GEOTECHNICAL INVESTIGATION
AND
DESIGN ANALYSIS
FOR

SAVERY DAM

August 1982

TO: THE WYOMING WATER DEVELOPMENT COMMISSION
GEOTECHNICAL INVESTIGATION
AND
DESIGN ANALYSIS
FOR

SAVERY DAM

August 1982

To: The Wyoming Water Development Commission

ROLLINS, BROWN AND GUNNELL, INC.
Professional Engineers
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I. PROJECT PURPOSE AND LOCATION

The proposed Upper Savery Dam is located in Section 35 and Section 36, Township 15 North, Range 89 West of the 6 P.M., as shown in Figure No. 1. The damsite is located on the Savery River approximately 12.5 miles north of the town of Savery, Wyoming. In 1980, Banner Associates, Inc. of Cheyenne, Wyoming, performed a feasibility study for "Stage 3" of the Little Snake River Water Management Project. This project, which includes the Upper Savery Dam and Reservoir, is part of Wyoming's effort to develop their land and water resources. Major work areas considered in the study by Banner Associates included hydrology, geology, engineering and construction costs. The geotechnical portion of the Banner study included only the surface geological conditions. The study recommended, however, that a detailed geotechnical investigation be performed at the site selected for the Upper Savery Dam and Reservoir.

The purpose of this investigation was to perform a detailed geotechnical investigation and to design an earth dam compatible with the subsurface conditions at the site and the available embankment material throughout the area. Since this investigation was limited to the geotechnical design aspects of the project, the spillway design and design of the outlet works shown on the drawings, follow the general designs prepared by Banner Associates.

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II. GENERAL SITE CONDITIONS AND FEATURES OF THE PROPOSED DAM AND RESERVOIR BASIN

The Upper Savery Reservoir is located in a wide valley in which the vegetative cover consists predominantly of brush and a few scrub trees along the river. The location selected for the Upper Savery Dam by Banner Associates is approximately one-half mile upstream from the damsite originally studied by the U.S. Bureau of Reclamation. The flood routing and optimizing studies performed by Banner Associates utilized USGS maps, and based on this data, the length of the dam was 3,600 feet and the height of the dam was 125 feet.

The crest of the dam was located at elevation 7,172 with a normal high water level at 7,150 and a maximum high water level at 7,166.9. The active storage within the reservoir basin was 57,510 acre-feet with a freeboard at normal high water level of 22 feet.

A side channel spillway with a cut and cover chute capable of passing 6,100 cfs was proposed for construction in the right abutment. The outlet works contemplated a six-foot pressure tunnel approximately 400 feet long, and an eight-foot non-pressure tunnel approximately 450 feet long. Control gates for the outlet works would be located between the six-foot and eight-foot conduits. A vertical shaft, extending from the top of the dam to the gate chamber, would provide
access to the gates. The maximum outlet capacity for the outlet works indicated above would be 775 cfs.

It was assumed in the Banner report that the outlet works would be located in the vicinity of the right abutment and that energy dissipation for both the spillway and the outlet works would be accomplished using a hydraulic-jump basin.
III. SCOPE OF THE GEOTECHNICAL INVESTIGATIONS AND THE DESIGN ANALYSIS

The work performed during this investigation has been completed in accordance with the contract between our organization and the Wyoming Water Development Commission, dated September 5, 1981.


The results of the investigation are outlined in the following sections of the report and the discussion generally follows the scope of the work summarized above.
IV. GEOLOGY OF DAMSITE AND RESERVOIR BASIN

1. Introduction

The Upper Savery Damsite is located on Savery Creek about one mile below the confluence with Little Savery Creek, in Southern Carbon County, Wyoming. This area is situated near the junction of two physiographic provinces—the Wyoming Basin and the Southern Rocky Mountains. At the Upper Savery site, the topography is dominated by a gently-arched peneplain capped by the Brown's Park formation and eroded discordantly by the major drainage. Both Savery and Little Savery Creeks are perennial streams which head in the Sierra Madre Mountains to the east and flow as meandering streams confined within wide but steep-walled valleys.

Bird Gulch and Coal Gulch are intermittent streams nourished by springs at the head of steep V-shaped valleys.

2. Site Geology

The dam foundation and abutment are located in the Cretaceous Steele shale. The Steele shale, described by Gill (1970), conformably overlies the Niobrara formation and consists of 3,000 to 4,000 feet of dark gray shale containing sparse layers of gray weathering limestone concretions and thin beds of fine-grained sandstone and siltstone. Drill hole logs reveal that, at the Upper Savery site, the Steele shale consists of dark gray shale, light gray sandstone and cross-
laminated shale, siltstone and sandstone. The Steele shale is of marine origin and weathers readily to form areas of low relief. The uppermost portion of the Steele shale contains sandy and silty shale and grades conformably into the overlying Haystack Mountain formation.

The Haystack Mountain formation, of Upper Cretaceous age, outcrops along the valley sidewalls of Coal Gulch, Bird Gulch and Savery Creek below the damsite. It is a lithologically diverse formation, deposited in a nearshore and shallow offshore marine environment, and consists of 800 to 2,500 feet of interbedded sandstones and shales. The sandstone is generally pale yellowish gray, fine grained and thin bedded. The Haystack Mountain formation contains three ledge-forming sandstone units, and the middle unit has been referred to as the Savery Creek sandstone (Barclay, 1974).

Unconformably overlying the gently dipping Cretaceous strata is the Miocene Brown's Park formation, a continental deposit laid down over the erosion surface formed on the Steele shale and Haystack Mountain formation. The Brown's Park is described by Bradley (1964) and is composed of a basal conglomerate overlain by white sandy volcanic tuff and tuffaceous sandstone, mudstone, and a few thin layers of greenish gray clayey mudstone. In the Upper Savery area, the basal conglomerate contains 10 to 20 feet of rounded pebbly gravel derived from the Sierra Madre highland to the east.

The valley floor along Savery Creek contains many abandoned meanders and oxbow depressions which have become
filled with silt and muck. The valley fill is composed of 2 to 26 feet of Quaternary sand and gravel, overlain by 4 to 8 feet of topsoil and muck containing many roots. The valley fill rests on a weathered Steele shale.

At the damsite, the valley floor rests 600 feet below the flood plain, whereas upstream near the confluence with Little Savery, the valley floor rests 250 feet below the plain. The sideslopes are covered with alluvial fan, colluvial, terrace gravel and landslide deposits, as shown in Figure No. 2. The alluvial and colluvial deposits consist of silty sandy clay with bits of weathered shale and mudstone. Terrace gravel deposits, remnants of former stream channels, occur as irregular patches of silty, sandy gravel at various elevations along the valley sides. Test pits dug in several of the terrace gravel deposits show them, on average, to contain 4 to 8 feet of the silty sandy gravel resting atop colluvium and weathered shale. The terrace gravel deposits are widely scattered and each contains from several thousand up to 50,000 cubic yards of material suitable for concrete aggregate. In addition to the terraces, a gravel bench 20 acres in area and estimated to contain 210,000 yards of sand and gravel was located by test pit excavation near the confluence of Savery Creek and Little Savery Creek. This gravel bench, together with the terrace just above it, should provide a source of concrete aggregate.
A major feature of the slopes in the Savery Creek area is the presence of numerous landslides. In the immediate vicinity of the Upper Savery Dam and Reservoir, there are some 23 discrete landslides varying in area from 3 acres to 83 acres as measured by planimeter. About one-half mile downstream from the proposed dam location, a landslide measuring 1.7 square miles in area has pushed out onto the Savery Creek floodplain. Test pits dug on several landslides show that the material is generally silty gravelly clay with occasional fragments of weathered shale and siltstone. Landslide material upstream is derived from the Steele shale and Brown's Park formation and is more homogeneous than landslide material downstream which is derived from Brown's Park, Haystack Mountain and Steele shale formations and contains more fragments of weathered siltstone and mudstone. From the test pit information, landslide material in the Bird Gulch area appears to be the most uniform deposit of impervious borrow material.

3. Geologic Hazards
A. Seismic Risk

The Savery Dam site is located within Seismic Risk Zone I, as shown in Figure No. 3. Although some mild earthquake can be expected, movement will be slight and damage minor. Minor earthquakes have occurred in this area; the nearest, a magnitude 4.2 shock, took place in 1977 at a distance of 14 miles, as shown in Figure No. 4. Both seismic wave amplitude and the resultant ground acceleration are

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attenuated with increasing distance from an earthquake epicenter. McDonal (1958) reports that in shale, wave amplitude decreases by a relation $A = T^{-n}$, $1.98 < n < 2.26$, where $T$ is the travel time. Attenuation curves for the maximum acceleration recorded on rock sites have been published by Schnabel and Seed (1973), and show that an earthquake of magnitude 5.2 occurring at a distance of 14 miles will produce an acceleration of about 0.04 g at the damsite. The preliminary geologic map of the Tullis quadrangle published by Barclay (1976) shows the presence of three faults in the Sandstone Creek area about eight miles from the Savery Damsite. These faults are all short, being less than one mile each in length. The length of a fault rupture is proportional to the magnitude of the earthquake caused by the rupture, and this length is used to determine the maximum credible earthquake (Slosson, 1974). From the above considerations, it can be concluded that there is a low potential for damaging earthquakes and that earthquakes that do occur will have minor impact on the Savery Creek Damsite.

B. Landslides

A geologic hazard is created by the presence of numerous landslides in the reservoir area. About 23 numerous landslides have been mapped upstream from the Savery Damsite, varying in area from 3 to 83 acres. Fourteen of these slides will be partially covered by water at the normal high water level. Zaruba (1969) and Sharpe (1938) have classified landslides by origin and type of movement. The Cretaceous strata
dip at a three to four degree angle to the west-southwest, while the Miocene Brown's Park, in angular unconformity, dips two to three degrees to the east. The large slides along Savery Creek and Bird Gulch are earthflows along shearing planes developed in the gently dipping Cretaceous strata. Differential weathering of the interbedded sandstones and shales has produced planes of separation along the original bedding planes, which allows sliding of the overlying material down the dip slope. Some of the smaller slides on the upper slopes and along Little Savery Creek are slumps developed as material slips downward on relatively steep slopes with backward rotation of the slide blocks. These slumps have developed where slopes have been undercut by erosion. Slump slides are indicated by the backward rotation of head blocks; earthflows are indicated by the presence of tension cracks and the hummocky, poorly drained topography developed on the surface. In both the earthflow and slump landslides movement has been slow, as indicated by the lack of sheet flow onto the flood plain and the lack of any riding of the slide material up the opposing slopes. Sliding of sandstones superjacent to claystones has been shown to depend on the amount of rainfall (Zarabu 1969, Hardy 1966). Of the many factors acting simultaneously on a slide mass, the effect of groundwater is generally considered most important. Berry (1960) reports that the basal conglomerate of the Brown's Park formation is a good aquifer and supports many springs at its contact with underlying less permeable formations. Such springs along the upper
slopes increase pore pressure for the sliding plane at the base of some of the slides along the upper slopes of Savery and Little Savery Creeks. The opening of tensile cracks in the upper reaches of earthflows increases the porosity and admits rainwater. This reported availability of groundwater must be reconciled with the fact that, except for two slumps along Little Savery Creek, there is no evidence of recent movement in any of the slide masses. Studies done on the large landslide downstream from the dam (Crandall, 1971) reveal that no significant movement has occurred in the last 30 years. Of the earthflows along Savery Creek and Bird Gulch, none show evidence of recent movement and some have developed drainage patterns indicating long-term stability. It is possible that most of the landslides are dormant and fossil landslides which developed on slopes cut by Savery Creek when it flowed at higher levels and deposited the terrace gravels. These slopes have since stabilized. The terrace gravels were probably deposited during the Pleistocene, and the increased precipitation at that time provided sufficient groundwater lubrication to get the landslides started. The drier climatic regime today (Berry, 1960) no longer provides sufficient moisture to the slide planes, and the slides have stabilized. This stability may be affected by either increasing the available water, or by changing the load.

As indicated above, a number of slides within the reservoir basin will be inundated by reservoir storage. There will likely be some renewed movement of some of the slides
with the drawdown of the reservoir; however, they should pose no threat to the project, since they are too small and move too slowly.
V. SUBSURFACE GEOLOGICAL CONDITIONS

1. Subsurface Investigations Along the Dam Axis

The characteristics of the subsurface material along the proposed dam axis were defined by drilling 12 test holes to depths varying from 60 to 135 feet. The locations of the test holes along the axis of the dam, along with a profile of the ground surface and the bedrock defined by the test borings, are shown in Figure No. 5. The logs for each of the test holes drilled along the axis are also presented in Figure No. 5.

It will be noted from the boring logs and the profile shown in Figure No. 5 that the maximum depth of overburden in both abutments and in the river bottoms is about 28 feet. It is significant to note, however, that the depth of the overburden in the left abutment between Station 21+60 and Station 15+45 is substantially less than the depth of the overburden in the right abutment between Station 30+78 and Station 33+50. The latter area on the right abutment corresponds to the general area where the outlet works were contemplated in the preliminary investigation prepared by Banner Associates, Inc.

During the subsurface investigation, field permeability tests were performed at approximately 10-foot intervals in all test borings drilled along the axis of the dam. Field permeability tests were performed both in the overburden
and the bedrock in accordance with Designation E-18 of the U.S. Bureau of Reclamation's Earth Manual. The results of the permeability tests are tabulated at the bottom of each test hole in terms of the permeability coefficient associated with Darcy's Law.

It will be observed from the boring logs that the overburden material in the river bottoms generally consists of a silt to silty clay layer varying in thickness from about 5 to 7 feet, underlain by a sand to sandy gravel which extends to the depth at which the bedrock was encountered. The surface silt layer is relatively impervious; however, the permeability coefficient for the sand to sandy gravel varies from about 7,000 to 17,000 feet per year. The bedrock material in the river bottoms and in the right abutments consists predominantly of shale which grades from a siltstone to a claystone, while the right abutment consists of laminated light gray fine sandstone and dark gray shale.

In the river bottoms, bedrock material was relatively impervious with permeability coefficients varying from about 200 feet per year to as low as 2 feet per year, with the major portion of the bedrock having permeability coefficient of less than 100 feet per year.

The permeability coefficients of the bedrock in the abutments in the upper portion of the profile were somewhat greater than the permeability coefficients encountered in the bedrock in the river bottoms. Test Hole 1, which was located
outside of the end of the dam, indicated permeability coefficients of between 100 and 240 feet per year below a depth of 19 feet below the ground surface. Test Hole 12, located outside of the dam on the right abutment, had permeability coefficients varying from 584 to 1,350 feet per year to a depth of 50 feet. Below this depth, the permeability coefficient varied from 8 to 276 feet per year. It should be noted, however, that the water surface elevation in the reservoir is below the area where the higher permeability coefficients in this hole exist. The highest permeability coefficients in the bedrock occurred in Test Hole 4 in the left abutment, where the permeability coefficients to a depth of 61 feet below the ground surface varied from 1,590 feet per year to nearly 4,000 feet per year. In essentially all of the abutment holes, the permeability coefficients approach small values at depths of below 50 to 60 feet.

It should be noted that the permeability coefficients shown in Figure No. 5 have been calculated on the basis of the water levels observed in the drill holes during the field operations. In order to insure that the water level in the drill holes consisted of the natural groundwater level, each test hole in the abutments was blown free of water using an air compressor. Perforated PVC pipe was installed in Test Holes 1, 4, 10, 11 and 12 to obtain a better indication of the actual groundwater surface along the axis of the dam.

In order to obtain a better indication of the characteristics of the overburden material between Station 0+00 and
Station 12+60, four test pits were excavated in the overburden material. The logs for the test pits are designated as 501 through 504 and are presented in Figure No. 14 along with the station at which each test pit was dug. The characteristics of this material will be discussed in a subsequent section of this report.

It should also be observed from the test pit logs that sampling was performed in the overburden material at approximately 5-foot intervals throughout the overburden depth. Both disturbed and undisturbed samples were obtained during the field investigations. Disturbed samples were obtained by driving a 2-inch, split-spoon sampling tube through a distance of 18 inches using a 140-pound weight dropped from a distance of 30 inches. The number of blows to drive the sampling spoon through each 6 inches of penetration is presented on the boring logs. The sum of the last two blow counts, which represents the number of blows to drive the sampling spoon through 12 inches, is defined as the standard penetration value. The standard penetration value provides a reasonably good indication of the in-place density of sandy-type materials, however, it must be used with some caution in interpreting the in-place density of gravelly-type soils, particularly where the particle size exceeds the inside diameter of the sampling spoon.

Undisturbed samples were obtained at various locations throughout the overburden material by pushing a 2.5-inch, thin-walled Shelby tube into the subsurface material using the hydraulic pressure on the drill rig. Each sample of the overburden material obtained in the field was classified in the laboratory according to the Unified Soil Classification...
System. The symbol designating the soil type according to this system is presented on the boring logs. A description of the Unified Soil Classification System is presented in Figure No. 6, and the meaning of the various symbols shown on the boring logs can be obtained from this figure.

2. **Subsurface Investigation Upstream and Downstream From the Axis and Throughout the Damsite Area**

The characteristics of the material upstream and downstream from the axis were defined by drilling four test borings to depths varying from 20 to 64 feet at locations as shown in Figure No. 7. The logs for the four test holes are also presented in Figure No. 7. It will be noted from the boring logs for Test Holes 13 and 14 that the depth of the overburden varied from 20 feet in Test Hole 13 to 30 feet in Test Hole 14. The depths of the overburden in these two test holes compared favorably with the depths of the overburden determined along the axis of the dam.

Test Holes 15 and 16 were drilled downstream from the axis of the dam to obtain more information on the characteristics of the granular deposits throughout the profile in this area. Sampling was performed in the overburden material at between 3- and 5-foot intervals, and the locations at which the samples were obtained are presented on the boring logs. The same general information was obtained for these test holes as was obtained during the drilling of the overburden material along the axis of the dam. The standard penetration values and the soil classifications, according to the Unified Soil Classification System, are presented on the boring logs.
The results of the standard penetration tests indicate that the granular material is generally in a medium-dense condition; however, some low density spots appear to exist throughout the site. Field permeability tests were performed in each of the test holes in accordance with Designation E-18 of the U.S. Bureau of Reclamation's Earth Manual, and the results of the permeability tests, expressed in terms of the permeability coefficient, are shown below the boring logs.

It will be observed that a relatively pervious zone exists in the bedrock in Test Hole 13 at a depth of between 22 and 40 feet below the existing ground surface, with the permeability coefficient between 22 and 33 feet exceeding 104,000 feet per year. Between 33 and 44 feet below the ground surface, the permeability coefficient was about 1,500 feet per year. At all other locations tested in the bedrock, the permeability coefficient was generally less than 54 feet per year. The permeability coefficient in the overburden in this test hole compared favorably with the permeability coefficient in the granular materials along the axis of the dam. In Test Holes 15 and 16, however, the permeability characteristics of the sand and gravel were relatively low, indicating that this material must contain an appreciable quantity of material in the silt- and clay-size range.

