PHASE II - SOIL MECHANICS REPORT

BULLDOG FLOODWAY & APACHE JUNCTION FLOOD CONTROL PROJECT PHOENIX, ARIZONA

Property of
Flood Control District of MC Library
Please Return to
2801 W. Durango
Phoenix, AZ 85009

PREPARED FOR

USDA SOIL CONSERVATION SERVICE
PHOENIX, ARIZONA

NOVEMBER 1985

EBASCO
EBASCO SERVICES INCORPORATED
Two World Trade Center, New York, N.Y. 10048

PHASE II - SOIL MECHANICS REPORT

BULLDOG FLOODWAY AND APACHE JUNCTION

FLOOD CONTROL PROJECT

PHOENIX, ARIZONA

PREPARED FOR

USDA SOIL CONSERVATION SERVICE

PHOENIX, ARIZONA

NOVEMBER 1985

EBASCO SERVICES INCORPORATED

0395v

TABLE OF CONTENTS

Section	Title	Page
I	Introduction	1
II-A	Summary of Tests Performed	4
II-B	Summary of Test Results	8
II-B.1	Field Tests and Sampling	8
II-B.2	Index Tests - Laboratory	10
II-B.3	Strength, Consolidation and Permeability Tests - Laboratory	12
II-B.4	Soil Design Parameters Obtained from Field and Laboratory Testing	13
II-C	Analysis and Interpretation	19
II-C.1	Slope Stability	19
II-C.2	Settlement Potential - General	25
II-C.3	Settlement Potential - Embankment	25
II-C.4	Settlement Potential - Foundation Soils	26
II-C.5	Collapsibility Potential - Foundation Soils	27
II-C.6	Seepage Analysis	31
II-C.7	Embankment Material and Compaction Criteria	32
II-C.8	Central Transition Zone and FAD Diaphragm	33
II-C.9	Foundation Preparation	35
II-C.10	Design of Concrete Walls	36
III	Conclusions and Recommendations	38

TABLES

TABLE I	Subsurface Investigation Summary: Drilling and Test Pits
TABLE II	Detailed Subsurface Investigation Summary as Presented in the SCS Geological Investigation Report
TABLE III	Subsurface Investigation: Field Testing Summary
TABLE IV	Subsurface Investigation: Laboratory Testing Summary
TABLE V	Field Permeability Test Results
TABLE VI	Field Density and Associated Laboratory Tests
TABLE VII	Results of Laboratory Tests Conducted on Compacted Samples
TABLE VIII	Results of Laboratory Tests Conducted on Undisturbed Foundation Samples
TABLE IX	Laboratory Permeability Tests Conducted on Borrow Soils
TABLE X	Freeboard Hydrograph (FBH): Duration of Flood Stages Above Various Elevations

FIGURES

FIGURE 1	Subsurface Profile Along Centerline of FRS
FIGURE 2	Embankment Cross-Section
FIGURE 3	Critical Slip Circles for Reservoir at Freeboard or End of Construction (Downstream Slope)
FIGURE 4	Critical Slip Circles for Reservoir at Crest of Emergency Spillway with Steady State Seepage (Downstream Slope)
FIGURE 5	Critical Slip Circle for Complete Sudden Drawdown

I INTRODUCTION

In April 1985, Ebasco Services Incorporated was awarded a contract for the design of the Bulldog Floodway and Apache Junction Floodway, Flood Retarding Structure (FRS), and Outlet. The contract consists of five phases: Phase I - Site Characterization and Comparative Design Studies, Phase II - Soil Mechanics Report, Phase III - Preliminary Design, Phase IV - Final Design and Phase V - Service During Construction. This Soil Mechanics Report is submitted in fulfillment of the Phase II contract requirements.

The main project features include the Apache Junction Floodway, Flood Retarding Structure, Outlet Channel, and Bulldog Floodway. These features are part of the Buckhorn-Mesa Watershed Project in Maricopa and Pinal Counties, Arizona. The Buckhorn-Mesa Watershed has a total area of approximately 108 square miles.

The Apache Junction Floodway and Flood Retarding Structure control a drainage area of 5.72 square miles. The total watershed to the end of Bulldog Floodway is 11.4 square miles. The Apache Junction Floodway intercepts flow from the drainage area between the Apache Trail Highway and the Apache Junction FRS and conveys those flows to the FRS. The Bulldog Floodway intercepts flows from the drainage area between the Apache Junction FRS and the Signal Butte FRS and conveys those flows, along with the outflow from the Apache Junction FRS, to the Signal Butte reservoir.

The Apache Junction Floodway consists of a rectangular concrete lined channel, energy dissipator, and trapezoidal earth channel which outlets into the Apache Junction FRS. The concrete portion of the floodway varies in width from

8.5 feet to 37.5 feet and is 1,645 feet in length. It intercepts two major drainage basins for which weir inlets will be provided. The major portion of the floodway is designed for a flow of 1,046 cfs. The concrete channel outlets into a grouted rock chute energy dissipator which in turn discharges into the 130 foot wide trapezoidal earth channel. Flows in both the concrete and earth channels are subcritical.

The Apache Junction Flood Retarding Structure is an earth embankment 8,500 feet long with a maximum height of approximately 22 feet. The dam has been classified as a Class "C" structure according to the requirements of Technical Release No. 60. The embankment has a 14 foot top width, 2.5H to 1V upstream slope and 2H to 1V downstream slope. The principal spillway consists of a 30 inch diameter reinforced concrete pipe with a baffle riser. A baffle chute with a 100 foot crest length will serve as the emergency spillway.

The principal spillway discharges into the Apache Junction Outlet Channel. The maximum discharge from the principal spillway is 81 cfs during passage of the 100 year flood. The outlet channel extends from the Apache Junction FRS to the Bulldog Floodway. The Apache Junction Outlet Channel and Bulldog Floodway, which discharge into the Signal Butte FRS, consist of a rectangular concrete channel 10,250 feet in length, a grouted riprap chute energy dissipator, and a trapezoidal earth channel having 100 foot width and 3H to 1V side slopes. The bottom width of the concrete channel varies from 3.5 feet to 50 feet and has a maximum 100 year discharge of 4,670 cfs. Flow in the concrete channel is supercritical. The earth channel is designed for 5,466 cfs. There are fourteen major side inlets along the floodway, including side channel and weir inlets.

As noted in the Geological Investigation Report prepared by the Soil Conservation Service (SCS), the project is situated within the Sonoran Desert section of the Basin and Range physiographic province near the eastern edge of the Phoenix Basin. This basin is subdivided into a series of lesser structural and topographic basins by a number of fault block mountains.

The project is located on a pediment of primarily unconsolidated alluvial fan deposits of Quarternary-Tertiary age. Features of the project intersect numerous ephemeral washes which have their origin mainly in the Goldfield Mountains. Loose alluvial deposits of recent age are in the washes. Caliche is intermittently exposed throughout the project area, especially in the smaller washes, and generally is present at shallow depths elsewhere. Depth to groundwater is unknown, but it is at least several hundred feet deep.

II-A SUMMARY OF TESTS PERFORMED

As presented in the Geological Investigation Report, dated July 1983, the initial subsurface investigation conducted by the Soil Conservation Service (SCS) for the Apache Junction Flood Retarding Structure (FRS), Apache Junction Floodway and Bulldog Floodway consisted of drilling 50 boreholes and digging 377 test pits. The field investigation was performed between June 14 and July 12, 1983 with a few test pits being completed from December 12 to 14, 1983. The boreholes were drilled using a CME Model 75 rig with hollow stem augers. Standard Penetration tests were performed on some of the holes drilled to obtain representative samples, while in others the soil identification was made from visual inspection of the auger cuttings. The trenches were excavated using either a Ford Model 555 backhoe or a Case Model 980-B backhoe. The soil sample list included in the Geological Investigation Report shows that 68 disturbed samples were collected from the various test pits and boreholes. The testing conducted during the field investigation also included three permeability tests on the caliche materials. Table I summarizes the scope of the subsurface exploration conducted by the SCS for the various project features. Table II summarizes in detail the drilling and test pit work performed as presented in the Geological Investigation Report.

The laboratory soil tests performed and presented in the Soil Test Results report dated Nov. 20, 1984 include 67 index tests (grain size, atterberg limits and soluble salts); four dispersion tests; 13 moisture contents; eight consolidation, triaxial shear with pore water pressure measurements, and

compaction; and four falling head permeability tests on the consolidation samples. The report indicates that a total of 74 disturbed soil samples were collected in the field of which seven were not received by the laboratory.

The soil samples for consolidation, permeability and triaxial strength testing were prepared by compacting the disturbed soil to approximately 95 percent of the maximum density previously obtained for the individual samples using the Standard Proctor Test (ASTM D698 - Method A). The moisture contents for the samples prepared were between 1.7 percent and 4.2 percent above optimum. One of the soil samples was obtained from the centerline of the dam (Station 52+50) and the seven others from the borrow area.

Subsequent to this laboratory testing program, three undisturbed samples from the centerline of the dam (Station 52+50) were subjected to index (grain size and atterberg limits), consolidation, permeability (falling head tests on the consolidation samples), and triaxial shear with pore water pressure measurements testing.

During the initial review by Ebasco of the above described field and laboratory test programs it was recommended to the SCS that additional compaction and permeability tests be performed on soils from the borrow area. The purpose of these tests was to determine the permeability coefficients of the soils when compacted at optimum moisture according to ASTM D698 - Method C instead of the previously used Method A, since the materials contained approximately 15 percent fine gravel (soil retained on No. 4 sieve and passing through 3/4 inch sieve).

These tests were performed in June 1985 using four disturbed samples collected during the initial field exploration program, and the results reported on June 24, 1985.

As part of the Phase I scope of work for the subject project, Ebasco subsequently conducted a thorough review and evaluation of the geological and soil mechanics testing information provided by the SCS. The Phase I Suitability Assessment: Soil Mechanics report submitted to the SCS recommended that additional field permeability and in-situ density tests be performed on the overburden soils along with laboratory classification and compaction tests on representative soil samples.

The purpose of the field permeability tests was to ascertain the permeability coefficients of the foundation soils in order to perform a seepage analysis and determine the cutoff requirements for the dam. As stated earlier, during the initial subsurface investigation only three permeability tests were conducted on the caliche material underlying the overburden soils. The in-situ density tests, laboratory classification and compaction tests were performed in order to determine the collapsibility potential of the foundation soils upon wetting and addition of the embankment loading. The boring logs indicate that the soils are generally cemented to some degree and that calcium carbonate ($CaCo_3$) is present. Therefore, it is possible that the soil structure is very strong in the dry state (therefore high blow counts were recorded), but may have a low in-situ density and propensity for collapse.

The actual test program, requested by Ebasco, was outlined to the SCS in a letter dated August 13, 1985 (Ebasco File Ref. USDA 3767-L0021). The field testing was perfomed on August 21 and 22, 1985 and consisted of six permeability tests and six in-situ density tests. Six index (grain size, atterberg limits, moisture content) and Standard Proctor tests were also conducted in the laboratory.

Summaries of the field and laboratory testing performed are presented on Tables III and IV respectively.

II-B SUMMARY OF TEST RESULTS

II-B.1 Field Tests and Sampling

Borings and test pits located along the center line of the Apache Junction FRS show the subsurface materials to consist mainly of silty sands (SM) and clayey sands (SC), with various degrees of cementation and presence of calcium carbonate ($CaCo_3$), underlayed by caliche. Along the few visible washes which cut across the dam, the soils are poorly graded sands (SP) generally extending to a depth of two to four feet. The depth to caliche varies significantly along the length of the structure from one foot to approximately twenty-four feet. Generally, as shown on Figure 1, caliche is found at shallow depths near the west abutment and fluctuates to maximum depths of 24 feet near the east abutment. The caliche varies from fairly soft to hard and ranges from poorly to moderately cemented. Standard Penetration tests show that the overburden soils are loose to medium dense extending from ground surface to a depth of approximately three feet. From this depth to the top of caliche, the subsurface soils appear to be dense to very dense as indicated by the blowcounts recorded. However, five in-situ density tests performed at depths ranging from five to nine feet indicate that the soils are loose to medium dense with the dry inplace densities being only 76 to 92 percent of the maximum densities obtained from Standard Proctor compaction tests. The caliche is very dense and breaks down to a gravelly, silty, fine to coarse sand when augered.

Borings and test pits located along the centerline of the Apache Junction Floodway and Bulldog Floodway show the subsurface conditions to be similar to those encountered along the centerline of the FRS, mainly silty sands (SM) with varying degrees of cementation and calcium carbonate ($CaCO_3$) content underlain by caliche.

The depths to caliche varies from zero to seventeen feet along the Apache Junction Floodway and from zero to twelve feet along the Bulldog Floodway.

Standard Penetration tests were conducted in two of the four boreholes drilled along the Apache Junction Floodway. The blow counts recorded indicate that the overburden soils are loose to medium dense to a depth of approximately three feet. From this depth to the top of caliche the soils appear to be dense to very dense based on the Standard Penetration tests conducted. In the two boreholes drilled along the Bulldog Floodway, Standard Penetration tests were not performed.

During the initial field testing program conducted in June, 1983, three permeability tests were performed along the centerline of the Apache Juntion FRS. It was noted by the SCS during a meeting held on May 23, 1985, that the test locations given in the Geological Investigation Report were incorrect. The correct locations were Stations 97+00 and 91+00 respectively. The permeability test at Station 97+00 was conducted on the caliche material encountered from a depth of three to thirty feet. The permeability rate reported for this test is 0.37 ft/day (1.2×10^{-4} cm/sec). Two other permeability tests also conducted in the caliche at Station 91+00, at depth

intervals of 2.5 ft to 10 ft and 2.5 ft to 20 ft, yielded coefficients of 1.8 ft/day (6.4×10^{-4} cm/sec) and 0.43 ft/day (1.5×10^{-4} cm/sec), respectively.

The results of the five permeability tests subsequently conducted in the SM-SC soils along the centerline of the FRS are presented on Table V. The permeability coefficients range between 1.6×10^{-3} and 4.9×10^{-4} cm/sec. Table VI presents the results of the field density tests conducted during the same time period and the associated laboratory index and compaction tests.

II-B.2 Index Tests - Laboratory

Mechanical analysis were performed on a total of twelve samples from the foundation area. Gradation characteristics indicate that the foundation soils range from silty sands (SM) with a minimum of 25 percent fines to clays (CL) with a minimum of 60 percent fines. The plasticity index for all the soils tested range between zero and 24, with eight of the samples having plasticity index less than 10. It must be noted that the logs of borings and test pits indicate that the majority of the foundation soils consist of silty sands (SM) and clayey sands (SC).

Four double hydrometer dispersion tests conducted on the foundation materials along the Apache Junction FRS yielded the following results:

Sample Number	Location	Classification	Percent Passing No. 200 Sieve	Dispersion (Percent)
84W - 2912	Dam 58+90	SC	25	20
84W - 2914	Dam 53+90	CL	64	20
84W - 2915	Dam 53+90	CL	60	13
84W - 2916A	Dam 53+90	ML	53	6

Dispersion values greater than 20 to 25 percent are usually indicative that dispersion in the field may be a problem while values grater than 50 percent strongly indicate that the soils are susceptible to erosion damage. Erosion resulting from the dispersion phenomenon may be seen in the form of either surface erosion or erosion tunnels extending through the dam, and takes place as the clay particles go into suspension in flowing water.

As noted in the Phase I Design Review Report prepared by the SCS, dispersion is generally associated with fine grained soils. Since the majority of the soils are coarse grained (SM or SC), and given the test percentages presented above, it does not appear that dispersion is a likely design concern. These tests, however, were performed in the laboratory using the double hydrometer procedure. It is suggested that during construction, pinhole tests be performed, as required, to determine whether dispersive clay material is present in the borrow soils to be used as embankment fill.

Forty-one mechanical analysis were conducted on soil samples obtained from the borrow area. Gradation characteristics show the samples to be predominantly silty sands (SM) and clayey sands (SC) with, on average, 84 percent of the material passing the number 4 sieve and 29 percent passing the number 200 sieve. The plasticity index for these soils ranges from zero to 25 with 34 of the samples tested having plasticity index of less than ten.

Eleven soil samples collected along the Bulldog Floodway and three samples from the Apache Junction Floodway were subject to mechanical analysis. The materials were in all cases silty sands (SM) or clayey sands (SC) and the plasticity index were fairly low ranging from zero to a maximum of 12.

A total of thirteen moisture content tests were performed on the soil samples obtained from the Apache Junction FRS, borrow area, Bulldog Floodway and Apache Junction Floodway. Results indicate that the natural moisture content of the silty sands (SM) and clayey sands (SC) ranges between one and four percent approximately. The moisture content of the silts (ML) and clays (CL) ranges between three and seven percent approximately.

II-B.3 Strength, Consolidation and Permeability Tests - Laboratory

Eight consolidated undrained triaxial shear tests with pore pressure measurements and eight one-dimensional consolidation tests were initially performed on compacted samples, seven of which were obtained from the borrow area and one from the centerline of the Apache Junction FRS. Compaction tests according to ASTM D698 - Method A were first conducted to determine maximum dry density and moisture content. The soil samples for strength and consolidation testing were then prepared by compacting the disturbed material to approximately 95 percent of the maximum density obtained as per ASTM D698 - Method A with moisture contents on the wet side of optimum. Four of the eight consolidation samples were subject to falling head permeability tests. Table VII presents the detailed results of all these tests.

Subsequently, consolidated undrained triaxial shear, one-dimensional consolidation, and falling head permeability tests were conducted for three undisturbed silt samples (ML) obtained from the foundation of the Apache Junction FRS (Station 52+50). Table VIII presents the detailed results of these tests.

Additionally compaction and permeability tests were conducted on four samples obtained form the borrow area which were prepared by compacting the material to 95 percent of the maximum density obtained as per ASTM D698 - Method C. The test results are presented in Table IX.

II-B.4 Soil Design Parameters Obtained from Field and Laboratory Testing

Standard compaction curves developed for the four samples from the borrow area in accordance with ASTM D698 - Method C, show the maximum dry density of the borrow soils to range between 125.5 pcf and 127.5 pcf with optimum moisture contents ranging between 9.0 and 10.5 percent. Since the compaction criteria to be established for construction of the embankment will require a minimum density of 95 percent of the maximum obtained using ASTM D698 - Method C, and if optimum moisture content is assumed, a wet density value of 130 pcf will be used as representative of the compacted embankment material. The results of the compaction testing are summarized in Table IX.

The slope stability analysis to be conducted on the Apache Junction FRS will require strength properties for both the embankment and foundation soils. The seven consolidated undrained triaxial shear tests with pore water pressure measurements presented in Table VII indicate that the following values should be used in the analysis for the embankment soils:

$$\bar{\phi} = 37^\circ$$

$$\bar{c} = 0 \text{ psf}$$

The effective cohesion and angle of friction parameters will be used for design since they yield conservative shear strength values under the normal stresses caused by the embankment loads.

Based on the description of the soils provided in the boring logs, field density tests, blow count data (Standard Penetration Tests), results of the consolidated undrained triaxial shear tests on the recompacted silty sand samples (Table VII) and undisturbed silt samples (Table VIII) the following values will be used to represent the strength properties of the foundation materials:

$$\bar{\phi} = 30^\circ$$

$$\bar{c} = 0 \text{ psf}$$

The seepage analysis required to determine flow through the embankment materials will use a conservative permeability coefficient of 1.0×10^{-4} cm/sec. The highest test value reported on Table IX for the SM compacted material (Field Sample Number 36.3) containing 19 percent fines under a surcharge load of 1,000 psf was 8.8×10^{-5} cm/sec. The coefficients shown on Table VII which range from 1.4×10^{-4} cm/sec to 4.6×10^{-4} cm/sec are not representative since they were obtained under a "no-load" condition. Additional falling head permeability tests perfomed on these

consolidation samples under pressures of 2,000 psf, show the coefficient to range between 1.8×10^{-4} cm/sec to 8.8×10^{-5} cm/sec. However, these samples were prepared by compacting according to ASTM D698 - Method A which resulted in lower maximum densities than those subsequently obtained using Method C.

The dry densities of the samples prepared as per Method A criteria ranged between 101.2 pcf and 113.9 pcf while those prepared as per Method C ranged between 119.2 pcf and 122.6 pcf. Similarly, the wet densities of the samples prepared as per Method A criteria ranged between 122.0 pcf and 130.7 pcf while those prepared as per Method C would have ranged between 131.7 pcf and 135.5 pcf if they were compacted at optimum moisture content (the moisture content for preparation of these samples was not noted in the laboratory report). In addition, the permeability coefficient of 1.0×10^{-4} cm/sec compares well with those values published for typical properties of compacted silty sand (SM) materials (NAVFAC presents a typical value of 5×10^{-5} cm/sec) and is representative of the overall range of values obtained from the laboratory tests.

As stated earlier and shown on Table V, the permeability coefficients for the foundation materials tested along the centerline of the Apache Junction FRS ranged between 1.6×10^{-3} cm/sec and 4.9×10^{-4} cm/sec. A value of 1.0×10^{-3} cm/sec will be used as the representative permeability coefficient for the foundation soils.

One-dimensional consolidation tests performed on the seven compacted soil samples obtained from the borrow area indicate that the pre-consolidation pressures (p_c) will in general be greater than the maximum overburden pressure caused by the embankment loads. The maximum embankment load assuming a height of 22 feet and a compacted wet density of 130 pcf will be 2860 psf. The apparent pre-consolidation pressures for the compacted borrow area soils tested range between 1,350 psf to 4,200 psf with an average of 2,770 psf.

The values of void ratio for the borrow area materials, as presented in Table VII, range between 0.391 and 0.644 with an average of 0.457. The recompression index, C_r, ranges between 0.007 and 0.012 with an average of 0.008, and the compression index, C_c, ranges between 0.068 and 0.104 with an average of 0.083. Therefore, the following values will be used if a settlement analysis based on one-dimensional consolidation theory is to be conducted:

preconsolidation pressure, p_c	= 2,750 psf
initial void ratio, e_o	= 0.460
recompression index, C_r	= 0.008
compression index, C_c	= 0.080

It must be noted that the above values accurately represent the overall narrow range of test results obtained in the laboratory. The lowest laboratory test values obtained were not used in the settlement analysis, since the results of the particular test widely differ from all the rest. In any case, irrespective of the test values used, the calculated settlements will not change significantly.

The one-dimensional consolidation tests performed on the three silt (ML) samples obtained from the foundation area of the Apache Junction FRS (Station 52+50) indicate that these soils have also been preconsolidated. As shown on Table VIII the overburden pressures for the samples, as provided by SCS, ranged between 400 psf and 1,200 psf while the preconsolidation pressures ranged betweem 4,000 psf and 6,700 psf. It must be noted that even if the lowest preconsolidation pressure of 4,000 psf is chosen, the embankment loads distributed to the foundation soils would not cause the total pressures to significantly increase beyond the minimum preconsolidation pressures.

The void ratio values for the three samples tested ranged between 0.811 and 1.030 with an average of 0.935. The recompression values, C_r, were 0.008, 0.020 and 0.023 while the compression values, C_c, were 0.094, 0.445 and 0.545. The following parameters obtained for Field Sample Number 11.4 will be used if a settlement analysis based on one-dimensional consolidation theory is to be conducted for the foundation soils:

minimum preconsolidation pressure, p_c	= 6,000 psf
initial void ratio, e_o	= 1.030
recompression index, C_r	= 0.023
compression index, C_c	= 0.545

It will be specified that after construction of the concrete walls for the various project structures the excavated areas be backfilled with a random fill material. The properties of these soils after compaction will be conservatively assumed as follows:

$$\gamma_\omega = 130.0 \text{ pcf}$$

$$\bar{\phi} = 30^\circ$$

$$\bar{c} = 0$$

The maximum lateral pressure for cohesionless materials is likely to be near the at rest pressure, K_o, at the bottom of the wall and decrease to the active pressure, K_a, towards the top of the wall where lateral movement of the wall is permitted. For the analysis of the walls on this project it will be assumed, given the wall thickness to height ratio, that the structures are non-yielding. Thus the at-rest pressure coefficient for level backfill as presented in the Far West States Engineering Design Standards, Figure 2-12, for a friction angle of 30° degrees will be used:

$$K_o = 0.60$$

In addition, any surcharge loading on the wall due to construction or maintenance equipment will also be taken into account during the design of the walls.

II-C ANALYSIS AND INTERPRETATION

II-C.1 Slope Stability

An analysis of slope stability using Bishop's Simplified Method was conducted for a critical cross-section of the Apache Junction FRS utilizing the SLOPE-II computer program. After performing an overall evaluation of the stability using the SLOPE-II program, Ebasco's in-house Slip Circle computer program (also based on Bishop's Simplified Method) was used to determine the stability of a few selected cases. In addition, an infinite slope stability analysis was performed.

The analysis focused on the more critical downstream slope since: (1) it is steeper than the upstream (2H to 1V versus 2.5H to 1V), and (2) the upstream slope will be submerged for most of the cases analyzed and therefore its stability will be increased by the water impounded against the slope. Still a few runs were made for the upstream slope for all conditions considered.

Upon review of Ebasco's Phase I Design Criteria, the SCS suggested that:

1 - Steady state and rapid drawdown conditions be analyzed in accordance with TR-60 based on the potential for the riser to plug.

2 - An infinite slope stability analysis be conducted for partial seepage and sudden drawdown conditions at the upstream slope.

