraries. A wealth of information is also available from geomembrane manufacturers and marketing groups. Those readers who wish to gain first hand knowledge on geomembranes should attend the International Conference on Geomembranes to be held in Denver, Colorado, 20–24 June 1984. For further information and a bulletin, please contact:

International Conference on Geomembranes
IFAI
Suite 450
345 Cedar Street
St. Paul, Minnesota 55101 USA

*Acknowledgements*

*The authors are indebted to C. Ah-Line, J. Becquart, R. G. Carroll, Jr, B. R. Christopher, J. E. Fluet, J. J. Fontaine, J. S. Goldstein, H. Haxo, B. V. Jones, K. H. Kastman, A. L. Meader, J. Y. Meynard, R. K. Schmidt, C. E. Staff, F. L. Stoller, J. VanderVoort, B. Venkataraman, W. E. Witherow, and R. M. Woodley for many valuable comments*

*Jean-Pierre Giroud and Ronald K. Frobel are well qualified to present geomembranes to our readers. They are co-chairmen of the International Conference on Geomembranes, to be held in June 1984, sponsored by the Industrial Fabrics Association International (IFAI).*

*Dr Jean-Pierre Giroud, who in 1977 coined the terms "geotextile" and "geomembrane", is director of the Geotextiles and Geomembranes Group of Woodward-Clyde Consultants. A former professor of geotechnical engineering, J. P. Giroud has been instrumental, in the past fifteen years, for the development of the terminology related to geotextiles and geomembranes and the design of their applications. Dr Giroud, who has been consulting in all continents, is one of the very few internationally recognized experts in the field of geotextiles and geomembranes.*

*Ronald K. Frobel, MSCE, P.E., is materials engineer for the U.S. Bureau of Reclamation, Engineering and Research Center, Denver, Colorado. For the past eight years, R. K. Frobel has been involved in design, development and research on geomembranes and has authored several technical papers and reports on the subject. Mr. Frobel is currently co-chairman of ASTM subcommittee D18.20 (Impermeable Barriers) and chairman of ASTM task group D18.20.02 (Flexible Membrane Liners).* □

# Calendar

**23–25 January 1984**

ASTM D18 Committee on Soil and Rock in San Diego. Meeting includes sub-committees on impermeable barriers and geotextiles. Contact ASTM at 215-299-5498.

**20–24 June 1984**

International Conference on Geomembranes in Denver, Colorado, USA. Sponsored by Industrial Fabrics Association International and 10 other organizations. Approximately 100 papers will be delivered on all aspects of geomembranes. Trade exhibition will feature fabrics and films, services, and equipment. Contact Secretary General, International Conference on Geomembranes, % IFAI, 345 Cedar Bldg., Suite 450, St. Paul, MN 55101, USA.

**9–12 July, 1984**

International conference on the Development of Low-Cost and Energy Saving Construction Materials and Applications will be held in Rio de Janeiro, Brazil at Pontificia Universidade Catolica. For more information contact: Dr. H. Y. Fang, Department of Civil Engineering, Fritz Engineering Laboratory, Lehigh University, Bethlehem, PA 18015.

# O & M PLAN

# PLAN FOR

# OPERATIONS AND MAINTENANCE

# of

# SIGNAL BUTTE FRS

This guide applies to the Signal Butte Floodwater Retarding Structure and all associated works of improvement (Buckhorn-Mesa Watershed).

## GENERAL

Signal Butte FRS was designed as a flood-control dam. It collects diverted water from Pass Mountain Diversion and Outlet, Apache Junction FRS, Bulldog Floodway and its own uncontrolled watershed. This collected water is released into Signal Butte Floodway at a controlled discharge for all storms less than the 100-year event. Runoff greater than the 100-year design storm will be discharged into the natural channel (normally dry) just west of Meridian Road.

A regular system of inspection and maintenance will assure that this structure performs as designed.

The following suggestions are to be used as a guide to safe operation and maintenance of the dam and all its associated structures.

## EMERGENCY PREPAREDNESS

A plan for means of notification and recommended actions should be coordinated with organizations responsible for the safety of people downstream from Signal Butte FRS. The dam is designed to spill no water unless the storm is greater than a 100-year event. Emergency spillway discharge will, in that event, travel down the channel that passes the south east toe of Signal Butte (the hill). This natural channel is already braided and of restricted size in several locations. A formal Emergency Action Plan is recommended.