3. **Subsurface Investigations Along the Proposed Spillway Alignment**

The characteristics of the subsurface material along the proposed spillway alignment were defined by drilling three
additional test holes, designated as Test Holes 19, 20 and 21. The locations and logs for these three holes are presented in Figure No. 7. Test Hole 11 is also shown in Figure No. 7, even though it was a centerline hole, since it is essentially on the spillway alignment.

The most significant characteristic of these holes is the depth of the overburden which exists along the right abutment. It will be noted that the depth of overburden varied from about 12 feet in Test Hole 20 to about 36 feet in Test Hole 19. The depth of the overburden in Test Hole 21, which is generally located along the alignment of the outlet works, as proposed in the Banner report, is over 24 feet. Test Hole 9, which is fairly close to the proposed alignment of the outlet works, as proposed in the Banner report, indicates an overburden depth of 25 feet.

Sampling, logging and classification procedures performed in the overburden followed the procedures previously outlined in this report. It will be noted that the subsurface material in the overburden in Test Hole 19 consisted predominantly of clay material classifying as CL-2 material according to the Unified Soil Classification System.

The bedrock underlying the spillway alignment consisted predominantly of brown to gray shale. Permeability tests performed in the drill holes along the spillway alignment indicate some variability in the permeability coefficient; however, in general, the overburden material, as well as the bedrock, has relatively low permeabilities, characteristic of the bedrock materials encountered at other locations throughout the damsite area.
4. Subsurface Investigations Along the Outlet Alignment

Inasmuch as the depth of the overburden along the alignment proposed for the outlet structure in the Banner report consisted of at least 25 feet of compressible clay, it is our opinion that a more favorable location for the outlet conduit would in the vicinity of the left abutment, where the bedrock, as defined by Test Hole 5, was within 6 feet of the existing ground surface.

Two additional test holes were drilled upstream and downstream from Test Hole 5, and the locations of these test holes along with the boring logs are presented in Figure No. 7. In Test Hole 17, which is located upstream from the dam axis, the depth to bedrock was about 9 feet, while in Test Hole 18, which is located downstream from the dam axis, the bedrock was approximately 11 feet deep. Laminated dark gray shale and light gray fine sandstone characterized the bedrock in the left abutment.

Field permeability tests were performed in both Test Holes 17 and 18, and the results of the field permeability tests are shown below the boring logs. It will be noted that the permeability coefficient is relatively low in all of these test holes and is characteristic of the sandstones and shales in most of the damsite area.
VI. AVAILABLE EMBANKMENT MATERIAL

The available embankment material defined during this investigation is primarily located in four major areas. The area designated as Borrow Area 100 is located up Bird Gulch. The borrow area designated as 200 is located immediately upstream of the left abutment. The borrow area designated as 300 is located at the upstream end of the reservoir basin, and the borrow area designated as 400 is located downstream from the left abutment.

During the initial investigation to determine suitable sources of available embankment material, a number of other test holes were excavated downstream and upstream from the proposed dam axis. These test pits do not constitute a major source of available material; however, it is possible that some embankment material could be obtained from these areas if it becomes necessary during construction. A total of 152 test pits have been excavated during the investigation including test pits in the miscellaneous area indicated above.

1. Borrow Area 100 - Bird Gulch

The general area investigated for available embankment material in Bird Gulch is shown in Figure No. 8. The locations at which the test pits were excavated in this area are also defined by coordinates in Figure No. 8. The materials available for use in the embankment in the Bird Gulch area
consist of alluvial deposits on the northerly side of the stream and landslide materials immediately uphill from the alluvial deposits. The alluvial deposits in Bird Gulch represent the primary source of the impervious material for the dam.

Thirty-nine test pits were excavated throughout the entire area, and the logs for these test pits are presented in Figure No. 9. In general, the alluvial deposits consist of medium to low plasticity cohesive soils generally classifying as CL-ML, CL-1 and CL-2 materials according to the Unified Soil Classification System. The test pits varied in depth from approximately 10 to 18 feet, and we estimate that a little over 2,000,000 yards of the alluvial material from Bird Gulch is available for use in the dam.

A substantial amount of slide material is also located in Bird Gulch, and the test pits excavated in this area indicate that this material has basically the same characteristics as the alluvial deposits. Considering both the landslide material and the alluvial deposits in Bird Gulch, we estimate that the total available material is over 2,600,000 yards.

2. **Borrow Area 200 - Left Abutment Upstream**

The general location of Borrow Area 200 is presented in Figure No. 10, and the locations of the test pits along with their logs are also presented in this figure. The alluvial terrace deposits are quite heterogeneous and classify as CL-ML, CL-1, CL-2, GC and GW materials according to the
Unified Soil Classification System. This means that the cohesive materials have medium to low plasticity characteristics. It will be noted from Figure No. 10 that the test pits excavated in this area varied from about 10 to 15 feet. We estimate that approximately 450,000 yards of material are available in this area.

3. Borrow Area 300 – Upstream End of Reservoir Basin

The general location of Borrow Area 300 is shown in Figure No. 11, and the locations at which test pits were excavated in this area are defined by coordinates. Borrow Area 300 represents the primary source of granular material for use in the proposed facility. Thirty-nine test pits, generally varying in depth from 10 to 12 feet, were excavated in this area. The logs for these test pits are presented in Figure No. 12, and it will be noted that a surface layer of silt or clay 2 to 3 feet thick covers the granular material in a number of the test pit areas. The granular material throughout the site is predominantly sand and gravel; however, a few test holes were encountered where the subsurface material consisted of sand, silt and clay.

In order to define the particle-size distribution characteristics of the subsurface materials in this area, bulk samples were analyzed in 10 test pits excavated throughout this site. We estimate that approximately 210,000 yards of granular material are available in this area.
4. **Borrow Area 400 - Left Abutment Downstream**

The general location of Borrow Area 400 is presented in Figure No. 13. The locations of the 30 test pits excavated in this area are also presented in Figure No. 13. The logs for the test pits excavated in Borrow Area 400 are presented in Figure No. 14, and it will be noted that the terrace deposits located on the northerly portion of the borrow area are predominantly cohesive-type materials; however, some granular soils are located in this region. The cohesive soils throughout the terrace deposit area of Borrow Area 400 generally classify as ML, CL-ML or CL-2 type materials, while the granular material generally classifies as GM or GW type soils according to the Unified Soil Classification System.

The southerly portion of Borrow Area 400 consisted primarily of slide material. The test pits excavated in the slide indicated that these materials are quite heterogeneous and generally consist of gravelly-type soils interbedded with medium to low plasticity clays. It should be noted that the slide material extends for a substantial distance in a southerly direction from the slide area shown in Figure No. 13, and an abundance of this material exists throughout this general area. We estimate that approximately 1,000,000 yards of material exist in Borrow Area 400.

5. **Miscellaneous Areas**

As indicated earlier in this report, a number of test pits have been excavated in miscellaneous areas both upstream
and downstream from the proposed axis of the dam. The location of these test pits, along with their logs, are presented in Figure No. 15. A number of these test pits were excavated in the river bottoms, and the logs for these test holes compare favorably with the boring logs drilled along the axis of the dam. A surface silt layer overlies sand and gravel throughout the reservoir basin. The granular materials underlying the surface silt generally consist of sand and sandy gravel and are a possible source of material for use in filters throughout the proposed facility.

Most of the granular materials existing in the river bottoms are below the water table, and excavating and stockpiling these materials will be more expensive than obtaining granular materials from Borrow Area 300 in the upper end of the reservoir basin.

6. **Available Material for Slope Protection**

Two possible sources of slope protection have been considered during this investigation. Source No. 1 is the sandstone caprock located north of the reservoir basin. The second source of riprap is located approximately 6 to 9 miles from the damsite up Little Savery Creek. The material in this area is metamorphic rock with low abrasion characteristics.
RESULTS OF LABORATORY TESTS
VIII. RESULTS OF LABORATORY TESTS

1. Foundation Materials in the Embankment Area

A. Classification Tests

Classification tests were performed on the foundation material obtained from Test Holes 2, 4, 6, 7, 9, 11, 13, 14, 19 and 21 and Test Pits 501, 502, 503 and 504. The classification tests performed on samples obtained from Test Holes 6, 7, 13 and 14 generally define the characteristics of the surface silt layer in the river bottoms and the granular layer existing between the surface silt layer and the shale. Atterberg limits were performed on the surface cohesive zone and on the shale materials underlying the granular zone. Mechanical analyses were performed on the granular zone between the surface cohesive zone and the shale.

A summary of all of the classification tests performed on the drill holes and the test pits indicated above is presented in Table No. 1, Summary of Test Data. The Atterberg limits performed on the surface cohesive material in the river bottoms indicate that this zone classifies as a CL-ML or a CL-1 type material according to the Unified Soil Classification System. The results of the mechanical analyses performed on the granular zone in the river bottoms indicate that this material varies from an SM to a GW or a GM type material and that the amount of material in the silt- and clay-size range is generally less than 20 percent.
The general characteristics of the overburden material in the right abutment were defined by performing classification tests on representative samples obtained from Drill Holes 9, 11, 19 and 21. The results of Atterberg limits performed on these samples indicate that the cohesive materials in the right abutment generally classify as CL-1 or CL-2 type clays. The results of the Atterberg limits for the right abutment are presented in Table No. 1, Summary of Test Data.

The characteristics of the overburden material in the left abutment were defined by performing classification tests on samples obtained from Drill Hole 2 and Test Pits 501 through 504. The results of the Atterberg limits performed on these samples are presented in Table No. 1, and it will be noted that the overburden cohesive material in this area classifies as a CL-1 or a CL-2 type soil according to the Unified Soil Classification System. The results of a bulk gradation test performed on the granular material from Test Pit 503 is presented in Figure 42.

The Atterberg limits performed on samples of the bedrock material indicated that the shale materials generally classified as CL-ML or CL-1 type materials. Some samples classifying as a CL-2 or a CH material were noted in the soil profile, however. The results of the classification tests are presented in Table No. 1, Summary of Test Data.

B. Consolidation Tests on the Overburden Material

The compressibility characteristics of the overburden throughout the proposed damsite were defined by
performing consolidation tests on representative samples obtained from Drill Holes 7, 9, 11, 14, 19, 21 and Test Pits 501, 502 and 503. The results of these tests are presented in Figures 16 through 30. The compressibility characteristics of the overburden materials on the right abutment were defined by samples obtained from Test Holes 9, 11, 19 and 21. The results of these tests indicate that the overburden materials in this area have low to moderate compressibility characteristics. Samples defined by Figures 20, 25 and 26 showed some swell potential.

The characteristics of the overburden material in the river bottoms were defined by samples obtained from Test Holes 7 and 14. The samples obtained from Test Hole 14 are characteristic of the surface cohesive zone in the river bottoms, and it will be observed that these materials have low to medium compressibility characteristics.

The compressibility characteristics of the fine-grained soils on the left abutment were defined by samples obtained from Test Pits 501 through 503, and the results of these tests are presented in Figures 27 through 30. These tests also have low to medium compressibility characteristics; however, these materials all appear to be substantially overconsolidated.

It should be noted that during the performance of the consolidation tests all samples were permitted to absorb water to determine the affect of moisture on the compressibility characteristics of these materials.
C. Swell Tests on the Foundation Shale

In order to obtain an indication of the expansive characteristics of the Steele shale throughout the damsite area, swell tests were performed on representative samples obtained from Test Holes 7, 9, 11 and 14, and the results of these tests are presented in Figures 31 through 39. During the conduct of the swell tests, each sample was permitted to absorb water. It will be observed from the swell tests that all samples exhibited some expansive characteristics. The unit weight of the shale material generally varied from about 143 to 153 pounds per cubic foot; and, as indicated earlier in this report, these materials generally classify as CL-1 type soils and have relatively low plasticity characteristics.

Based upon the results of the swell tests, it appears as if shales above the water table throughout the site can experience some swelling if they become wet or saturated.

D. Unconfined Compression Tests

In order to more fully define the characteristics of the Steele shale throughout the foundation area, unconfined compression tests were performed on samples obtained from Test Holes 2, 4, 7, 9, 11, 14 and 19, and the results of these tests are presented in Table No. 1, Summary of Test Data. The unconfined compressive strength of the shale varied from about 50 pounds per square inch to nearly 5,400 pounds per square inch with the compressive strength for most of the samples exceeding 1,500 pounds per square inch.
E. Triaxial Shear Tests

In order to obtain an indication of the strength characteristics of the granular material in the soil profile in the river bottoms, three consolidated drained triaxial shear tests were performed on composite samples consisting of the minus one-half inch material. Each sample was densified to the estimated unit weight of the foundation material. The results of the triaxial shear tests are presented in the form of a Mohr envelope in Figure No. 40, and it will be observed that a friction angle for this material is 42.6 degrees with zero cohesion. The gradation characteristics of the composite sample is shown in Figure 42.

In order to obtain an indication of the strength characteristics of the cohesive material in the right abutment, three consolidated drained triaxial shear tests were performed on representative samples obtained from Drill Hole 9 at a depth of 6 to 7 feet below the existing ground surface. Each sample was saturated by back-pressure techniques and sheared under drained conditions. The results of the triaxial shear tests are presented in Figure No. 41 in the form of a Mohr envelope, and it will be observed that a friction angle of 36 degrees was obtained for this material.

F. Slacking Tests

Twenty-two samples of the Steele shale in the foundation for the proposed facility were subjected to alternate wetting and drying tests. Twelve-hour wetting and
twelve-hour drying cycles were used in the tests. The locations at which the samples were obtained in the drill holes, along with their reaction to the slacking tests, are presented in Table No. 2. It will be noted from this table that most of the samples slacked readily when they were immersed in water for the first time. The remainder of the shales slacked on the second wetting cycle.

Two of the samples subjected to the slacking tests were sandstone with thin shale lenses. After the second wetting cycle, the shale lenses in these samples also slacked. It is our opinion that these samples provide a reasonable statistical indication of the performance of the shale material throughout the foundation area when it is subjected to wetting and drying conditions.

G. Sulfate Tests

Inasmuch as it is anticipated that the spillway will be in contact with earth materials, the sulfate content was determined on four samples obtained from Drill Holes 19 and 20. The results of these tests are tabulated below:

<table>
<thead>
<tr>
<th>Drill Hole - Depth</th>
<th>Sulfate (PPM)</th>
</tr>
</thead>
<tbody>
<tr>
<td>19 5-10'</td>
<td>17.1</td>
</tr>
<tr>
<td>19 10-20'</td>
<td>2.14</td>
</tr>
<tr>
<td>20 0-15'</td>
<td>1.60</td>
</tr>
<tr>
<td>21 5-10'</td>
<td>10.8</td>
</tr>
</tbody>
</table>

It is apparent from the above tabulation that the amount of sulfate in the subsurface materials in the right
abutment in the area where the proposed spillway will be located is relatively small.

H. Soluble Salts

In order to obtain an indication of the amount of soluble salts in the overburden materials in the right abutment, soluble salt determinations were performed on the same samples used for the sulfate determinations. The amount of soluble salts in Drill Hole 19, along with the sample obtained in Drill Hole 20 from 0 to 15 feet, were essentially zero. The amount of soluble salts in the sample obtained at a depth of 5 to 10 feet in Drill Hole 21 was 50 PPM. These tests are compatible with the sulfate tests, and it is not anticipated that any adverse reactions will occur between the concrete and the soil in the area of the right abutment.

2. Available Embankment Materials

A. Classification Tests

Classification tests consisting of either mechanical analyses or Atterberg limits were performed on borrow areas designated as 100, 200, 300 and 400. The results of the classification tests for Borrow Area 100, which is the primary source of the impervious zone, is presented in Table No. 3, Summary of Test Data. A summary of the classification tests for Borrow Areas 200, 300 and 400, are presented in Tables 4, 5 and 6, respectively.

It will be noted from Table No. 3 that the plastic index of the subsurface material in this area generally
varies from about 4 to 21 percent and that the material classifies as a CL-1 or a CL-2 type soil. It should be noted, however, that most of the CL-2 type soils are near the borderline between a CL-1 and a CL-2 type soil according to the Unified Soil Classification System.

The plastic index for the fine-grained soils in Borrow Area 200 varies from about 6 to 19 percent with the major portion of the soils classifying as CL-1 type soils. Those soils which classify as a CL-2 type material are generally on the borderline between a CL-1 and a CL-2 type soil.

The major portion of Borrow Area 300 consists of granular-type soils, and the results of mechanical analyses performed in this area are presented in Table No. 5, Summary of Test Data. It will be noted from this table that the amount of material in the silt- and clay-size range is generally less than 6 to 7 percent. However, some samples were encountered where the percent of materials in the silt- and clay-size range was equal to about 23 percent. The granular materials generally classified as an SM, a GP or a GW type material according to the Unified Soil Classification System.

The plastic index of the available material in Borrow Area 400 generally varies from about 4 to 25 percent, and these materials generally classify as a CL-1 or a CL-2 type soil. Again, it should be noted that the CL-2 type soils have a plastic index only a few percentage points above the borderline between the CL-1 and CL-2 type materials.
The results of all of the classification tests indicate that the available embankment materials have relatively low plasticity characteristics.

B. Particle-Size Distribution Analyses

The particle-size distribution analyses consist of a hydrometer analysis for the silt- and clay-size fraction throughout the area and mechanical analyses for the granular material characterizing Borrow Area 300. The results of hydrometer analyses performed on representative samples of the fine-grained soils obtained from Borrow Area 100 are presented in Figures 42 through 50. These tests were performed in pairs with the particle distribution for one sample determined for the natural material, while the particle-size distribution for the other sample was determined using a dispersion agent. It is readily apparent from these curves that the use of a dispersion agent has a significant affect on the particle-size distribution for the clay-size fraction of these samples. It will also be noted that the amount of material in the silt- and clay-size range generally varies from about 60 to 92 percent.

It is also apparent from the results of the hydrometer analyses presented in Figures 42 through 50 that the filter design should be based upon the particle-size distribution curve for the natural material, rather than the particle-size distribution curve for the dispersed sample.
No particle-size distribution curves were performed for Borrow Area 200, since the fine-grained material in this area was very similar to the fine-grained material in Borrow Area 100.

In order to obtain a realistic indication of the particle-size distribution characteristics for Borrow Area 300, 11 bulk samples were subjected to an analysis in Borrow Area 300. In performing the bulk analysis, several hundred pounds of a representative sample of the existing granular material was placed on a plastic membrane and separated into particle sizes. A representative sample of the minus No. 4 material for each sample was returned to the laboratory for completion of the test.

In addition to the bulk sample analysis, several representative samples of the smaller size granular material were obtained from the borrow area and returned to the laboratory for conventional mechanical analysis.