3 - An infinite slope stability analysis be conducted on the cohensionless embankment soils rather than the normal slip circle or random type analysis.

The stability of the embankment was investigated for the following conditions using both the slip circle and infinite slope stability analysis to meet the requirements of Technical Release No. 60 (TR-60) and Ebasco's Phase I Design Criteria:

1 - End of construction without seismic forces.

2 - Reservoir at freeboard storm elevation (EL +1809.4 ft) without seismic forces.

3.a - Reservoir at crest of emergency spillway (EL +1799.7 ft) and steady state seepage without seismic forces.

3.b - Reservoir at crest of emergency spillway (EL +1799.7 ft) and steady state seepage with a horizontal ground acceleration of 0.10g (pseudostatic condition).

4 - Complete sudden drawdown condition with reservoir initially at crest of emergency spillway (EL +1799.7 ft).

The following table presents the minimum factors of safety obtained for the slip circle and infinite slope stability analysis performed for the Apache Junction FRS and, in parentheses, the required safety factors as per TR-60.

APACHE JUNCTION FRS EMBANKMENT SLOPE STABILITY

Loading Condition	Slip Circle Analysis		Infinite Slope Analysis	
	W/O Seismic	W/Seismic	W/O Seismic	W/Seismic
1) End of Construction (DS slope)	1.65 (1.4)	Not Applicable	1.51 (1.1)	Not Applicable
2) Reservoir at free-board storm elevation (DS slope)	1.65 (Not Req'd)	1.32 (Not Req'd)	Not Required	Not Required
3) Reservoir at crest of emergency spill-way and steady state seepage (DS slope)	1.24 (1.5)	0.97 (1.1)	0.78 (1.1)	0.62 (1.1)
4) Sudden drawndown (US slope)	0.87 (1.3)	Not Applicable	0.83 (1.1)	Not Applicable

Note: Failure modes for loading conditions 3 and 4 are shallow circles. Figures 3 to 5 present the critical failure circles.

0395v

The slip circle analysis conducted indicates that shallow failure circles within the saturated embankment zones of both the upstream and downstream slopes may occur for the steady state seepage condition with seismic forces. Shallow sloughs may also occur at the upstream slope in a sudden drawdown condition.

It is Ebasco's opinion that the steady state seepage and rapid drawdown conditions (loading conditions 3 and 4) are not applicable in the analysis of the stability of the Apache Junction FRS under the current operating criteria which states that the "FRS will mitigate the flood peak by storing water behind the embankment over a period of 10 days or less".

Table X presents the duration of water impoundment behind the FRS, at various elevations, for the freeboard hydrograph (FBH). It can be observed that during the FBH, the duration of water impoundment above elevation +1799 feet will be under 10 hours. Under normal operating conditions the impoundment of water behind the FRS above elevation +1795 feet will be under five days (116.2 hrs). Therefore given the unsaturated conditions of the embankment, the penetration of water into the fill material above elevation +1799 feet will only be a few inches. During the freeboard hydrograph the penetration of water into the embankment soils, will at maximum, be a few feet.

The short impoundment durations, coupled with a relatively impervious and unsaturated embankment will prevent the development of steady state seepage along the cross-section of the FRS and thus the possible occurrence of a rapid drawdown condition along a considerable portion of the upstream slope.

It must be noted that if cracks develop in the upstream face and some water penetrates through the embankment, the water flow will be terminated at the filter and the zone of penetration will be restricted to the localized crack area. Therefore, neither steady state seepage or saturation of the embankment will occur.

The seepage analysis performed indicates that if the riser were to plug with the reservoir at the highest possible elevation (1799.7 ft) it would take nearly seventeen months for a steady state seepage condition to be established and then only the lower four feet of material in the downstream slope near the toe becomes saturated. Assuming that the riser were to be plugged for this large period of time and neglecting other factors such as evaporation and bank storage, it is expected that the reservoir level will be significantly lowered since calculations show that water flow through the natural soils beneath the foundation cut-off trench will occur after a relatively short period of three months. The loss of water through the natural soils underneath the foundation will decrease the elevation of the impoundment, unless the reservoir is continuously replinished, and will further decrease the probability of the development of steady state seepage. Therefore it is doubtful that a steady state seepage condition will ever be established in the embankment soils.

Even though it is theoretically possible that the upstream face of the embankment may become saturated at very shallow depths, it is not foreseen that near surface sloughs will occur. The soils used for construction of the embankment will contain at least 15 percent to 20 percent fines (passing No. 200 sieve) and will be compacted at or near optimum moisture content. Thus the soils will exhibit some cohesion after compaction which will increase as the materials become dessicated and cemented. In addition, even if it is

assumed that the soils are cohesionless and a drawdown condition does occur at the upstream face, it is expected that the water retained in the voids would be held by capillary forces and the pore water pressures developed in the fill would be negative thereby making the embankment temporarily more stable than it was at the outset. The triaxial shear strength test results conducted on the compacted borrow area soils indicate that negative pore water pressures do indeed develop in these soils and thus the total strength parameters have a significant temporary cohesion component.

Based on the discussions presented above, it is believed that: (1) the conventional steady state and rapid drawdown conditions are extremely unlikely to occur at the Apache Junction FRS, (2) the infinite slope stability analysis for cohesionless soils is not totally representative of actual field conditions, and (3) sloughs of the near surface materials will be prevented by the limited cohesion properties of the soils and the temporary development of negative pore water pressures upon the possible occurrence of a drawdown condition.

In any case as shown on Figures 4 and 5, if steady state seepage or sudden drawdown conditions were to occur, the only damage to the FRS would be surface sloughs or shallow failure circles of the saturated zones in either slope. This unlikely occurrence can be easily rectified through maintenance rather than drastically decreasing the slopes at the time of construction to satisfy the stability safety factors required by TR-60. The slopes mandated by the safety factor criteria established in TR-60 are approximately 4.5H to 1V downstream and 3.5H to 1V upstream.

For the embankment slopes considered, surface erosion due to rainfall runoff may occur as gullies are formed. In order to minimize this damage, it is possible to provide either a rip-rap cover along the entire dam or some type of protective vegetative cover. Alternatively, a maintenance program can be established to periodically repair the erosion damage in the embankment.

II-C.2 Settlement Potential - General

Deformation of the embankment or foundation soils due to imposed loads generally occurs through either elastic distortion of the soil particles or dissipation of high pore water pressure. These two mechanisms are commonly termed elastic settlement and consolidation settlement.

Consolidation settlement occurs when an external force applied to a saturated soil mass causes the pore water pressure to increase beyond the hydrostatic pressure thus forcing water to flow out of the soil until the pressures are equalized and thereby producing a reduction in soil volume. In order for the theory of consolidation to apply the soils must be fine grained (silts and clays) and be at a saturation level of 90 percent or higher.

II-C.3 Settlement Potential - Embankment

The borrow soils used for construction of the embankment will be silty sands (SM) and clayey sands (SC). Standard proctor (D698-Method C) compaction tests performed on these materials show the optimum moisture content to vary between 9.0 percent and 10.5 percent (Table IX), while saturation water contents are in the order of 18.0 percent (thus only about 50 percent saturation).

During construction of the embankment these relatively impervious soils will be compacted at or near optimum moisture content and, over time, will become dessicated near the surface. Since the soils are mainly composed of coarse grained material (sand) and their natural moisture content will be much lower than the water content at saturation, a settlement analysis of the embankment subjected to its own load based on consolidation theory is not totally appropriate.

However, in order to compare the maximum settlement value calculated for the embankment under its own weight using assumed elastic parameters, a consolidation analysis was performed using the results of the laboratory tests presented in Table VII.

The settlement analysis conducted for the embankment section indicates that settlement will be under one inch and due to the relatively coarse grained nature of the soils, it should take place during construction.

II-C.4 Settlement Potential - Foundation Soils

The overburden soils along the Apache Jundtion FRS mainly consist of silty sands (SM) and clayey sands (SC) with moisture contents ranging between one and four percent. The natural moisture contents of the silt material encountered at Station 52+50 ranged between 5.5 percent and 6.9 percent, while the water content at complete saturation ranged between 30 percent and 38 percent.

Therefore, given the in-situ moisture conditions which are much lower than those at saturation and the grain size distribution of the majority of the soils, a settlement analysis based on consolidation theory is not totally appropriate for the foundation soils.

The potential settlement of the FRS foundation soils was calculated for a section of maximum overburden height using the consolidation test results as well as assumed elastic parameters in order to compare the values obtained. The settlement analysis conducted for the worst case situation indicates that settlement of the foundation soils due to the embankment loading will be under two inches and with the majority of the settlement occurring during construction.

Since at most locations the embankment will be directly supported on the caliche or within close proximity of it, the potential total and differential settlement of the FRS is expected to be a maximum of two inches.

II-C.5 Collapsibility Potential - Foundation Soils

In addition to elastic and consolidation settlement certain soils undergo a sudden and appreciable change in volume when they become saturated and are subjected to a certain load. These collapsible soils usually exhibit the following characteristics:

1 - In-situ dry densities range from 65 pcf to 105 pcf.

2 - In-situ water contents range from 5 percent to 30 percent and are generally below 10 percent.

3 - The clay content of the materials range from 10 percent to 30 percent with the remaining material being silt or fine sand.

4 - Liquid limits are generally below 45 percent and plasticity index below 25 percent.

Due to the dessicated state of the foundation soils along the FRS and the presence of calcium carbonate, the collapsibility potential of these soils was investigated.

The limited field density and laboratory index testing conducted in August 1985 indicates that there is a possibility of certain foundation soils being prone to collapse. The low in-situ dry densities of the materials tested negate the assumption that the high blow counts recorded during the initial subsurface investigation were due to the high densities of the soils. Instead, it appears likely that the high blow counts were caused by the cemented strength of the soil structure in its dry state condition.

It must be noted that the available empirical relationships, based on the index characteristics of the soils, only indicate whether the materials are susceptible to collapse. The available data does not allow a quantitative analysis of the settlements which may possibly occur due to collapse or even certify that collapse will indeed occur.

In order to ascertain the load and the volume change which a collapse prone soil will undergo when saturated and loaded, it is necessary to obtain undisturbed soil samples and conduct a laboratory testing program. Given the

varied nature of soils along the FRS, the collection and testing of samples would require an elaborate and extensive investigation.

To address the issue of collapsibility three alternatives are presented for the SCS's consideration:

1 - Conduct a field and laboratory testing program to determine whether the soils are indeed collapsible and ascertain the amount.

2 - During construction of the embankment, remove all the overburden material to caliche.

3 - Remove the upper twenty-four inches of loose material, compact the foundation surface near optimum moisture content and rely on the filter transition zone to accommodate any movement which may result from the potential collapse.

Removal of the upper twenty-four inches of loose material, and then wetting and compacting the entire foundation area appears to be the most effective solution when considering technical, economic and construction schedule issues and will serve to mitigate any potential for collapse. It is recommended that this alternative be selected due to the following:

1 - A cut-off trench extending below the removed overburden to a five foot depth or to caliche (whichever is less) and having a bottom width of approximately fifteen feet will be required along the length of the FRS. The entire embankment foundation area will be compacted

at a near optimum moisture content using a large vibrating roller (rubber tires or smooth wheel drum) and it is expected that some settlement will occur as a result of this effort.

2 - The construction of the five-foot-deep trench will place a significant portion of the FRS directly on the caliche. Where the FRS still rests on overburden material a large section of the soil layer, which may be susceptible to collapse, would have been removed and the remaining material will have been compacted. Therefore, both the possibility of collapse and the associated amount of settlement would be greatly reduced.

3 - The foundation soils will only be subjected to a small stress increase due to the embankment loads.

4 - All the empirical relationships which determine the collapsibility potential of the soils based on their index characteristics indicate that each of the three silt samples obtained at Station 52+50 (Table VIII) are susceptible to collapse. However, the consolidation tests performed on these undisturbed saturated samples show that a reduction in volume does not occur until relatively high consolidation pressures are applied (4,000 psf, 6,000 psf and 6,700 psf, respectivelly). Furthermore, the percent consolidation for these samples at the maximum applied normal pressure of 16,000 psf were 3.2 percent, 9.0 percent and 11.8 percent, respectivelly. This limited data suggests, assuming the soils are indeed prone to collapse, that: (1) high normal pressures must be exerted to the saturated soils in order for the soil structure to

settle, and (2) the volume changes caused by the collapse are relatively minor and therefore may be controlled by the central transition zone.

II-C.6 Seepage Analysis

A flow net seepage analysis was performed for the subsurface soils using a maximum depth under the embankment of twenty-two feet (maximum depth to caliche) and a permeability coefficient of 1.0×10^{-3} cm/sec. Assuming that the riser becomes plugged and the reservoir water elevation remains at 1799.7 feet, it will take nearly four months (113 days) for water to reach the downstream toe of the embankment. The calculated volume of water flowing at the toe after this period of time is 58 gallons per day per foot length of embankment. The piping potential and seepage pressure calculations indicate that a toe drain is not required.

A seepage analysis conducted for the FRS using a permeability coefficient of 1.0×10^{-4} cm/sec and assuming that the riser becomes plugged (therefore the reservoir water elevation remains at 1799.7 feet) indicates that it will take nearly 17 months for a steady state seepage condition to develop in the embankment. The expected volume of water after this time would be less than two gallons per day per foot length of embankment. It must be noted that if cracks develop in the upstream slope, they will only extend to the transition zone. Therefore possible water flow through the cracks will be terminated at the face of the transition filter. The flow of water through the cracks will shorten the time required for the water to reach the downstream toe to six months in a worst case analysis, however the volume will still be minimal (3

gallons per day per foot length of embankment). Thus horizontal drains connected to the transition filter and extending to the toe are not required.

II-C.7 Embankment Material and Compaction Criteria

A homogenous fill embankment meeting the following gradation criteria is suggested:

SIEVE SIZE	% PASSING BY WEIGHT
3"	100
1"	90-100
1/2"	85-100
#4	75-100
#10	50-100
#40	30-95
#200	15-60
.02	5-40
.001	0-30

The grain size analysis performed on soil samples from the borrow area show that the majority of the available soils will meet the above gradation criteria.

The embankment and transition fill material (Section II-C.8) should be placed at densities not less than 95 percent of maximum density as determined by the applicable method of ASTM-D698. The soils may be placed with a moisture content of plus or minus three percentage points from optimum moisture contents.

II-C.8 Central Transition Zone and FAD Diaphragm

In order to control the cracking which may occur in the embankment due to either potential foundation movement, dessication, or any other phenomenon, a central transition zone along the entire centerline of the FRS will be provided. This transition zone shall be four to five feet wide and extend from a depth of approximately two feet below the embankment crest to two feet above the bottom of the cutoff trench. The purpose of burying the transition zone below the crest is to minimize the seepage of rainfall into this pervious zone and therefore reduce moisture changes inside the embankment. Similarly the transition does not extend to the bottom of the cutoff trench in order to avoid the entrance of water into this zone if reservoir water flows downstream through the foundation soils subject to the pressure head in the reservoir.

The seepage analysis conducted for the FRS indicates that a drain is not required and therefore the purpose of the transition zone is to prevent the possible movement of the embankment soils caused by flowing water. The cohesionless characteristics of the transition material would also permit the readjustment of particles in this zone if the foundation underneath were to move considerably.

Calculations presented in Appendix B include the gradation design of the transition filter material according to the recently revised Soil Mechanics Note No. 1. The transition filter material requires the following approximate gradation:

SIEVE SIZE	% PASSING BY WEIGHT
2"	100
3/4"	90-100
#4	60-100
#10	40-100
#20	20-75
#40	0-55
#60	0-40
#100	0-25
#200	0-5

In addition during design, a filter and drainage (FAD) diaphragm should be provided for both the principal spillway conduit and the AT&T cable crossing the embankment.

II-C.9 Foundation Preparation

As stated in Section II-C.5, it is recommended that the FRS foundation be prepared as follows:

1 - Removal of the upper twenty-four inches of existing base material. At the discretion of the inspector this amount may be varied during construction to reflect actual field conditions encountered.

2 - Compaction of the embankment foundation area at near optimum moisture contents (-2% to 2%) with six to eight passes of a large vibratory roller (rubber tires or smooth wheel drum exerting a minimum energy of 40,000 ft-lbs). The maximum densities of the compacted foundation materials at a depth of six inches should not be less than 98 percent of maximum density as determined by the appropriate Method of ASTM-D698.

3 - Excavation of the cut-off trench extending to a depth of five feet below the removed material or one foot into the caliche, whichever depth is less. The bottom width of the trench shall be fifteen feet with 1.5H to 1V minimum slopes. The purpose of the cut-off trench is primarily to identify and remove buried washes of pervious coarse material. Therefore if the bottom of the cut-off trench is resting on a buried wash of sand and gravel, this material shall be removed regardless of the excavation depth required. The buried channel shall also be removed in the upstream and downstream directions extending to the outermost limits of the FRS foundation using practical widths and side slopes to permit production oriented backfilling operations.

4 - Upon removal of the soils in the cut-off trench, the area shall be compacted to the same requirements previously specified for the overall foundation area. If the excavated soils meet the gradation

requirement of the embankment fill or transition filter material, they can be used during construction.

5 - The recommended location of the centerline of the cut-off trench is coincident to the centerline of the FRS.

II-C.10 Design of the Concrete Walls

The concrete walls for the Apache Junction Floodway, FRS Outlet Channel, and Bulldog Floodway will be designed for lateral pressures exerted by the random backfill soils. Since the depth to caliche is shallow along major portions of the channels, the caliche must be excavated for construction of the walls and the bottom slab will be resting on it. The area between the walls and the excavated side slopes which will be partially caliche, will be backfilled with available random material. In order to prevent the saturation of the backfill, during a significant storm on which water may become pounded on top of the more impervious caliche, weep holes will be provided at the bottom of the walls and spaced at approximately eight feet. Properly graded filter material, located at the bottom of the walls, should be used to provide continuous drainage along the entire outside length of the channel walls and thus serve to hydraulically connect the weep holes. Proper protection should also be provided to prevent the potential loss of soil through the weep holes. By providing weep holes through which water pressures will be relieved, the possibility of both hydrostatic pressures acting against the wall and uplift pressures against the bottom slab are minimized.

As stated earlier in this report all walls on the project, including the Emergency Spillway, will be designed for lateral earth pressures caused by a random backfill material with an empty channel.

The walls are also currently being designed to carry a full head of water in the channels without any support from the structural backfill on the other side. It is not expected that erosion of the structural backfill against the walls will be significant due to the relatively flat ground slopes and thus low velocities of any runoff flow. Since the structural integrity of the channel walls would not be affected by erosion of the structural backfill, periodic maintenance should be used to correct any aesthetic damage caused by erosion.

To prevent erosion on the outside of the Emergency spillway walls, it is suggested that a twelve inch layer of rip-rap, having an approximate width of ten feet be provided.

III CONCLUSIONS AND RECOMMENDATIONS

The following conclusions and recommendations are offered for the design of the various project structures:

1 - Embankment Configuration - As shown in Figure 2, the recommended side slopes of the embankment are 2.5H to 1V upstream and 2H to 1V downstream. A five foot wide transition zone will extend along the centerline of the dam. In addition, a cut-off trench extending to either a depth of five feet in the subsurface soils or one foot into caliche will be required to identify the possible existence of buried washes. The width of the bottom of the cut-off trench shall be fifteen feet with minimum slopes of 1.5H to 1V. The centerline of the trench shall coincide with the centerline of the dam.

2 - Foundation Preparation - All buried washes of coarse material which are uncovered during trench excavation, shall be removed extending to the outer limits of the FRS foundation. The entire foundation area shall be compacted with six to eight passes of a large vibratory roller at near optimum moissure contents (-2% to +2%).

The minimum densities of the compacted foundation materials at a depth of six inches should not be less than 98 percent of the maximum density as determined by the appropriate method of ASTM-D698.

3 - Embankment Compaction - The embankment and transition fill material should be placed at densities not less than 95 percent of maximum density as determined by the applicable method of ASTM-D698. The

soils may be placed with a moisture content of plus or minus three percentage points from optimum moisture content.

4 - Settlement and Seepage - The settlement analysis conducted indicates that over building or providing camber to the crest of the FRS to account for future total or differential settlement is not required. Similarly, horizontal drains extending from the transition zone to the toe of the downstream slope in order to reduce seepage pressures and carry water are not necessary.

5 - Principal Spillway and AT&T Cable Crossing - A filter and drainage diaphragm to meet the requirements of Soil Mechanics Note No. 1 must be provided. In addition, if the pipe is not resting on caliche, the conduit joints must be designed to remain watertight under the maximum probable joint opening computed using TR-18.

6 - Maintenance - Unless the FRS slopes are protected, periodic maintenance will be required to repair the damages caused by erosion due to surface water runoff. Similar maintenance will be required for the backfill material along the channel walls.
Maintenance of the FRS slopes will also be required, if sloughs or shallow failures of the near surface soils occur. If this possible maintenance requirement is not acceptable, the slopes must be flatten or other alternatives pursued.

TABLES

TABLE I

SCS - APACHE JUNCTION AND BULLDOG PROJECT

SUBSURFACE INVESTIGATION SUMMARY

DRILLING AND TEST PITS

(Source: USDA - SCS Geological Investigation Report)

AREA	BOREHOLES DRILLED	TEST PITS EXCAVATED
Centerline of Dam	16	38
Principal Spillway	2	--
Emergency Spillway	13	37
Alternate Emergency Spillway	6	--
*Emergency Spillway Alternate No. 1	--	10
Emergency Spillway Alternate No. 2	--	21
Borrow Area	7	204
Apache Junction Floodway	4	5
Centerline of Outlet Channel (Principal Spillway)	--	7
Bulldog Floodway	2	55
TOTAL	50	377

* Location of Ebasco proposed Emergency Spillway

0395v

TABLE II

SCS - APACHE JUNCTION AND BULLDOG PROJECT

DETAILED SUBSURFACE INVESTIGATION SUMMARY

AS PRESENTED IN THE SCS GEOLOGICAL INVESTIGATION REPORT

Area	Geological Investigation Report Pages	Date Started	Date Completed	Equipment Used	Work Performed
Centerline of Dam	p. 9-p. 19	6/26/83	6/29/83	CME-75	Borings 1-16
	p. 20-p. 26	6/14/83	6/15/83	Ford 555 Backhoe	TP 6-TP 23
	p. 27-p. 36	7/7/83	7/8/83	Ford 555 Backhoe	TP330-TP349
Bulldog Floodway	p. 37-p. 38	6/17/83	6/17/83	Ford 555 Backhoe	TP 44-TP 52
	p. 39-p. 44	6/30/83	7/1/83	Ford 555 Backhoe	TP278-TP299
	p. 45-p. 47	12/14/83	12/14/83	Case 980-B Backhoe	TP396-TP406
Emergency Spillway Alternate No. 2	p. 48-p. 52	12/12/83	12/12/83	Case 580-B Backhoe	TP408-TP428
Emergency Spillway Alternate No. 1	p. 53-p. 54	7/12/83	7/12/83	Ford 555 Backhoe	TP350-TP359
Alternate Emergency Spillway	p. 55-p. 57	7/6/83	7/8/83	CME-75	Borings 201-206
Emergency Spillway	p. 58-p. 60	7/1/83	7/5/83	CME-75	Borings 21-32
	p. 61	12/13/83	12/13/83	Case 980-B Backhoe	TP392-TP395
	p. 62	7/8/83	7/8/83	CME-75	Boring 207
	p. 63-p. 68	7/11/83	7/11/83	Ford 555 Backhoe	TP360-TP392
Bulldog Floodway	p. 69	7/8/83	7/8/83	CME-75	Borings 33-34
	p. 70-p. 73	6/16/83	6/17/83	Ford 555 Backhoe	TP 31-TP 43
Apache Jct Floodway	p. 74	6/14/83	6/14/83	Ford 555 Backhoe	TP 1-TP 5
	p. 75-p. 77	6/29/83	6/29/83	CME-75	Borings 17-20
Centerline of Outlet Channel	p. 78-p. 79	6/15/83	6/15/83	Ford 555 Backhoe	TP 24-TP 30
Principal Spillway	p. 80	7/12/83	7/12/83	CME-75	Borings 301-302
Borrow Area	p. 81-p. 90	6/20/83	6/21/83	Ford 555 Backhoe	TP101-142
	p. 91-p.117	6/21/83	6/28/83		TP146-277
	p.118-p.126	7/1/83	7/6/83	Ford 555 Backhoe	TP300-TP329
	p.127-p.128	7/12/83	7/12/83	CME-75	Borings 35-41

TABLE III

SCS-APACHE JUNCTION AND BULLDOG PROJECT

SUBSURFACE INVESTIGATION: FIELD TESTING SUMMARY

Area	Permeability	Density	Approx. Date	Source
Centerline of Dam	3	--	6/83	USDA - SCS Geological Investigation Report
Centerline of Dam	5	6	8/21/85	

0395v

TABLE IV

APACHE JUNCTION AND BULLDOG PROJECT

SUBSURFACE INVESTIGATION: LABORATORY TESTING SUMMARY

Area	Grain Size	Atterberg Limits	Soluble Salts	Dispersion	Natural Moisture	Consolidation	Permeability	Triaxial w/ Pore Ppressure	Compaction	Source
Centerline of Dam	12	12	12	4	6	1	1	1	1	USDA - SCS Soil Test Results Dated 11/20/84 File Code 210-22
Bulldog Floodway	11	11	11	-	2	-	-	-	-	
Borrow Area	41	41	41	-	4	7	3	7	7	
Apache Junction Floodway	3	3	3	-	1	-	-	-	-	
Centerline of Dam	3	3	-	-	-	3	3	3	-	USDA -SCS Soil Test Results Dated 2/5/85 File Code 210-22
Borrow Area	-	-	-	-	-	-	4	-	4	USDA - SCS Soil Test Results Dated 6/24/85 File Code 210-22
Centerline of Dam	6	6	-	-	6	-	-	-	6	