## OPERATION

The dam is designed to function without supervision. There is only one operational structure, the gated outlet.

The gated outlet serves two design purposes. One is to act as a means of totally evacuating the reservoir and borrow area. The other is to supply additional water to downstream vegetation.

Since floods on such a relatively small watershed can be sudden and at inconvenient times, the gate is intended to remain closed except under controlled circumstances. As this area is developed, some kind of warning to people downstream may be required before the gate is opened. In any case, the gate and its outlet should be under responsible supervision at all times when the gate is open.

## MAINTENANCE

### The Dam

Inspect the dam annually and after every major storm.

The top of the dam has been designed with a uniform cross slope for surface drainage and ease of maintenance. Maintain this surface to prevent ponding of rainwater on the top of the dam.

Check the top and side slopes for any signs of distress, such as cracks landslides or gullies. Any transverse cracks that cross the top of dam should be checked for depth or severity. Check to see if High-Density Polyethylene (HDPE) curtain is exposed by the crack. Check its condition. Repair any gullies, rills and small slides. Any complicated cracking or exposure of the HDPE curtain should be brought to the attention of the Soil Conservation Service for evaluation of cause and recommended repair.

## THE PRINCIPAL SPILLWAY

The principal spillway consists of a covered reinforced concrete inlet, a 36-inch conduit and a reinforced concrete impact basin at the outlet (Signal Butte Floodway). Check this structure for general condition every year. Remove any trash, debris, and sediment after every major storm or annually, whichever occurs first. Sediment is not expected to be a problem, since the outlet is normally self-cleaning, but long periods with no appreciable flow may permit sediment to become unusually resistant to removal by low to moderate discharge velocities.

### The Gated Outlet

The gated outlet consists of a 12-inch slide gate and trashrack with a long gate stem and wheel on the top of the dam; a 12-inch monolithic concrete pipe with a steel liner; a small "PWD" outlet structure; an inlet channel and an outlet channel with a short section of riprap. Open and close the gate periodically to assure that it is functional at all times. Clear the trash rack and "PWD" basin of any debris or obstruction as often as experience shows is necessary. Check both channels, the riprap and the structures for any visible evidence of erosion or deterioration. Make repairs as needed.

### The Emergency Spillway

The emergency spillway consists of a compacted earth apron, a reinforced concrete baffled apron drop structure and an outlet channel to a large natural wash. Check the apron and outlet channel for general condition, obstruction and erosion. Make any necessary repairs. Check the baffled apron drop for any obstructions, cracks or signs of structural distress. Clear any debris. Notify the Soil Conservation Service if any signs of structural distress are noted.

The walls at the inlet and outlet are designed to function as retaining walls. Some separation at the articulation joints is to be expected and is acceptable. If the waterstop tears or pulls away from the joint, notify the Soil Conservation Service for recommendations.

### The Diversion

The diversion is a minor channel at the east side of the reservoir that has been constructed to protect the end of the dam until Bulldog Floodway is completed. Bulldog Floodway will divert water that presently flows through a culvert under Meridian Road, at which time the diversion will no longer serve any design function. Until that time, check the channel and dike for any serious erosion and clear the channel if any major obstruction is noted.

Vegetation

Vegetation is to be encouraged to thrive and spread. Any large dead or dying vegetation that appears to affect the maintenance of the gated outlet should be cleared. Remove any plants on the dam that agronomists know to have a deep root system. Otherwise, bushes and grass are expected to increase the stability of the relatively coarse-grained embankment slopes.

## Critical Items

The dam, principal spillway, gated outlet and emergency spillway are important not only to this structure, but to the design function of the total system upstream from Signal Butte Floodway.

The diversion is not expected to require maintenance. Check for vandalism (dumping debris near the road) and remove if a potential exists for the material to reach the trash rack.

## O & M Inspection & Followup

It is the current practice of the Flood Control District of Maricopa County to make visual checks of structures on a quarterly basis with an annual inspection of all flood retarding structures on the east side of Maricopa County every fall. Include Signal Butte FRS in this excellent program.