The results of all particle-size distribution analysis performed on the material from Borrow Area 300 are presented in Figures 51 through 58. It will be noted that the material in Borrow Area 300 is relatively well graded with a maximum size of about 6 inches and with the amount of material in the silt- and clay-size range in the vicinity from about 4 to 10 percent. One or two of the samples, however, contained as much as 22 percent of the material in the silt- and clay-size range. The suitability of this material for use as concrete aggregate will be discussed in a separate section of this report.
Both mechanical analyses and hydrometer analyses were performed on representative samples obtained from Borrow Area 400. The hydrometer analysis, performed on the fine-grained material in this borrow area, was performed on pairs in a manner previously discussed for Borrow Area 100, while the granular material was sieved in a conventional manner.

The results of the particle-size distribution analysis for Borrow Area 400 are presented in Figures 59 through 65. It will be observed from these curves that the amount of material in the silt- and clay-size range for the hydrometer tests varies from about 78 to 94 percent, while the amount of material in the silt- and clay-size range for the granular soils varies from about 5 to 45 percent. It is anticipated that materials in this borrow area will be utilized in Zone II of the embankment.

C. Soil-Moisture Density Relationships

Based upon the classification tests performed for each borrow area, as discussed in an earlier section of this report, samples were selected to determine the soil-moisture density relationships. The samples were selected to cover the entire range of material types existing in a particular borrow area. Fifteen moisture density tests were performed on samples obtained from Borrow Area 100. The results of these tests are presented in Figures 66 through 80 and summarized in Table No. 3, Summary of Test Data.

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It will be noted that the maximum density for materials in Borrow Area 100 varied from about 101 to 121 pounds per cubic foot, while the optimum moisture content varied from about 11 to 20 percent. The majority of the samples, however, had a maximum density in excess of 110 pounds per cubic foot.

Three moisture density relationships were performed on representative samples obtained from Borrow Area 200, and the results of these tests are presented in Figures 81 through 83. The maximum density of these materials varied from about 108 pounds per cubic foot for the fine-grained soils to 140 pounds per cubic foot for the granular materials. The optimum moisture content varied from about 6.5 percent for the granular materials to about 14 percent for the fine-grained soils.

Three moisture density relationships were determined for representative samples in Borrow Area 300, and the results of these tests are presented in Figures 84 through 86. The results shown in Figures 84 and 85 correspond to soils containing a substantial amount of gravelly-type materials, while the results shown in Figure 86 correspond to granular material containing a substantial amount of material in the fine sand range. It will be noted from this data that the maximum density of the gravelly-type soils in Borrow Area 300 will approach at least 140 pounds per cubic foot.
The moisture density relationships for Borrow Area 400 were characterized by performing nine tests on the range of materials existing throughout the site. The results of these tests are presented in Figures 87 through 95, and it will be observed that the maximum density of these materials varies from 108 to 128 pounds per cubic foot and that the optimum moisture content varies from about 9 to 16 percent. It should be noted that the soil-moisture density relationships for all soils tested during this investigation were performed in accordance with ASTM D 698, and it is our opinion that all of the soils tested can be satisfactorily used for fill material in the proposed embankment.

D. Laboratory Permeability Tests

The coefficient of permeability for soils throughout the proposed borrow areas was determined on samples compacted to approximately 95 percent of the maximum density as determined by ASTM D 698. The sample used in the permeability tests was 4 inches high and 6 inches in diameter. Each sample was saturated by back-pressure techniques prior to performing the permeability test. The results of the permeability tests are tabulated in Tables 3 through 6, Summary of Test Data.

Six permeability tests were performed on representative samples obtained from Borrow Area 100, and the results of these tests indicate permeability coefficients...
varying from about 0.3 feet per year to 1.1 feet per year. Two permeability tests were performed on representative samples from Borrow Area 200, and the results of these tests varied from 0.3 feet per year to 15.7 feet per year.

The results of permeability tests performed on representative samples obtained from Borrow Area 300 indicated permeability coefficients ranging from 91 feet per year to 146 feet per year. The results of three permeability performed on representative samples obtained from Borrow Area 400 indicated permeability coefficients varying from 0.2 to 0.47 feet per year.

Essentially all of the materials contemplated for Zones I and II, with exception of granular zones in these areas, have permeability coefficients less than 1 foot per year, which is entirely satisfactory for the proposed facility.

E. Direct Shear Tests

Consolidated drained direct shear tests were performed on four representative samples obtained from Borrow Area 100 and two representative samples obtained from Borrow Area 400. Each direct shear test sample was 1 inch high and 2.5 inches square and was saturated by back-pressure techniques prior to consolidation.

The results of the consolidated drained direct shear tests performed on four samples obtained from Borrow Area 400 are presented in Figures 96 through 99. It will be noted that the friction angle for these materials varied from
25.5 to 30.1 degrees, while the cohesion varied from about 2 to 3 psi.

The results of the consolidated drained direct shear tests performed on two representative samples obtained from Borrow Area 400 are presented in Figures 100 and 101. The friction angle for these materials varied from 27.7 to 29.3 degrees, while the cohesion varied from 5 to 5.9 psi.

The stress-strain curves, along with pertinent data relative to the tests, are presented with the results of the shear tests.

F. Triaxial Shear Tests

Triaxial shear tests performed to define the strength characteristics of the embankment material included unconsolidated undrained tests at the placement moisture content, isotropic and anisotropic consolidated undrained tests, and consolidated drained tests. Except for the unconsolidated undrained shear test, all samples were saturated by back-pressure techniques prior to consolidation and failure.

Consolidated drained triaxial shear tests were performed on three representative samples obtained from Borrow Area 100, and the results of these tests are presented in Figures 102 through 104. It will be observed that the friction angle for these tests varied from 29.7 degrees to 32.7 degrees with a cohesion value varying from 2 to 4 psi. Pertinent data relative to the sample characteristics and the results of these tests are also presented in these figures.
An unconsolidated undrained test, an isotropic and two anisotropic consolidated undrained tests, and one consolidated drained test were performed on a representative sample obtained from Test Pit 210 at a depth of 5 to 8 feet below the ground surface in Borrow Area 200. This sample classified as a CL-1 type soil and is generally representative of the CL-1 type materials encountered in Borrow Area 100.

The results of the unconsolidated undrained test at the placement moisture content for the sample from Borrow Area 200 is presented in Figure 105. It will be observed that a friction angle of 17.5 degrees and a cohesion of 17 psi was obtained for this test. The results of the consolidated undrained test on this same sample for isotropic consolidated conditions is presented in Figure 106. It will be noted that a friction angle of 16.5 degrees and a cohesion of 4 psi were obtained for this test.

The results of two anisotropic consolidated samples performed with the ratio of the major principal stress to the minor principal stress equal to 1.5 and 2 on the sample obtained from Borrow Area 200 are presented in Figures 107 and 108. It will be observed that the friction angle obtained for these two tests were 22.7 degrees and 27.3, respectively with a cohesion of 3 psi in each case. This series of consolidated undrained shear tests performed with the ratio of the major principal stress to the minor principal stress varying from 1 to 2 indicate the affect of anisotropic consolidation on
the friction angle of the cohesive material throughout this site.

The results of consolidated drained triaxial shear tests performed on the sample obtained from Borrow Area 200 is presented in Figure 109. It will be observed that the friction angle obtained for this case is 28.2 degrees with a cohesion of 3 psi. It is interesting to note that the friction angle for the consolidated drained test under an isotropic consolidation is only slightly greater than the consolidated undrained friction angle consolidated with the ratio of the major principal stress to the minor principal stress equal to 2.

One consolidated drained triaxial shear test was performed on a representative sample obtained from Borrow Area 400, and the results of this test are presented in Figure 110. The results of this test indicated that the material has a friction angle of 31.2 degrees and a cohesion of about 3 psi. It is our opinion that the results obtained from this sample are characteristic of the cohesive material existing in Borrow Area 400.

G. Sulfate Tests

In order to obtain an indication of the sulfate content of the available embankment material, a number of sulfate tests were performed on representative samples from Borrow Area 100, Borrow Area 200 and Borrow Area 400. The results of these tests are tabulated below as follows:
<table>
<thead>
<tr>
<th>Test Pit - Depth</th>
<th>Solubles (PPM)</th>
<th>Test Pit - Depth</th>
<th>Solubles (PPM)</th>
</tr>
</thead>
<tbody>
<tr>
<td>108 4-8'</td>
<td>1.74</td>
<td>210 5-8'</td>
<td>83.4</td>
</tr>
<tr>
<td>113 2-6'</td>
<td>8.10</td>
<td>410 4-10'</td>
<td>93.1</td>
</tr>
<tr>
<td>115 2-7'</td>
<td>93.1</td>
<td>413 0-2½'</td>
<td>1.60</td>
</tr>
<tr>
<td>118 2-5'</td>
<td>7.47</td>
<td>416 6-12'</td>
<td>19.4</td>
</tr>
<tr>
<td>121 7-11'</td>
<td>10.6</td>
<td>426 0-11'</td>
<td>83.4</td>
</tr>
<tr>
<td>209 2-7'</td>
<td>100.0</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

The results of these tests indicate that the sulfate quantities are relatively small and that no adverse reactions will occur between structures placed adjacent to or on the earth embankment materials.

H. Soluble Salts

The soluble salt content of the available embankment material was evaluated by performing 11 soluble salt determinations on representative samples obtained from the borrow areas. The results of these tests are presented below:

<table>
<thead>
<tr>
<th>Test Pit - Depth</th>
<th>Solubles (PPM)</th>
<th>Test Pit - Depth</th>
<th>Solubles (PPM)</th>
</tr>
</thead>
<tbody>
<tr>
<td>108 4-8'</td>
<td>200</td>
<td>210 5-8'</td>
<td>400</td>
</tr>
<tr>
<td>113 2-6'</td>
<td>0</td>
<td>410 4-10'</td>
<td>600</td>
</tr>
<tr>
<td>115 2-7'</td>
<td>1050</td>
<td>413 0-2½'</td>
<td>0</td>
</tr>
<tr>
<td>118 2-5'</td>
<td>100</td>
<td>416 6-12'</td>
<td>150</td>
</tr>
<tr>
<td>121 7-11'</td>
<td>0</td>
<td>426 0-11'</td>
<td>900</td>
</tr>
<tr>
<td>209 2-7'</td>
<td>1500</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

The results of these tests indicate that, in general, the amount of soluble salts is relatively small, and it is not anticipated that sufficient soluble salts exist in the embankment material at this site to create problems associated with seepage through the embankment.
I. Dispersive Soil Tests

Cohesive-type soils, known as dispersive soils, have been encountered at various locations throughout the United States. These soils generally have low interparticle attraction and under relatively high hydrostatic conditions, they will exhibit erosive characteristics similar to cohesionless soils. A pinhole test has been developed by Sherard (1975) for determining the dispersive characteristic of clay materials. During this test, water, under various hydraulic gradients, is forced through a small hole drilled through a sample of a specified size. Pinhole tests were performed on several samples from Borrow Area 100, 200 and 400. The results of these tests are tabulated below:

<table>
<thead>
<tr>
<th>Test Pit</th>
<th>Depth</th>
<th>Dispersive Classification</th>
</tr>
</thead>
<tbody>
<tr>
<td>108</td>
<td>4-8'</td>
<td>Non-dispersive</td>
</tr>
<tr>
<td>116</td>
<td>2-11'</td>
<td>Non-dispersive</td>
</tr>
<tr>
<td>118</td>
<td>2-5'</td>
<td>Non-dispersive</td>
</tr>
<tr>
<td>121</td>
<td>7-11'</td>
<td>Non-dispersive</td>
</tr>
<tr>
<td>206</td>
<td>3-9'</td>
<td>Dispersive</td>
</tr>
<tr>
<td>210</td>
<td>5-8'</td>
<td>Non-dispersive</td>
</tr>
<tr>
<td>410</td>
<td>4-10'</td>
<td>Non-dispersive</td>
</tr>
<tr>
<td>413</td>
<td>0-2'</td>
<td>Non-dispersive</td>
</tr>
</tbody>
</table>

It will be noted that only one sample out of eight exhibited dispersive characteristics, and based upon the tests performed above, it is our opinion that overall the available materials for the proposed embankment at this site will not exhibit dispersive characteristics.
3. Available Slope Protection
   A. Abrasion Tests

   As indicated earlier in this report, two sources of material, which could possibly serve as slope protection on the upstream portion of the dam, were described. Source No. 1 consisted of the caprock sandstone in the immediate vicinity of the reservoir basin, while Source No. 2 consisted of a metamorphic rock located approximately 6 to 7 miles northeast of the damsite.

   In order to evaluate the suitability of these two sources to serve as riprap for the proposed structure, abrasion tests performed in accordance with ASTM D 535 were performed on one sample of the sandstone and four samples of the metamorphic rocks. The results of these tests indicated that the percent wear for the sandstone material was 91 percent, while the percent wear of the four samples of the metamorphic rock varied from 19.7 to 20.8 percent.

   B. Bulk Specific Gravity

   The bulk specific gravity was determined for each of the possible riprap sources. The bulk specific gravity of the sandstone was 2.17, while the bulk specific gravity of the metamorphic rock was 2.97.

4. Available Concrete Materials
   A. Sodium Sulfate Soundness

   The sodium soundness of the representative samples from Borrow Area 300 were performed in accordance with
ASTM C88-76. Two representative samples of fine aggregate and two representative samples of coarse aggregate were subjected to the sodium sulfate soundness test. The percent loss for the fine and coarse aggregate for Sample No. 1 was 4.91 and 3.1%, respectively. The percent loss for Sample No. 2 was 4.59 for the fine aggregate and 1.99 for the coarse aggregate. Fine aggregate having a loss of 8% and coarse aggregate having a loss of 10% are generally considered to be satisfactory soils.

B. Alkali Reactivity

The potential reactivity of the aggregates in Borrow Area 300 was determined in accordance with ASTM C 289-81. Six samples of representative material obtained from Borrow Area 300 were prepared in accordance with the ASTM specifications for potential reactivity analysis. The results of these tests are tabulated below:

<table>
<thead>
<tr>
<th>Sample No.</th>
<th>$S_C$</th>
<th>$R_C$</th>
<th>Sample No.</th>
<th>$S_C$</th>
<th>$R_C$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>23.31</td>
<td>54.4</td>
<td>4</td>
<td>29.97</td>
<td>71.6</td>
</tr>
<tr>
<td>2</td>
<td>33.30</td>
<td>56.0</td>
<td>5</td>
<td>26.64</td>
<td>70.4</td>
</tr>
<tr>
<td>3</td>
<td>19.98</td>
<td>94.4</td>
<td>6</td>
<td>16.65</td>
<td>93.6</td>
</tr>
</tbody>
</table>

If $R_C$ is plotted as a function of $S_C$, as shown in Figure 2 of ASTM C289, all points plot in the area indicating that the aggregates do not have potential alkali reactivity.

C. Abrasion

Six abrasion tests were performed in accordance with ASTM C535-81 on the coarse aggregate obtained from Borrow Area 300. The percent wear of this material after 1,000 revolutions varied from 22.7 to 34.8. It is generally recognized that concrete aggregates should have a percent wear of less than 40 after 500 revolutions. It is our opinion, therefore, that the concrete aggregates in Borrow Area 300 have the required degree or hardness and toughness for concrete structures for the proposed facility.

The specific gravity of a representative sample of fine aggregate obtained from Borrow Area 300, determined in accordance with ASTM C128, was 2.0%. The specific gravity of the coarse aggregate from Borrow Area 300 was 2.79% with an absorption of 0.9%, as determined in accordance with ASTM C127.
DESIGN ANALYSIS
IX. FOUNDATION TREATMENT

Based upon the results of the subsurface investigation throughout the damsite area, the following conclusions can be made:

(1) The subsurface profile in the river bottoms consists of a surface cohesive zone varying in thickness from 6 to 8 feet, a sand and gravel zone underlying the surface zone extending to a depth of between 20 and 28 feet below the ground surface, and a rock zone consisting of the Steele shale which extends to the depth at which the test borings were drilled throughout the site.

(2) The surface cohesive zone throughout the river bottoms contains a considerable amount of organic matter and is relatively soft and compressible.

(3) The granular zone between the cohesive material and the Steele shale is moderately permeable and will transmit a considerable amount of water.

(4) The Steele shale below the overburden materials in the river bottoms is relatively impervious and will not transmit excess water quantities.

(5) The Steele shale in the left abutment appears to be relatively impervious, except for a location at the crest of the slope, where the water loss in the drill holes was moderately high.
(6) The permeability characteristics of the Steele shale below the water level in the right abutment appear to be relatively impervious, except for one location.

(7) The Steele shale generally has low plasticity characteristics, relatively high unit weight, possesses some swell potential, and slacks quickly and breaks down into a clay under one or two wetting and drying cycles.

Based upon the above characteristics of the foundation materials throughout the damsite, at the proposed location, the following foundation treatment is recommended:

1. **Cutoff Trench**

A compacted clay cutoff trench extending from the existing ground surface through the weathered shale at the top of the Steele shale formation is contemplated along the entire length of the dam. The cutoff trench should have a bottom width of at least 50 feet and should have sideslopes of 2 horizontal to 1 vertical.

Dewatering of the foundation area throughout the proposed damsite will be required to permit construction operations in this area. The cutoff trench should be backfilled with CL-1 or CL-2 type clay materials obtainable from Borrow Area 100.

It is anticipated that the shale material in the bottom of the cutoff trench will weather quickly to its original low plasticity clay material following a few cycles of wetting and drying.
Since the weathered clay material from the shale formation has similar plasticity characteristics to the proposed backfill material, no difficulty should be encountered in making a satisfactory bond between the natural backfill material and the weathered shale material in the bottom of the cutoff trench.

All earth materials placed in the cutoff trench should be densified to an in-place unit weight equal to 95 percent of the maximum laboratory density as determined by ASTM D 698.

2. Left Abutment Preparation

Since possible leakage problems exist in the left abutment, and since the depth of overburden on the left abutment is relatively small, we recommend that the entire left abutment downstream from the cutoff trench be excavated and that granular filter drain material be placed immediately between the abutment and the earth materials composing the dam. Any water seeping through the abutment can be readily collected into the filter drain on the abutment and disposed of outside of the dam without causing high hydrostatic pressure. Material stripped from the left abutment may be used as fill material in the embankment area.

3. Grouting of the Left Abutment

As indicated earlier in this report, the left abutment appears to be relatively permeable to a depth of over 60 feet, measured from the existing ground surface in the vicinity of
the crest of the left abutment. Since it is unlikely that the Steele shale can be excavated to the entire depth of the permeable zone, a grout curtain will be required in this area. Because of the uncertainty as to the extent of the grout curtain required in this area, we recommend that exploration grout holes be drilled at 100 feet on center within Station 11+00 and Station 15+00. The exploration holes would be NX size and would be water tested and grouted to a depth of approximately 80 feet.