0395v

TABLE V

SCS - APACHE JUNCTION AND BULLDOG PROJECT

FIELD PERMEABILITY TEST RESULTS

(SOURCE: USDA - SCS LETTER DATED 9/24/85)

Test Location Station)	Depth Test Conducted (ft)	Soil Classification	Permeability (cm/sec)
DAM 67+00	5' - 15'	SC-SM	9.3×10^{-4}
DAM 80+40	5' - 10'	SM	1.5×10^{-3}
DAM 81+00	2' - 7'	SC-SM	1.6×10^{-3}
DAM 87+50	5' - 10'	SC	4.9×10^{-4}
DAM 98+30	1' - 6'	SC	7.9×10^{-4}

NOTES: 1) Test Data not yet received from SCS

TABLE VI

SCS-APACHE JUNCTION AND BULLDOG PROJECT

FIELD DENSITY AND ASSOCIATED LABORATORY TESTS

(SOURCE: USDA - SCS TEST RESULTS TRANSMITTED ON 9/24/85)

Test Location (Station)	Depth Test Conducted (ft)	Soil Classification	In-situ Dry Density (pcf)	In-situ Moisture Content (%)	Liquid Limit	Plasticity Index	Percent Passing No. 200	Percent Passing No. 4	STANDARD PROCTOR d max (pcf)	STANDARD PROCTOR w_o (%)
32+50	5	SM	99.0	4.8	NP	NP	2.6	50.4	119.2	12.2
42+50	5	SM	103.4	6.0	33.4	14.2	21.0	88.1	122.8	10.9
53+90	3	SM	75.0	3.8	NP	NP	17.1	69.2	127.1	9.8
53+90	9	SM	85.9	10.5	NP	NP	29.5	93.1	112.0	14.5
70+00	5	SM-SC	94.0	4.9	24.5	4.3	32.9	91.7	122.9	11.5
87+50	8	SM	101.5	10.8	NP	NP	9.3	88.8	109.8	16.7

TABLE VII

SCS - APACHE JUNCTION AND BULLDOG PROJECT

LABORATORY TEST RESULTS CONDUCTED ON COMPACTED SAMPLES

(SOURCE: USDA - SCS SOIL TEST RESULTS DATED 11/20/84)

Sample Location (Station)	Field Sample Number	Sample Depth (feet)	Classification	Liquid Limit	Plasticity Index	Percent Passing No. 200	Percent Passing No. 4	STANDARD PROCTOR		SAMPLE PREPARATION		TRIAXIAL SHEAR				PERME-ABILITY
								d max	w_o	Sample Molding Moisture	Sample Compaction	ϕ	c (psf)	$\bar{\phi}$	$\bar{c}$ (psf)	(cm/sec)
DAM 52+00	11.1	5'-10'	SC	36	12	37%	86%	118.0 pcf	12.0%	16.2%	95.3%	19.0°	1575	37.0°	0	2.5×10^{-4}
BORROW 91+00 400 US	152.1	0'-4'	SC-SM	21	5	36%	91%	125.5 pcf	9%	12.3%	95.3%	22.0°	1875	37.0°	0	
BORROW 52+00 600 US	38.1	10'-15'	SC	30	11	16%	76%	123.5 pcf	11%	13.3%	95.2%			39.5°	0	
BORROW 100+00 600 US	160.1	2.5'-3'	SM	NP	NP	23%	81%	120.0 pcf	11.5%	14.7%	94.9%	32.5°	1500	37.0°	375	2.8×10^{-4}
BORROW 92+00 600 US	164.1	0.5'-2.5'	SM	NP	NP	35%	87%	106.5 pcf	16.5%	20.6%	95.0%	29.5°	1575	38.0°	100	1.4×10^{-4}
BORROW 62+00 800 US	36.3	10'-15'	SM	28	5	19%	80%	122.0 pcf	11.5%	13.2%	94.9%	41.0°	1350	39.0°	100	
BORROW 62+00 800 US	36.4	15'-20'	SM	28	2	26%	86%	119.5 pcf	12.0%	14.4%	95.0%	21.0°	1900	35.5°	275	4.6×10^{-4}
BORROW 78+00 1000 US	40.1	10'-15'	SM	28	4	29%	83%	121.5 pcf	11.5%	13.6%	95.0%	24.5°	1250	38.5°	0	

NOTES: 1) Standard Proctor as psr ASTM D698 - Method A.

2) Permeability coefficients reported above were obtained from consolidation tests under a "no load" condition.

3) Values for p_c, C_c, and C_r were determined by Ebasco based on results of consolidation tests.

0395v

TABLE VII (Cont'd)

SCS - APACHE JUNCTION AND BULLDOG PROJECT

LABORATORY TEST RESULTS CONDUCTED ON COMPACTED SAMPLES

(SOURCE: USDA - SCS SOIL TEST RESULTS DATED 11/20/84)

Sample Location (Station)	Field Sample Number	Sample Depth (feet)	Classification	Liquid Limit	Plasticity Index	Percent Passing No. 200	Percent Passing No. 4	STANDARD PROCTOR		SAMPLE PREPARATION			CONSOLIDATION		
								d max	w_o	Sample Molding Moisture	Sample Compaction	p_c (psf)	e_o	C_c	C_r
DAM 52+00	11.1	5'-10'	SC	36	12	37%	86%	118.0 pcf	12.0%	16.2%	95.3%	4,200	0.486	0.096	0.008
BORROW 91+00 400 US	152.1	0'-4'	SC-SM	21	5	36%	91%	125.5 pcf	9.0%	12.3%	95/3%	1,300	0.391	0.077	0.008
BORROW 52+00 600 US	38.1	10'-15'	SC	30	11	16%	76%	123.5 pcf	11.0%	13.3%	95.2%	2,400	0.410	0.075	0.007
BORROW 100+00 600 US	160.1	2.5'-3'	SM	NP	NP	23%	81%	120.0 pcf	11.5%	14.7%	94.9%	2,300	0.444	0.078	0.008
BORROW 92+00 600 US	164.1	0.5'-2.5'	SM	NP	NP	35%	87%	106.5 pcf	16.5%	20.6%	95.0%	4,000	0.644	0.104	0.012
BORROW 62+00 800 US	36.3	10'-15'	SM	28	5	19%	80%	122.0 pcf	11.5%	13.2%	94.9%	4,200	0.420	0.068	0.007
BORROW 62+00 800 US	36.4	15'-20'	SM	28	2	26%	86%	119.5 pcf	12.0%	14.4%	95.0%	2,700	0.460	0.084	0.007
BORROW 78+00 1000 US	40.1	10'-15'	SM	28	4	29%	83%	121.5 pcf	11.5%	13.6%	95.0%	2,500	0.430	0.101	0.008

0395v

TABLE VIII

SCS - APACHE JUNCTION AND BULLDOG PROJECT

LABORATORY TEST RESULTS CONDUCTED ON UNDISTURBED FOUNDATION SAMPLES

(SOURCE: USDA - SCS SOIL TEST RESULTS DATED 2/5/85

Sample Location (Station)	Field Sample Number	Sample Depth (feet)	Classification	Liquid Limit	Plasticity Index	Percent Passing No. 200	Percent Passing No. 4	TRIAXIAL SHEAR TEST						CONSOLIDATION				PERMEABILITY (cm/sec)
								ϕ	c (psf)	$\bar{\phi}$	$\bar{c}$ (psf)	p_o	p_c	e_o	C_c	C_r		
DAM 52+50	11.2	3.5'-4.7'	ML	25	3	80%	100%	9°	5470	45°	0	400	4,000	0.811	0.094	0.008		7.0×10^{-4}
DAM 52+50	11.3	12'-13.2'	ML	29	5	85%	100%	4°	2370	40°	0	1,200	6,700	0.976	0.445	0.020		2.8×10^{-4}
DAM 52+50	11.4	12'-13.2'	ML	29	4	88%	100%	10.5°	2300	41°	0	1,200	6,000	1.030	0.545	0.023		2.5×10^{-4}

NOTES: 1) Permeability coefficients reported above were obtained from consolidation tests under a "no load" condition.

2) Values of p_o, p_c, e_o and C_c were provided by SCS in the consolidation curve plots. Values of C_r were determined by Ebasco based on results of consolidation tests.

0395v

TABLE IX

SCS - APACHE JUNCTION AND BULLDOG PROJECT

LABORATORY PERMEABILITY TESTS CONDUCTED ON BORROW SOILS

(SOURCE: USDA - SCS SOIL TEST RESULTS DATED 6/24/85)

Sample Location (Station)	Field Sample Number	Sample Depth (feet)	Classification	Liquid Limit	Plasticity Index	Percent Passing No. 200	Percent Passing No. 4	STANDARD PROCTOR		SAMPLE PREPARATION		PERMEABILITY (cm/sec)	
								d max	w o	Sample Molding Moisture	Sample Compaction	Surcharge Load: 1,000 psf	Surcharge Load: 2,000 psf
Borrow 52+00 600 US	38.1	10'-15'	SC	30	11	16%	76%	127.5 pcf	9.0%	--	95%	3.2×10^{-5}	2.1×10^{-5}
Borrow 100+00 600 US	160.1	2.5'-3'	SM	NP	NP	23%	81%	125.5 pcf	10.5%	--	95%	7.1×10^{-6}	----
Borrow 62+00 800 US	36.3	10'-15'	SM	28	5	19%	80%	126.0 pcf	10.5%	--	95%	8.8×10^{-5}	4.6×10^{-5}
Borrow 78+00 1000 US	40.1	10'-15'	SM	28	4	29%	83%	127.5 pcf	9.5%	--	95%	1.1×10^{-6}	----

NOTES: 1) Standard Proctor as per ASTM D698 - Method C.

2) Sample molding moisture not reported.

TABLE X

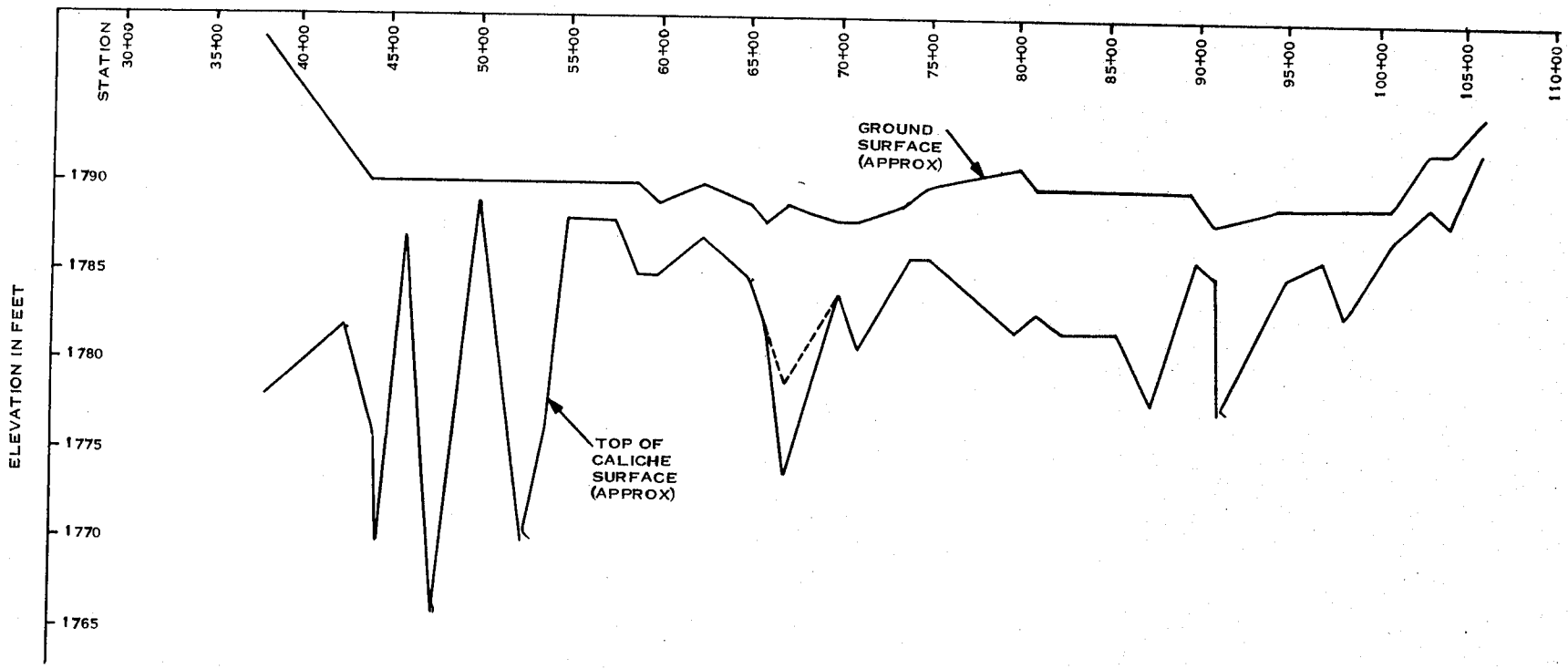
SCS - APACHE JUNCTION AND BULLDOG PROJECT

FREEEBOARD HYDROGRAPH (FBH)

DURATION OF FLOOD STAGES ABOVE VARIOUS ELEVATIONS

Elevation (feet)	Duration (hrs)
1809	0.9
1808	1.7
1807	2.3
1806	2.9
1805	3.5
1804	4.2
1803	5.0
1802	6.0
1801	7.2 (estimated)
1800	8.4 (estimated)
1799	41.4 (estimated)
1798	65.7 (estimated)
1797	83.8 (estimated)
1796	102.0 (estimated)
1795	116.2 (estimated)
1794	218.0 (estimated)

FIGURES



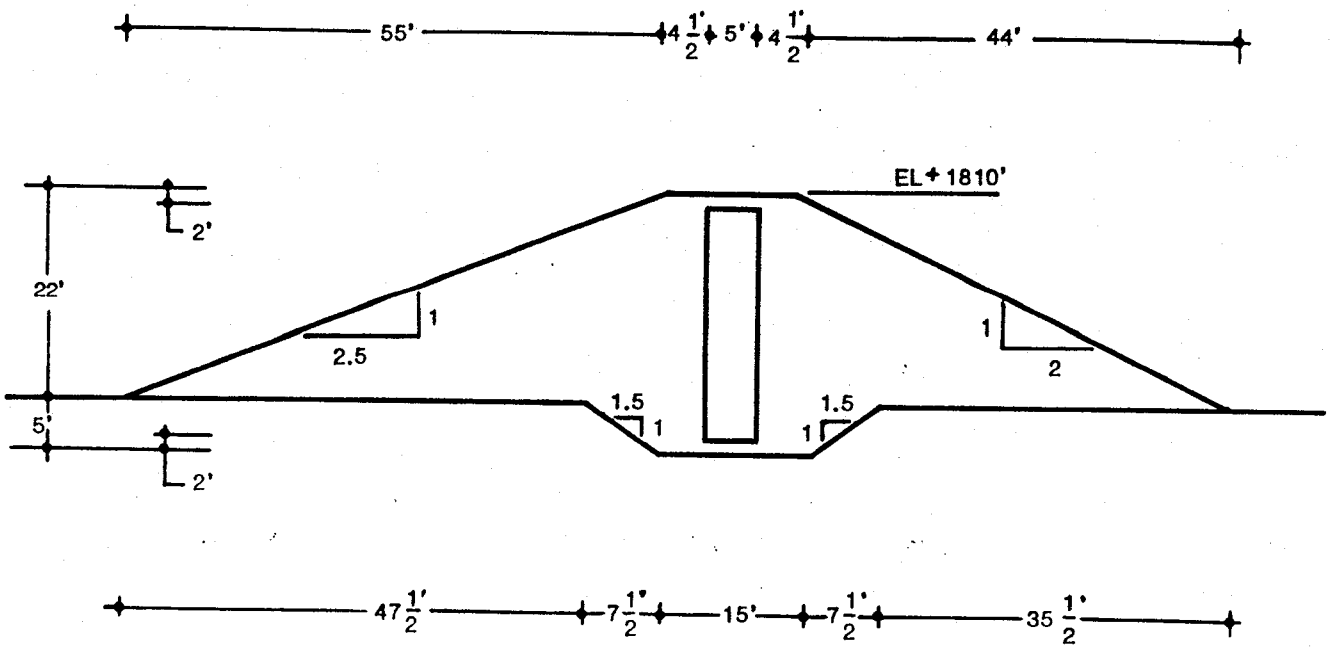
USDA-SOILS CONSERVATION SERVICE

APACHE JUNCTION FRS

SUBSURFACE PROFILE ALONG CENTERLINE OF FRS

FIGURE I

EBASCO SERVICES INCORPORATED



USDA-SOILS CONSERVATION SERVICE

APACHE JUNCTION FRS

EMBANKMENT
CROSS - SECTION
FIGURE 2

EBASCO SERVICES INCORPORATED

EBASCO SERVICES INCORPORATED

SHEET ____ OF ____

CLIENT USDA-SCS

PROJECT APACHE JUNCTION FRS

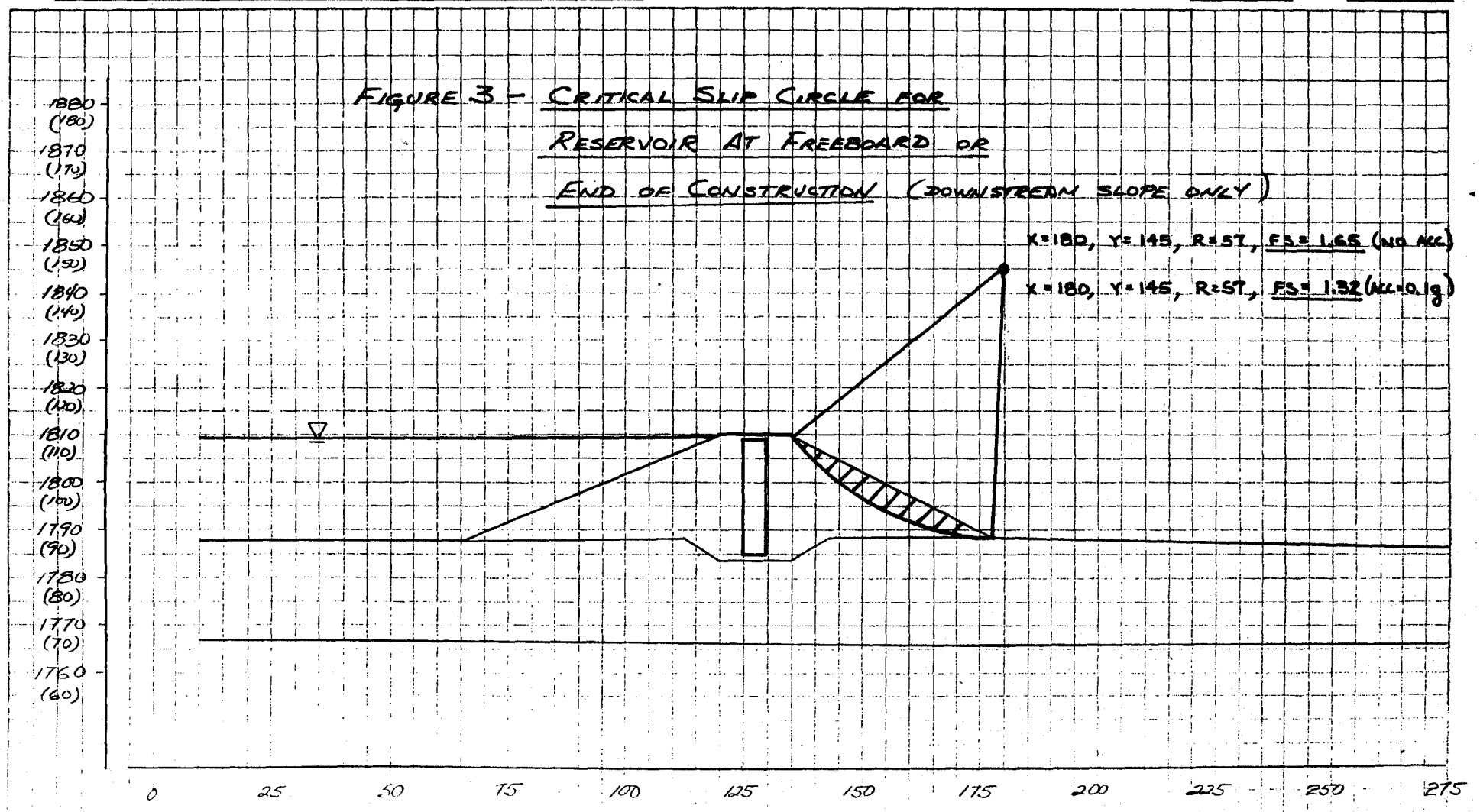
SUBJECT STABILITY ANALYSIS - RESERVOIR AT FREEBOARD

OFS NO. ____ DEPT. NO. ____

BY V BOLANO DATE 11/7/85

CHECKED BY ____ DATE ____

FIGURE 3 - CRITICAL SLIP CIRCLE FOR RESERVOIR AT FREEBOARD OR END OF CONSTRUCTION (DOWNSTREAM SLOPE ONLY)