The book entitled "State of Arizona Watersheds Operation and Maintenance Handbook" for projects installed with assistance from the Soil Conservation Service, U.S. Department of Agriculture, Soil Conservation Service dated May 1971 is herein made a part of this O & M Guide.

## Funds for O & M

Funds for O & M shall be provided by the Flood Control District of Maricopa County (the Sponsors).

# COST ESTIMATE

## ENGINEERS COST ESTIMATE
## SIGNAL BUTTE FLOODWATER RETARDING STRUCTURE

| Item No. | Work or Material | Spec. No. | Quantity | Unit | Unit Price | Amount |
|---|---|---|---|---|---|---|
| 1. | Mobilization | 8 | One | Lump Sum | XXXX | $65,000.00 |
| 2. | Water | 10 | 24,000 | 1000 Gal. | 1.26 | 30,240.00 |
| 3. | Cutoff Trench Excavation, Common | 21 | 42,580 | Cu. Yd. | 1.20 | 51,096.00 |
| 4. | Structure Excavation, Common | 21 | 5,339 | Cu. Yd. | 4.00 | 21,356.00 |
| 5. | Channel Excavation, Common, (E.S.) | 21 | 21,106 | Cu. Yd. | 1.20 | 25,327.20 |
| 6. | Channel Excavation, Common, (G.O.) | 21 | 1,044 | Cu. Yd. | 1.20 | 1,252.80 |
| 7. | Channel Excavation, Common, (P.S.) | 21 | 1,411 | Cu. Yd. | 1.20 | 1,693.20 |
| 8. | Channel Excavation, Common, (Div.) | 21 | 1,071 | Cu. Yd. | 1.70 | 1,820.70 |
| 9. | Earthfill | 23 | 500,636 | Cu. Yd. | 2.15 | 1,076,367.40 |
| 10. | Structure Backfill | 23 | 4,584 | Cu. Yd. | 10.00 | 45,840.00 |
| 11. | Concrete, Class 4000 (Colored) | 31 | 645 | Cu. Yd. | 285.00 | 183,825.00 |
| 12. | Concrete, Class 4000 | 31 | 103 | Cu. Yd. | 225.00 | 23,175.00 |
| 13. | Steel Reinforcement (E.S.) | 34 | 82,590 | Lbs. | 0.35 | 28,906.50 |
| 14. | Steel Reinforcement (other) | 34 | 12,108 | Lbs. | 0.38 | 4,601.04 |
| 15. | 36-Inch Pipe | 41 | One | Lump Sum | XXXX | 30,000.00 |
| 16. | Rock Riprap | 61 | 6.3 | Cu. Yd. | 20.00 | 126.00 |
| 17. | 12-Inch Slide Gate Assembly | 71 | One | Lump Sum | XXXX | 9,000.00 |
| 18. | Identification Sign | 81 | One | Lump Sum | XXXX | 2,000.00 |
| 19. | Gate and Guard Fence | 91 | One | Lump Sum | XXXX | 4,000.00 |
| 20. | Fence | 92 | 7,400 | Lin. Ft. | 1.50 | 11,100.00 |
| 21. | Surveys | 401 | One | Lump Sum | XXXX | 50,000.00 |
| 22. | HDPE Curtain | 402 | 162,516 | Sq. Ft. | 2.00 | 325,032.00 |
| | | | | **Total....** | | $1,991,758.84 |

AZ Buckhorn-Mesa WPP - Signal Butte FRS

SL 6-13-84

Engrs. Est. - Mobilization & water. 1 10

B.I.1 Mobilization -

1) Close to Phoenix - no great difficulty
HDPE sub-contractor might be slight problem (not expected to be local)
use <u>3½%</u> of total cost of contract.

2) B.I.2 Water - 1000 gal.

No water available on site

All estimates by guess - way too many uncontrollable variables.
Am including a bid item by <u>directive</u>
Do not recommend.