If the results of the water tests and the grout take is sufficiently great, it is recommended that primary grout holes be drilled to a depth of 60 feet at locations as shown in Figure 111. Depending upon the nature of the grout take in the primary holes, it may be necessary to drill secondary holes to approximately 60 feet at locations shown in Figure 111.

If the results of the grouting, as indicated above, do not appear to provide a completely satisfactory grout curtain, it is recommended that a second line of grout holes be constructed, as shown in Figure 111. Careful supervision will be required in the grouting operations at this location to insure that the permeability characteristics of the left abutment zone are satisfactorily reduced and to insure that money is not wasted in grouting operations where it will not provide benefits in reducing seepage in the left abutment area.
4. Stripping Requirements in the River Bottom Area

As a general guide, the subsurface material beneath an earth dam embankment should have strength characteristics equal to the material within the embankment. The surface cohesive zone throughout the river bottoms does not possess a strength equivalent to the proposed embankment materials, and as indicated earlier in this report, it contains a considerable amount of organic material. Furthermore, a drainage blanket beneath the dam downstream from the cutoff trench will not perform satisfactorily if it is placed directly on the cohesive material.

For the above reasons, it is our opinion that the surface cohesive zone in the river bottoms should be stripped to the elevation of the sand and gravel zone underlying the cohesive material. The sand and gravel will provide a relatively strong foundation for the earth embankment and will permit a reduction in the thickness of the drainage blanket downstream from the compacted earth cutoff. It is anticipated that the surface cohesive zone will be removed between Station 16+00 and Station 31+00.
X. THE USE OF THE AVAILABLE MATERIAL IN THE DAM

The results of the investigation performed to determine the available embankment material throughout the region adjacent to the dam and reservoir indicate that the major portion of the available material is cohesive-type soils. This is particularly true of the cohesive material which exists in Bird Gulch. It should be noted, however, that the landslide material and the terrace deposits may have granular-type soils intermixed with the cohesive-type soil. It appears to us that the most feasible cross section for the Upper Savery Dam will be a central impervious dam constructed of materials obtained from Bird Gulch with exterior zones consisting of materials obtained from the terrace and slide areas. It is expected, however, that the exterior zones will be relatively impervious because of the sizable amount of cohesive materials which exist in the borrow areas containing the terrace deposits and the slide material.

Based upon the recommendations presented in the section on foundation treatment and the information presented above, the cross section of the proposed facility at various locations along the axis of the dam will appear as shown in Figure 112. The proposed embankment will have a crest width of 30 feet, a freeboard of 22 feet, and a cutoff trench extending to a depth of between 30 and 35 feet below the existing ground surface. Sideslopes of 3 horizontal to 1 vertical

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will be required for the major portion of the upstream slope, while sideslopes of 2 horizontal to 1 vertical are contemplated for the downstream slope.

The Zone I material and the Zone II material will be separated by a chimney filter drain, while the slope of the Zone I material in the upstream portion of the dam will be 1.5 horizontal to 1 vertical.
XI. INTERNAL DRAINAGE SYSTEM

It is our opinion that a satisfactory internal drainage system is essential for the safety of any earth dam. The essentials of the internal drainage system, discussed in this section of the report and shown in Figure 111, include a chimney filter drain, a horizontal underdrain and vertical drains and drainage conduits.

1. Chimney Filter Drain

If the surface cohesive material in the river bottoms is removed so that the proposed embankment is located directly on the sand and gravel, foundation settlement will be relatively small. Differential movement throughout the embankment will not be sufficient to cause cracking of the proposed facility. It is recommended, however, that the cohesive materials within the embankment be compacted to a moisture content equal to at least the optimum value.

While cracking of the proposed embankment is unlikely, a chimney filter drain will be required within the proposed facility to intercept any water which may flow through the embankment. Only if the chimney filter drain functions properly and the downstream portion of the dam remains unsaturated can be a stable slope constructed in this area at side-slopes of 2 horizontal to 1 vertical.
In order for the filter drain to perform satisfactorily, the gradation of the filter should satisfy the filter requirements with respect to the Zone I material. The gradation requirements for the chimney filter are presented in Figure 113, along with the typical gradation curves for the Zone I material and the sand and gravel in the bottom of the stream channel. It is apparent from Figure 113 that the gradation characteristics of the sand and gravel in the bottom of the river channel will not meet filter requirements with respect to the material in the cutoff trench. Hence, a filter drain, as shown in Figure 112, will be required along the back side of the cutoff trench to prevent the movement of any cutoff material into the granular material downstream from the cutoff.

A filter at this location is particularly important for the proposed structure, as will be discussed later in this report, since the full hydrostatic pressure will exist along the upstream face of the cutoff trench.

2. **Horizontal Underdrain**

If the cohesive material in the river bottoms is stripped from the proposed embankment area, the granular material in the river bottoms will tend to serve as a drain for the embankment soils. As a result of this condition, the thickness of the horizontal drain may be reduced compared to its magnitude if the granular material were absent.
It will be noted from Figure 112 that a 2-foot thick horizontal drain is contemplated between the bottom of the chimney filter drain and the downstream toe of the dam. The horizontal underdrain should have the same gradation characteristics as recommended for the chimney filter drain. The horizontal underdrain, as shown in Figure 111, should lap up and cover the entire contact surface between the embankment and the left abutment downstream from the cutoff. The thickness of the filter drain in this area should be increased to 3 feet.

3. **Vertical Drains and Drainage Conduits**

   It should be recognized that granular drains are not capable of transmitting large quantities of water, and that in order for appreciable quantities of water to be intercepted in the abutments or the internal portion of an earth dam, conduits are necessary. In order to intercept any water which may pass through the grout curtain, we recommend that a series of 6-inch drainage wells be constructed downstream from the grout curtain in the left abutment, as shown in Figures 111 and 112. The drainage wells will be spaced at 10-foot centers and will extend to a depth of 60 feet below the existing ground surface.

   As indicated earlier in this report, one location was observed in the right abutment where high permeability coefficients were obtained. Since no grouting is contemplated in the right abutment, and since the overburden material is too
thick to be economically removed, we recommend that a series of 6-inch drainage wells be constructed in the right abutment, as shown in Figure 111. The use of drainage wells, as recommended herein, has been criticized by some engineers on the basis that the wells cannot be cleaned and may cease to function over a period of time. It is our contention that any leakage which may occur through the foundations of an earth dam will likely occur following the first filling of the reservoir or within a few years thereafter. The installation of the drain wells at critical areas in the foundation material permits an early detection of any critical seepage problems which may develop and will provide sufficient time for more elaborate corrective action to be taken. We, therefore, strongly recommend that the drainage wells discussed above be constructed in accordance with the recommendations provided herein.

In order to provide sufficient capacity for the drainage wells and any seepage which may occur into the bottom of the cutoff trench, we recommend that a 12-inch perforated, corrugated steel pipe be constructed in the bottom of the cutoff trench, as shown in Figures 111 and 112. Connection details between the vertical drains and the conduit in the bottom of the cutoff trench are presented in Figure 111.

In order to discharge water collected in the corrugated steel pipe in the bottom of the cutoff trench, riser pipes, located on 400-foot intervals, will be constructed up the back slope of the cutoff trench as shown in Figure 111.
In order to supplement the flow in the horizontal underdrain, we recommend that a 12-inch diameter corrugated steel pipe be laid at 200-foot intervals throughout the length of the drainage blanket. It will be noted from Figure 111 that alternate drain pipes in the horizontal underdrain connect to the drains leading up the back slope of the cutoff trench.

The location of the drainage wells in the left abutment relative to the grout holes in that area are presented in Figure 112 at the cross section for Station 12+00.
XII. SEEPAGE ANALYSIS

1. Seepage Through the Dam and Foundation

In order to obtain some indication of the equal hydraulic headlines and the flow patterns through the earth dam and the foundation material, various assumptions have been made relative to the permeability characteristics of the foundation and the earth dam. Three different idealized cases have been considered to obtain some indication of the nature of the equal hydraulic headlines and the flowline distribution.

Case 1 considers that the embankment is completely homogeneous with a permeability coefficient of 0.25 feet per year. The granular zone above the bedrock is assumed to have a permeability coefficient of 10,000 feet per year, while the bedrock is assumed to have a permeability coefficient of 15 feet per year to a depth of 50 feet below the granular zone.

Using this information, the equal hydraulic headline pattern was determined from a computer program utilizing a finite element solution, developed by the Corps of Engineers at the Hydrologic Engineering Center in Davis, California. The distribution of the equal hydraulic headlines along with the approximate flowline pattern is presented in Figure 114. It is significant to note that the full hydrostatic pressure
acts along the upstream portion of the cutoff trench and that the full reservoir head is dissipated within the width of the trench. It is also significant to note that the loss in head all occurs in the Zone I material and that under steady-state seepage conditions, nearly the full hydrostatic pressure exists in the upstream portion of the dam.

In Case 2, the Zone I, Zone II, the sand and gravel zone and the bedrock were all assumed to have different permeabilities; however, the permeability in both the horizontal and the vertical direction for each zone were assumed to be the same. The permeability of Zone I was assumed to be equal to 0.1 feet per year, the permeability of Zone II was assumed to be equal to 0.5 feet per year, the permeability of the sand and gravel was assumed to be 10,000 feet per year, and the permeability of the bedrock was assumed to be 15 feet year. The equal hydraulic headline distribution for this case is presented in Figure 115. It will be noted from this figure that the equal hydraulic headline distribution is very similar to the homogeneous and isotropic case. The phreatic surface is slightly higher and the equal hydraulic headline distribution is located further inside of the Zone II material.

Case 3 assumed that each zone within the embankment and foundation had anisotropic conditions. The Zone I material was assumed to have a permeability in the X direction of 0.5 feet per year and a permeability in the Y direction of 0.10 feet per year. The Zone II material was assumed to have
a permeability in the X direction of 2.5 feet per year and a permeability coefficient in the Y direction of 0.5 feet per year. The sand and gravel layer was assumed to have a permeability coefficient in the X direction of 12,000 feet per year and a permeability coefficient in the Y direction of 2,400 feet per year. The shale formation was assumed to have a permeability coefficient in the X direction of 15 feet per year and a permeability coefficient in the Y direction of 3 feet per year.

The equal hydraulic headline distribution for this condition is presented in Figure 116. The equal hydraulic headline distribution for this condition is also similar to the other two cases. However, it will be noted that the equal hydraulic headlines extend further upstream in the Zone I material and in the foundation area, and the phreatic surface is somewhat higher than for the homogeneous case.

No flowlines were constructed for either Case 2 or Case 3. Inasmuch as the flow net for Cases 2 and 3 will not be dramatically different than the flow net for Case 1, this case has been used to estimate the seepage quantity through the embankment and the foundation. The results of the seepage calculations indicate that the total quantity of seepage passing through the embankment and the foundation in the river bottom area will be approximately 570,000 cubic feet per year. This quantity of flow is tantamount to a flow of 0.02 cubic feet per second or about 10 gallons per minute. The exact quantity of seepage which will occur through the left abutment
is not subject to a rational analysis. However, it is our opinion that if the grout curtain is constructed in this area in accordance with the recommendations outlined in an earlier section of this report, the amount of seepage will not exceed 0.2 cubic feet per minute. It is our opinion, therefore, that the total quantity of seepage through the embankment and the foundation material for the entire length of the dam will not likely exceed 0.3 to 0.4 cubic feet per second.

2. Seepage From the Reservoir Basin

The seepage from the reservoir basin is not subject to a quantitative estimate. It should be noted, however, that the proposed dam is on a live stream and that the drainage occurs from the upland areas towards the stream. It should also be noted that the Steele shale provides an envelope completely surrounding the mass of water within the reservoir basin. The results of the subsurface investigation along the axis of the dam indicates that the Steele shale is relatively impervious, except for zones in the abutments where the subsurface materials have likely been subjected to stress release or weathering. It is anticipated that beyond these near surface zones, the Steele shale in the abutments will have characteristics similar to the shale in the bottom of the stream channel and that seepage losses from the reservoir basin will be relatively small.
XIII. EMBANKMENT SLOPES AND STABILITY CONSIDERATIONS

1. Introduction

As indicated previously in this report, the foundation conditions, along with the available embankment material, suggested that the proposed facility could be constructed using sideslopes of 2 horizontal to 1 vertical downstream and sideslopes of 3 horizontal to 1 vertical upstream, and that the available material would be placed within the embankment in accordance with Figure 112. It should be noted that the type of material contemplated for the various zones within the embankment cross sections is also shown in Figure 112.

The slope protection for the upstream face of the dam is assumed to consist of 1 foot of granular material plus 30 inches of riprap obtained from a source previously described in this report.

In the initial cross sections for the proposed facility, the bottom width of the cutoff trench was assumed to be equal to 40 feet. This value was used in preparing the equal hydraulic headline distributions, as shown in Figures 114 through 116, with the full hydrostatic head acting on the upstream face of the cutoff trench. The hydraulic gradient through the contact between the bedrock and the compacted earth fill was approximately 3.0. In order to reduce the hydraulic gradient in this zone, the width of the cutoff trench was extended to 50 feet.
In performing the stability analysis for the proposed facility, consideration has been given to stability of the downstream slope at the end of construction, the stability of the downstream slope under steady-state seepage conditions, and the upstream slope under conditions of sudden drawdown. Consideration has also been given in the stability analysis to the affect of anisotropic consolidation on the strength characteristics of the soil and its affect on the stability of the embankment. It should be noted that the proposed structure is located in Seismic Zone I and that the likelihood of seismic activity in this region is relatively small. Hence, no detailed seismic analysis was contemplated during this investigation. A pseudostatic analysis has been performed, however, using a seismic coefficient of .1 g.

In performing the stability computations for the proposed facility, both the effective analysis and total stress analysis have been used. Spencer's method, adapted to a computer program developed by Stephen Wright at the University of Texas, has been used in stability computations. It should be noted that Spencer's method satisfies both force and moment equilibrium. The Modified Bishop's method has also been used to check the factor of safety for the critical circle for the downstream slope under steady-state seepage conditions and for the pseudostatic analysis.
2. Stability of the Downstream Slope End of Construction Case Maximum Section

The stability of the downstream slope end of construction case has been calculated using a total stress analysis. The shearing strength under these conditions has been assumed to be equal to the unconsolidated undrained shear strength. The unconsolidated undrained test results shown in Figure 105 have been used for this analysis. The unit weight of the embankment material has been assumed equal to the placement unit weight, which corresponds to approximately 95 percent of the maximum laboratory density as determined by ASTM D 698. The results of this test indicate a factor of safety of 17.2. This factor of safety is relatively high, as would be expected, since the construction pore pressures are generally relatively small for an embankment of this height constructed from soils having low plasticity characteristics.


The factor of safety for the downstream slope under steady-state seepage conditions was calculated using an effective stress analysis. Pore pressures throughout the embankment were estimated from the flow net for the homogeneous case. The shear strength parameters and the unit weight for the various zones throughout the maximum cross section are shown in Figure 117. It will be noted that the friction angle and the cohesion for Zone I was 28 degrees and 300 pounds per square foot, respectively, while the friction angle and the cohesion for Zone II materials were 30 degrees.
and 300 pounds per square foot, respectively. The results of the factor of safety for the downstream slope under steady-state seepage conditions was 1.52, which in our opinion is satisfactory for the proposed facility.

4. Stability of the Upstream Slope Under Sudden Drawdown Conditions Maximum Section

The stability analysis performed for the upstream slope under steady-state seepage conditions considered both an effective stress analysis and a total stress analysis. The piezometric line under drawdown conditions used in the effective stress analysis is presented in Figure 117. This line corresponds very close to the pore pressures estimated by Bishop's method for the case of sudden drawdown using equal hydraulic headline distribution prior to drawdown. Shear strength parameters used in the sudden drawdown case are tabulated in Figure 117 and are the same as those used for the stability analysis for the downstream slope under steady-state seepage conditions.

The results of the effective stress analysis indicates a factor of safety of approximately 1.4. The location of the failure surface for the sudden drawdown condition using the effective stress analysis is presented in Figure 117.

A total stress analysis was also performed for the sudden drawdown case using the Mohr envelope presented in Figure 106. It will be noted that the friction angle in Figure 106 is 16.5 degrees and the cohesion is 4 psi. The results of the stability analysis for the sudden drawdown case using a total stress analysis is 1.37. Since a factor
of safety of 1.3 is normally considered satisfactory for the sudden drawdown condition, and since the factors of safety for both the total stress analysis and the effective stress analysis are very close, it is our opinion that the upstream slope is sufficiently stable under conditions of sudden drawdown.

5. The Affect of Anisotropic Consolidation on the Stability of Earth Embankments

It is generally recognized that anisotropic consolidation of soil samples in the triaxial shear apparatus increases in the strength characteristics of the material. As indicated earlier in this report, isotropic and anisotropic tests were performed on a sample of cohesive material obtained from Borrow Area 200. The results of these tests are shown in Figures 106, 107 and 108, and have been previously discussed. It will be noted from these figures that the friction angle for a major principal stress ratio of 1, 1.5 and 2, are 16.5, 22.3 and 27.7, respectively.

In order to obtain an indication of the principal stress ratio within an earth embankment, a finite element study has been performed on the cross section for the proposed embankment using a computer program developed by Duncan, et. al. at the University of California. This program makes use of non-linear relationships for both the modulus of elasticity and Poisson's ratio. The mathematical expression for the stress-strain curve and for Poisson's ratio were derived from the consolidated drained triaxial shear test performed during investigation.
Contours showing the maximum principal stress ratio for the cross section of the proposed dam are presented in Figure 118, and it will be observed that the principal stress ratio in the region of the failure surface for both the upstream and the downstream zones is equal to or greater than 2. It should be noted that the stress contours correspond to the end of construction case. It appears unlikely, however, that the principal stress ratios will change substantially for the case of sudden drawdown. Assuming that anisotropic consolidation has occurred in the earth embankment under a principal stress ratio of approximately 2, it is apparent that a substantial increase in the factor of safety would occur for the case of sudden drawdown. Using the shear strength parameters defined in Figure 108, the results of a stability analysis performed for the case of sudden drawdown indicates a factor of safety of 1.95.

Based upon the above analysis, it is our opinion that the actual factor of safety existing within the embankment for both the steady-state seepage case and the sudden drawdown case are somewhat greater than would be indicated using the shearing strength parameters determined under isotropic conditions.