5810/5-74

EBASCO SERVICES INCORPORATED

SHEET ____ OF ____

CLIENT USDA-SCS

PROJECT APACHE JUNCTION FRS

SUBJECT STABILITY ANALYSIS - RESERVOIR AT CREST OF EMERGENCY SPILLWAY

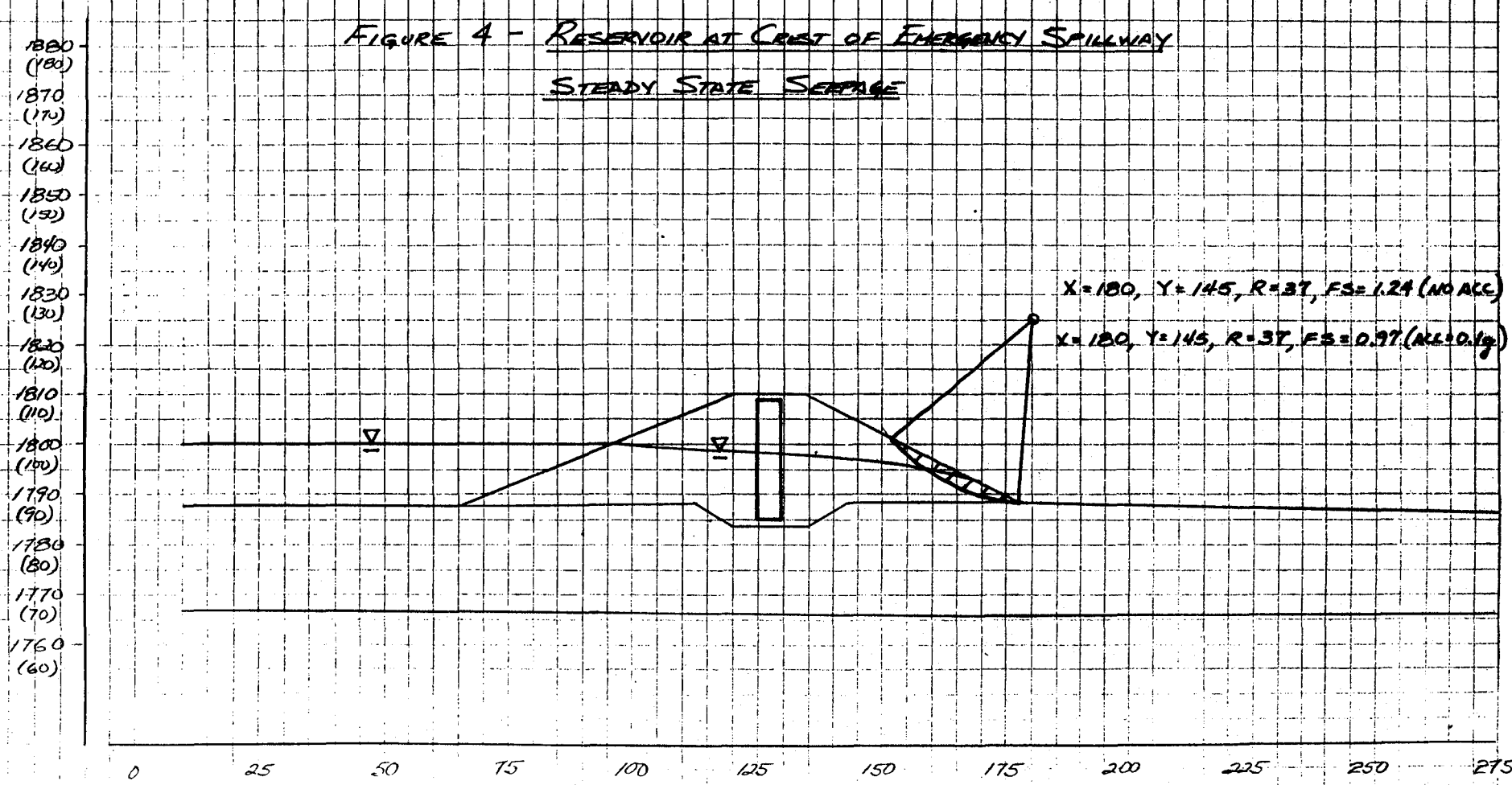
OFS NO. ____ DEPT. NO. ____

BY Y. BOLANO DATE 11/7/85

CHECKED BY ____ DATE ____

FIGURE 4 - RESERVOIR AT CREST OF EMERGENCY SPILLWAY

STEADY STATE SEEPAGE



581D/5-74

EBASCO SERVICES INCORPORATED

SHEET ____ OF ____

CLIENT USDA-SCS

PROJECT APACHE JUNCTION FRS

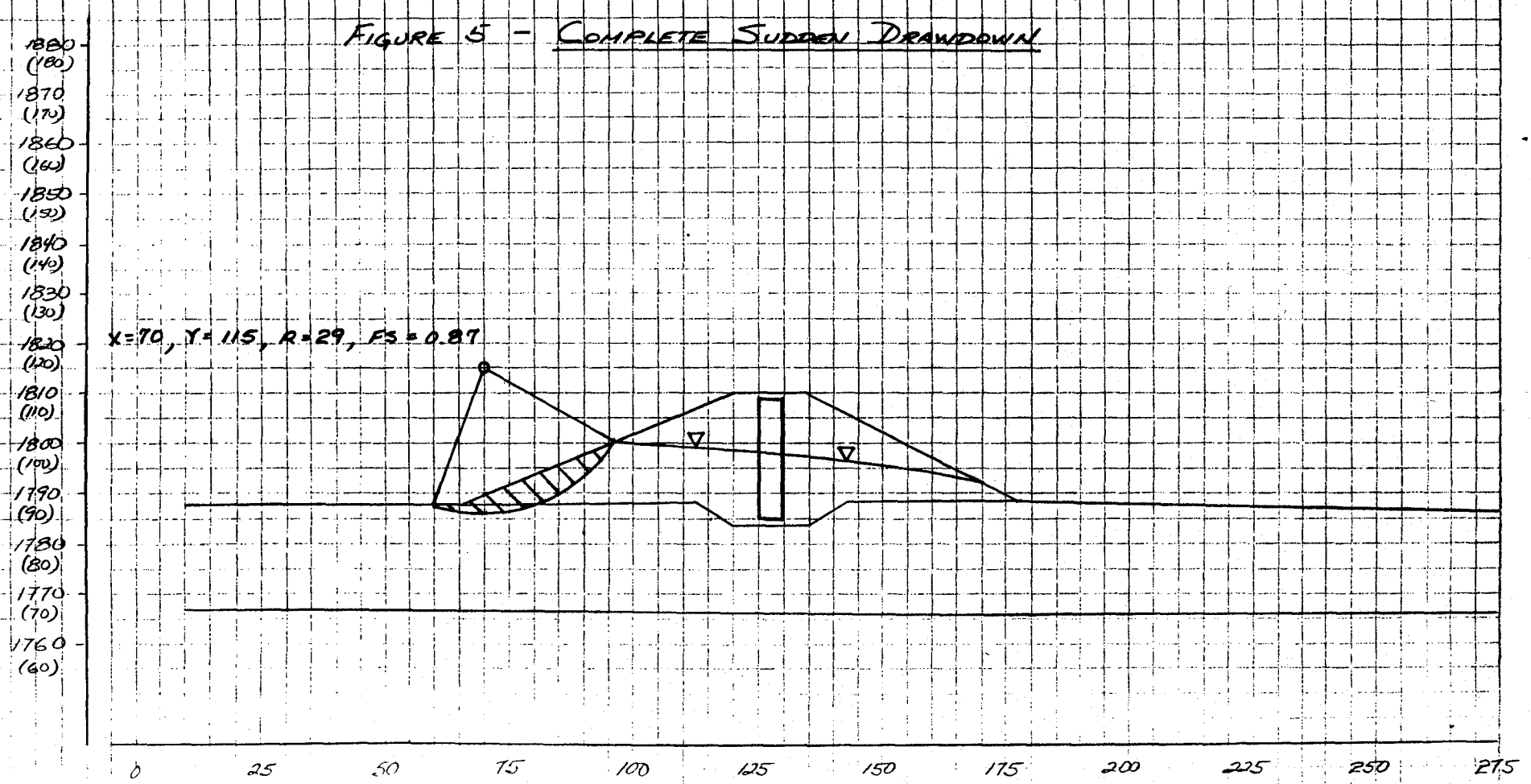
SUBJECT STABILITY ANALYSIS - COMPLETE SUDDEN DRAWDOWN

OFS NO. ____ DEPT. NO. ____

BY V BOLANO DATE 11/7/85

CHECKED BY ____ DATE ____

FIGURE 5 - COMPLETE SUDDEN DRAWDOWN



581D/5-74

APPENDIX A - CALCULATIONS

EBASCO SERVICES INCORPORATED

BY V BOLANO DATE 10/13/85 SHEET 1 OF 1

CHKD. BY _____ DATE _____ OFS NO. _____ DEPT. NO. _____

CLIENT USDA-SCS

PROJECT APACHE JCT FRS

SUBJECT DATA PROVIDED BY SCS

A) FIELD AND LABORATORY TEST DATA PROVIDED BY SCS

1 - Geological Investigation Report, Apache Jct FRS Apache Junction Floodway, Bulldog Floodway, July 1983

2 - Soil Test Results, Apache Jct FRS, Apache Junction Floodway, Bulldog Floodway, November 20, 1984

3 - Compaction and permeability test results, letter dated 6/24/85 from MNTC Soil Mechanics Lab to Arizona State Conservation Engineer

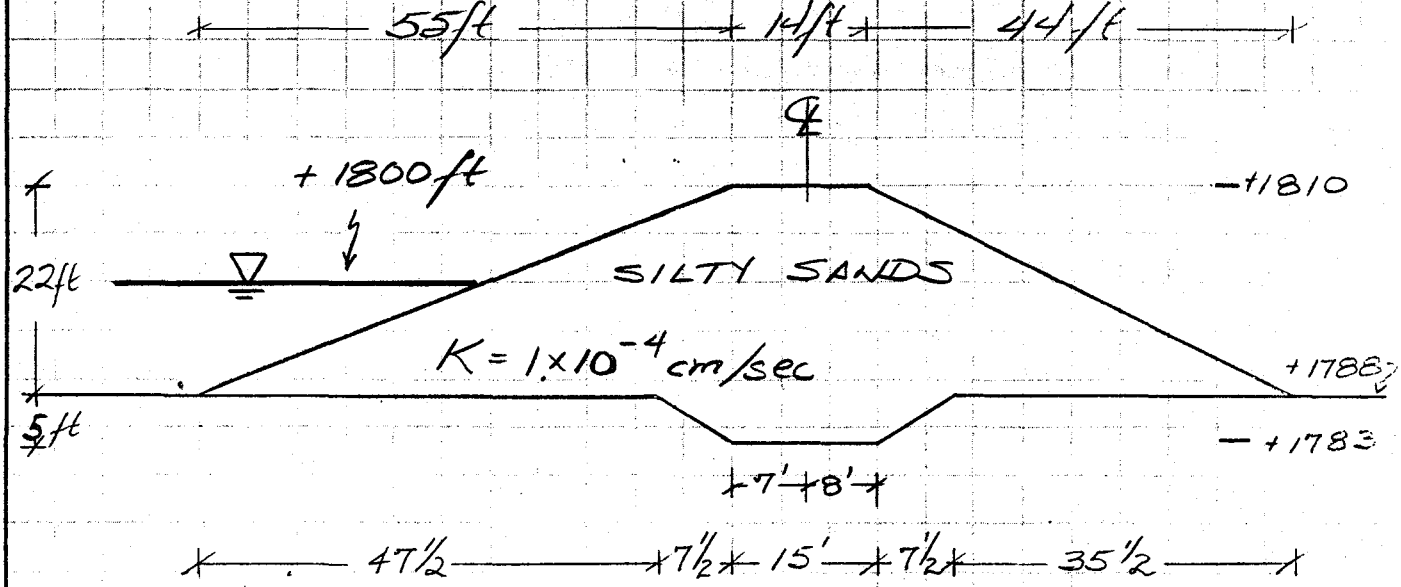
4 - Shear and consolidation test results, letter dated 2/5/85 from MNTC Soil Mechanics Lab to Arizona State Conservation Engineer

5 - Letter from Arizona Soil Conservation Service to Ebasco dated 9/24/85 containing a) permeability test results, b) log of test holes, c) grain size distribution, d) in-place moisture and density determination, e) compaction, and f) atterberg limits.

B) PUBLICATIONS

1) Technical Release No. 60, June 1976
2) Soil Mechanics Note No. 1 (Revision in progress)
3) Soil Mechanics Note No 3, May 1971
4) Soil Mechanics Note No 7, October 1979
5) Far West States Eng'g Design Standards, July 1979

DEVELOPMENT OF PHREATIC SURFACE THROUGH THE EMBANKMENT

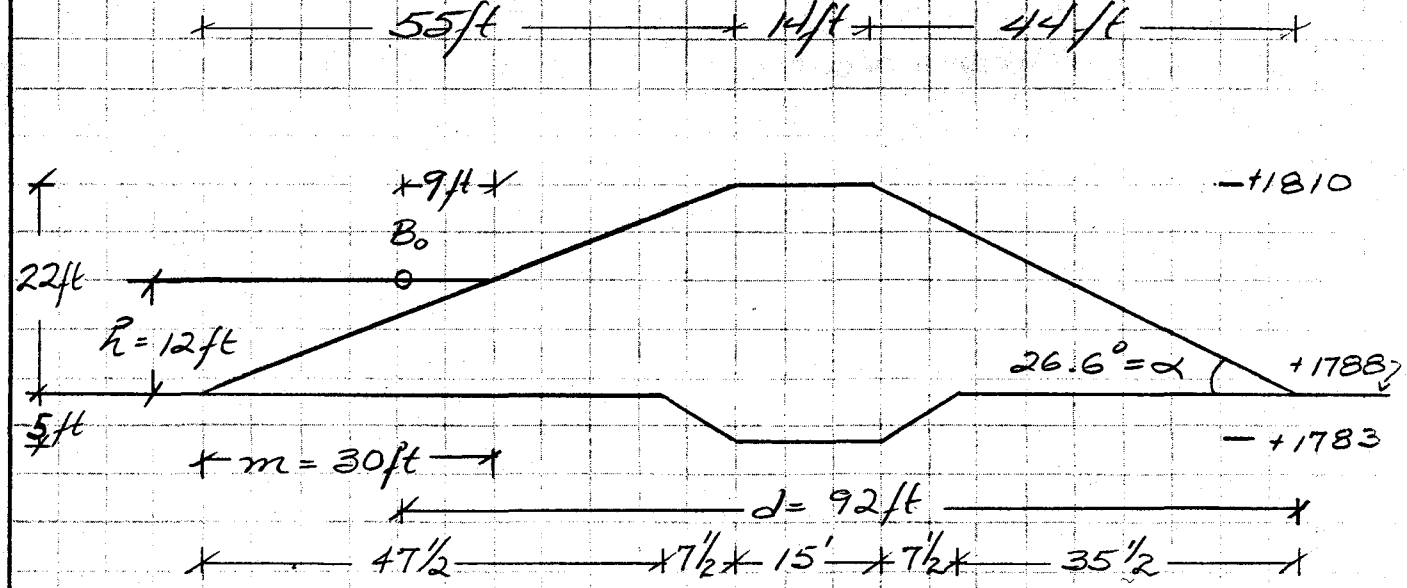


Emergency Spillway is @ EL + 1799.7 ∴ If riser becomes plugged and assuming a storm greater than the principal spillway hydrograph flood, then the highest long term water impoundment elevation will be @ +1799.7 ft. For simplicity the calculations will assume this elevation to be +1800 ft.

Calculate the time required for water to travel through the embankment and exit at toe

(Note: In case of a freeboard hydrograph storm the time for the reservoir impoundment to drop from above EL + 1809 ft to EL + 1800 is approximately 8 hrs (table X) only. Steady state seepage can not occur during this short time)

(SOURCE: Soil Mechanics Note No 7, The Mechanics of Seepage Analyses, October, 1979)



$$B_o = 0.3m = (0.3)(30ft) = 9ft$$

1) $y_o = (h^2 + d^2)^{1/2} - d$ where

$h = 12ft$ and $d = 92ft$

$$y_o = (12^2 + 92^2)^{1/2} - 92 = 0.8ft$$

2) $a = (h^2 + d^2)^{1/2} - (d^2 - h^2 \cot^2 \alpha)^{1/2}$ where

$h = 12ft$, $d = 92ft$ and $\alpha = 26.6°$

$$a = (12^2 + 92^2)^{1/2} - \left[92^2 - (12^2)(\cot 26.6^\circ)^2\right]^{1/2}$$

$$a = 92.8\,ft - 88.8\,ft = 4\,ft$$

3) $y = (2y_0 x + y_0^2)^{1/2}$ where $y_0 = 0.8\,ft$

x (ft)	y (ft)
10	4.1
20	5.7
30	7.0
40	8.0
50	9.0
60	9.8
70	10.6
80	11.3
90	12.0

4) $q = Ka \sin^2 \alpha$ where

$a = 4\,ft$, $K = 1 \times 10^{-4}$ cm/sec or $0.283\,ft/day$, and $\alpha = 26.6^\circ$

$$q = (0.283\,ft/day)(4\,ft)(\sin^2 26.6^\circ)$$

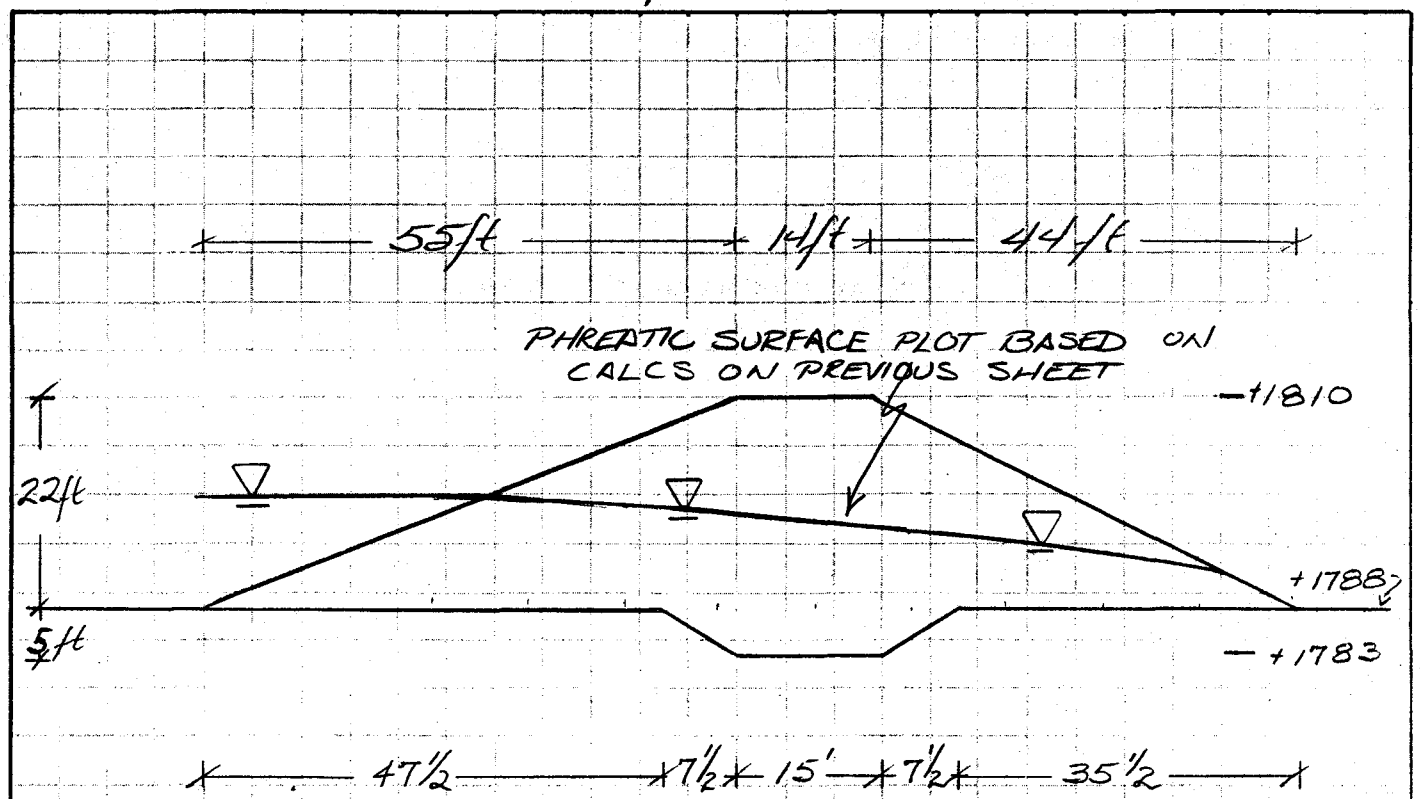
$$q = 0.23\,ft^3/day \text{ per ft length}$$

$$q = (0.23\,cfd/ft)(448.9\,gpm/cfs)\left(\frac{1\,min}{60\,sec}\right) = 1.72\,gpd/ft \text{ length}$$

CLIENT USDA

PROJECT APACHE JCT FRS

SUBJECT EMBANKMENT SEEPAGE ANALYSIS



Minimum flow path (scaled) = 76 ft

$$V = \frac{K \Delta h}{l n}$$ where

K = soil permeability (1×10^{-4} cm/sec)

Δh = 12 ft (EL 1800 - EL 1788)

l = 76 ft (scaled from figure above)

n = porosity (0.30 Groundwater by Freeze & Cherry. For silt n = 0.35 to 0.50, for sand n = 0.25 to 0.50)

$$V = \frac{K \Delta h}{l n} = \frac{(1 \times 10^{-4} \text{ cm/sec})(12 \text{ ft})(1 \text{ in}/2.54 \text{ cm})(8.64 \times 10^{4} \text{ sec/day})}{(76 \text{ ft})(0.30)}$$

$$V = 1.79 \text{ in/day}$$

$$t = \frac{l}{V} = \frac{76 \text{ ft}}{(1.79 \text{ in/day})(1 \text{ ft}/12 \text{ in})} = 509 \text{ days}$$

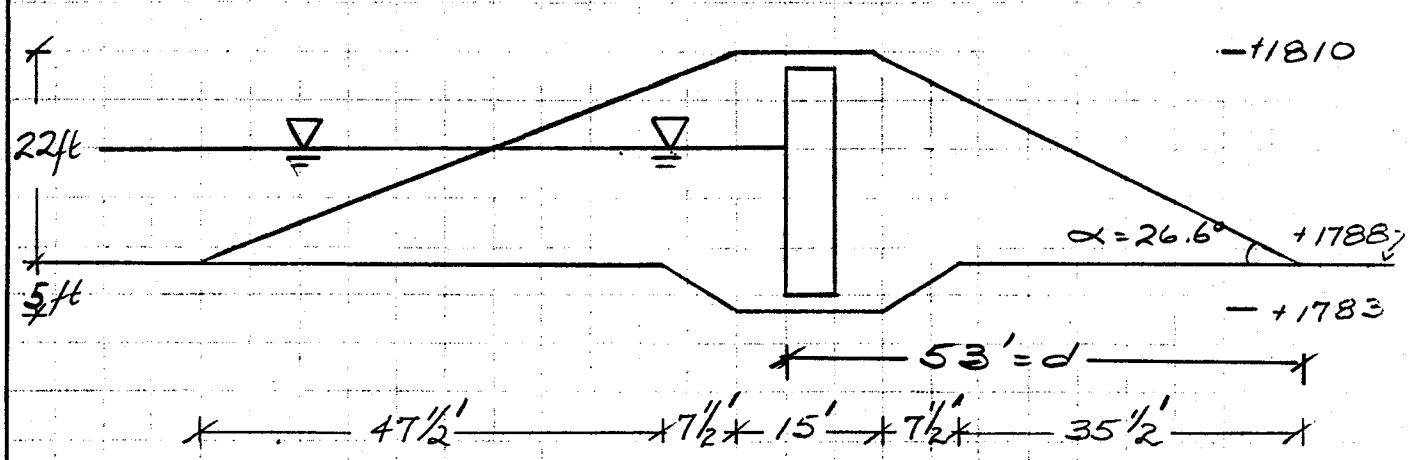
$$\left(\text{Note: } t = \frac{l^2 n}{K \Delta h} \right)$$

CONCLUSIONS

Based on the previous calculations it appears highly unlikely that a steady state seepage condition will be established even if one assumes that the riser is plugged. (It will take more than one year for water to travel through the embankment soils and exit at the downstream face near the toe of the dam).

Therefore it is not reasonable to assume a steady state seepage condition when analyzing the downstream slope stability of the dam. However, saturation of the upstream slope and the occurrance of a rapid drawdown will be analyzed for the stability of the upstream

ASSUME CRACKING EXTENDING TO TRANSITION



Assume $d = 53$ feet and $h = 12$ feet

1) $y_o = (h^2 + d^2)^{1/2} - d$

$y_o = [(12)^2 + (53)^2]^{1/2} - 53 = 54.34 - 53 = 1.34 ft$

2) $a = (h^2 + d^2)^{1/2} - (d^2 - h^2 \cot^2 \alpha)^{1/2}$ where $\alpha = 26.6°$

$a = [(12)^2 + (53)^2]^{1/2} - [(53)^2 - (12)^2 \cot^2(26.6°)]^{1/2}$

$a = 54.34' - 47.27' = 7.07 ft$

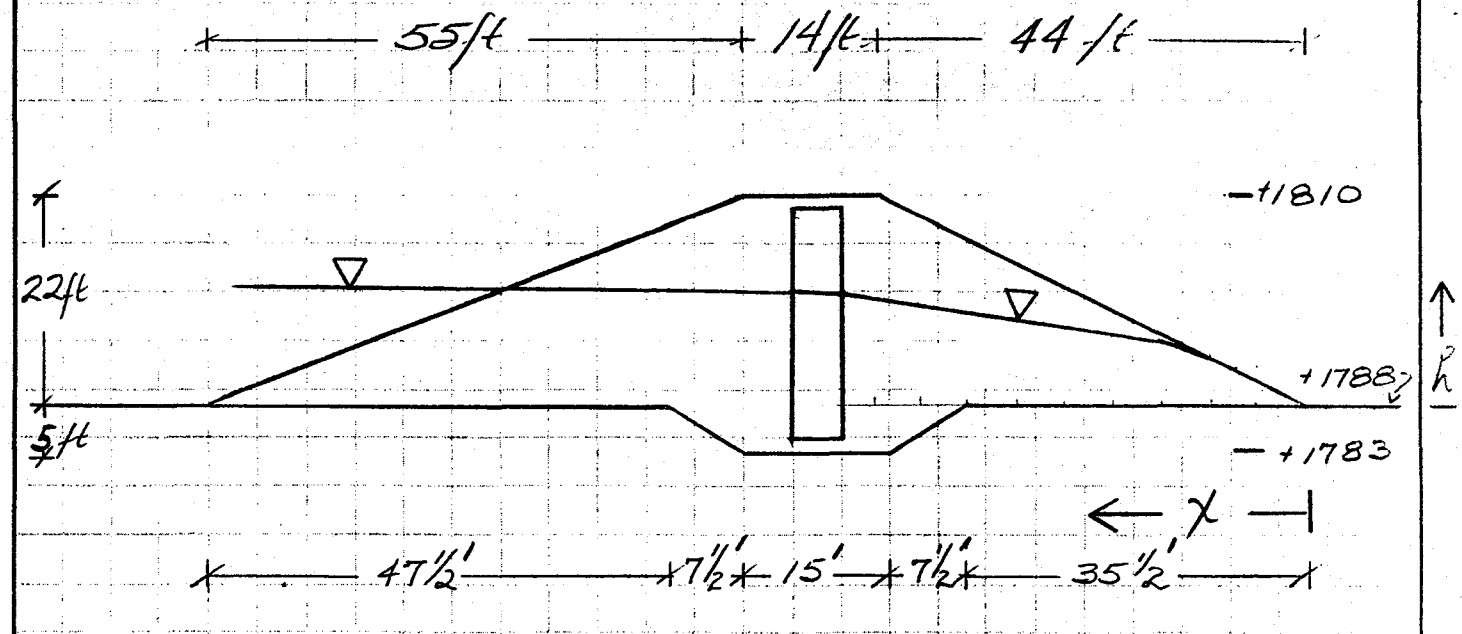
3) $q = K a \sin^2 \alpha$

$= (0.283 ft/day)(7.07 ft)(\sin^2 26.6°)$

$= 0.40 ft^3/day$ per ft length or 3gpd/ft length

4) $y = (2y_0 x + y_0^2)^{1/2}$ where $y_0 = 1.34$ ft

x (ft)	y (ft)
5	3.9
10	5.3
15	6.5
20	7.4
25	8.3
30	9.1
35	9.8
40	10.4
45	11.1
50	11.7
55	12.2



Minimum flow path = 45 ft (scaled)

$$t = \frac{l}{V} = \frac{l^2 n}{K \Delta h} = \frac{(45\,ft)^2(0.3)}{(0.283\,ft/day)(12\,ft)} = 179 \text{ days}$$

From SHEET 9

$N_f = 5$

$N_D = 22$

$K = 1 \times 10^{-3}$ cm/sec (field test results, Table V)

$h_{max} = 12$ ft (assumes riser plugged and water @ EL + 1800)

$$q = K h N_f / N_D = \left(1 \times 10^{-3} \text{ cm/sec}\right)\left(\frac{1 \text{ in}}{2.54 \text{ cm}}\right)\left(\frac{1 \text{ ft}}{12 \text{ in}}\right)(12 \text{ ft})(5)/(22)$$

$$q = 8.95 \times 10^{-5} \text{ cfs/ft}$$

$$q = \left(8.95 \times 10^{-5} \text{ cfs/ft}\right)\left(448.9 \text{ gpm/cfs}\right) = 4.02 \times 10^{-2} \text{ gpm/ft}$$

NOTE: This calculation represents a worst case analysis since:

1) The height of the silty sand layer above the caliche was assumed to be 22 ft. This large height was only found near STATION 47+50. Generally depth to caliche is considerably less (average of 6.5 feet)

2) It is assumed that the riser becomes plugged and a steady state seepage condition is established through the foundation soils with the reservoir water level at elevation + 1800

3) Permeability coefficient used in the analysis was the highest value obtained during the field testing program (Table V)

Calculate how long riser must be plugged for water to travel through the foundation soils to the toe of the dam

$$V = \frac{K \Delta h}{l\, n} \quad \text{and} \quad t = l/v$$

V = velocity
K = soil permeability (1×10^{-3} cm/sec or 3.28×10^{-5} ft/sec)
Δh = difference in water head ($\approx$ 12 ft)
l = travel distance ($\approx$ 113 ft)
n = porosity (0.3, Groundwater by Freeze & Sherry, p 37)
t = time

$$V = \frac{(3.28 \times 10^{-5}\ \text{ft/sec})\,(12\ \text{ft})}{(113\ \text{ft})\,(0.3)} = 1.16 \times 10^{-5}\ \text{ft/sec}$$

$$t = \frac{113\ \text{ft}}{1.16 \times 10^{-5}\ \text{ft/sec}} = 9.7 \times 10^{6}\ \text{sec or } 113\ \text{days}$$

Thus, using a permeability coefficient of 1×10^{-3} cm/sec and assuming that the riser is plugged, it will take more than three months for a steady state condition to develop.