505,220

Soil to be compacted. -

| | | |
|---|---|---|
| from Pass Mtn - | 310,552 | cy. excav. |
| | - 180,655 | cy. fill (PM) |
| | - 27,098 | est. shrink (?) |
| Total: | 102,799 | cy. (use 15% - PM) |

Need 505,220 cy. earthfill to const. SB. FRS
assume excavate additional 20% (dam)
505,220 + 101,044 = 606,264 cy. (excavate

Assume most fill caliche - SM, field condition dry ~ 2% m.
optimum moisture generally 9 to 15%, use 12
12% - 2% = 10%, design min $\gamma_d = 112 \#/ft^3$
assume ave. $118 \#/ft^3$

$W_w = W_s(m) = 118(.1) = 11.8 \#/ft^3$

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Buckhorn-Mesa WPP, Signal Butte FRS

Engrs. Est - water, cont. 2 10

Embankment

$11.8\ \#/ft^3$ water × 505,220 cy. × $\frac{27\ ft^3}{cy}$ = 1[illegible]

160,963,092 # $H_2O$

$\frac{160{,}963{,}092\ \#}{62.4\ \#/ft^3} \times \frac{7.48\ gal}{ft^3}$ = 19,294,935 gal.

or 19,295 Mgal.

Replace evap. losses est. 20%

23,154 Mgal.

est. $1.46/cy. 10.00/

Dust Abatement (?)

606,264 cy. excavate, ave. haul ~ [illegible]

102,800 $\left(\frac{3000\ ft}{2} + 1650'\right)$ = 323,820,000

503,464 (1200 ft) = 604,156,800

927,976,800 cy.-ft.

ave. haul = 1,530.65 ft.

$1 + [illegible]/mo + tax 5%

peak use Mgal/mo 5,600 gal/day 20,000 gal/day precut.

assume 10,000 cy. /day ~ 50 wkg. days.

@ 5 days. /week, 10 weeks plus start and stop time — use 65 days.

Say 2 months @ 250 Mgal/mo. = 500 Mgal

4 months @ 80 Mgal/mo. = 320 Mgal.

6 months. 820 Mgal.

first qd. 25,000 gal/mo. 80,000 reg./mo.

(based on contractor @ Signal Butte Floodway)

23,154 + 820 = 23,974 or 24,000 MGal

AZ Buckhorn Mesa WPP – Signal Butte FRS
SL 6-13-84
Engrs. Est. – water, cont. 3 10

Contracta @ Signal Butte Floodway
pays $1/Mgal + $4/month + 5% tax
6-13-84 Telcon. Tom Tayo

(1)(24,000) + (4)(6 months) + 5%($24,000)
= $25,224 unit price/Mgal = $1.051
+ 15% for profit & oh.
+ 5% for unknowns
1.2 × 1.051 = $1.2612 use $1.26

BI. 3 Cutoff excavation, common.

expect easy material except @ bottom – Contractor working adjacent for $1.45/cy. – no problems. Most excavation wasted, use $1.20/cy.

BI 4 Structure excavation
est. 4.00
most should be easy, em. spy. difficult?

BI 5, 6 & 7 essentially the same as 3
same as Signal Butte Floodway. $1.20

BI 8 Diversion, may hit something more difficult – use $1.70

BI 9 Earthfill est 90-103% Std. Proctor
workable mix – no fill removed if
112#/ft³ or more –

AZ Buckhorn-Mesa WPP, Signal Butte FRS

SL 6-14-84

Engrs. Est. 4 10

Earthfill - BI 9 cont.

Start w/ 50¢/cy; regular fill using pd. excav.

505,220 cy. compacted fill, 606,264 exc.

102,799 cy. from Pass Mtn. - excav.
72,551 cy. from SB excav.

175,350 cy. paid excav. -

1071
1411
1044
2106
5339
42,580

leaves 430,914 cy. to be excavated.
assume 1.10 to excavate
(no neat lines)
exc. $474,005.40

if regular fill would be 50¢/cy.
(95% SPD & ±2% m, homogeneous fill)

SB FRS has 2 zones (each side of line)
which requires special routing

min 112#/ft³ which is 90-102% STP
no moisture req't, but will need to
work caliche & ... carefully

say double 50¢ to $1.00 on embankment

175,350 cy (1.00) = $175,350
430,914 cy (1.1+1.0) = 904,919.4

505,220 cy. fill @ $1,080,269.4

is $2.138 / cy. use $2.15

AZ Signal Butte FRS, Buckhorn Mesa WPP
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Engrs. Est. -cont. 5 10

BI-10 Structure Backfill, use $10^{00}$, not much of it

BI-11 Concrete - E5 (colored)

645 cy.

reviewed bids on RWCD Reach II and Signal Butte Floodway.