6. Stability for the Left Abutment Section

Since the left abutment section contained a substantial deposit of cohesive material in the foundation soils, stability analysis for the downstream slope under steady-state conditions and for the upstream slope under sudden drawdown
were performed in this area. Using a friction angle of 27 degrees and a cohesion of 150 pounds per square foot for the foundation materials and shear strength parameters as shown in Figure 117 for the earth embankment, the factor of safety for the downstream slope under steady-state seepage conditions was 1.51. Using a friction angle of 27 degrees and a cohesion of 200 pounds per square foot for the foundation material and shear strength parameters for the embankment, as shown in Figure 117, the upstream slope under conditions of sudden drawdown indicated a factor of safety of 1.31. Based upon this analysis, it is our opinion that the embankment in the left abutment is stable for the operating conditions contemplated for this facility.

7. **Pseudostatic Dynamic Analysis**

As indicated earlier in this report, a pseudostatic dynamic analysis was performed for the maximum section using a seismic coefficient of 0.1 g. The analysis was performed for the critical circle determined for the downstream slope under steady-state seepage conditions. The results of the analysis indicated a factor of safety of 1.23, which in our opinion is satisfactory for the proposed facility.

8. **Instrumentation**

As indicated earlier in the report, the full hydrostatic pressure of the reservoir will exist along the upstream surface of the cutoff trench, and the greatest loss in head
through the structure occurs in the cutoff section. In order
to determine the effectiveness of the earth fill cutoff, we
recommend that Casagrande type piezometers be installed down­
stream from the cutoff trench at locations as shown in Figure
112. It is contemplated that the piezometers would be spaced
at 150-foot intervals and that one half of the piezometers
would be located in the granular material downstream from the
cutoff and that the other half of the piezometers would be
located in the bedrock a few feet below the interface between
the shale and the granular material. It is our opinion that
no other instrumentation will be required for the proposed
facility.
XIV. SPILLWAY AND OUTLET CONSIDERATIONS

1. Introduction

As indicated earlier in this report, design considerations associated with the spillway and the outlet works were not a part of this investigation, except as they were affected by the foundation conditions. Consideration is given, therefore, to the foundation conditions for each of these facilities in the following section of this report.

It should be noted that the topographic survey performed during this investigation indicated some discrepancies between the topography obtained from the USGS maps and the topography developed by our organization. It appears that the actual height of the dam is approximately 7 feet greater according to our surveys than the height determined using the USGS maps. This discrepancy would be expected because of the small scale of the USGS maps used during the preliminary work. The discrepancy suggests the need for accurate topographic maps of the reservoir basin in completing the final design for the proposed facility. It is entirely possible that rerouting the maximum probable flood through the reservoir basin may result in some modification in the crest elevation of the dam, as well as in some changes in the spillway design.
2. Spillway Considerations

The location of the spillway adapted to the topography determined during this investigation is presented in Figure 119. The geometrics of the spillway are identical to those proposed in the Banner Report. As indicated earlier in this report, the depth of the overburden of the bedrock in the area where the spillway will be located is approximately 25 feet below the existing ground surface, and the materials in this area are all cohesive-type soils. No foundation problems appear to exist in this area, however, since the stress release associated with the excavation for the spillway will exceed the spillway loads. Appropriate underdrains should be provided for the spillway and provisions should be made for appropriate anchors in the control section of the spillway.

It is also recommended that in the final design consideration be given to the construction of a principal spillway to pass the 100-year flood and an emergency spillway to pass the probable maximum flood. It appears to us that an unlined spillway of a substantial width extending to bedrock could be constructed in the left abutment to serve as an emergency spillway and that a concrete chute-type spillway could be constructed in the center of the emergency spillway to pass the 100-year flood. A draw located immediately downstream from the left abutment could be used as a discharge point for the spillway. It is possible that this type of a facility would be more economical than the side channel spillway recommended in the Banner report.

-72-
3. Outlet Works Consideration

It is our opinion that where possible the outlet conduit should be located on bedrock. Since the depth to bedrock in the area where the outlet conduit was proposed in the Banner report is approximately 25 feet below the ground surface, it does not appear possible to locate the outlet works in this area and still construct the facility on bedrock. The bedrock is relatively close to the ground surface in the left abutment, and we believe that the outlet conduit could be placed on bedrock throughout the major portion of its length.

Assuming that the inlet elevation for the outlet conduit will be located at 7060, which corresponds to the same elevation used in the Banner report, the proposed location of the outlet conduit is presented in Figure 119. The profile for the outlet conduit is also presented in Figure 119, and it will be observed that a significant cut through the shale is required for the proposed alignment.

While the depth of the excavation in the bedrock could be reduced by extending the alignment further towards the stream, it is our opinion that the alignment presented in Figure 119 is the most plausible approach.

It should be noted that the size of the outlet conduit and the location of the gate chamber are the same as contemplated in the Banner report. It is recommended, however, that in the final design, consideration be given to an access to the gate chamber other than a cut and cover structure extending upwards through the embankment.
XV. PROJECT COST ESTIMATES

A cost estimate for the proposed project has been prepared and is shown in Table No. 7. Items which were not a part of this contract included: Spillway Design, Outlet Works Design, Road Relocation, and Land Acquisition. The cost for these items were taken from the Banner Associates, Inc., report.

Our preliminary estimates for the cost of these structures were somewhat lower than those indicated in the Banner report. It will be observed from Table No. 7 that 15 percent has been included for contingencies and engineering, which we believe to be adequate in view of the information which is currently known about the proposed structure. Assuming the cost for the Spillway and Outlet Works as shown in the Banner report, our estimated cost for the project will be $24,607,400.
BIBLIOGRAPHY


Sharpe, C.F., Landslides and Related Phenomena, Columbia University Press (1938)

Sherard, J.L., Dunnigan, L.P., Decker, R.S., and Steele, E.F., Pinhole Test for Identifying Dispersive Soils, ASCE (1975)


Seismic Zone Map of the United States

ZONE 0 - No damage.
ZONE 1 - Minor damage, distant earthquakes may cause damage to structures with fundamental periods greater than 10 seconds; corresponds to Intensities V and VI of the MM Scale.
ZONE 2 - Moderate damage, corresponds to Intensity VII of the MM Scale.
ZONE 3 - Major damage, corresponds to Intensity VIII and higher of the MM Scale.

This map is based on the known distribution of damaging earthquakes and the MM intensities associated with these earthquakes, evidence of strain release; and consideration of mapped geologic structures and provinces believed to be associated with earthquake activity. The probable frequency of occurrence of damaging earthquakes in each zone was not considered in assigning ratings to various zones.

*Modified Mercalli Intensity Scale of 1931

(After Algarmissian, 4th World Conference on Earthquake Engineering, 1969)
Earthquake Epicenter with Year and Magnitude (Richter Scale)

Source: National Geophysical and Solar-Terrestrial Data Center, Denver, Colorado
**Figure No. 6**

**Unified Soil Classification System**

<table>
<thead>
<tr>
<th>Major divisions</th>
<th>Group symbols</th>
<th>Typical names</th>
<th>Laboratory classification criteria</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Coarse grained soils</strong>&lt;br&gt;(More than half of material is larger than No. 200 sieve size)</td>
<td>GW</td>
<td>Well-graded gravels, gravel-sand mixtures, little or no fines</td>
<td>$D_{10} &gt; 60$, $C_{u} = \frac{D_{60}}{D_{10}} &gt; 4$, $C_{c} = \frac{(D_{60})^2}{D_{10} - D_{85}}$ between 1 and 3</td>
</tr>
<tr>
<td>Gravels&lt;br&gt;(More than half of material is larger than No. 4 sieve size)</td>
<td>GP</td>
<td>Poorly graded gravels, gravel-sand mixtures, little or no fines</td>
<td>Not meeting all gradation requirements for GW</td>
</tr>
<tr>
<td>(Appreciable amount of fines)</td>
<td>GM*</td>
<td>Silty gravels, gravel-sand-silt mixtures</td>
<td>Above &quot;A&quot; line with P.I. greater than 7 and between 4 and 7 are borderline cases requiring use of dual symbols.</td>
</tr>
<tr>
<td>Clean gravels (little or no fines)</td>
<td>GC</td>
<td>Clayey gravels, gravel-sand-clay mixtures</td>
<td>Atterberg limits above &quot;A&quot; line and P.I. less than 4</td>
</tr>
<tr>
<td><strong>Sand with fines</strong>&lt;br&gt;(More than half of material is smaller than No. 4 sieve size)</td>
<td>SW</td>
<td>Well-graded sands, gravelly sands, little or no fines</td>
<td>Atterberg limits below &quot;A&quot; line or P.I. less than 4</td>
</tr>
<tr>
<td>Clean sands (little or no fines)</td>
<td>SP</td>
<td>Poorly graded sands, gravelly sands, little or no fines</td>
<td>Above &quot;A&quot; line with P.I. between 4 and 7 are borderline cases requiring use of dual symbols.</td>
</tr>
<tr>
<td>(Appreciable amount of fines)</td>
<td>SM*</td>
<td>Silty sands, sand-silt mixtures</td>
<td>Atterberg limits above &quot;A&quot; line and P.I. less than 4</td>
</tr>
<tr>
<td>Clean sands (little or no fines)</td>
<td>SC</td>
<td>Clayey sands, sand-clay mixtures</td>
<td>Limits plotting in hatched zone with P.I. between 4 and 7 are borderline cases requiring use of dual symbols.</td>
</tr>
<tr>
<td><strong>Silt and clays</strong>&lt;br&gt;(Liquid limit less than 50)</td>
<td>ML</td>
<td>Inorganic silts and very fine sands, rock flour, silt or clayey fine sands or clayey silts with slight plasticity</td>
<td>Atterberg limits below &quot;A&quot; line or P.I. less than 4</td>
</tr>
<tr>
<td>Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, clayey clays, lean clays</td>
<td>CL</td>
<td>1</td>
<td></td>
</tr>
<tr>
<td>MH</td>
<td>Inorganic clays of high plasticity, fat clays</td>
<td></td>
<td></td>
</tr>
<tr>
<td>OL</td>
<td>Organic silts and organic silty clays of low plasticity</td>
<td></td>
<td></td>
</tr>
<tr>
<td>CH</td>
<td>Organic clays of medium to high plasticity, organic silts</td>
<td></td>
<td></td>
</tr>
<tr>
<td>OH</td>
<td>Organic clays of medium to high plasticity, organic silts</td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Silt and clay</strong>&lt;br&gt;(Liquid limit greater than 50)</td>
<td>CH</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>MH</td>
<td>Inorganic silts, micaeous or diatomaceous fine sands or silty soils, silty clays</td>
<td></td>
<td></td>
</tr>
<tr>
<td>CH</td>
<td>Inorganic clays of high plasticity, fat clays</td>
<td></td>
<td></td>
</tr>
<tr>
<td>OH</td>
<td>Organic clays of medium to high plasticity, organic silts</td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Highly organic soils</strong></td>
<td>Pt</td>
<td>Peat and other highly organic soils</td>
<td></td>
</tr>
</tbody>
</table>

*Division of GM and SM groups into subdivisions of d and u for roads and airfields only. Subdivision is based on Atterberg limits, suffix d used when L.L. is 28 or less and the P.I. is 6 or less, the suffix u used when L.L. is greater than 28. ** Borderline classifications, used for soils possessing characteristics of two groups, are designated by combinations of group symbols. For example: GW-GC, well-graded gravel-sand mixture with clay binder.*
CONSULTING ENGINEERS

BORROW AREA NO. 100

ESTIMATE QUANTITY = 2,600,000 CU. YDS.

BORROW AREA NO. 100

LOCATION OF TEST PITS FROM BORROW AREA NO. 100 LOCATED IN BIRD GULCH

Rollins, Brown & Gannell, Inc.
Consulting Engineers

Wyoming Water Development Commission
Cheyenne, Wyoming

Savery Dam

Figure 8
<table>
<thead>
<tr>
<th>Test Pit</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Test Pit 101</td>
<td>Brown clay with some silt, gravel, and coarse sand (very hard-dry)</td>
</tr>
<tr>
<td>Test Pit 102</td>
<td>Yellow clay with some silt, gravel, and coarse sand (very hard-dry)</td>
</tr>
<tr>
<td>Test Pit 103</td>
<td>Gray silty clay with gravel and coarse sand (very hard-dry)</td>
</tr>
<tr>
<td>Test Pit 104</td>
<td>Brown clay with some silt, gravel, and coarse sand (very hard-dry)</td>
</tr>
<tr>
<td>Test Pit 105</td>
<td>Yellow clay with some silt, gravel, and coarse sand (very hard-dry)</td>
</tr>
<tr>
<td>Test Pit 106</td>
<td>Brown clay with some silt, gravel, and coarse sand (very hard-dry)</td>
</tr>
<tr>
<td>Test Pit 107</td>
<td>Yellow clay with some silt, gravel, and coarse sand (very hard-dry)</td>
</tr>
<tr>
<td>Test Pit 108</td>
<td>Brown clay with some silt, gravel, and coarse sand (very hard-dry)</td>
</tr>
<tr>
<td>Test Pit 109</td>
<td>Yellow clay with some silt, gravel, and coarse sand (very hard-dry)</td>
</tr>
<tr>
<td>Test Pit 110</td>
<td>Brown clay with some silt, gravel, and coarse sand (very hard-dry)</td>
</tr>
<tr>
<td>Test Pit 111</td>
<td>Yellow clay with some silt, gravel, and coarse sand (very hard-dry)</td>
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<td>Test Pit 112</td>
<td>Brown clay with some silt, gravel, and coarse sand (very hard-dry)</td>
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<td>Test Pit 113</td>
<td>Yellow clay with some silt, gravel, and coarse sand (very hard-dry)</td>
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<td>Test Pit 114</td>
<td>Yellow clay with some silt, gravel, and coarse sand (very hard-dry)</td>
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<tr>
<td>Test Pit 115</td>
<td>Yellow clay with some silt, gravel, and coarse sand (very hard-dry)</td>
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<td>Test Pit 116</td>
<td>Yellow clay with some silt, gravel, and coarse sand (very hard-dry)</td>
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<td>Test Pit 117</td>
<td>Yellow clay with some silt, gravel, and coarse sand (very hard-dry)</td>
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<td>Yellow clay with some silt, gravel, and coarse sand (very hard-dry)</td>
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<td>Yellow clay with some silt, gravel, and coarse sand (very hard-dry)</td>
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<td>Yellow clay with some silt, gravel, and coarse sand (very hard-dry)</td>
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<td>Test Pit 122</td>
<td>Yellow clay with some silt, gravel, and coarse sand (very hard-dry)</td>
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<td>Test Pit 123</td>
<td>Yellow clay with some silt, gravel, and coarse sand (very hard-dry)</td>
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<td>Test Pit 134</td>
<td>Yellow clay with some silt, gravel, and coarse sand (very hard-dry)</td>
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<td>Test Pit 135</td>
<td>Yellow clay with some silt, gravel, and coarse sand (very hard-dry)</td>
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<td>Test Pit 136</td>
<td>Yellow clay with some silt, gravel, and coarse sand (very hard-dry)</td>
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<td>Test Pit 137</td>
<td>Yellow clay with some silt, gravel, and coarse sand (very hard-dry)</td>
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<td>Test Pit 138</td>
<td>Yellow clay with some silt, gravel, and coarse sand (very hard-dry)</td>
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<tr>
<td>Test Pit 139</td>
<td>Yellow clay with some silt, gravel, and coarse sand (very hard-dry)</td>
</tr>
<tr>
<td>Test Pit 140</td>
<td>Yellow clay with some silt, gravel, and coarse sand (very hard-dry)</td>
</tr>
</tbody>
</table>
**BORROW AREA 200**

ESTIMATED QUANTITY: 450,000 CU.YDS.

**LOCATION OF TEST PITS FROM BORROW AREA 200**

**BORROW AREA 200**

**TEST PIT 200**
- **BROWN CLAY**
- **CLAY (DRY)**
- **SILTY MUDSTONE (DRY)**
- **YELLOW CLAY**
- **SANDY GRAVEL**
- **CLAY**
- **GRAVELLY MUDSTONE**
- **GRAY CLAY**
- **SILTY CLAY**
- **CLAY**
- **SANDY GRAVEL**
- **CLAY**
- **GRAVELLY MUDSTONE**

**TEST PIT 201**
- **BROWN CLAY**
- **CLAY (DRY)**
- **SILTY MUDSTONE (DRY)**
- **YELLOW CLAY**
- **SANDY GRAVEL**
- **CLAY**
- **GRAVELLY MUDSTONE**
- **GRAY CLAY**
- **SILTY CLAY**
- **CLAY**
- **SANDY GRAVEL**
- **CLAY**
- **GRAVELLY MUDSTONE**

**TEST PIT 205**
- **BROWN CLAY**
- **CLAY (DRY)**
- **SILTY MUDSTONE (DRY)**
- **YELLOW CLAY**
- **SANDY GRAVEL**
- **CLAY**
- **GRAVELLY MUDSTONE**
- **GRAY CLAY**
- **SILTY CLAY**
- **CLAY**
- **SANDY GRAVEL**
- **CLAY**
- **GRAVELLY MUDSTONE**

**TEST PIT 207**
- **BROWN CLAY**
- **CLAY (DRY)**
- **SILTY MUDSTONE (DRY)**
- **YELLOW CLAY**
- **SANDY GRAVEL**
- **CLAY**
- **GRAVELLY MUDSTONE**
- **GRAY CLAY**
- **SILTY CLAY**
- **CLAY**
- **SANDY GRAVEL**
- **CLAY**
- **GRAVELLY MUDSTONE**

**LOG AND LOCATION OF TEST PITS FROM BORROW AREA 200**

**SAVERY DAM**

Rollins, Brown & Gunnell, Inc.
Consulting Engineers
Cheyenne, Wyoming

Wyoming Water Development Commission

FIGURE 10
BORROW AREA 300
ESTIMATED QUANTITY = 210,000 CU.YDS.

LOCATION OF BORROW AREA 300

APPROXIMATE BOUNDARY OF GRAVEL TERRACE DEPOSIT

SAVERY DAM

WYOMING WATER DEVELOPMENT COMMISSION
CHEYENNE, WYOMING

ROLLINS, BROWN & GUNNELL, INC.
CONSULTING ENGINEERS
LOCATION OF BORROW AREA 400

BORROW AREA 400
ESTIMATED QUANTITY = 1,000,000 CU YDS.