Therefore assuming that a steady state condition does develop the calculated seepage flows (maximum) of 4.02×10^{-2} gpm/ft or 58 gpd/ft) are acceptable.

PIPING POTENTIAL (SM # 7, 10/79)

$$i_{dmax} = \frac{\Delta h}{\Delta l} \quad \text{where} \quad \Delta h = \frac{h}{N_D}$$

$h = 12$ ft
$N_D = 22$
$\Delta l \approx 2.5$ ft (approx measured)

$$i_{dmax} = \frac{h}{\Delta l \, N_D} = \frac{12 \text{ ft}}{(2.5 \text{ ft})(22)} = 0.22$$

$$i_{dcrit} = \frac{\gamma_{buoyant}}{\gamma_{water}} = \frac{\gamma_{wet} - \gamma_{water}}{\gamma_{water}} \text{ (for low PI soils) where}$$

$\gamma_{wet} = 94.9$ pcf (Table VI, STATION 70+00)

$$i_{dcrit} = \frac{94.9 \text{ pcf} - 62.4 \text{ pcf}}{62.4 \text{ pcf}} = 0.52$$

$$i_{d_{allow}} = \frac{i_{d\,crit}}{FS} = \frac{0.52}{2} = 0.26$$

$$i_{d_{allow}}\ (0.26) > i_{d_{req}}\ (0.22)$$

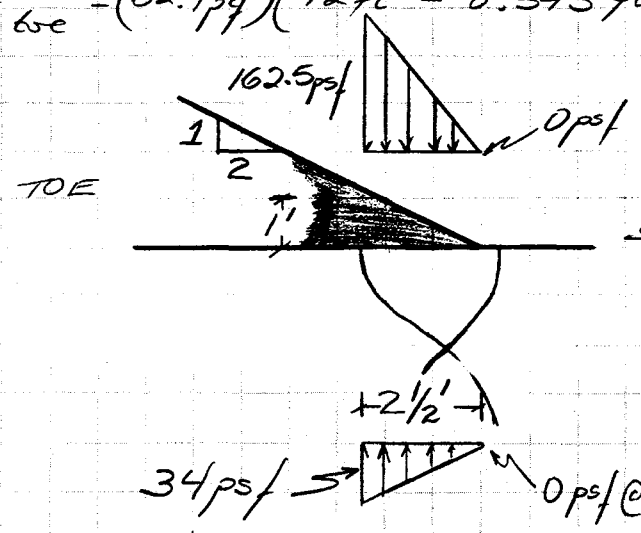
∴ Piping not a concern for foundation soils

SEEPAGE PRESSURE AT TOE

$$P_{s_{toe}} = \gamma_w \left(h - \Delta h\, N_{D_x} \right) \quad \text{where}$$

$\gamma_w = 62.4\,pcf$
$h = 12\,ft$
$\Delta h = 0.545\,ft \quad (h/N_D = 12ft/22)$
$N_{D_x} = 21$

$$P_{s\,toe} = (62.4\,pcf)(12\,ft - 0.545\,ft \times 21) = 34\,psf$$



Downward force due to soil weight = $\frac{2.5ft}{2} \times 130\,pcf \times 1\,ft = 162.5\,psf$

∴ TOE DRAIN IS NOT REQUIRED

$$FS = \frac{162.5\,psf}{34\,psf} = 4.8$$

CONCLUSIONS:

Given the low probability that a steady state seepage condition is established and the minimal flow quantities which may result, it is not viewed as necessary to construct a cut-off to caliche to prevent seepage through the foundation materials.

A five foot deep cutoff (15 ft wide at the bottom with 1.5 H to 1 V slopes) will be required in order to remove any buried washes which may exist near the ground surface.

EBASCO SERVICES INCORPORATED

BY V BOLAND DATE 10/13/85 SHEET 15 OF 66

CHKD. BY S G DATE 11-6-85 OFS NO. DEPT. NO.

CLIENT USDA-SCS

PROJECT APACHE JCT FRS

SUBJECT STABILITY ANALYSIS

Embankment Slope Stability

Minimum Factors of Safety as per TR 60 requirements

LOADING CONDITION	FACTOR OF SAFETY MINIMUM	
	WITHOUT SEISMIC FORCES	WITH SEISMIC FORCES
1) End of Construction	1.4 (3)	NOT APPLICABLE
2) Steady Seepage, Reservoir Elevation @ Highest Principal Spillway Crest	1.5 (3)	1.1
3) Sudden Drawdown	1.3 (3)	NOT APPLICABLE

NOTES: 1) The Apache Jct FRS is classified as a Class C structure

2) The elevation of the highest principal spillway crest is +1793.5 ft (aprox 6 feet above ground surface)

3) If an infinite slope analysis is conducted, the minimum safety factor shall be 1.1

Infinite slope analysis

1) End of construction:

Downstream slope (critical) 1V TO 2H

$\bar{\phi} = 37°$ (Table VII)

No seepage pressures, No seismic

SF = 1.51 (see following sheets, SH 18)

$$\left(SF = \frac{\tan \bar{\phi}}{\tan \text{slope angle}} = \frac{\tan 37°}{\tan 26.6°} = 1.51 \right)$$

2) Steady seepage (parallel to outer slope)

$\bar{\phi} = 37°$, $\gamma_{sat} = 130$ pcf

Downstream slope (critical) 1V TO 2H

Non-seismic SF = 0.78

Seismic SF = 0.62 (acc = 0.10g)

For a SF ≥ 1.1 the following slopes are required

1) Non-seismic 3H TO 1V

2) Seismic 4.5H TO 1V

3) Sudden Drawdown (seepage assumed horizontal to outer slope)

Upstream Slope 1V to 2.5H

$\bar{\phi} = 37^\circ$, $\gamma_{sat} = 130$ pcf

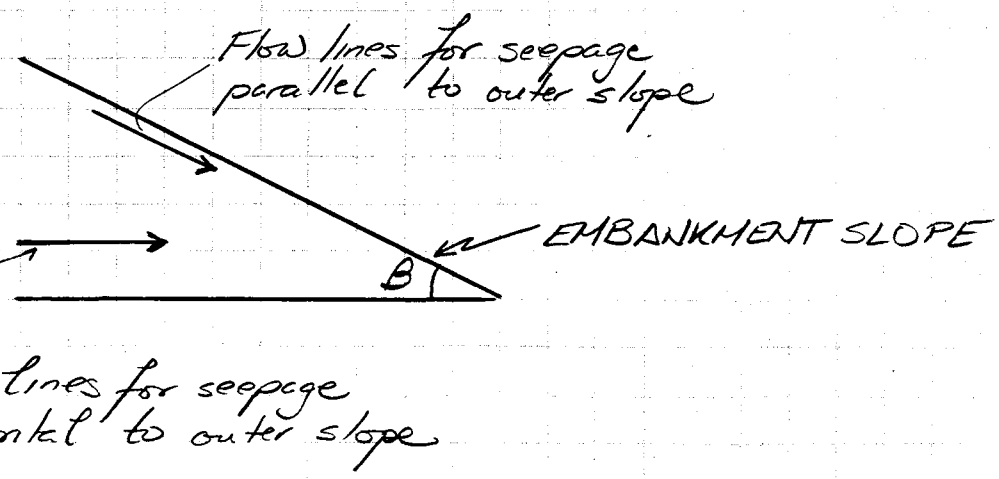
SF = 0.83

For a SF $\geq$ 1.1 it would require a slope of 3.5H to 1V

If seepage was assumed to be parallel to outer slope then the SF for the upstream slope would be

SF = 0.98 and a slope of 3H to 1V would be required to yield a SF > 1.1

NOTE:



Tabular Computations

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

INFINITE SLOPE ANALYSIS

NO SEEPAGE PRESSURES

FACTOR OF SAFETY WITHOUT SEISMIC

SLOPE (H:1)	28°	29°	30°	31°	32°	33°	34°	35°	36°	37°	38°	39°
1.5	0.80	0.83	0.87	0.90	0.94	0.97	1.01	1.05	1.09	1.13	1.17	1.21
2.0	1.06	1.11	1.15	1.20	1.25	1.30	1.35	1.40	1.45	1.51	1.56	1.62
2.5	1.33	1.39	1.44	1.50	1.56	1.62	1.69	1.75	1.82	1.88	1.95	2.02
3.0	1.60	1.66	1.73	1.80	1.87	1.95	2.02	2.10	2.18	2.26	2.34	2.43
3.5	1.86	1.94	2.02	2.10	2.19	2.27	2.36	2.45	2.54	2.64	2.73	2.83
4.0	2.13	2.22	2.31	2.40	2.50	2.60	2.70	2.80	2.91	3.01	3.13	3.24
4.5	2.39	2.49	2.60	2.70	2.81	2.92	3.04	3.15	3.27	3.39	3.52	3.64

Ø EFFECTIVE STRESS

DOWNSTREAM CRITICAL

END OF CONSTRUCTION

NO SEISMIC

SH 18

Tabular Computations

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

INFINITE SLOPE ANALYSIS

PARALLEL SEEPAGE

γ_{SAT} = 130 FACTOR OF SAFETY WITHOUT SEISMIC

SLOPE (H:1)	28°	29°	30°	31°	32°	33°	34°	35°	36°	37°	38°	39°
	Φ EFFECTIVE STRESS											
1.5	0.41	0.43	0.45	0.47	0.49	0.51	0.53	0.55	0.57	0.59	0.61	0.63
2.0	0.55	0.58	0.60	0.62	0.65	0.68	0.70	0.73	0.76	0.78	0.81	0.84
2.5	0.69	0.72	0.75	0.78	0.81	0.84	0.88	0.91	0.94	0.98	1.02	1.05
3.0	0.83	0.86	0.90	0.94	0.97	1.01	1.05	1.09	1.13	1.18	1.22	1.26
3.5	0.97	1.01	1.05	1.09	1.14	1.18	1.23	1.27	1.32	1.37	1.42	1.47
4.0	1.11	1.15	1.20	1.25	1.30	1.35	1.40	1.46	1.51	1.57	1.63	1.68
4.5	1.24	1.30	1.35	1.41	1.46	1.52	1.58	1.64	1.70	1.76	1.83	1.89

NO SEISMIC

DOWNSTREAM CRITICAL

STEADY SEEPAGE

SH 19

Tabular Computations

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

INFINITE SLOPE ANALYSIS
PARALLEL SEEPAGE

γ_{SAT} = 130 FACTOR OF SAFETY WITH SEISMIC ψ=0.10

SLOPE (H:1)	28°	29°	30°	31°	32°	33°	34°	35°	36°	37°	38°	39°
1.5	0.34	0.35	0.37	0.38	0.40	0.41	0.43	0.44	0.46	0.48	0.49	0.51
2.0	0.44	0.46	0.48	0.49	0.51	0.53	0.56	0.58	0.60	0.62	0.64	0.67
2.5	0.53	0.55	0.58	0.60	0.62	0.65	0.67	0.70	0.73	0.75	0.78	0.81
3.0	0.62	0.64	0.67	0.70	0.72	0.75	0.78	0.81	0.84	0.87	0.91	0.94
3.5	0.70	0.73	0.76	0.79	0.82	0.85	0.88	0.92	0.95	0.99	1.02	1.06
4.0	0.77	0.80	0.84	0.87	0.91	0.94	0.98	1.01	1.05	1.09	1.13	1.17
4.5	0.84	0.87	0.91	0.95	0.99	1.02	1.06	1.10	1.15	1.19	1.23	1.28

Φ EFFECTIVE STRESS

STEADY SEEPAGE
DOWNSTREAM CRITICAL
SEISMIC (a = 0.10g)

SH 20

Tabular Computations

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

INFINITE SLOPE ANALYSIS
HORIZONTAL SEEPAGE

$\gamma_{SAT} = 130$ FACTOR OF SAFETY WITHOUT SEISMIC

SLOPE (H:1)	28°	29°	30°	31°	32°	33°	34°	35°	36°	37°	38°	39°
1.5	0.24	0.25	0.27	0.28	0.29	0.30	0.31	0.32	0.33	0.35	0.36	0.37
2.0	0.43	0.44	0.46	0.48	0.50	0.52	0.54	0.56	0.58	0.60	0.63	0.65
2.5	0.59	0.61	0.64	0.67	0.69	0.72	0.75	0.78	0.81	0.83	0.87	0.90
3.0	0.74	0.78	0.81	0.84	0.87	0.91	0.94	0.98	1.02	1.05	1.09	1.13
3.5	0.89	0.93	0.97	1.01	1.05	1.09	1.14	1.18	1.22	1.27	1.31	1.36
4.0	1.04	1.09	1.13	1.18	1.22	1.27	1.32	1.37	1.42	1.48	1.53	1.59
4.5	1.19	1.24	1.29	1.34	1.40	1.45	1.51	1.56	1.62	1.68	1.74	1.81

Φ EFFECTIVE STRESS

NO SEISMIC

UPSTREAM SLOPE

SUDDEN DRAWDOWN

SH21

Tabular Computations

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

INFINITE SLOPE ANALYSIS
PARALLEL SEEPAGE

$\gamma_{SAT} = 130$ FACTOR OF SAFETY WITHOUT SEISMIC

SLOPE (H:1)	28°	29°	30°	31°	32°	33°	34°	35°	36°	37°	38°	39°
	ϕ EFFECTIVE STRESS											
1.5	0.41	0.43	0.45	0.47	0.49	0.51	0.53	0.55	0.57	0.59	0.61	0.63
2.0	0.55	0.58	0.60	0.62	0.65	0.68	0.70	0.73	0.76	0.78	0.81	0.84
2.5	0.69	0.72	0.75	0.78	0.81	0.84	0.88	0.91	0.94	0.98	1.02	1.05
3.0	0.83	0.86	0.90	0.94	0.97	1.01	1.05	1.09	1.13	1.18	1.22	1.26
3.5	0.97	1.01	1.05	1.09	1.14	1.18	1.23	1.27	1.32	1.37	1.42	1.47
4.0	1.11	1.15	1.20	1.25	1.30	1.35	1.40	1.46	1.51	1.57	1.63	1.68
4.5	1.24	1.30	1.35	1.41	1.46	1.52	1.58	1.64	1.70	1.76	1.83	1.89

NO SEISMIC

UPSTREAM SLOPE

SUDDEN DRAWDOWN

SH22

SUMMARY OF RESULTS

LOADING CONDITION	MINIMUM FACTOR OF SAFETY	
	WITHOUT SEISMIC FORCES	WITH SEISMIC FORCES
1) Without seismic forces (DOWNSTREAM SLOPE)	1.51	NOT APPLICABLE
2) Steady seepage, reservoir elevation @ highest principal spillway crest (DOWNSTREAM SLOPE)	0.78 (For FS > 1.1; 3H to 1V)	0.62 (For FS > 1.1; 4.5 H to 1V)
3) Sudden Drawdown (UPSTREAM SLOPE)	0.83 (For FS > 1.1; 3.5H to 1V)	NOT APPLICABLE

CONCLUSIONS

The flatter slopes (3.0 H to 1V downstream without seismic forces or 4.5 H to 1V with seismic forces and 3.5 H to 1V upstream) required to meet the minimum factor of safety (1.1) in the infinite slope analysis are not reasonable since:

1) It is highly unlikely as shown on Sheets 5 & 8 that a steady state condition will ever develop and reach the downstream slope. In addition, if one did develop, only the lower four feet (Sh 3 & 4, embankment seepage analysis) will be subject to localized sloughing).

2) If it is assumed that the riser is plugged for a significant period of time thereby leading to saturation of the upstream slope it could be possible that a "sudden drawdown" condition may occur when the riser is unplugged and the reservoir is emptied. However, once again, this would only lead as a maximum, to localized sloughing on the upstream slope.

Given the high unlikelyhood that either steady state seepage will develop or a sudden drawdown will occur, it is recommended that the upstream slope (2.5H to 1V) and downstream slope (2.0H to 1V) remain unchanged. If localized sloughing occurs it will be taken care by maintenance during operations.

APPENDIX V

Infinite Slope Analysis for Cohesionless Soils

1. Infinite Slope Computations. For cohesionless materials ($c = 0$), equations applicable to an infinite slope may be used to obtain an estimate of the stability of the slope of an embankment where seepage is involved. It is assumed that the seepage flow is uniform throughout the soil mass.

2. General Case. The safety factor for the general case where seepage flow is neither parallel nor horizontal to the outer slope is

$$F.S. = \frac{\gamma' - \left(\gamma_w \frac{\tan \alpha}{\cot \beta}\right)}{\gamma_{sat}} \cot \beta \tan \phi$$

where

γ' = submerged unit weight of soil

γ_w = unit weight of water

α = angle between seepage flow line and embankment slope

β = angle of inclination of embankment slope with horizontal ($\cot \beta = b$)

γ_{sat} = saturated unit weight of soil.

ϕ = angle of internal friction

3. Seepage Parallel to Slope. For seepage flow parallel to and coincident with the embankment slope ($\alpha = 0$), the safety factor becomes

$$F.S. = \frac{\gamma'}{\gamma_{sat}} \cot \beta \tan \phi = \frac{\gamma'}{\gamma_{sat}} b \tan \phi$$

where

$b = \cot \beta$

4. Horizontal Seepage. Where seepage flow is horizontal ($\alpha = \beta$), the factor of safety is

$$F.S. = \frac{\gamma' - \frac{\gamma_w}{\cot^2 \beta}}{\gamma_{sat}} (\cot \beta \tan \phi) = \frac{b^2 \gamma' - \gamma_w}{b \gamma_{sat}} (\tan \phi)$$

5. No Seepage. Where no seepage forces exist, i.e. for a dry slope, the factor of safety is

$$F.S. = \frac{\tan \phi}{\tan \beta} = b \tan \phi$$

6. Earthquake. The effects of an earthquake loading can be applied to all of the previous equations for factor of safety by replacing b with the term b' where

$$b' = \frac{b - \psi}{\underbrace{1 + b\psi}_{1 + b \cdot \psi}}$$

ψ = seismic coefficient (see fig. 6, main text)

7. Example. An example of the influence of the direction of seepage flow on the factor of safety is illustrated in the following tabulation.

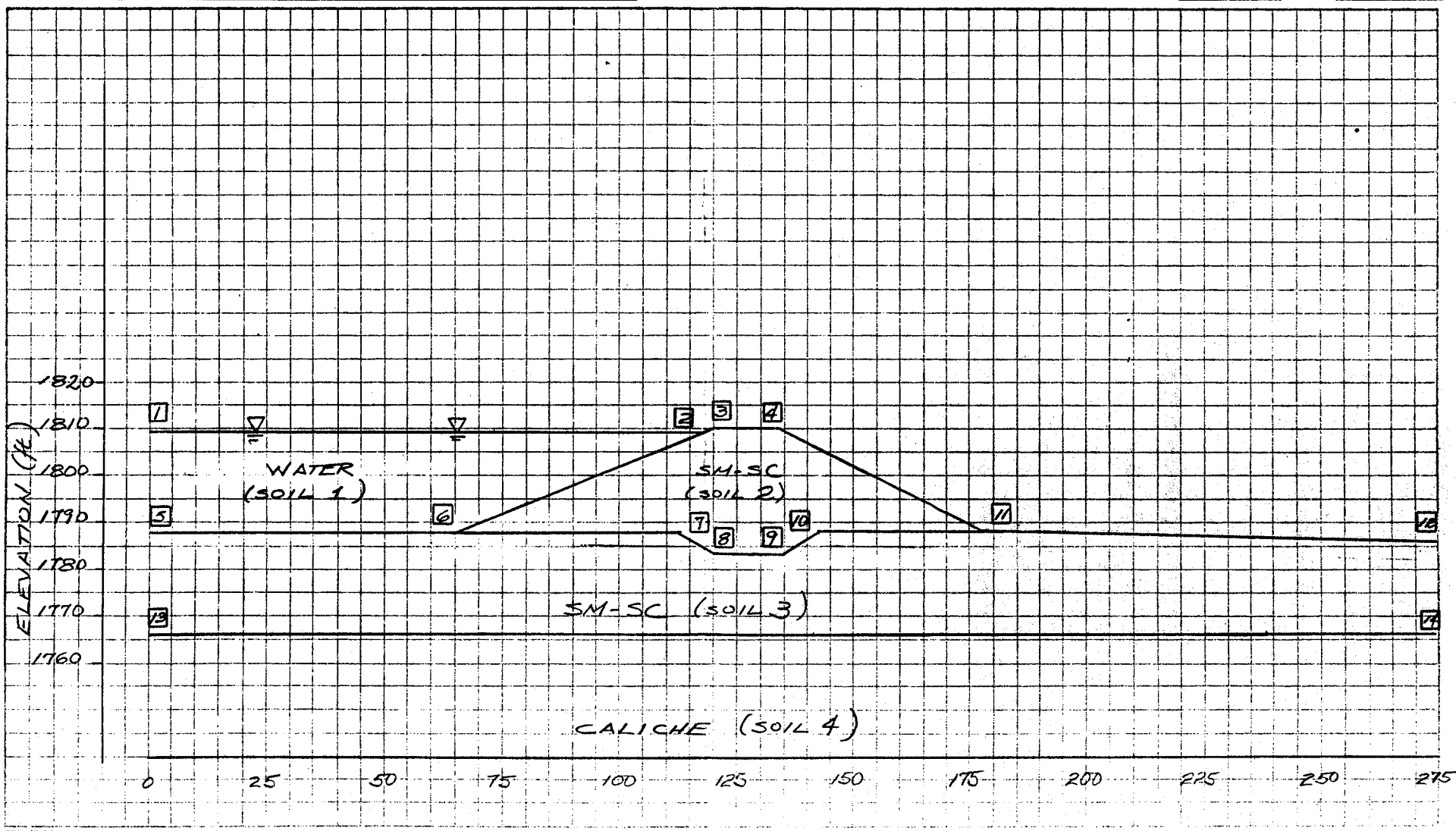
Assumed design values	Factor of safety for: Seepage parallel to outer slope	Horizontal seepage	No seepage
$b = 3.5$			
$\gamma_{sat} = 2\gamma_w$	1.23†	1.13†	2.45†
$\tan \phi = 0.7$			
$\psi = 0.1$	0.88††	0.74††	1.76††
$b' = 2.52$			

† Without earthquake loading.
†† With earthquake loading.

EBASCO SERVICES INCORPORATED

CLIENT USDA-SCS
PROJECT APACHE JUNCTION FRS
SUBJECT STABILITY ANALYSIS MODEL - RESERVOIR AT FREEBOARD

OFS NO. _____ DEPT. NO. _____
BY V BOLANO DATE 9/4/85
CHECKED BY S G DATE 11-17-85



581D/5-74

EBASCO SERVICES INCORPORATED

BY V M BOLAND DATE 9/4/85 SHEET 28 OF 66

CHKD. BY SG DATE 11-17-85 OFS NO. DEPT. NO.