RWCD Reach II bids ranged 147.77 to 157.71 (1st 4)

the installation part of that bid ranged from $75 to $110

-RWCD concrete quantity was 11,077 c.y.

forming emergency spillway blocks is complicated and expensive.

$160_{base} - 80_{forming} + 3(80_{forming}) = \$320$

add $10 for color, $$330^{00}$ (supplier recom.)

Signal Butte Floodway - bids, (1st 5) ranged $205 to 220/c.y. more forming and 6,334 c.y.

Structural forming $160 to 300/c.y. forming still not as complicated, but close and only 177 cy.

$160_{base} + 160_{forming} = \$320^{00} + 10$

AZ Signal Butte FRS - Buckhorn-Mesa WPP
SL 8-24-84.
Engrs. Est. 6 10

BI-11 Concrete - ES cont.

\$330.00/cy. appears reasonalbe for the colored baffled apron -
-\$45 for steel = 285

BI 12 - concrete other

forming not as complicated, but lots of it.
still rel. small quantity.

use ave. \$210/cy. (SB Floodway) for ALL concrete

ave. for steel and cement was \$75
80 \$135 ave for forming ALL concrete.

use \$200 forming
75 steel & cement
\$275 Total est.
-\$50 steel reinf. = \$225.00

BI-13 Steel for ES

\$45/cy. x 645 cy = 29,025
÷ 82,590# = 0.35xx/#

0.35 (82,590) = 28,906.50 OK.
Very reasonable.

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8-24-84
Engrs. Est.

Signal Butte FRS - Buckhorn-Mesa WPP

7 10

BI-14 steel for "other" concrete.

@ $50 approx cost / cy.

50 (64) = $3,200

÷ 8390# = 0.38

too much temp. steel - not reasonable - (modify. BI 12)

use $0.55 (8207) = $4,513.85

25#/cy.

add gated outlet pipe -

note - 261 cy. non-reinf. concrete @ 225.00/cy. = $58,725.00

subsidiary to HDPE curtain. BI # 22

÷ 162,516 ft² = 36¢/ft²

1.64 + .36 = $2.00

BI 15. 36-inch Pipe -

148 LF. Price Bros
FOB Apache Junction
roughly 95.00 to contractor

assume 200.00 installed
x 148
$29,600 use $30,000

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Signal Butte FRS - Buckhorn-Mesa WPP

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Engrs. Est,

8 10

BI-16 rock riprap —

can go from $8 to $40

very small quantity —

assume $20.00/cy.

BI-17 12-inch gate assembly

discussed with Bill Johnson, ARMCO

use flat-back gate and bolt to concrete

approx. 102' gate stem.

crank lift.

sold 18" gate SD-10, similar, recent for $4619 (hardware)

use that & double for $9000.00 installed cost.

BI-18 ID Sign —

bids range widely —

assume 1 sign — $2000

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Signal Butte FRS - Buckhorn-Mesa W

8-24-84
Engrs. Est.

9 10

BI-19 Gate & Guard fence
use lump sum

<200' chain link fence
lots of small fittings and
end posts.
assume $4000

BI-20 fence 7,400 ft.

prior bids $1.00 to $1.50 (SB, Floodway)
on 30,000 ft. of fence.
use $1.50

BI-21 Surveys - nebulous

checked bids on 2 jobs this
was a bid item on,
generally 1.5-2% of total
contract less Mobilization -

Because HDPE curtain may increase
survey demands, use approx 2.5%
or $50,000.00

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Signal Butte FRS

8-24-84

Engrs. Est.

10

BI-22 HDPE Curtain

The Army COE uses $1.00/ft² for installed cost estimates on 100 mil HDPE —
This installation included continuous feed of a 34' x 500' roll of HDPE in a 34' trench with slurry (bentonite) in cold weather.
The manufacturer has quoted anything from 73¢ to $1.00/ft² installed.

Since it is a subcontract with additional administrative costs, a type of installation that is very unusual and with roughly $.36/ft² of anchor expense to add on, I use
$2.00/ft² for estimate.