BORROW AREA 400
LOCATION OF TEST PITS IN BORROW AREA 400
CONSULTING ENGINEERS
OF MISCELLANEOUS TEST PITS

LOCATION OF MISCELLANEOUS TEST PITS

TEST PIT 1
BLACK CLAY WITH GRAVEL, ROOTS TO 8' CL-1 (WET)
GRAY WEATHERED SHALE

TEST PIT 2
BLACK CLAY
GRAY GRAY CLAYEY SILT
GRAY GRAY CLAYEY SILT (SATURATED)
GRAY GRAY CLAYEY SILT
GRAY WEATHERED SHALE

TEST PIT 3
LIGHT BROWN Silt CLAY (ROOTS)
GRAY GRAY CLAYEY SILT
GRAY WEATHERED SHALE

TEST PIT 4
BROWN Silt CLAY
GRAY WEATHERED SHALE
LIGHT BROWN Silt CLAY

TEST PIT 5
BLACK CLAYEY Silt
GRAY GRAY CLAYEY SILT
GRAY WEATHERED SHALE

TEST PIT 6
LIGHT BROWN Silt
GRAY GRAY CLAYEY SILT
GRAY WEATHERED SHALE

TEST PIT 7
BLACK CLAYEY Silt
GRAY GRAY CLAYEY SILT
GRAY WEATHERED SHALE

TEST PIT 8
LIGHT BROWN Silt
GRAY CLAYEY Silt
GRAY WEATHERED SHALE

TEST PIT 9
BLACK CLAYEY Silt
GRAY CLAYEY Silt
GRAY WEATHERED SHALE

TEST PIT 10
BLACK CLAYEY Silt
GRAY CLAYEY Silt
GRAY WEATHERED SHALE

TEST PIT 11
BLACK CLAYEY Silt
GRAY CLAYEY Silt
GRAY WEATHERED SHALE

TEST PIT 12
BLACK CLAYEY Silt
GRAY CLAYEY Silt
GRAY WEATHERED SHALE

TEST PIT 13
BLACK CLAYEY Silt
GRAY CLAYEY Silt
GRAY WEATHERED SHALE

TEST PIT 14
BLACK CLAYEY Silt
GRAY CLAYEY Silt
GRAY WEATHERED SHALE

TEST PIT 15
BLACK CLAYEY Silt
GRAY CLAYEY Silt
GRAY WEATHERED SHALE

TEST PIT 16
BLACK CLAYEY Silt
GRAY CLAYEY Silt
GRAY WEATHERED SHALE

TEST PIT 17
BLACK CLAYEY Silt
GRAY CLAYEY Silt
GRAY WEATHERED SHALE

TEST PIT 18
BLACK CLAYEY Silt
GRAY CLAYEY Silt
GRAY WEATHERED SHALE

TEST PIT 19
BLACK CLAYEY Silt
GRAY CLAYEY Silt
GRAY WEATHERED SHALE

TEST PIT 20
BLACK CLAYEY Silt
GRAY CLAYEY Silt
GRAY WEATHERED SHALE

TEST PIT 21
BLACK CLAYEY Silt
GRAY CLAYEY Silt
GRAY WEATHERED SHALE

TEST PIT 22
BLACK CLAYEY Silt
GRAY CLAYEY Silt
GRAY WEATHERED SHALE

TEST PIT 23
BLACK CLAYEY Silt
GRAY CLAYEY Silt
GRAY WEATHERED SHALE

TEST PIT 24
BLACK CLAYEY Silt
GRAY CLAYEY Silt
GRAY WEATHERED SHALE

TEST PIT 25
BLACK CLAYEY Silt
GRAY CLAYEY Silt
GRAY WEATHERED SHALE

TEST PIT 26
BLACK CLAYEY Silt
GRAY CLAYEY Silt
GRAY WEATHERED SHALE

TEST PIT 27
BLACK CLAYEY Silt
GRAY CLAYEY Silt
GRAY WEATHERED SHALE

TEST PIT 28
BLACK CLAYEY Silt
GRAY CLAYEY Silt
GRAY WEATHERED SHALE

TEST PIT 29
BLACK CLAYEY Silt
GRAY CLAYEY Silt
GRAY WEATHERED SHALE

TEST PIT 30
BLACK CLAYEY Silt
GRAY CLAYEY Silt
GRAY WEATHERED SHALE

Rollins, Brown & Gunnell, Inc.
CONSULTING ENGINEERS

WYOMING WATER DEVELOPMENT COMMISSION
CHEYENNE, WYOMING

SAVERY DAM

TEST PIT LOGS 1-30
CONSOLIDATION TEST RESULTS

Figure No. 16  Boring No. 7

Surface Elev.  Depth Interval 25'

LL 31.9  % Pl 22.8  % PI 9.1  % W 32.4  %

Date Test  7/26/82  Dry Unit Wt. 92.8  pcf

Project  Savery Dam

PRESSURE IN TONS PER SQUARE FOOT

VOID RATIO
CONSOLIDATION TEST RESULTS
Figure No. 17  Boring No. 9
Surface Elev. 7045  Depth Interval 6\frac{1}{2}-7\frac{1}{2} foot
LL 31.4 %  PL 14.3 %  PL 17.1 %  W 17.6 %
Date Test 6-15-82  Dry Unit Wt. 108.6 pcF
Project Savery Dam
CONSOLIDATION TEST RESULTS

Figure No. 18  Boring No. 9
Surface Elev. 7045  Depth Interval 11 1/2-12'
LL 27.6%  PL 16.3%  PI 11.3%  W 21.6%
Date Test 6-15-82  Dry Unit Wt. 102.2 pcf
Project Savery Dam

PRESSURE IN TONS PER SQUARE FOOT
CONSOLIDATION TEST RESULTS

Figure No. 19  Boring No. 9
Surface Elev. 4075  Depth Interval 16½-17½'
LL 32.1  % PI 17.0  % PI 15.1% W 26.4%
Date Test  Dry Unit Wt. 96.8 pcf
Project Savery Dam

PRESSURE IN TONS PER SQUARE FOOT
CONSOLIDATION TEST RESULTS
Figure No. 20  Boring No. 11
Surface Elev.  Depth Interval  6-7'
LL  34.1% PL  20.0% PI  14.1% W  10.9%  
Date Test  7/7/82  Dry Unit Wt.  110.4 pcf
Project  Savery Dam
CONSOLIDATION TEST RESULTS

Figure No. 21  Boring No. 14
Surface Elev. 7039  Depth Interval 3.4-4.6'
LL 39.0 %  PL 24.5 %  PI 14.5 %  W 35.7 %
Date Test  Dry Unit Wt. 80.9  pcf
Project Savery Dam

PRES OF VERT. LOAD (pounds per square inch)

PRES IN TONS PER SQUARE FOOT
CONSOLIDATION TEST RESULTS

Figure No. 22  Boring No. 14
Surface Elev. 7039  Depth Interval 6-7'
LL   % PL   % PI   % W  29.7%
Date Test  Dry Unit Wt. 89.9 pcf
Project  Savery Dam

PRESSURE IN TONS PER SQUARE FOOT
CONSOLIDATION TEST RESULTS

Figure No. 23  Boring No. 19

Surface Elev.  Depth Interval  6-7'
LL  27.6 % PL  20.6 % PI  7.0 % W  19.0 %
Date Test  7/7/82  Dry Unit Wt.  101.3  pcf
Project  Savery Dam
CONSOLIDATION TEST RESULTS

Figure No. 24  Boring No. 19
Surface Elev. ___  Depth Interval ___15-16___
LL 33.1  % PL 19.5  % PI 13.6  % W 25.4  %
Date Test 7/7/82  Dry Unit Wt. 98.5  pcf
Project Savery Dam

PRESSURE IN TONS PER SQUARE FOOT

VOID RATIO
CONSOLIDATION TEST RESULTS

Figure No. 25  Boring No. 19
Surface Elev.  Depth Interval 24-25'
LL 33.5% PL 20.1% PI 13.4% W 17.7%
Date Test  Dry Unit Wt. 109.6 pcf
Project  Savery Dam

PRESSURE IN TONS PER SQUARE FOOT
VOID RATIO
CONSOLIDATION TEST RESULTS
Figure No. 26  Boring No. 21
Surface Elev. _____  Depth Interval 3-4'
LL 37.0 %  PL 13.3 %  PI 23.7 %  W 11.9 %
Date Test _______  Dry Unit Wt. 135.5 pcf
Project _______  Savery Dam

0.01  0.1  1.0  10
PRESSURE IN TONS PER SQUARE FOOT
CONSOLIDATION TEST RESULTS

Figure No. 27 Test Pit 501
Surface Elev. 501 Depth Interval 4'
LL 27.0 % PL 19.4 % PI 9.6 % W 12.7 %
Date Test 7/15/82 Dry Unit Wt. 84.4 pcf
Project Savery Dam
CONSOLIDATION TEST RESULTS

Figure No. 28   Test Pit 502

Surface Elev.  Depth Interval 6'
LL 23.4%  PL 16.4%  PI 7.0%  W 11.4%
Date Test  7/15/82  Dry Unit Wt.  97.3  pcf
Project  Savery Dam
CONsolidation test results
figure no. 29 test pit 502
surface elev. depth interval 12'
ll 24.7% pl 15.6% pi 9.1% w 15.5%
date test 7/19/82 dry unit wt. 101.8 pcf
project savery dam
CONSOLIDATION TEST RESULTS

Figure No. 30  Test Pit 503
Surface Elev.  Depth Interval 3'
LL 35.6  % PL 16.9  % PI 18.7  % W 14.2  %
Date Test 7/15/82  Dry Unit Wt. 102.7  pcf
Project Savery Dam

VOID RATIO

0.01  0.1  1.0  10
PRESSURE IN TONS PER SQUARE FOOT
CONSOLIDATION TEST RESULTS
Figure No. 31  Boring No. 7
Surface Elev. __  Depth Interval __ 45'
LL  % PL  % PI  % W  1.9 %
Date Test 7-27-82  Dry Unit Wt. 145.8 pcf
Project Savery Dam
CONSOLIDATION TEST RESULTS

Figure No. 32  Boring No. 7
Surface Elev.  Depth Interval 65'
LL %  PL %  PI %  W %  2.2 %
Date Test  8-02-82  Dry Unit Wt. 150.1 pcf
Project  Savery Dam

CONSOLIDATION TEST RESULTS

PRESERVE IN TONS PER SQUARE FOOT
CONSOLIDATION TEST RESULTS

Figure No. 33  Boring No. 9
Surface Elev.  Depth Interval 25'
LL  % PL  % PI  % W  1.4 %
Date Test  8-10-82  Dry Unit Wt. 142.8 pcf
Project  Savery Dam
CONSORTIATION TEST RESULTS

Figure No. 34   Boring No. 9
Surface Elev.   Depth Interval 45'
LL 55.8%   PL 17.1%   PI 38.7%   W 3.0%
Date Test 8-10-82   Dry Unit Wt. 149.6 pcf
Project  Savery Dam
CONSOLIDATION TEST RESULTS
Figure No. 35  Boring No. 11  
Surface Elev.  Depth Interval 10'
LL 35.7%  PL 20.5%  PI 15.2%  W 4.6%
Date Test 8-03-82  Dry Unit Wt. 130.5 pcf
Project Savery Dam

PRESSURE IN TONS PER SQUARE FOOT
CONSOLIDATION TEST RESULTS
Figure No. 36  Boring No. 11
Surface Elev.  Depth Interval  30'
LL  25.8  %  PL  18.3  %  PI  7.5  %  W  3.7  %
Date Test  8-03-82  Dry Unit Wt.  147.5  pcf
Project  Savery Dam

CONsolidation Test Results

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<th>PRESSURE IN TONS PER SQUARE FOOT</th>
<th>VOID RATIO</th>
</tr>
</thead>
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<td>0.01</td>
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<tr>
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<td>0.15</td>
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<tr>
<td>10</td>
<td>0.14</td>
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</table>
CONsolidation test results

Figure No. 37  Boring No. 11

Surface Elev.  Depth Interval  50'
LL  24.2%  PL  14.5%  PI  9.7%  W  6.1%
Date Test  8-04-82  Dry Unit Wt.  144.8 pcf
Project  Savery Dam

Void Ratio

Pressure in Tons per square foot
CONSOLIDATION TEST RESULTS

Figure No. 38  Boring No. 11

Surface Elev.  Depth Interval  70'

LL  30.6%  PL  16.8%  PI  13.8%  W  5.0%

Date Test  8-3-82  Dry Unit Wt.  151.7 pcf

Project  Savery Dam

PRESSURE IN TONS PER SQUARE FOOT
CONSOLIDATION TEST RESULTS

Figure No. 39  Boring No. 14

Surface Elev.  Depth Interval  30'

LL  26.4%  PL  20.2%  PI  6.2%  W  2.1%

Date Test  8/10/82  Dry Unit Wt.  152.8 pcf

Project  Savery Dam

PRESSURE IN TONS PER SQUARE FOOT
*Saturated using back-pressure techniques
### CONSOLIDATED DRAINED TRIAXIAL SHEAR TEST

<table>
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<tr>
<th>Test no. or symbol</th>
<th>Boring no. or depth</th>
<th>Sample data</th>
<th>Degree of saturation (%)</th>
<th>Confining pressure (psi)</th>
<th>Maximum deviator stress (psi)</th>
<th>Strength values at failure</th>
<th>Sample size, L/D (inches)</th>
<th>Strain rate (inches/minute)</th>
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<td>111</td>
<td>17.6</td>
<td>*</td>
<td>30</td>
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<td>36.0</td>
<td>0.5</td>
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<tr>
<td>9</td>
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<td>109</td>
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<td>116.1</td>
<td>36.0</td>
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</table>

*Satrated using back-pressure techniques

**Figure No. 41**
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</table>

**STANDARD SIEVE SIZES**

- 15
- 12
- 10
- 8
- 6
- 4
- 3
- 2
- 1½
- 1
- ¾
- 1/2
- 3/8
- 4
- 10
- 20
- 40
- 60
- 140
- 200

**GRAIN SIZE DISTRIBUTION CURVE**

Project: Savery Dam
Location: Test Hole 105, 3'-10'

- = dispersing agent
- = no dispersing agent for Test
- = Hole 105 at 3'-10'
- = Hole 503 at 6'-10'
- = Hole 6 at 6'-25'

ROLLINS, BROWN AND GUNNELL, INC.
PROFESSIONAL ENGINEERS

FIGURE NO. 42
<table>
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<th>GRAVEL</th>
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<td>COARSE</td>
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<td></td>
<td>MEDIUM</td>
</tr>
<tr>
<td></td>
<td></td>
<td>FINE</td>
</tr>
<tr>
<td>3&quot;</td>
<td>2&quot;</td>
<td>1½&quot;</td>
</tr>
<tr>
<td>1&quot;</td>
<td>⅜&quot;</td>
<td>⅓&quot;</td>
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<tr>
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</table>

**Standard Sieve Sizes**

**Grain Size Distribution Curve**

*Project: Savery Dam*

*Location: Test Hole 108, 4-8'*

- O dispersing agent
- ● non-dispersing agent
GRAVEL

<table>
<thead>
<tr>
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<tbody>
<tr>
<td>3'</td>
<td>1'</td>
</tr>
<tr>
<td>2</td>
<td>3/4</td>
</tr>
<tr>
<td>1 1/2</td>
<td>3/8</td>
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</table>

SAND

<table>
<thead>
<tr>
<th>COARSE</th>
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<th>FINE</th>
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<tbody>
<tr>
<td>10</td>
<td>20</td>
<td>40</td>
</tr>
<tr>
<td>60</td>
<td>140</td>
<td>200</td>
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</table>

SILT OR CLAY

GRAIN SIZE DISTRIBUTION CURVE

Project: Savery Dam
Location: Test Hole 115, 2-7'

○ dispersing agent
● non-dispersing agent

ROLLINS, BROWN AND GUNNELL, INC.
PROFESSIONAL ENGINEERS

FIGURE NO. 46
GRAVEL SAND
<table>
<thead>
<tr>
<th>COARSE</th>
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<th>MEDIUM</th>
<th>FINE</th>
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</table>

SILT OR CLAY

STANDARD SIEVE SIZES

GRAIN SIZE DISTRIBUTION CURVE

ROLLINS, BROWN AND GUNNELL, INC.
PROFESSIONAL ENGINEERS

Project: Savery Dam
Location: Test Hole 115, 7-11'

FIGURE
No. 47
GRAVEL

<table>
<thead>
<tr>
<th>COARSE</th>
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</table>

SAND

<table>
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<th>COARSE</th>
<th>MEDIUM</th>
<th>FINE</th>
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</table>

SILT OR CLAY

STANDARD SIEVE SIZES

GRAIN SIZE DISTRIBUTION CURVE

Project: Savery Dam
Location: Test Hole 116, 2-11'

ROLLINS, BROWN AND GUNNELL, INC.
PROFESSIONAL ENGINEERS

FIGURE NO. 48
GRAVEL

SAND

SILT OR CLAY

<table>
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</table>

STANDARD SIEVE SIZES

GRAIN SIZE DISTRIBUTION CURVE

Project: Savery Dam
Location: Test Hole 118, 2-5'

ROLLINS, BROWN AND GUNNELL, INC.
PROFESSIONAL ENGINEERS

FIGURE NO. 49
<table>
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<tr>
<td>COARSE</td>
<td>FINE</td>
<td>COARSE</td>
</tr>
<tr>
<td>3'</td>
<td>2'</td>
<td>1 1/2'</td>
</tr>
<tr>
<td>1'</td>
<td>3/8'</td>
<td>4</td>
</tr>
<tr>
<td>20</td>
<td>60</td>
<td>140</td>
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</tbody>
</table>

**Standard Sieve Sizes**

**GRAIN SIZE DISTRIBUTION CURVE**

*Project:* Savery Dam  
*Location:* Test Hole 121, 7-11'  

**FIGURE NO. 50**

**ROLLINS, BROWN AND GUNNELL, INC.**  
**PROFESSIONAL ENGINEERS**
<table>
<thead>
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<tr>
<td></td>
<td>15'</td>
<td>12'</td>
<td>10'</td>
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</table>

**STANDARD SIEVE SIZES**

**GRAIN SIZE DISTRIBUTION CURVE**

- Project: Savery Dam
- Location:

**FIGURE NO. 51**

*Bulk Sample*
GRAIN SIZE DISTRIBUTION CURVE

- **Project:** Savery Dam
- **Location:**

**SILT OR CLAY**

<table>
<thead>
<tr>
<th>GRAVEL</th>
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**STANDARD SIEVE SIZES**

- 15, 12, 10, 8, 6, 4, 3, 2, 1, 1/2, 3/8
- 4, 10, 20, 60, 140, 200

**GRAIN SIZE IN MILLIMETERS**

- 0.001, 0.01, 0.1, 1, 2, 3, 4, 5, 10, 20, 40, 60

**PERCENT FINE BY WEIGHT**

- 0, 10, 20, 30, 40, 50, 60, 80, 100

- **Symbols:**
  - ◇ #307, 5-7'
  - ◇ #309, 1-8'
  - ◇ #310, 1-8'
  - *Bulk Sample

**Figure No. 53**
<table>
<thead>
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<th>SILT OR CLAY</th>
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</table>

**STANDARD SIEVE SIZES**

- 15
- 12
- 10
- 8
- 6
- 4
- 3
- 2
- 1 1/2
- 1
- 3/4
- 1/2
- 3/8
- 1/4
- 1/8

**GRAIN SIZE DISTRIBUTION CURVE**

- Project: Savery Dam
- Location: 