CLIENT USDA-SCS

PROJECT APACHE JCT FRS

SUBJECT STABILITY ANALYSIS MODEL - RESERVOIR AT FREEBOARD

POINT	X-COORD	Y-COORD	ELEVATION (ft)
1	0.0	109.4	1809.4
2	118.5	109.4	1809.4
3	120.0	110.0	1810.0
4	134.0	110.0	1810.0
5	0.0	88.0	1788.0
6	65.0	88.0	1788.0
7	112.5	88.0	1788.0
8	120.0	83.0	1783.0
9	135.0	83.0	1783.0
10	142.5	88.0	1788.0
11	178.0	88.0	1788.0
12	275.0	86.0	1786.0
13	0.0	66.0	1766.0
14	275.0	66.0	1766.0

LINE	POINTS JOINED
1	1, 2, 3, 4, 11, 12
2	5, 6, 2, 3, 4, 11, 12
3	5, 6, 7, 8, 9, 10, 11, 12
4	13, 14

CLIENT USDA - SCS

PROJECT APACHE JUNCTION FRS

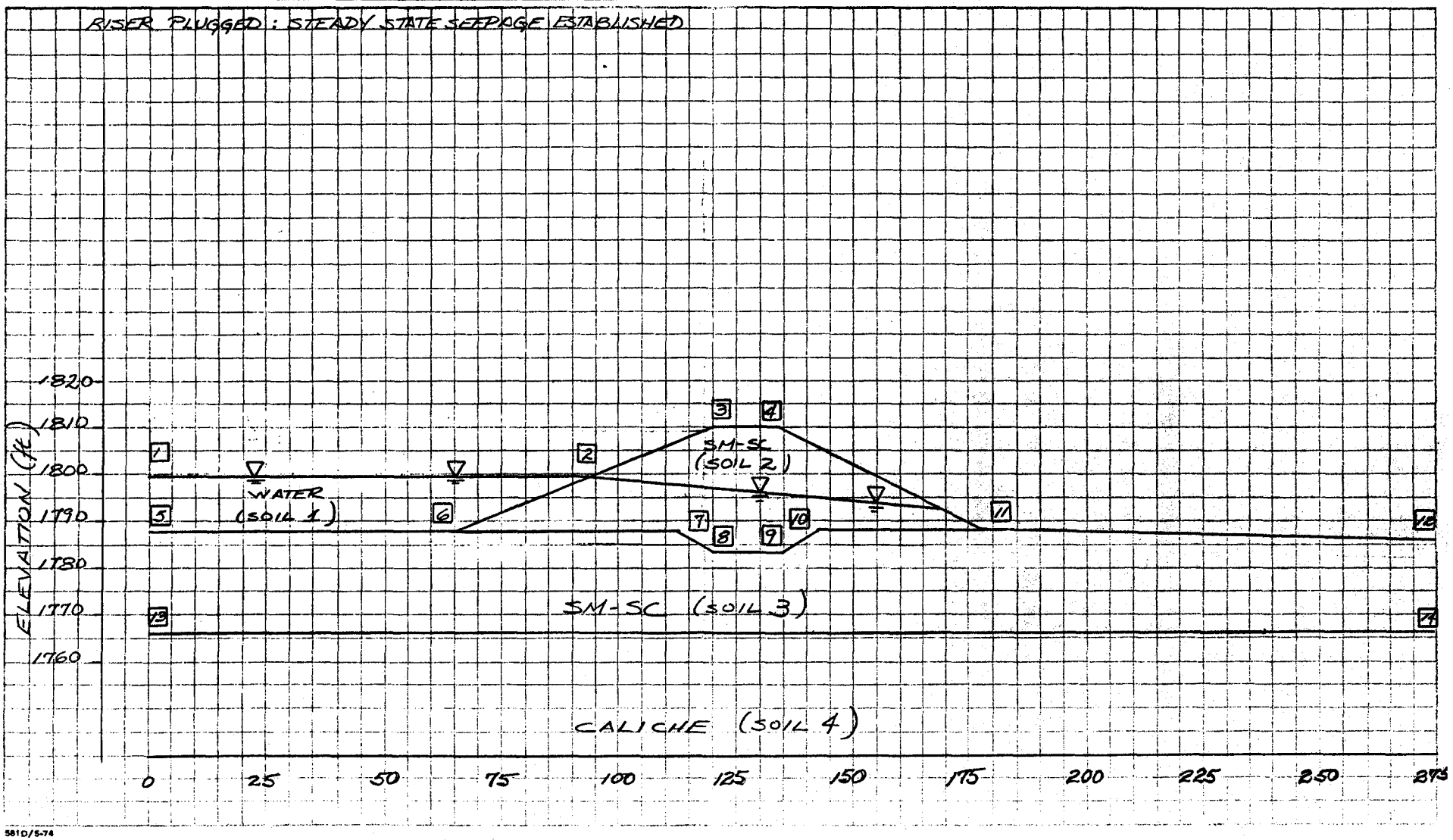
SUBJECT STABILITY ANALYSIS MODEL - RESERVOIR AT CREST OF EMERGENCY SPILLWAY

OFS NO. _____ DEPT. NO. _____

BY V BOLANO DATE 9/4/85

CHECKED BY S G DATE 11-17-85

RISER PLUGGED : STEADY STATE SEEPAGE ESTABLISHED



581D/5-74

EBASCO SERVICES INCORPORATED

BY V M BOLANO DATE 9/4/85 SHEET 30 OF 66

CHKD. BY S G DATE 11-11-85 OFS NO. DEPT. NO.

CLIENT USDA-SCS

PROJECT APACHE JUNCTION FRS

SUBJECT STABILITY ANALYSIS-RESERVOIR AT CREST OF EMERGENCY SPILLWAY

RISER PLUGGED: STEADY STATE SEEPAGE ESTABLISHED

POINT	X-COORD	Y-COORD	ELEVATION (ft)
1	0.0	99.7	1799.7
2	94.2	99.7	1799.7
3	120.0	110.0	1810.0
4	134.0	110.0	1810.0
5	0.0	88.0	1788.0
6	65.0	88.0	1788.0
7	112.5	88.0	1788.0
8	120.0	83.0	1783.0
9	135.0	83.0	1783.0
10	142.5	88.0	1788.0
11	178.0	88.0	1788.0
12	275.0	86.0	1786.0
13	0.0	66.0	1766.0
14	275.0	66.0	1766.0

LINE	POINTS JOINED
1	1, 2, 3, 4, 11, 12
2	5, 6, 2, 3, 4, 11, 12
3	5, 6, 7, 8, 9, 10, 11, 12
4	13, 14

EBASCO SERVICES INCORPORATED

CLIENT USDA-SCS

PROJECT APACHE JUNCTION FRS

SUBJECT STABILITY ANALYSIS MODEL - SUDDEN DRAWDOWN CONDITION

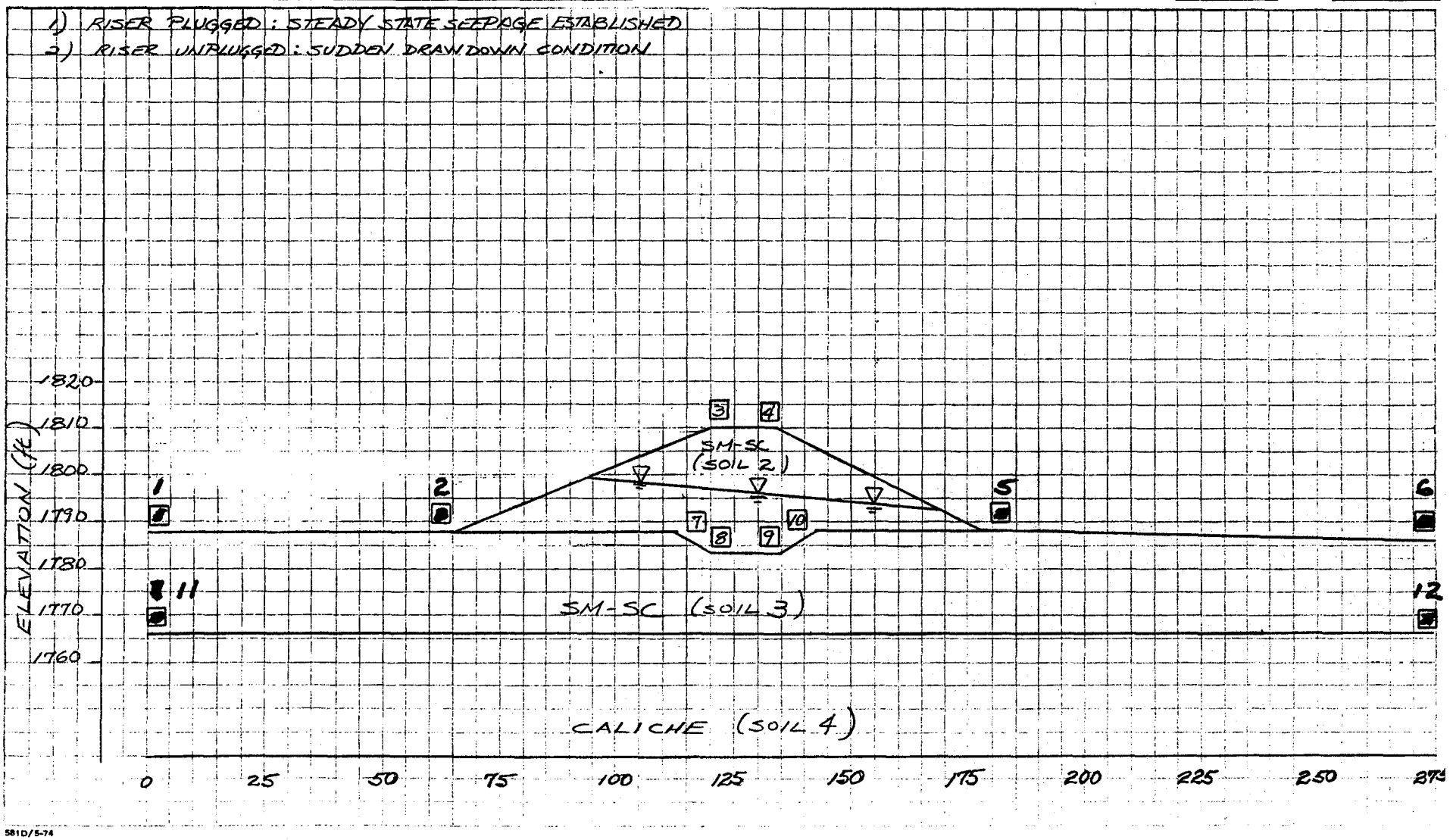
OFS NO. _____ DEPT. NO. _____

BY V BOLANO DATE 9/4/85

CHECKED BY SG DATE 11-17-85

1) RISER PLUGGED: STEADY STATE SEEPAGE ESTABLISHED

2) RISER UNPLUGGED: SUDDEN DRAWDOWN CONDITION



581D/5-74

EBASCO SERVICES INCORPORATED

BY V M BOLAND DATE 9/4/85 SHEET 32 OF 66

CHKD. BY SG DATE 11-17-85 OFS NO. DEPT. NO.

CLIENT USDA - SCS

PROJECT APACHE JCT FRS

SUBJECT STABILITY ANALYSIS MODEL - SUDDEN DRAWDOWN CONDITION

RISER PLUGGED: STEADY STATE SEEPAGE ESTABLISHED
RISER UNPLUGGED: SUDDEN DRAWDOWN CONDITION

POINT	X-COORD	Y-COORD	ELEVATION (ft)
3	120.0	110.0	1810.0
4	134.0	110.0	1810.0
1	10.0	88.0	1788.0
2	65.0	88.0	1788.0
7	112.5	88.0	1788.0
8	120.0	83.0	1783.0
9	135.0	83.0	1783.0
10	142.5	88.0	1788.0
5	178.0	88.0	1788.0
6	275.0	86.0	1786.0
11	10.0	66.0	1766.0
12	275.0	66.0	1766 0

LINE	POINTS JOINED
1	1, 2, 3, 4, 5, 6
2	1, 2, 7, 8, 9, 10, 5, 6
3	11, 12

SOIL PROPERTIES FOR ALL CASES

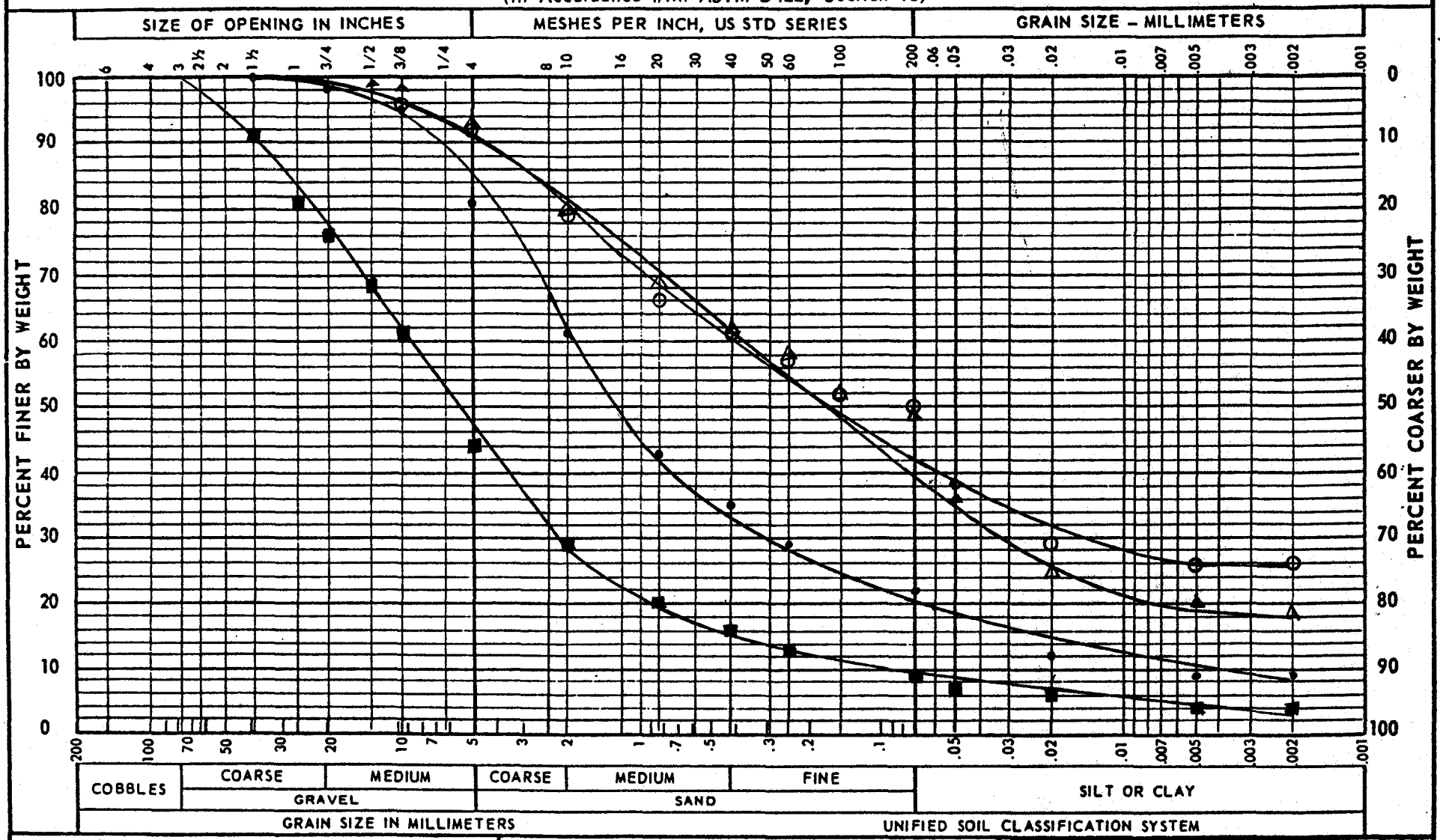
"SOIL #"	DESCRIPTION	UNIT WEIGHT (pcf)	COHESION (psf)	FRICTION ANGLE (°)
1	WATER	62.4	—	—
2	COMPACTED SM-SC	130.0	0	37°
3	FOUNDATION SM-SC	110.0	0	30°
4	CALICHE ①	—	—	—

NOTE: 1) SLIP CIRCLE FAILURE IS NOT PERMITTED TO PASS THROUGH THE CALICHE MATERIAL FOUND BELOW ELEVATION + 1766 FT.

FORM #14

MECHANICAL ANALYSIS GRAPH OF GRANULAR MATERIALS

(In Accordance With ASTM D422, Section 16)



TEST NO.	
SAMPLE NO.	
SAMPLE LOCATION	
SAMPLE ELEVATION	

SAMPLE DESCRIPTION

- ● LAB # 2928
- ○ LAB # 2932
- △ LAB # 2941
- ■ LAB # 2940

EBASCO SERVICES INCORPORATED

APACHE JCT FRS
PROJECT

DRAWN BY	CHECKED BY	DATE
V BOLANO	SG	27-8-85

GRADATION RANGE OF 'BORROW AREA SOILS

SH 34

FORM #14

EBASCO SERVICES INCORPORATED

BY VBOLANO DATE 10/14/85 SHEET 35 OF 66

CHKD. BY SG DATE 11-8-85 OFS NO. DEPT. NO.

CLIENT USDA-SCS

PROJECT APACHE JCT FRS

SUBJECT GRADATION BORROW AREA SOILS

SIEVE SIZE	% PASSING (FINE MATERIAL)	% PASSING REGRADED	% PASSING (COARSE MATERIAL)	% PASSING REGRADED
3"	100		100	
1½"	100		91	
1"	100		81	
3/4"	99		76	
½"	99		68	
3/8"	98		61	
#4	92	100	44	100
#10	79	86	29	66
#20	66	72	20	45
#40	61	66	16	36
#60	57	62	13	30
#140	52	57	10	23
#200	50	54	9	20
0.06	38	41	7	16
0.02	29	32	6	14
0.005	26	28	4	9
0.002	26	28	4	9

"FINE MATERIAL" REPRESENTED BY SAMPLE #2932

"COARSE MATERIAL" REPRESENTED BY SAMPLE #2940

CORRECTION FACTOR FOR FINE MATERIAL $= \frac{100}{92} = 1.087$

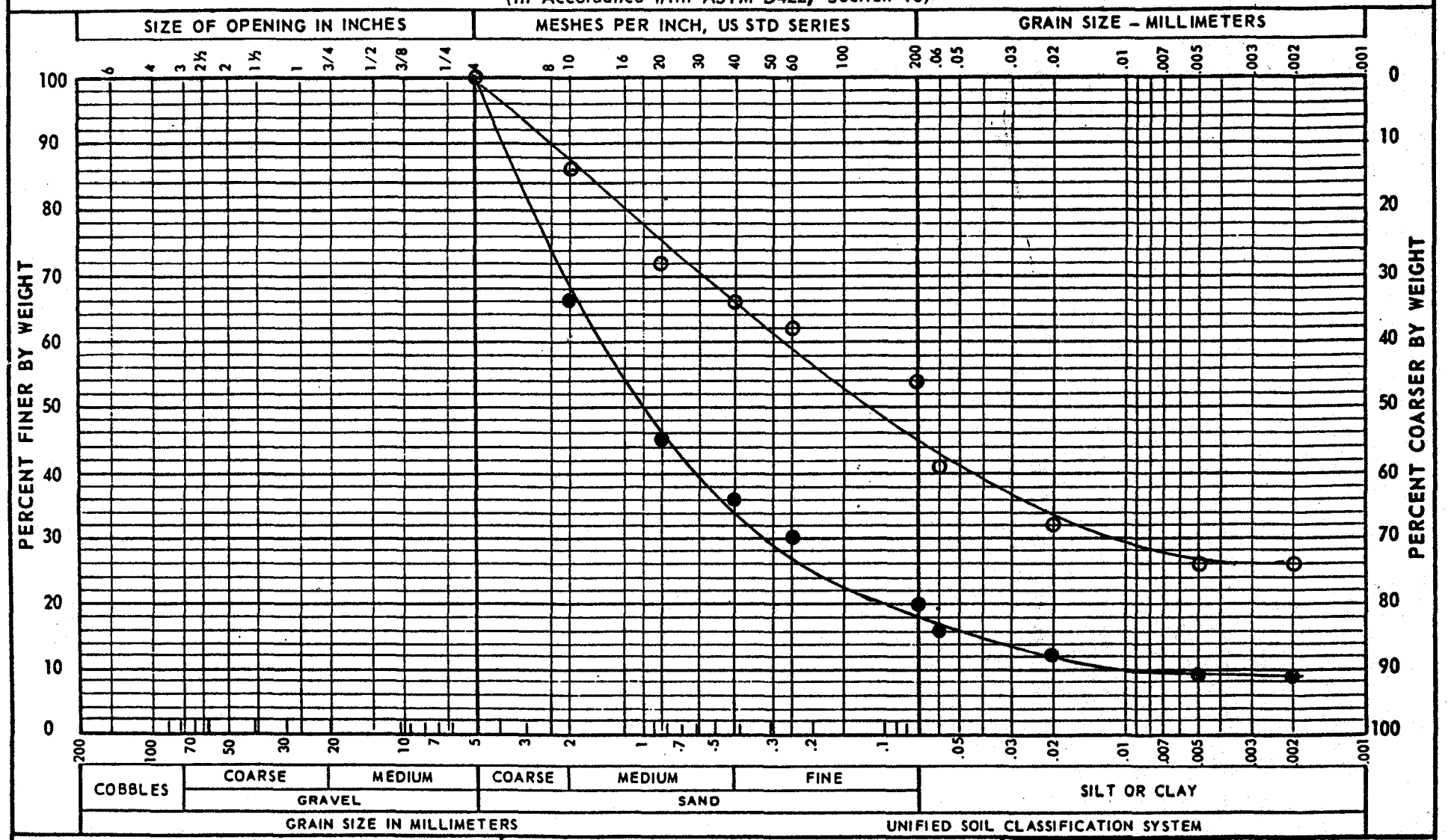
CORRECTION FACTOR FOR COARSE MATERIAL $= \frac{100}{44} = 2.273$

581/8-81

FORM #14

MECHANICAL ANALYSIS GRAPH OF GRANULAR MATERIALS

(In Accordance With ASTM D422, Section 16)



TEST NO.	SAMPLE DESCRIPTION	EBASCO SERVICES INCORPORATED
SAMPLE NO.	REGRADED GRADATION CURVES FOR "FINE" AND "COARSE" SOILS	APACHE JCT FRS (PROJECT)
SAMPLE LOCATION		DRAWN BY / CHECKED BY SG / DATE 11-8-85
SAMPLE ELEVATION		

SH 36

FORM #14

(SOURCE: USDA SOIL MECHANICS NOTE #1, JULY 1985)

FOR FINER MATERIAL

Determine filter gradation limits as follows

1) Smallest D_{15} size ≈ 0.001 mm

2) Percent passing #200 sieve from adjusted gradation curve $\approx$ 54%

$\therefore$ Base soil is in category 2

From Table III for Base soil category 2

$D_{15\,MAX} \leq 0.7$ mm

3) Filter minimum D_{15} size $\geq 4 \times d_{15}$ of base soil or 0.1 mm

$D_{15\,MIN} \geq 0.1$ mm

4) $D_{5\,MAX} \geq 0.075$ mm (200 sieve)

5) Maximum $D_{85} = 15$ mm (Since $D_{15\,MIN} < 0.5$ mm)

Suggested reasonable gradation limits for filter material:

SIEVE SIZE	% PASSING	SIEVE SIZE	% PASSING
3 in	100%	#20	20-75%
2 in	100%	#40	0-55%
3/4 in	90-100%	#60	0-40%
#4	60-100%	#100	0-25%
#10	40-100%	#200	0-5%

FOR COARSER MATERIAL

Determine filter gradation limits as follows

1) Smallest D_{15} size $\approx$ 0.4 mm

2) Percent passing #200 sieve from adjusted gradation curve $\approx$ 20%

$\therefore$ Base soil is in category 3

From Table III for Base soil category 3

$$D_{15\,MAX} \leq \left[\left(\frac{40-A}{40-15}\right)\left(4 \times d_{85} - 0.7\,mm\right)\right] + 0.7\,mm$$

A = % passing the No 200 sieve after regrading

$A \approx 20$

d_{85} for regraded base soil curve is approx 3.5 mm

$$D_{15\,MAX} \leq \left(\frac{40-20}{40-15}\right)\left(4 \times 3.5\,mm - 0.7\,mm\right) + 0.7\,mm = 11.3\,mm$$

3) Filter minimum D_{15} size $\geq 4 \times d_{15}$ of base soil or 0.1 mm

d_{15} for regraded base soil curve is approx 0.05 mm

$$D_{15\,MIN} \geq 4 \times 0.05\,mm = 0.2\,mm$$

4) $D_{5\,MAX} \geq 0.075\,mm$ (200 sieve)

EBASCO SERVICES INCORPORATED

BY V BOLAND DATE 10/14/85 SHEET 39 OF 66

CHKD. BY SG DATE 11-8-85 OFS NO. DEPT. NO.