**FIGURE NO. 55**

- #322, 1-8'
- #322, 8-11'
- #326, 3-7'*

*Bulk Sample
<table>
<thead>
<tr>
<th>GRAVEL</th>
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<tr>
<td>Coarse</td>
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<td>15'</td>
<td>12'</td>
<td>10'</td>
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**GRAIN SIZE DISTRIBUTION CURVE**

- **Project:** Savery Dam
- **Location:**
- **FIGURE NO.:** 57

- **O:** #337, 1-5'
- **□:** #337, 5-11'
- **△:** #339, 1-6'

**GRAIN SIZE IN MILLIMETERS**

**PERCENT FINER BY WEIGHT**
GRAVEL COBBLES

SAND

SILT OR CLAY

STANDARD SIEVE SIZES

GRAIN SIZE DISTRIBUTION CURVE

Project: Savery Dam
Location:

ROLLINS, BROWN AND GUNNELL, INC.
PROFESSIONAL ENGINEERS

FIGURE NO. 58

ROLLINS, BROWN AND GUNNELL, INC.
PROFESSIONAL ENGINEERS

GRAIN SIZE DISTRIBUTION CURVE

Project: Savery Dam
Location:

ROLLINS, BROWN AND GUNNELL, INC.
PROFESSIONAL ENGINEERS

FIGURE NO. 58

GRAIN SIZE DISTRIBUTION CURVE

Project: Savery Dam
Location:

ROLLINS, BROWN AND GUNNELL, INC.
PROFESSIONAL ENGINEERS

FIGURE NO. 58

GRAIN SIZE DISTRIBUTION CURVE

Project: Savery Dam
Location:
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STANDARD SIEVE SIZES

GRAIN SIZE DISTRIBUTION CURVE

Project: Savery Dam
Location:

HOLE NO. 405
DEPTH: 3–8'
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<td>COARSE</td>
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<tr>
<td>MEDIUM</td>
<td>FINE</td>
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</table>

**Standard Sieve Sizes**

- 3"
- 2"
- 1 1/2"
- 1"
- 3/4"
- 1/2"
- 3/8"
- 4"
- 10"
- 20"
- 40"
- 60"
- 140"
- 200"

**Grain Size Distribution Curve**

- Project: Savery Dam
- Location:
- Hole No.: 411
- Depth: 5–10'
### Grain Size Distribution Curve

**Project:** Savery Dam  
**Location:** Test Hole 413, 0-2½'

<table>
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<th>SAND</th>
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</tr>
<tr>
<td>FINE</td>
<td>MEDIUM</td>
<td>FINE</td>
</tr>
</tbody>
</table>

#### Standard Sieve Sizes

- 3"  
- 2" 1½"  
- 1"  
- ¾"  
- ½"  
- ¼"  
- ⅛"  
- 4"  
- 10"  
- 20"  
- 40"  
- 60"  
- 140"  
- 200"  

**Graph Details:**
- **X-axis:** Grain size in millimeters  
- **Y-axis:** Percent finer by weight  
- **Symbols:**  
  - ○ dispersing agent  
  - ● non-dispersing agent

---

**Source:** Rollins, Brown and Gunnell, Inc.

**Figure No.:** 62
GRAIN SIZE DISTRIBUTION CURVE

Rollins, Brown and Gunnell, Inc.
Professional Engineers

Project: Savery Dam
Location:

Figure No. 65
SOIL MOISTURE DENSITY RELATIONSHIP
ASTM D 698

Maximum Density 113.9 lbs. per cubic foot
Optimum Moisture 14.2 %
SOIL MOISTURE DENSITY RELATIONSHIP
ASTM D 698
Maximum Density 116.7 lbs. per cubic foot
Optimum Moisture 13.8 %
SOIL MOISTURE DENSITY RELATIONSHIP
ASTM D 698
Maximum Density 119.6 lbs. per cubic foot
Optimum Moisture 11.6%
SOIL MOISTURE DENSITY RELATIONSHIP
ASTM D 698
Maximum Density 110.0 lbs. per cubic foot
Optimum Moisture 14.8 %

Project: Savery Dam
Location: #108 @ 4' - 8'
Figure No. 69
SOIL MOISTURE DENSITY RELATIONSHIP

ASTM D 698

Maximum Density 108.7 lbs. per cubic foot

Optimum Moisture 17.0 %

ROLLINS, BROWN AND GUNNELL, INC.
PROFESSIONAL ENGINEERS

Project: Savery Dam
Location: #110 @ 4'-10'
Figure No. 70
SOIL MOISTURE DENSITY RELATIONSHIP
ASTM D 698
Maximum Density 111.7 lbs. per cubic foot
Optimum Moisture 14.5°
SOIL MOISTURE DENSITY RELATIONSHIP
ASTM D 698
Maximum Density 112.3 lbs. per cubic foot
Optimum Moisture 14.4 %
SOIL MOISTURE DENSITY RELATIONSHIP
ASTM D 698
Maximum Density 114.9 lbs. per cubic foot
Optimum Moisture 13.3 %

Project: Savery Dam
Location: TP 115, 7-11'
Figure No. 73
SOIL MOISTURE DENSITY RELATIONSHIP
ASTM D 698

Maximum Density $101.7$ lbs. per cubic foot
Optimum Moisture $20.3\%$

- MOISTURE IN PERCENT
- DRY UNIT WEIGHT IN LBS. PER CUBIC FOOT

Project: Savery Dam
Location: #116 @ 2'-11'
SOIL MOISTURE DENSITY RELATIONSHIP
ASTM D 698

Maximum Density 122.3 lbs. per cubic foot
Optimum Moisture 11.4 %
SOIL MOISTURE DENSITY RELATIONSHIP
ASTM D 698
Maximum Density 114.2 lbs. per cubic foot
Optimum Moisture 14.0 %
SOIL MOISTURE DENSITY RELATIONSHIP
ASTM D 698

Maximum Density: 121.1 lbs. per cubic foot
Optimum Moisture: 11.3 %

ROLLINS, BROWN AND GUNNELL, INC.
PROFESSIONAL ENGINEERS

Project: Savery Dam
Location: TP 131, 8-15'
Figure No. 77
SOIL MOISTURE DENSITY RELATIONSHIP
ASTM D 698

Maximum Density 112.1 lbs. per cubic foot
Optimum Moisture 15.0 %

DRY UNIT WEIGHT IN LBS. PER CUBIC FOOT

MOISTURE IN PERCENT

ROLLINS, BROWN AND GUNNELL, INC.
PROFESSIONAL ENGINEERS

Project: Savery Dam
Location: TP 134, 0-17'
Figure No. 78
SOIL MOISTURE DENSITY RELATIONSHIP

ASTM D 698

Maximum Density 118.0 lbs. per cubic foot
Optimum Moisture 13.0 %
SOIL MOISTURE DENSITY RELATIONSHIP
ASTM D 698
Maximum Density 141.3 lbs. per cubic foot
Optimum Moisture 8.5 %
SOIL MOISTURE DENSITY RELATIONSHIP
ASTM D 698
Maximum Density 117.4 lbs. per cubic foot
Optimum Moisture 12.7 %
SOIL MOISTURE DENSITY RELATIONSHIP
ASTM D 698

Maximum Density 108.4 lbs. per cubic foot
Optimum Moisture 14.4 %

Figure No. 82
SOIL MOISTURE DENSITY RELATIONSHIP
ASTM D 698

Maximum Density 140.0 lbs. per cubic foot
Optimum Moisture 6.6 %

MOISTURE IN PERCENT

DRY UNIT WEIGHT IN LBS. PER CUBIC FOOT

ROLLINS, BROWN AND GUNNELL, INC.
PROFESSIONAL ENGINEERS

Project: Savery Dam
Location: TP 207, 0-6'
Figure No. 81
SOIL MOISTURE DENSITY RELATIONSHIP
ASTM D 698
Maximum Density 117.5 lbs. per cubic foot
Optimum Moisture 12.8 %

ROLLINS, BROWN AND GUNNELL, INC.
PROFESSIONAL ENGINEERS

Project: Savery Dam
Location: TP 138, 0-16'
Figure No. 80
SOIL MOISTURE DENSITY RELATIONSHIP
ASTM D 698

Maximum Density 142.5 lbs. per cubic foot
Optimum Moisture 6.7 %

ROLLINS, BROWN AND GUNNELL, INC.
PROFESSIONAL ENGINEERS

Project: Savery Dam
Location: #339 @ 1'-5'
Figure No. 85
SOIL MOISTURE DENSITY RELATIONSHIP

**ASTM D 698**

Maximum Density 125.7 lbs. per cubic foot
Optimum Moisture 10.0%
SOIL MOISTURE DENSITY RELATIONSHIP
ASTM D 698

Maximum Density 123.1 lbs. per cubic foot
Optimum Moisture 9.4 %
SOIL MOISTURE DENSITY RELATIONSHIP

ASTM D 698

Maximum Density $114.7$ lbs. per cubic foot
Optimum Moisture $14.5\%$

ROLLINS, BROWN AND GUNNELL, INC.
PROFESSIONAL ENGINEERS

Project: Savery Dam
Location: #410 @ 4'-10'
Figure No. 88
SOIL MOISTURE DENSITY RELATIONSHIP
ASTM D 698
Maximum Density 113.2 lbs. per cubic foot
Optimum Moisture 16.5 %

DRY UNIT WEIGHT IN LBS. PER CUBIC FOOT

MOISTURE IN PERCENT

ROLLINS, BROWN AND GUNNELL, INC.
PROFESSIONAL ENGINEERS

Project: Savery Dam
Location: #413 @ 0'-2.5'
Figure No. 89
SOIL MOISTURE DENSITY RELATIONSHIP
ASTM D 698
Maximum Density 117.9 lbs. per cubic foot
Optimum Moisture 12.3 %

Project: Savery Dam
Location: #413 @ 10'-12'
Figure No. 90
SOIL MOISTURE DENSITY RELATIONSHIP

ASTM D 698

Maximum Density 108.3 lbs. per cubic foot
Optimum Moisture 15.3 %
SOIL MOISTURE DENSITY RELATIONSHIP
ASTM D 698

Maximum Density 119.0 lbs. per cubic foot
Optimum Moisture 13.2 %

Project: Savery Dam
Location: #418 @ 5'-11'
Figure No. 92
SOIL MOISTURE DENSITY RELATIONSHIP

ASTM D 698

Maximum Density 109.6 lbs. per cubic foot
Optimum Moisture 16.0 %

DRY UNIT WEIGHT IN LBS. PER CUBIC FOOT

MOISTURE IN PERCENT
SOIL MOISTURE DENSITY RELATIONSHIP
ASTM D 698
Maximum Density 128.0 lbs. per cubic foot
Optimum Moisture 9.8 %
SOIL MOISTURE DENSITY RELATIONSHIP

ASTM D

Maximum Density 125.2 lbs. per cubic foot
Optimum Moisture 10.5%
## CONSOLIDATED DRAINED DIRECT SHEAR TEST

**HOLE NO. 108**
**DEPTH: 3'-8'**
**PROJECT: Savery Dam**

### Test Values

<table>
<thead>
<tr>
<th>Test no. or symbol</th>
<th>Boring no. or depth</th>
<th>Sample data</th>
<th>Degree of saturation (%)</th>
<th>Normal stress, $\sigma_n$ (psi)</th>
<th>Maximum shear stress, $\tau$ (psi)</th>
<th>Strength values at failure</th>
<th>Sample size (inches)</th>
<th>Strain rate (inches/minute)</th>
</tr>
</thead>
<tbody>
<tr>
<td>108</td>
<td>3'-8'</td>
<td>108.8</td>
<td>15.4</td>
<td>*</td>
<td>31.1</td>
<td>21.8</td>
<td>3</td>
<td>2.5</td>
</tr>
<tr>
<td>108</td>
<td>3'-8'</td>
<td>108.8</td>
<td>15.4</td>
<td>*</td>
<td>43.2</td>
<td>31.2</td>
<td>3</td>
<td>2.5</td>
</tr>
<tr>
<td>108</td>
<td>3'-8'</td>
<td>108.8</td>
<td>15.4</td>
<td>*</td>
<td>65.1</td>
<td>42.7</td>
<td>3</td>
<td>2.5</td>
</tr>
</tbody>
</table>

*Saturated using back-pressure techniques*
Volume change (%) vs Axial strain (%)

Horizontal displacement, \( \delta_h \) (in. \( \times 10^{-2} \))

Shear stress, \( \tau \) (psi)

Normal stress (psi)

Shear stress, \( \tau \) (psi)

<table>
<thead>
<tr>
<th>Test no. or symbol</th>
<th>Boring no. or depth</th>
<th>Sample density (pcf)</th>
<th>Moisture content (%)</th>
<th>Degree of saturation (%)</th>
<th>Normal stress, ( \sigma_n ) (psi)</th>
<th>Maximum shear stress, ( \tau ) (psi)</th>
<th>Strength values at failure</th>
<th>Sample size (inches)</th>
<th>Strain rate (inches/minute)</th>
</tr>
</thead>
<tbody>
<tr>
<td>113</td>
<td>2'-6'</td>
<td>102.0</td>
<td>14.3</td>
<td>*</td>
<td>33.5</td>
<td>19.05</td>
<td>25.5</td>
<td>3</td>
<td>2.5</td>
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<tr>
<td>113</td>
<td>2'-6'</td>
<td>102.0</td>
<td>14.3</td>
<td>*</td>
<td>47.6</td>
<td>26.3</td>
<td>25.5</td>
<td>3</td>
<td>2.5</td>
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<tr>
<td>113</td>
<td>2'-6'</td>
<td>102.0</td>
<td>14.3</td>
<td>*</td>
<td>66.3</td>
<td>35.4</td>
<td>25.5</td>
<td>3</td>
<td>2.5</td>
</tr>
</tbody>
</table>

* Saturated using back-pressure techniques
**Direct Shear Test**

**Project:** Savery Dam

**Hole No.:** 115

**Depth:** 2'-7'

---

**Table:**

<table>
<thead>
<tr>
<th>Test no. symbol</th>
<th>Boring no. or depth</th>
<th>Sample data</th>
<th>Degree of saturation (%)</th>
<th>Normal stress, $\sigma_n$ (psi)</th>
<th>Maximum shear stress, $\tau$ (psi)</th>
<th>Strength values at failure</th>
<th>Sample size (inches)</th>
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</thead>
<tbody>
<tr>
<td>115</td>
<td>2'-7'</td>
<td>106.2</td>
<td>15.2</td>
<td>32.8</td>
<td>18.7</td>
<td>25.7</td>
<td>3</td>
</tr>
<tr>
<td>115</td>
<td>2'-7'</td>
<td>106.2</td>
<td>15.2</td>
<td>49.3</td>
<td>27.5</td>
<td>25.7</td>
<td>3</td>
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<tr>
<td>115</td>
<td>2'-7'</td>
<td>106.2</td>
<td>15.2</td>
<td>65.4</td>
<td>34.7</td>
<td>25.7</td>
<td>3</td>
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</table>

*Saturated using back-pressure techniques*
**Shear stress, \( \tau \) (psi)**

<table>
<thead>
<tr>
<th>Test no. or symbol</th>
<th>Boring no. or depth</th>
<th>Sample data</th>
<th>Degree of saturation (%)</th>
<th>Normal stress, ( \sigma_n ) (psi)</th>
<th>Maximum shear stress, ( \tau ) (psi)</th>
<th>Strength values at failure</th>
<th>Sample size (inches)</th>
<th>Strain rate (inches/minute)</th>
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</thead>
<tbody>
<tr>
<td>121</td>
<td>7-11'</td>
<td>108.6</td>
<td>*</td>
<td>33</td>
<td>20.7</td>
<td>29.9</td>
<td>1.5</td>
<td>2( \frac{3}{8} ) x 2( \frac{3}{8} )</td>
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<tr>
<td></td>
<td></td>
<td>14.0</td>
<td>*</td>
<td>49.2</td>
<td>30.0</td>
<td>29.9</td>
<td>1.5</td>
<td>2( \frac{3}{8} ) x 2( \frac{3}{8} )</td>
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<tr>
<td></td>
<td></td>
<td></td>
<td>*</td>
<td>65.5</td>
<td>39.4</td>
<td>29.9</td>
<td>1.5</td>
<td>2( \frac{3}{8} ) x 2( \frac{3}{8} )</td>
</tr>
</tbody>
</table>

*Saturated using back-pressure techniques*
**Shear stress, \( \tau \) (psi)**

- **Horizontal displacement, \( \delta_h \) (in \( \times 10^{-2} \))**
- **Volume change (%)**
- **Axial strain (%)**

**Consolidated Drained Direct Shear Test**

<table>
<thead>
<tr>
<th>Test no. or symbol</th>
<th>Boring no. or depth</th>
<th>Sample data</th>
<th>Degree of saturation (%)</th>
<th>Normal stress, ( \sigma_n ) (psi)</th>
<th>Maximum shear stress, ( \tau ) (psi)</th>
<th>Strength values at failure</th>
<th>Sample size (inches)</th>
<th>Strain rate (inches/minute)</th>
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</thead>
<tbody>
<tr>
<td>410 4-10</td>
<td>109.4</td>
<td>14.5</td>
<td>*</td>
<td>31.4</td>
<td>22.9</td>
<td>29.3</td>
<td>5</td>
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<tr>
<td>410 4-10</td>
<td>109.4</td>
<td>14.5</td>
<td>*</td>
<td>48.9</td>
<td>32.3</td>
<td>29.3</td>
<td>5</td>
<td>-2.5</td>
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<tr>
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<td>14.5</td>
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<td>64.2</td>
<td>41.4</td>
<td>29.3</td>
<td>5</td>
<td>2.5</td>
</tr>
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*Saturated using back-pressure techniques*

---

**Rollins, Brown and Gunnell, Inc.**

Professional Engineers

**Project:** Savery Dam

**Consolidated Drained**

**HOLE NO. 410**

**DEPTH:** 4'-10'
### CONSOLIDATED DRAINED DIRECT SHEAR TEST

**Project:** Savery Dam  
**HOLE NO.:** 413  
**DEPTH:** 0'-2.5'

<table>
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<tr>
<th>Test no. or symbol</th>
<th>Boring no. or depth</th>
<th>Sample data</th>
<th>Degree of saturation (%)</th>
<th>Normal stress, ( \sigma_n ) (psi)</th>
<th>Maximum shear stress, ( \tau ) (psi)</th>
<th>Strength values at failure</th>
<th>Sample size (inches)</th>
<th>Strain rate (inches/minute)</th>
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<tbody>
<tr>
<td>413 0-2.5</td>
<td>107.9 16.5</td>
<td>*</td>
<td>33.0</td>
<td>23.4</td>
<td>27.7</td>
<td>5.9</td>
<td>2.5</td>
<td>.0014</td>
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<tr>
<td>413 0-2.5</td>
<td>107.9 16.5</td>
<td>*</td>
<td>48.4</td>
<td>30.9</td>
<td>27.7</td>
<td>5.9</td>
<td>2.5</td>
<td>.0014</td>
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<tr>
<td>413 0-2.5</td>
<td>107.9 16.5</td>
<td>*</td>
<td>65.3</td>
<td>40.2</td>
<td>27.7</td>
<td>5.9</td>
<td>2.5</td>
<td>.0014</td>
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</tbody>
</table>

*Saturated using back-pressure techniques*
**Consolidated Drained Triaxial Shear Test**

**Project:** Savery Dam

**Test Pit 108**

**Depth:** 4'-8'

**FIGURE NO. 102**

<table>
<thead>
<tr>
<th>Test no. symbol</th>
<th>Boring no. or depth</th>
<th>Sample data</th>
<th>Degree of saturation (%)</th>
<th>Confining pressure (psi)</th>
<th>Maximum deviator stress (psi)</th>
<th>Strength values at failure</th>
<th>Sample size, L/D (inches)</th>
<th>Strain rate (inches/minute)</th>
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<tr>
<td>108 4'-8'</td>
<td>108.5</td>
<td>15.4</td>
<td>*</td>
<td>20</td>
<td>45.4</td>
<td>29.7</td>
<td>2</td>
<td>2.12</td>
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<tr>
<td>108 4'-8'</td>
<td>108.9</td>
<td>15.4</td>
<td>*</td>
<td>40</td>
<td>82.6</td>
<td>29.7</td>
<td>2</td>
<td>2.12</td>
</tr>
<tr>
<td>108 4'-8'</td>
<td>108.2</td>
<td>15.4</td>
<td>*</td>
<td>50</td>
<td>105.7</td>
<td>29.7</td>
<td>2</td>
<td>2.12</td>
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*Saturated using back-pressure techniques*
**CONSOLIDATED DRAINED TRIAXIAL SHEAR TEST**

<table>
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<th>Test no. or symbol</th>
<th>Boring no. or depth</th>
<th>Sample data</th>
<th>Degree of saturation (%)</th>
<th>Confining pressure (psi)</th>
<th>Maximum deviator stress (psi)</th>
<th>Strength values at failure</th>
<th>Sample size, L/D (inches)</th>
<th>Strain rate (inches/minute)</th>
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<td>2.8/1.3</td>
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<td>2</td>
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<td>116.3</td>
<td>11.4</td>
<td>30</td>
<td>87.5</td>
<td>31.6</td>
<td>4</td>
<td>2.8/1.3</td>
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<tr>
<td>3</td>
<td></td>
<td>116.5</td>
<td>11.4</td>
<td>40</td>
<td>105.7</td>
<td>31.6</td>
<td>4</td>
<td>2.8/1.3</td>
</tr>
</tbody>
</table>

*Saturated using back-pressure techniques*
**CONSOLIDATED DRAINED TRIAXIAL SHEAR TEST**

**TEST PIT 138**
**DEPTH: 0-16'**

**Project:** Savery Dam

*Saturated using back-pressure techniques*
**UNCONSOLIDATED UNDRAINED-UNSATURATED TRIAXIAL SHEAR TEST**

**Test Pit No.: 210**  
**Depth: 5-8\textquoteleft**

**Project:** Savery Dam

*Saturated using back-pressure techniques*

<table>
<thead>
<tr>
<th>Test no. or symbol</th>
<th>Boring no. or depth</th>
<th>Sample data (pcf)</th>
<th>Moisture content (%)</th>
<th>Degree of saturation (%)</th>
<th>Confining pressure (psi)</th>
<th>Maximum deviator stress (psi)</th>
<th>Strength values at failure</th>
<th>Sample size, $l/\bar{f}$ (inches)</th>
<th>Strain rate (inches/minute)</th>
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<tbody>
<tr>
<td>210</td>
<td>5-8\textquoteleft</td>
<td>112.8</td>
<td>12.6</td>
<td>*</td>
<td>30</td>
<td>72.1</td>
<td>17.5</td>
<td>17</td>
<td>2.8 / 1.32 / .0012</td>
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<tr>
<td>210</td>
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<td>112.5</td>
<td>12.6</td>
<td>*</td>
<td>60</td>
<td>97.6</td>
<td>17.5</td>
<td>17</td>
<td>2.8 / 1.32 / .0012</td>
</tr>
<tr>
<td>210</td>
<td>5-8\textquoteleft</td>
<td>112.4</td>
<td>12.6</td>
<td>*</td>
<td>90</td>
<td>124.2</td>
<td>17.5</td>
<td>17</td>
<td>2.8 / 1.32 / .0012</td>
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</table>
Volume change (%) - Deviator stress, $\sigma_1 - \sigma_3$ (psi) - Normal stress (psi) - Shear stress at maximum deviator stress (psi)

<table>
<thead>
<tr>
<th>Test no. symbol</th>
<th>Boring no. or depth</th>
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<th>Moisture content (%)</th>
<th>Degree of saturation (%)</th>
<th>Confining pressure (psi)</th>
<th>Maximum deviator stress (psi)</th>
<th>Friction angle $\phi$ (degrees)</th>
<th>Cohesion $c$ (psi)</th>
<th>Strength values at failure</th>
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<td>5-8'</td>
<td>111.8</td>
<td>12.6</td>
<td>*</td>
<td>30</td>
<td>37.7</td>
<td>16.5</td>
<td>4</td>
<td>2.8</td>
</tr>
<tr>
<td>210</td>
<td>5-8'</td>
<td>111.6</td>
<td>12.6</td>
<td>*</td>
<td>60</td>
<td>58.9</td>
<td>16.5</td>
<td>4</td>
<td>2.8/1.3</td>
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<td>112.3</td>
<td>12.6</td>
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<td>80</td>
<td>74.5</td>
<td>16.5</td>
<td>4</td>
<td>2.8/1.3</td>
</tr>
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</table>

*Saturated using back-pressure techniques

CONSOLIDATED UNDRAINED $K_c = 1.0$

TRIAXIAL SHEAR TEST

PROJECT: Savery Dam

TEST PIT 210

DEPTH: 5-8'

FIGURE NO. 106
**CONSOLIDATED UNDRAINED Kc = 1.5**

**TRIAXIAL SHEAR TEST**

**Project:** Savery Dam

**Test Pit No. 210**

**Depth:** 5-8'

**Figure No. 107**

<table>
<thead>
<tr>
<th>Test no. or symbol</th>
<th>Boring no. or depth</th>
<th>Sample data</th>
<th>Degree of saturation (%)</th>
<th>Confining pressure (psi)</th>
<th>Maximum deviator stress (psi)</th>
<th>Strength values at failure</th>
<th>Sample size, L/D (inches)</th>
<th>Strain rate (inches/minute)</th>
</tr>
</thead>
<tbody>
<tr>
<td>210</td>
<td>5-8'</td>
<td>111.9</td>
<td>12.6</td>
<td>*</td>
<td>20</td>
<td>24.2</td>
<td>22.7</td>
<td>3</td>
</tr>
<tr>
<td>210</td>
<td>5-8'</td>
<td>112.0</td>
<td>12.6</td>
<td>*</td>
<td>40</td>
<td>24.2</td>
<td>22.7</td>
<td>3</td>
</tr>
<tr>
<td>210</td>
<td>5-8'</td>
<td>112.6</td>
<td>12.6</td>
<td>*</td>
<td>60</td>
<td>55.5</td>
<td>22.7</td>
<td>3</td>
</tr>
</tbody>
</table>

*Saturated using back-pressure techniques*
**Consolidated Undrained $k_c = 2.0$ Triaxial Shear Test**

**Project:** Savery Dam

**Test Pit 210**

**Depth:** 5-8′

---

<table>
<thead>
<tr>
<th>Test no. or symbol</th>
<th>Boring no. or depth</th>
<th>Sample data</th>
<th>Degree of saturation</th>
<th>Confining pressure (psi)</th>
<th>Maximum deviator stress (psi)</th>
<th>Strength values at failure</th>
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<tr>
<td></td>
<td></td>
<td>Dry density (pcf)</td>
<td>Moisture content (%)</td>
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<td>Friction angle $\phi$ (degrees)</td>
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<td>210</td>
<td>5-8′</td>
<td>111.8</td>
<td>12.6</td>
<td>*</td>
<td>20</td>
<td>42.2</td>
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<tr>
<td>210</td>
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<td>112.0</td>
<td>12.6</td>
<td>*</td>
<td>30</td>
<td>61.9</td>
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<td>112.0</td>
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<td>*</td>
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</table>

*Saturated using back-pressure techniques

---

\[ \phi = 27.3^\circ \]

\[ C = 3 \text{ psi} \]
**CONSOLIDATED DRAINED TRIAXIAL SHEAR TEST**

**Test Pit No.:** 210  
**DEPTH:** 5-8'  
**Project:** Savery Dam  

<table>
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<tr>
<th>Test no. or symbol</th>
<th>Boring no. or depth</th>
<th>Sample data</th>
<th>Degree of saturation (%)</th>
<th>Confining pressure (psi)</th>
<th>Maximum deviator stress (psi)</th>
<th>Strength values at failure</th>
<th>Sample size, L/D (inches)</th>
<th>Strain rate (inches/minute)</th>
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<td>20</td>
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<td>3</td>
<td>2.8</td>
<td>1.32</td>
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<tr>
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<td>1.32</td>
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*Saturated using back-pressure techniques*
**CONSOLIDATED DRAINED TRIAXIAL SHEAR TEST**

**Project:** Savery Dam

**Test Pit 426**

**Depth:** 0-11'

**FIGURE NO. 110**

*Saturated using back-pressure techniques*
ABUTMENT TO BE STRIPPED TO BEDROCK WITHIN THE FOUNDATION AREA DOWNSTREAM OF CUTOFF BETWEEN STA.13+25 AND STA.31+00.

NOTE: ALL GROUT HOLES SHALL BE INCLINED ISO-20° UPSTREAM, BOTTOM OF CUTOFF TRENCH TO TOP OF CUTOFF TRENCH.

DETAIL 1

TYPICAL CONNECTION OF CUTOFF DRAIN TO OUTFALL DRAIN.

TYPICAL OUTFALL DRAIN TO BE PLACED AT STA.13+25 AND BETWEEN STA.15+00 AND STA.31+00, 200'C.C.

PLAN

TYPICAL CONNECTION OF CUTOFF DRAIN TO OUTFALL DRAIN.

NOTE: GROUTING PROGRAM AS SHOWN IS FOR PLANNING PURPOSES. FINAL DEPTH OF HOLES AND SPACING WILL BE DETERMINED DURING CONSTRUCTION BY THE ENGINEER.
1. Cutoff trench to extend to competent bedrock as determined by the engineer at the time of construction.
2. Cohesive material existing in the upper 8 feet of the embankment foundation shall be removed between STA. 16+00 and STA. 31+00.
3. The filter drain shall be placed between STA. 14+00 and STA. 35+50.
4. Zone I material placed in cutoff trench shall be limited to CL-1 and CL-2 type materials as defined by the Unified Soil Classification System.
5. Riprap shall consist of reasonably well graded rock fragments having a max. size of 24" and 95% larger than 6".
6. Piezometers to be installed at 150' intervals between STA. 15+50 and 30+50. Even number piezometers to extend into the bedrock. Odd number piezometers to terminate in the granular overburden.
FILTERS-------
FREE
DRAINING

SILTY SAND & GRAVEL
K_x = 10,000
K_y = 10,000

ZONE I
K_x = 0.1 ft/yr
K_y = 0.1 ft/yr

ZONE II
K_x = 0.5 ft/yr
K_y = 0.5 ft/yr

SHEELE SHALE FORMATION
K_x = 15 ft/yr
K_y = 15 ft/yr

ROLLINS, BROWN & GUNNELL, INC.
CONSULTING ENGINEERS
WYOMING WATER DEVELOPMENT COMMISSION
CHEYENNE, WYOMING
SAVERY DAM
EQUIPOTENTIAL HEAD LINE DISTRIBUTION—NONHOMOGEOUS & ISOTROPIC CASE
FIGURE NO. 115
SOIL STRENGTH PARAMETERS

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<th>ZONE</th>
<th>k+ (pcf)</th>
<th>C (psf)</th>
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<td>1</td>
<td>125</td>
<td>28&quot;</td>
</tr>
<tr>
<td>2</td>
<td>128</td>
<td>30&quot;</td>
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<tr>
<td>FILTERS</td>
<td>125</td>
<td>35&quot;</td>
</tr>
<tr>
<td>FOUNDATION</td>
<td>130</td>
<td>34&quot;</td>
</tr>
</tbody>
</table>

ZONE I:
- SOIL STRENGTH PARAMETERS
  - FILTERS: 125, 35"
  - FOUNDATION: 130, 34"

ZONE II:
- SOIL STRENGTH PARAMETERS
  - FILTERS: 125, 35"
  - FOUNDATION: 130, 34"

DOWNSTREAM STEADY-STATE FAILURE SURFACE
F.S. = 1.5

CREST EL. 7172

STeadY-STATE PIEZOMETRIC LINE

Sudden-Drawdown Piezometric Line

N.H.W. EL. 7150

ZONE II

ZONE I

NORTHWEST EL. 7150

ZONE I:
- SOIL STRENGTH PARAMETERS
  - FILTERS: 125, 35"
  - FOUNDATION: 130, 34"

ZONE II:
- SOIL STRENGTH PARAMETERS
  - FILTERS: 125, 35"
  - FOUNDATION: 130, 34"

GLOBE ROLLINS, BROWN & GUNNELL, INC.
CONSULTING ENGINEERS

WYOMING WATER DEVELOPMENT COMMISSION
CHEYENNE, WYOMING

SAVERY DAM
CRITICAL FAILURE SURFACES

FIGURE NO. 117
Key: Principal Stress Ratio

A = 1.00
B = 1.25
C = 1.50
D = 1.75
E = 2.00
F = 2.25
G = 2.50
H = 2.75

CONTOURS OF THE PRINCIPAL STRESS RATIO

FIGURE NO. 118
INTAKE STRUCTURE WITH 10'X10' GRATES
6' PRESSURE CONDUIT

CONTROL BUILDING

OUTLET WORKS PROFILE

NOTE: DETAILS OF SPILLWAY AND OUTLET WORKS DESIGN ARE NOT SHOWN. THIS WORK TO BE COMPLETED IN FINAL DESIGN.

OUTLET WORKS INTAKE STRUCTURE

HYDRAULIC JUMP STILLING BASIN

NOTE: CROSS SECT. 35T15N, R89W, WEST OF 6P.M.

PROJECT COORDINATES
N 10,000 - E 10,000

SAVERY CREEK

NE COR. SEC.

SIDE CHANNEL SPILLWAY STRUCTURE 25' WIDE

OVERFLOW CREST EL. 7150

GROUND EL. -7200 - 7100 - 7000

HYDRAULIC JUMP STILLING BASIN

OVERFLOW CREST EL. 7150

LOCATION OF OUTLET WORKS & SPILLWAY
TABLES
<table>
<thead>
<tr>
<th>HOLE NO.</th>
<th>DEPTH BELOW GROUND SURFACE</th>
<th>STANDARD PENETRATION BLOWS PER FOOT</th>
<th>IN-PLACE</th>
<th>UNCONFINED COMPRESSIVE STRENGTH (PSI)</th>
<th>FRICTION ANGLE (°)</th>
<th>CONSISTENCY LIMITS</th>
<th>MECHANICAL ANALYSIS</th>
<th>UNIFIED SOIL CLASSIFICATION SYSTEM</th>
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<td>2</td>
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<td></td>
<td></td>
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<td>1797</td>
<td>22.1, 19.4, 2.7</td>
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<td>60'</td>
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<td></td>
<td></td>
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<td>4918</td>
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<td>4</td>
<td>20'</td>
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<td></td>
<td></td>
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<td>5659</td>
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<td>5933</td>
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<td>15-16½'</td>
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<td>31.9, 22.8, 9.1</td>
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### Table No. 1 Summary of Test Data

**Project:** Savery Dam  
**Feature:** Drill Holes  
**Location:** Foundation

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<th>HOLE NO.</th>
<th>DEPTH BELOW GROUND SURFACE</th>
<th>STANDARD PENETRATION BLOWS PER FOOT</th>
<th>UNIT WEIGHT LB/FT³</th>
<th>MOISTURE PERCENT</th>
<th>VOID RATIO</th>
<th>UNCONFINED COMPRESSIVE STRENGTH PSI</th>
<th>CONSISTENCY LIMITS</th>
<th>MECHANICAL ANALYSIS</th>
<th>UNIFIED SOIL CLASSIFICATION SYSTEM</th>
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<td>% Gravel 31.4 14.3 17.1</td>
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<tr>
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<td></td>
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<td>1.4</td>
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<td>6-7½</td>
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# TABLE NO. 1 SUMMARY OF TEST DATA

**PROJECT**  | Savery Dam  
**FEATURE**  | Test Pits  
**LOCATION** | Foundation  

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<tr>
<th>TEST PITS</th>
<th>DEPTH BELOW GROUND SURFACE</th>
<th>STANDARD PENETRATION BLOWS PER FOOT</th>
<th>IN-PLACE</th>
<th>UNCONFINED COMPRRESSIVE STRENGTH LF/FT</th>
<th>FRICTION ANGLE</th>
<th>CONSISTENCY LIMITS</th>
<th>MECHANICAL ANALYSIS</th>
<th>UNIFIED SOIL CLASSIFICATION SYSTEM</th>
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RESULTS OF SLACKING TESTS

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*All samples permitted to dry before wetting.
# TABLE NO. 3 SUMMARY OF TEST DATA

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**FEATURE** Borrow Area No. 100  
**LOCATION** Bird Gulch

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**TABLE NO. 3 SUMMARY OF TEST DATA**

**PROJECT** Savery Dam  
**FEATURE** Borrow Area No. 100  
**LOCATION** Bird Gulch

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**TABLE NO. 4 SUMMARY OF TEST DATA**

**PROJECT** Savery Dam  
**FEATURE** Borrow Area No. 200  
**LOCATION** Upstream of Left Abutment

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<th>LAB PERMEABILITY UNIT WEIGHT LB/FT³</th>
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<th>FI/YR</th>
<th>CONSISTENCY LIMITS L.L. %</th>
<th>P.L. %</th>
<th>P.I. %</th>
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**Project** | Savery Dam  
**Feature** | Borrow Area No. 300  
**Location** | Near Fork of Little Savery and Savery Creek

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CL-ML

GW

SM

GP

GW

SM
### TABLE NO. 5 SUMMARY OF TEST DATA

**PROJECT** Savery Dam  
**FEATURE** Borrow Area No. 300  
**LOCATION** near Fork of Little Savery and Savery Creek

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# TABLE NO. 6 SUMMARY OF TEST DATA

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