CLIENT USDA-SCS

PROJECT APACHE JCT FRS

SUBJECT GRADATION OF FILTER MATERIALS

5) Maximum D_{85} = 15 mm (since $D_{15\,min}$ < 0.5 mm)

Suggested reasonable gradation limits for filter material:

SIEVE SIZE	% PASSING	SIEVE SIZE	% PASSING
3 in	100%	#20	0-75%
2 in	100%	#40	0-55%
3/4 in	90-100%	#100	0-25%
#4	45-100%	#200	0-5%
#10	15-100%		

CONCLUSIONS

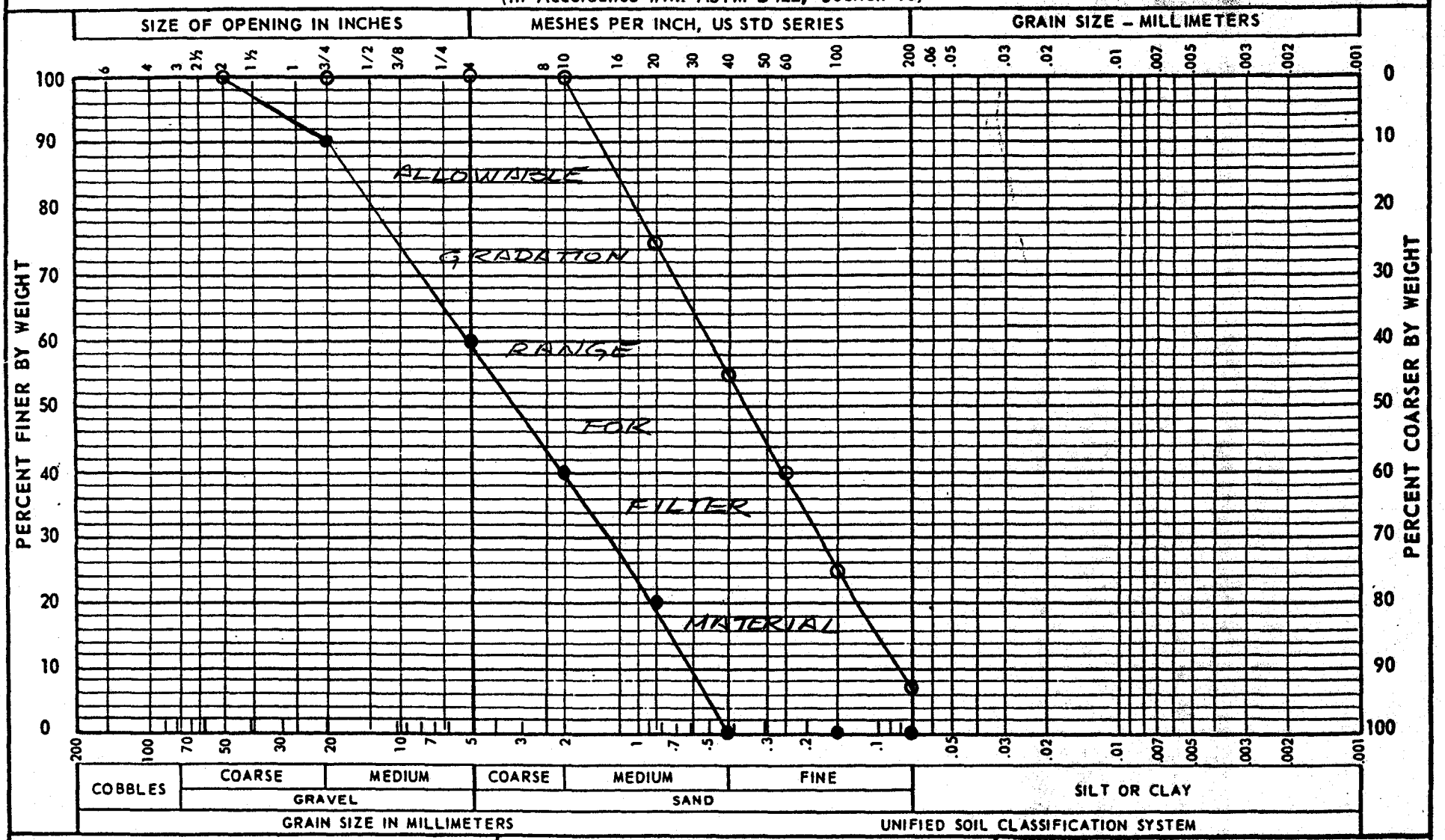
SIEVE SIZE	RECOMMENDED % PASSING FOR FINER SOIL	RECOMMENDED % PASSING FOR COARSER SOIL
3	100%	100%
2	100%	100%
3/4	90%-100%	90%-100%
#4	60%-100%	45%-100%
#10	40%-100%	15%-100%
#20	20%-75%	0-75%
#40	0-55%	0-55%
#60	0-40%	0-40%
#100	0-25%	0-25%
#200	0-5%	0-5%

USE A FILTER WITH A GRADATION AS RECOMMENDED FOR FINER SOIL SINCE IT MEETS THE REQUIREMENTS OF BOTH FINER SOIL (CATEGORY 2) AND COARSER SOIL (CATEGORY 3)

FORM #14

MECHANICAL ANALYSIS GRAPH OF GRANULAR MATERIALS

(In Accordance With ASTM D422, Section 16)



TEST NO.	SAMPLE DESCRIPTION	EBASCO SERVICES INCORPORATED
SAMPLE NO.	Allowable gradation range for filter material	
SAMPLE LOCATION		APACHE JCT FRS (PROJECT)
SAMPLE ELEVATION		DRAWN BY / CHECKED BY SG / DATE 11-8-85

SH 40

FORM #14

EBASCO SERVICES INCORPORATED

BY V BOLANO DATE 10/14/85 SHEET 40-A OF 66

CHKD. BY SG DATE 11-8-85 OFS NO. DEPT. NO.

CLIENT USDA-SCS

PROJECT APACHE JCT FRS

SUBJECT GRADATION OF EMBANKMENT MATERIALS

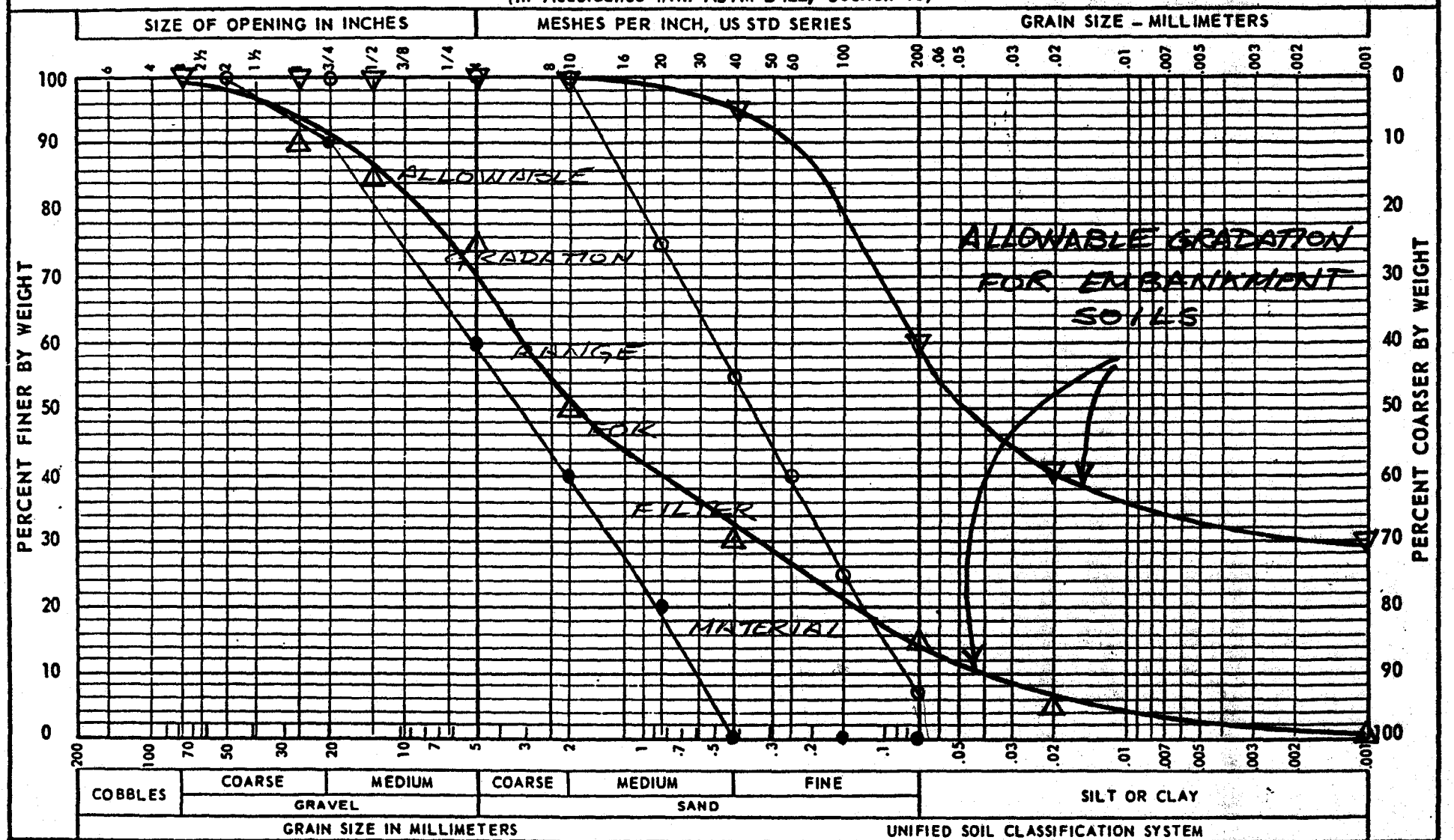
ALLOWABLE GRADATION FOR EMBANKMENT SOILS

SIEVE SIZE	% PASSING MAXIMUM	% PASSING MINIMUM
3"	100	100
1"	100	90
1/2"	100	85
#4	100	75
#10	100	50
#40	95	30
#200	60	15
0.02	40	5
0.001	30	0

FORM #14

MECHANICAL ANALYSIS GRAPH OF GRANULAR MATERIALS

(In Accordance With ASTM D422, Section 16)



TEST NO.	
SAMPLE NO.	
SAMPLE LOCATION	
SAMPLE ELEVATION	

SAMPLE DESCRIPTION

Allowable gradation range for filter material & embankment fill

EBASCO SERVICES INCORPORATED

APACHE JCT FRS
PROJECT

DRAWN BY	CHECKED BY	DATE
	SG	11-8-85

SH-40B

FORM #14

Compatibility of specified embankment fill and selected filter criteria.

$$\frac{D_{15}\text{ (of filter)}}{D_{85}\text{ (of soil)}} < 4 \text{ to } 5 < \frac{D_{15}\text{ (of filter)}}{D_{15}\text{ (of soil)}}$$

D_{15} (of filter) = 0.1 mm to 0.7 mm

D_{85} (of soil) = 0.2 mm to 15 mm (specified embankment fill)

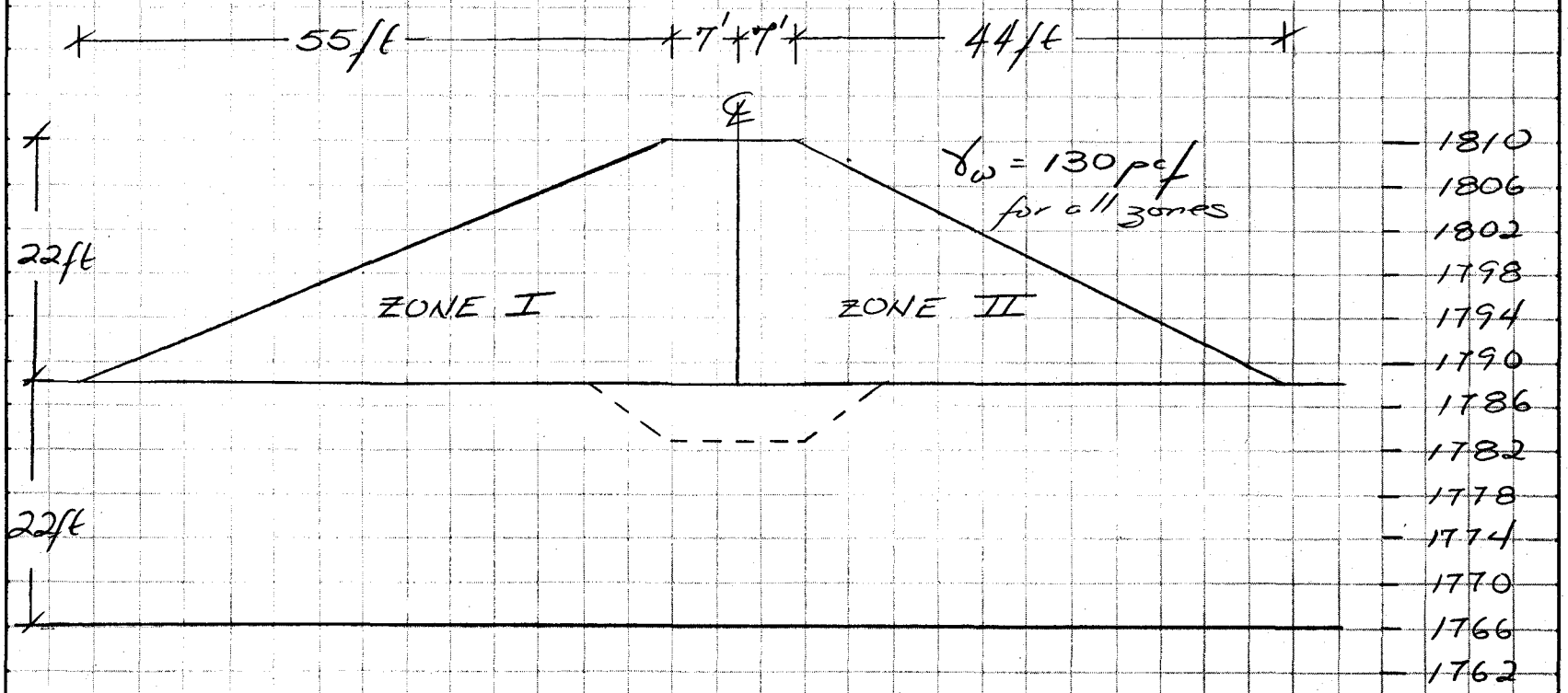
D_{15} (of soil) = <0.001 mm to 0.07 mm (specified embankment fill)

1) $\frac{D_{15}\text{ (of filter)}}{D_{85}\text{ (of soil)}}$ CRITICAL $= \frac{0.7\text{ mm}}{0.2\text{ mm}} = 3.5 < 4 \therefore$ OK

2) $\frac{D_{15}\text{ (of filter)}}{D_{15}\text{ (of soil)}}$ CRITICAL $= \frac{0.1\text{ mm}}{0.07\text{ mm}} = 1.4 < 4 \therefore$ NG

In order to meet the second criteria D_{15} (of filter) must not be less than 0.28 mm (4 × 0.07 mm)

It must be noted that the SM, SC soils predominant in the borrow area will satisfy the allowable gradation criteria for construction of the Apache JCT FRS. The use of gravelly sands will not be permitted for embankment material.



NOTES:

1) CUTOFF TRENCH WILL BE NEGLECTED AND INCREASE IN STRESS DUE TO EMBANKMENT LOADING WILL BE CALCULATED FOR THIS ZONE

2) MAXIMUM THICKNESS OF SILTY SAND LAYER IS ASSUMED AS 22 FT EXTENDING FROM ELEVATION +1776 TO ELEVATION + 1788

3) USE 4 FOOT LAYERS FOR DETERMINIG INCREASE IN STRESS

ZONE I: $a_I = 55 ft$, $b_I = 7 ft$

ZONE II: $a_{II} = 44 ft$, $b_{II} = 7 ft$

(Refer to attached Figure 5-5 from NAVFAC, p 7-5-7)

DEPTH z (ft)	a_I/z	b_I/z	I_I	a_{II}/z	b_{II}/z	I_{II}	$I_I + I_{II}$	$\Delta\sigma$
2	27.5	3.50	0.50	22.0	3.50	0.50	1.00	2,860
6	9.2	1.17	0.49	7.3	1.17	0.49	0.98	2,802
10	5.5	0.70	0.48	4.4	0.70	0.47	0.95	2,717
14	3.9	0.50	0.46	3.1	0.50	0.45	0.91	2,603
18	3.0	0.39	0.44	2.4	0.39	0.44	0.88	2,517
21	2.6	0.33	0.43	2.1	0.33	0.42	0.85	2,431

$$\Delta\sigma = (I_I + I_{II})\, P \quad \text{where}$$

$$P = \gamma_w H = (130 pcf)(22 ft) = 2860 psf$$

SH 43

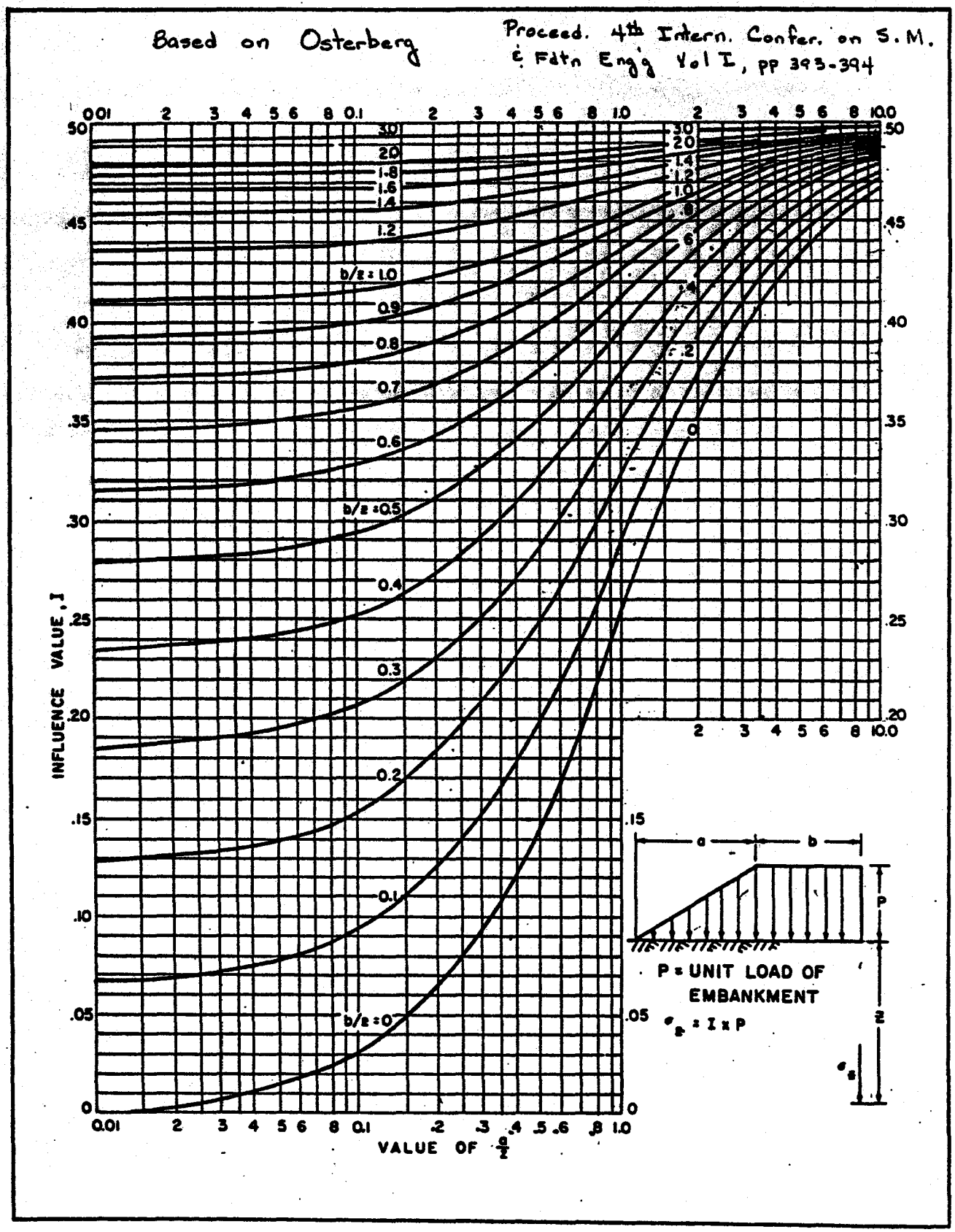


FIGURE 5-5
Influence Value for Vertical Stress Under Embankment Load of Infinite Length

EBASCO SERVICES INCORPORATED

BY V. BOLANO DATE 10/15/85 SHEET 44 OF 66

CHKD. BY S.G. DATE 11-8-85 DEPT. NO.

CLIENT USDA - SCS OFS NO.

PROJECT APACHE JCT FRS

SUBJECT SETTLEMENT ANALYSIS - FOUNDATION

DEPTH (ft)	γ_{wet} (pcf)	$\bar{\sigma}_o$, EXISTING OVERBURDEN PRESSURE (psf)	$\Delta\sigma$ INCREASE IN PRESSURE (psf)	$\log \frac{\bar{\sigma}_o + \Delta\sigma}{\bar{\sigma}_o}$	C_r	H (inches)	e_o	ΔS
2	100.0	200	2,860	1.185	0.020	48	0.976	0.58
6	100.0	600	2,802	0.754	0.020	48	0.976	0.37
10	100.0	1,000	2,717	0.570	0.020	48	0.976	0.28
14	110.0	1,440	2,603	0.448	0.020	48	0.976	0.22
18	110.0	1,880	2,517	0.369	0.020	48	0.976	0.18
21	110.0	2,210	2,431	0.322	0.020	24	0.976	0.08
								Σ = 1.71 inches

CONSOLIDATION SETTLEMENT = 1.71 inches

NOTES: 1) $\Delta S = \dfrac{C_r H \log \frac{\bar{\sigma}_o + \Delta\sigma}{\bar{\sigma}_o}}{1 + e_o}$ where

C_r = Recompression index (0.020 - Table VIII)
H = Layer height in inches
e_o = Original void ratio (0.976 - Table VIII)

581/8-81

EBASCO SERVICES INCORPORATED

BY V. BOLANO DATE 10/15/85 SHEET 45 OF 66

CHKD. BY S.G. DATE 11-17-85 DEPT. NO.

CLIENT USDA - SCS OFS NO.

PROJECT APACHE JCT FRS

SUBJECT SETTLEMENT ANALYSIS - FOUNDATION

DEPTH (ft)	γ_{wet} (pcf)	$\bar{\sigma}_o$, EXISTING OVERBURDEN PRESSURE (psf)	$\Delta\sigma$ INCREASE IN PRESSURE (psf)	$\log \frac{\bar{\sigma}_o + \Delta\sigma}{\bar{\sigma}_o}$	C_r	H (inches)	e_o	ΔS (inches)
2	100.0	200	2,860	1.185	0.023	48	1.030	0.64
6	100.0	600	2,802	0.754	0.023	48	1.030	0.41
10	100.0	1,000	2,717	0.570	0.023	48	1.030	0.31
14	110.0	1,440	2,603	0.448	0.023	48	1.030	0.24
18	110.0	1,880	2,517	0.369	0.023	48	1.030	0.20
21	110.0	2,210	2,431	0.322	0.023	24	1.030	0.09
								$\Sigma = 1.89$ inches

CONSOLIDATION SETTLEMENT = 1.89 inches

NOTES: 1) $\Delta S = \dfrac{C_r H \log \frac{\bar{\sigma}_o + \Delta\sigma}{\bar{\sigma}_o}}{1 + e_o}$ where

C_r = Recompression index (0.023 - Table VIII)
H = Layer height in inches
e_o = Original void ratio (1.030 - Table VIII)

2) Since the existing overburden pressure plus the increase in pressure due to the embankment loading is less than the preconsolidation pressure (lab test) ($\bar{\sigma}_o + \Delta\sigma < \sigma_p$) the recompression index C_r is used in performing the settlement calculations

Calculated settlement (per consolidation theory) = 1.7 in

ALTERNATE CALCULATION OF SETTLEMENT USING ELASTIC THEORY

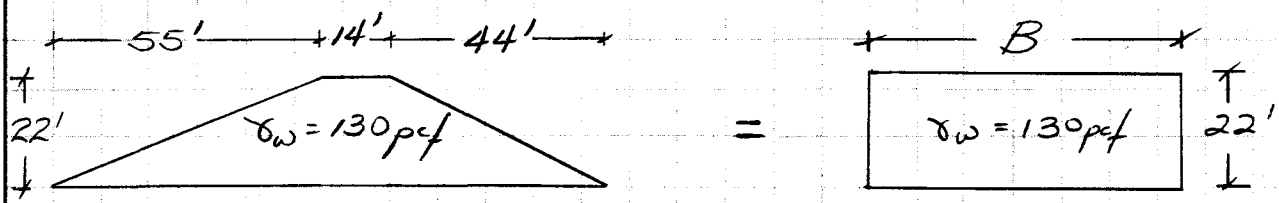
A.1) NAVFAC DM-7.1, MAY 1982, FIGURE 6, SH 7.1-219

$$\Delta H = \frac{2\,q\,B^2}{K_{v1}\,(B+1)^2} \quad \text{for } B \geq 40\,ft \quad \text{where}$$

ΔH = Elastic settlement
q = intensity of contact pressure (2,860 psf - Sheet 42)
B = least lateral dimension
K_{v1} = Modulus of vertical subgrade reaction

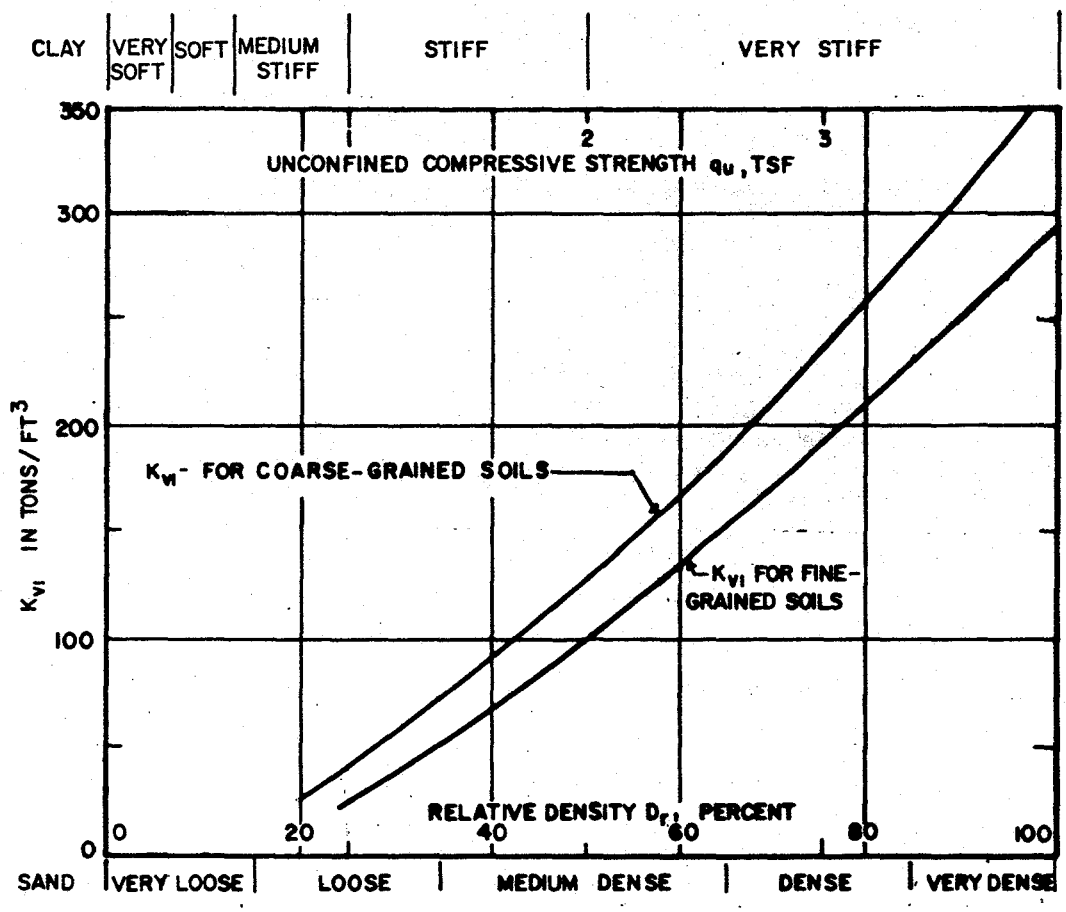
From NAVFAC Figure 6, for a loose sand, $K_{v1} = 40\ ^{Tons}/_{ft^3}$

Calculate B as follows:



$$55/2 + 14 + 44/2 = B$$

$$B = 63.5\,ft$$



DEFINITIONS

ΔH_i = IMMEDIATE SETTLEMENT OF FOOTING
q = FOOTING UNIT LOAD IN tsf
B = FOOTING WIDTH

D = DEPTH OF FOOTING BELOW GROUND SURFACE
K_{vi} = MODULUS OF VERTICAL SUBGRADE REACTION

COARSE-GRAINED SOILS

(MODULUS OF ELASTICITY INCREASING LINEARLY WITH DEPTH)
SHALLOW FOOTINGS $D \leq B$
FOR $B \leq 20$ FT:

$$\Delta H_i = \frac{4\,q\,B^2}{K_{vi}\,(B+1)^2}$$

FOR $B \geq 40$ FT:

$$\Delta H_i = \frac{2\,q\,B^2}{K_{vi}\,(B+1)^2}$$

INTERPOLATE FOR INTERMEDIATE VALUES OF B

DEEP FOUNDATION $D \geq 5B$
FOR $B \leq 20$ FT:

$$\Delta H_i = \frac{2\,q\,B^2}{K_{vi}\,(B+1)^2}$$

NOTES: 1. NONPLASTIC SILT IS ANALYZED AS COARSE-GRAINED SOIL WITH MODULUS OF ELASTICITY INCREASING LINEARLY WITH DEPTH.
2. VALUES OF K_{vi} SHOWN FOR COARSE-GRAINED SOILS APPLY TO DRY OR MOIST MATERIAL WITH THE GROUNDWATER LEVEL AT A DEPTH OF AT LEAST 1.5B BELOW BASE OF FOOTING. IF GROUNDWATER IS AT BASE OF FOOTING, USE $K_{vi}/2$ IN COMPUTING SETTLEMENT
3. FOR CONTINUOUS FOOTINGS MULTIPLY THE SETTLEMENT COMPUTED FOR WIDTH "B" BY 2.

FIGURE 6
Instantaneous Settlement of Isolated Footings on Coarse-Grained Soils

$$\Delta H = \frac{(2)(2{,}860\ psf)(63.5\ ft)^2(12\ in/ft)}{(40\ Tons/ft^3)(2000\ lbs/ton)(63.5\ ft + 1)^2}$$

$\Delta H = 0.8$ in

For continuous footings the settlement computed for width "B" (0.8 in) must be multiplied by 2

$\therefore S = 2 \times 0.8$ in $= \underline{1.6\ in}$

A.2) STRESS - STRAIN CONCEPT (BOWLES, FOUNDATION ANALYSIS AND DESIGN, p. 165)

$$S = \frac{\Delta q \Delta H}{E_s} = \int_0^H \varepsilon\, dH \quad \text{where}$$

Δq = increase in stress at certain depth due to surcharge (embankment) loading (see sheet 42)

ΔH = height of soil layer (48 inches, sheet 44)

E_s = Elastic soil modulus (Use 2.5 ksi for a loose sand or silty sand. Bowles, p. 35, Table 2-3)

$$S = \frac{(2{,}860 + 2{,}802 + 2{,}717 + 2{,}603 + 2{,}517)(48\ in) + (2431)(24\ in)}{(2.5\ Ksi)(1000\ lbs/k)(144\ in^2/ft^2)}$$

$S = \underline{2.0\ in}$

Settlement calculations performed using consolidation and elastic theory analysis indicate that the largest expected foundation settlement due to embankment loading is approximately 2 inches

It must be noted that this is a worst case analysis since:

1) the height of the "compressible" silty sand layer above caliche was assumed to be 22 ft (EL + 1766 to EL + 1788) and generally depth to caliche is only approx 6.5 feet.

2) The calculations were performed for a point at the centerline of the embankment.

It is expected that "settlement" (consolidation or elastic) of the subsurface soils will be minimal or non-existant along the majority of the foundation and therefore the maximum differential settlement would be in the order of 2 inches for the maximum depth of foundation. Since the soils are predominantly granular, the settlement will be elastic and will take place during construction of the embankment.

EBASCO SERVICES INCORPORATED

BY V. BOLANO DATE 10/15/85 SHEET 50 OF 66

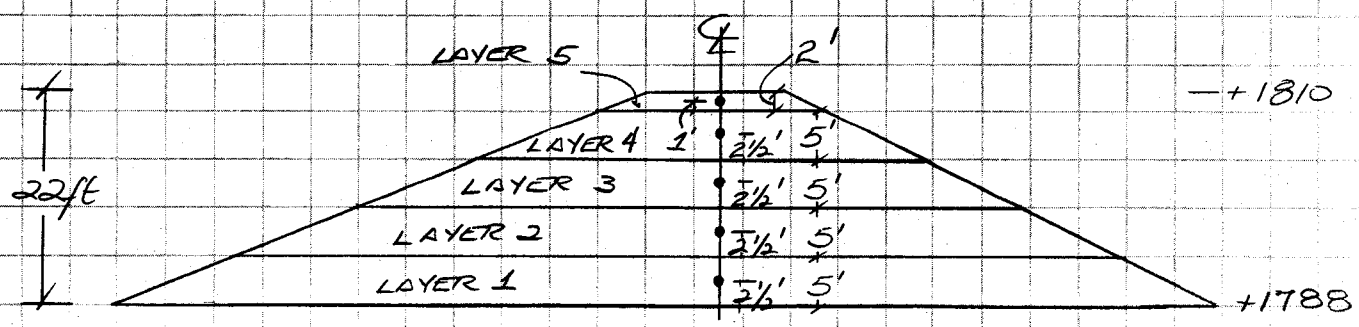
CHKD. BY S.G. DATE 11-8-85 OFS NO. _____ DEPT. NO. _____

CLIENT USDA-SCS

PROJECT APACHE JCT FRS

SUBJECT SETTLEMENT ANALYSIS - EMBANKMENT

CONSOLIDATION SETTLEMENT



LAYER	$\bar{\sigma}_o$ (psf)	$\Delta\sigma$ (psf)	$\log\frac{\bar{\sigma}_o+\Delta\sigma}{\bar{\sigma}_o}$	C_c	H (inches)	e_o	ΔS
1	325	2,535	0.944	0.008	60	0.460	0.31
2	325	1,885	0.833	0.008	60	0.460	0.27
3	325	1,235	0.681	0.008	60	0.460	0.22
4	325	585	0.447	0.008	60	0.460	0.15
5	130	130	0.301	0.008	24	0.460	0.04
							Σ=0.99 inches

NOTES: 1) $\bar{\sigma}_o = (130\,pcf)(2.5\,ft) = 325\,psf$

2) $\Delta\sigma = (130\,pcf)$(Height of soil layers above)

Example: $\Delta\sigma_1 = (130\,pcf)(19\tfrac{1}{2}\,ft) = 2535\,psf$

3) $$\Delta S = \frac{C_r H \left(\log \frac{\bar{\sigma}_o + \Delta\sigma}{\bar{\sigma}_o}\right)}{1 + e_o}$$ where

C_r = Recompression index (0.008 - Table VII)
H = Layer height in inches
e_o = Original void ratio (0.460 - Table VII)

4) The recompression index is used since the total pressures are generally less than the "apparent" preconsolidation pressures ($\bar{\sigma}_o + \Delta\sigma < p_c$).

ALTERNATE CALCULATION OF SETTLEMENT USING ELASTIC THEORY: STRESS - STRAIN CONCEPT (BOWLES, FOUNDATION ANALYSIS AND DESIGN, p. 165)

$$S = \frac{\Delta q \, \Delta H}{E_s} = \int_0^H \varepsilon \, dH$$ where

Δq = increase in stress at certain depth due to surcharge loading (see sheet

ΔH = height of soil layer

E_s = Elastic soil modulus (Use 5ksi for a silty sand or a dense sand. Bowles, p. 35, Table 2-3)

LAYER	Δq (psf)	ΔH (inches)	E_s (ksi)	ΔS (inches)
1	2,535	60	5	0.21
2	1,885	60	5	0.16
3	1,235	60	5	0.10
4	585	60	5	0.05
5	130	24	5	—
				Σ 0.52 inches

$$\Delta S_1 = \frac{(2{,}535\ lb/ft^2)(60\ in)(1\ ft^2/144\ in^2)}{(5\ k/in^2)(1000\ lbs/kip)} = 0.21\ inches$$

$$S_{TOTAL} = \Delta S_1 + \Delta S_2 + \Delta S_3 + \Delta S_4 + \Delta S_5$$

$$\underline{S_{TOTAL} = 0.52\ inches}$$

(Note: If a value of 3 ksi were used for E_s then the calculated total elastic settlement would be 0.87 inches. That is:

$$S_{TOTAL} = \frac{(2{,}535 + 1{,}885 + 1{,}235 + 585\ psf)(60\ in)(1\ ft^2/144\ in^2)}{(3\ k/in^2)(1000\ lbs/kip)}$$

$$S_{TOTAL} = 0.87\ inches$$)

Settlement calculations performed using consolidation and elastic theory analysis indicate that the largest expected settlement of the embankment under its own weight will be under one inch. This settlement will be mostly elastic and occur during construction.

SETTLEMENT CALCULATIONS - CONCLUSIONS

The maximum settlement of the foundation soils and the embankment under its own weight will be less than 3 inches (2 inches for foundation soils and 1 inch for embankment). It is expected that settlements of this magnitude will not occur (conservative assumptions in calcs) and if they did, it would take place during construction due to the elastic characteristics of the soils. Therefore it is not necessary to add any additional height to freeboard in order to compensate for future consolidation or elastic settlements.

(MAJOR SOURCE: ALFI, ABDULAZIZ ADNAN SHARIF, Mechanical and Electron Optical Properties of a Stabilized Collapsible Soil in Tucson, Arizona, PHD Thesis, University of Arizona, 1984)

Collapsible soils generally have the following properties:

1) Dry densities range from 65 pcf to 105 pcf reflecting porosities from 40% to 60%

2) Initial water content from 5% to 30%, usually less than 10%

3) Clay contents range from 10% to 30% with the balance of the soil composed of silt or fine sand particles (usually fairly uniformly graded)

4) Liquid limits below 45 and plasticity index below 25

Behavior of the soil upon wetting depends on the magnitude of the applied load and the change in water content.

Appreciable collapse of a soil requires the following conditions:

1) Highly porous, potentially unstable, partly saturated structure

2) High applied load to increase metastability

3) Strong bonding between interparticle contacts that is reducible upon wetting to trigger collapse

RESULTS OF FIELD DENSITY TESTS AND LABORATORY CLASSIFICATION TESTS PERFORMED TO ASCERTAIN COLLAPSIBILITY POTENTIAL OF FOUNDATION SOILS ARE PRESENTED ON TABLE VI (FOLLOWING PAGE)

EMPIRICAL METHODS USED TO DETERMINE COLLAPSIBILITY POTENTIAL

1) Denisov Method

$$K = \frac{e_L}{e_o} \quad \text{where}$$

e_L = void ratio at liquid limit

e_o = natural void ratio

If $K = 0.5$ to 0.75 ; highly collapsible soil
$K = 1.0$; non-collapsible loams
$K = 1.5$ to 2.0 non collapsible soils

$$e_L = w_L G_s \quad \text{and} \quad e_o = G_s \left(\gamma_w / \gamma_d \right) - 1 \quad \text{where}$$

w_L = liquid limit
G_s = specific gravity of soil solids
γ_w = unit weight of water
γ_d = dry unit weight of soil

STATION 70+00 SAMPLE (TABLE VI):

$\omega_L = 0.245$, $\gamma_d = 94.0$ pcf, $G_s = 2.65$ (assumed)

$e_L = \omega_L G_s = (0.245)(2.65) = 0.649$

$e_o = G_s(\gamma_w/\gamma_d) - 1 = (2.65)(62.4\text{pcf}/94.0\text{pcf}) - 1 = 0.759$

$K = e_L/e_o = 0.649/0.759 = 0.86$ ∴ Perhaps collapsible

STATION 42+50 SAMPLE (TABLE VI)

$\omega_L = 0.334$, $\gamma_d = 103.4$, $G_s = 2.65$ (assumed)

$e_L = \omega_L G_s = (0.334)(2.65) = 0.885$

$e_o = G_s(\gamma_w/\gamma_d) - 1 = (2.65)(62.4\text{pcf}/103.4\text{pcf}) - 1 = 0.599$

$K = e_L/e_o = 0.885/0.599 = 1.48$ ∴ Non-collapsible

2) Priklonskij Method

$$K_d = \frac{\omega_L - \omega_s}{I_p}$$ where

ω_L = liquid limit

ω_s = natural water content at saturation

I_p = plasticity index

If $K_d < 0$; highly collapsible soil
$K_d > 0.5$; non-collapsing soil
$K_d > 1.0$; swelling soil

STATION 70+00 SAMPLE (TABLE VI):

$\omega_L = 0.245$, $I_p = 0.043$

$$\omega_s = e_o/G_s = \frac{G_s(\gamma_w/\gamma_d) - 1}{G_s} = \left(\frac{\gamma_w}{\gamma_d}\right) - \left(\frac{1}{G_s}\right)$$

$\gamma_w = 62.4$ pcf, $\gamma_d = 94.0$ pcf, $G_s = 2.65$ (assumed)

$$\omega_s = \left(62.4/94.0\right) - \left(1/2.65\right) = 0.287$$

$$K_d = \frac{\omega_L - \omega_s}{I_p} = \frac{0.245 - 0.287}{0.043} = -0.98 \therefore \underline{\text{Highly collapsible}}$$

STATION 42+50 SAMPLE (TABLE VI)

$\omega_L = 0.334$, $I_p = 0.142$

$$\omega_s = \left(\gamma_w/\gamma_d\right) - \left(\frac{1}{G_s}\right)$$

$\gamma_w = 62.4$ pcf, $\gamma_d = 103.4$ pcf, $G_s = 2.65$ (assumed)

$$\omega_s = \left(62.4/103.4\right) - \left(1/2.65\right) = 0.226$$

$$K_d = \frac{\omega_L - \omega_s}{I_p} = \frac{0.334 - 0.226}{0.142} = 0.76 \therefore \underline{\text{Non collapsible}}$$

3) Gibbs Method

$$R_G = \omega_S / \omega_L \quad \text{where}$$

ω_S = water content @ saturation
ω_L = liquid limit

$R_G \geq 1$; collapsible soil

STATION 70+00

$\omega_S = 0.287$ (sheet 57), $\omega_L = 0.245$ (Table VI)

$R_G = \omega_S / \omega_L = 0.287 / 0.245 = 1.17$ ∴ Collapsible soil

STATION 42+50

$\omega_S = 0.226$ (sheet 57), $\omega_L = 0.334$ (Table VI)

$R_G = 0.226 / 0.334 = 0.68$ ∴ Non-collapsible soil

4) USSR 1962 Building Code

$$\lambda = \frac{e_o - e_L}{1 + e_o} \quad \text{where}$$

e_o = natural void ratio
e_L = void ratio at liquid limit

If $\lambda \geq -0.1$; collapsible soil

STATION 70+00

$e_o = 0.759$ (sheet 56), $e_L = 0.649$ (sheet 56)

$\lambda = \frac{e_o - e_L}{1+e_o} = (0.759 - 0.649)/(1+0.759) = 0.06$ ∴ Collapsible

STATION 42+50

$e_o = 0.599$ (sheet 56), $e_L = 0.885$ (sheet 56)

$\lambda = \frac{e_o - e_L}{1+e_o} = (0.599 - 0.885)/(1+0.599) = -0.18$ ∴ Non-collapsible

5) Feda Porosity Method

If $\eta_o = \frac{e_o}{1+e_o} \times 100 > 40\%$ then soil is susceptible to collapse where

e_o = natural void ratio

STATION 70+00

$e_o = 0.759$ (sheet 56)

$\eta_o = (0.759/(1+0.759)) \times 100 = 43\% > 40\%$ ∴ Collapsible

STATION 42+50

$e_o = 0.599$ (sheet 56)

$\eta_o = (0.599/1+0.599) \times 100 = 37\% < 40\%$ ∴ Non-collapsible

6) Dual Criteria Suggested by Abdulazij

Collapsible soils: 1) $e_o > 0.67$ and

2) $\dfrac{e_o - e_L}{(1+e_o)(w_o \gamma_d)} > -0.67$ where

e_o = natural void ratio
e_L = void ratio at liquid limit
w_o = natural moisture content as a fraction
γ_d = natural dry density in g/cm^3

STATION 70+00

$e_o = 0.759$ (Sheet 56), $e_L = 0.649$ (Sheet 56),

$w_o = 0.049$ (Table VI), $\gamma_d = 94.0$ pcf or $1.51\ g/cm^3$ (Table VI)

$e_o = 0.759 > 0.67$

$$\frac{e_o - e_L}{(1+e_o)(w_o \gamma_d)} = \frac{0.759 - 0.649}{(1+0.759)(0.049)(1.51)} = 0.85 > -0.67$$

∴ Collapsible

STATION 42+50

$e_o = 0.599$ (Sheet 56), $e_L = 0.885$ (Sheet 56)

$w_o = 0.06$ (Table VI, $\gamma_d = 103.4$ pcf or $1.66\ g/cm^3$ (Table VI)

$e_o = 0.599 < 0.67$ NG

$$\frac{e_o - e_L}{(1+e_o)(\gamma_d w_o)} = \frac{0.599 - 0.885}{(1+0.599)(1.66)(0.06)} = -1.80 < -0.67$$

∴ Non-collapsible

EBASCO SERVICES INCORPORATED

BY V BOLANO DATE 10/16/85 SHEET 61 OF 66

CHKD. BY SG DATE 11-8-85 OFS NO. DEPT. NO.

CLIENT USDA - SCS

PROJECT APACHE JCT FRS

SUBJECT COLLAPSIBILITY POTENTIAL

SUMMARY OF RESULTS: COLLAPSIBILITY POTENTIAL

METHOD	SAMPLE FROM STATION 70+00	SAMPLE FROM STATION 42+50
1- Denisov	C	NC
2- Priklonskij	C	NC
3- Gibbs	C	NC
4- USSR 1962 BC	C	NC
5- Feda Porosity	C	NC
6- Dual Criteria	C	NC

NOTES: 1) Of the six samples tested (as shown on Table VI), four were non-plastic

2) In order to determine whether the sample from STATION 70+00 is indeed collapsible it would be necessary to perform a consolidation (oedometer) test on an undisturbed sample.

3) C = collapsible, NC = Non-collapsible

581/8-81

Three undisturbed soil samples were obtained from Q Dam STATION 52+50 (Table VIII) and routine classification and consolidation tests were performed

The six methods previously used to determine the collapsibility potential of the soils will now be applied to these three samples and the results compared with the consolidation tests performed previously.

6- Dual Criteria By Abdulaziz

$$e_o > 0.67 \text{ and } A = \frac{e_o - e_L}{(1+e_o)(w_o \gamma_d)} > -0.67$$

STATION 52+50 SAMPLE	SOIL TYPE	e_o	w_L	G_s	w_o	γ_d (g/cc)	e_L	A	COLLAPSIBLE
11.2	ML	0.811	0.25	2.69	0.055	1.49	0.673	0.93	YES
11.3	ML	0.976	0.29	2.70	0.064	1.37	0.783	1.11	YES
11.4	ML	1.030	0.29	2.69	0.069	1.33	0.780	1.34	YES

NOTE: 1) All soil parameters were obtained from consolidation test results (partly presented on Table VIII).

2) $e_L = w_L G_s$

1) Denisov Method

$K = e_L / e_o$ If $K = 0.5$ to 0.75, Highly collapsible
0.75 to 1.0 Collapsible

STATION 52+50 SAMPLE	e_L (Sheet 62)	e_o (Sheet 62)	K	COLLAPSIBLE
11.2	0.673	0.811	0.83	Yes
11.3	0.783	0.976	0.80	Yes
11.4	0.780	1.030	0.76	Yes

2) Priklonskij Method

$K_d = \frac{w_L - w_s}{I_p}$ If $K < 0$; Highly collapsible

STATION 52+50 SAMPLE	w_L	e_o	G_s	w_s	I_p	K_d	COLLAPSIBLE
11.2	0.25	0.811	2.69	0.30	0.03	-1.7	YES
11.3	0.29	0.976	2.70	0.36	0.05	-1.4	YES
11.4	0.29	1.030	2.69	0.38	0.04	-2.3	YES

NOTE: 1) $w_s = e_o / G_s$

3) Gibbs Method

$R_G = \omega_S / \omega_L$ If $R_G \geq 1$; collapsible soil

STATION 52+50 SAMPLE	ω_S (Sheet 63)	ω_L (Sheet 63)	R_G	COLLAPSIBLE
11.2	0.30	0.25	1.20	Yes
11.3	0.36	0.29	1.24	Yes
11.4	0.38	0.29	1.31	Yes

4) USSR 1962 Building Code

$\lambda = \frac{e_o - e_L}{1 + e_o}$ If $\lambda \geq -0.1$; collapsible soil

STATION 52+50 SAMPLE	e_o (Sheet 62)	e_L (Sheet 62)	λ	COLLAPSIBLE
11.2	0.811	0.673	0.08	YES
11.3	0.976	0.783	0.10	YES
11.4	1.030	0.780	0.12	YES

5) Feda Porosity Method

If $n_o = \frac{e_o}{1+e_o} \times 100 > 40\%$ then susceptible to collapse

STATION 52+50 SAMPLE	e_o (sheet 62)	n_o	COLLAPSIBLE
11.2	0.811	45%	YES
11.3	0.976	49%	YES
11.4	1.030	51%	YES

SUMMARY OF RESULTS: COLLAPSIBILITY POTENTIAL

Method	STATION 52+50		
	SAMPLE 11.2	SAMPLE 11.3	SAMPLE 11.4
1) Denisov	C	C	C
2) Priklonskij	C	C	C
3) Gibbs	C	C	C
4) USSR 1962	C	C	C
5) Feda Porosity	C	C	C
6) Dal Criteria	C	C	C

C = Collapsible

CONSOLIDATION TEST RESULTS

STATION 52+50 SAMPLE	SOIL TYPE	P_c (psf)	% CONSOLIDATION @ P_c	MAXIMUM PRESSURE (psf)	% CONSOLIDATION @ P_{MAX}	ω_n	ω_{sat}
11.2	ML	4,000	0.7%	16,000	3.2%	5.5%	30%
11.3	ML	6,700	1.2%	16,000	9.0%	6.4%	36%
11.4	ML	6,000	3.2%	16,000	11.8%	6.9%	38%

where

P_c = preconsolidation pressure
ω_n = natural moisture content
ω_s = moisture content at saturation

The results of the consolidation tests performed contradict the empirical relationships used on the previous page to determine the collapsibility potential of the soils.

CONCLUSIONS

Given the extremely limited amount of test data, the collapsibility potential of the foundation soils can not be quantitatively ascertained. However, based on the three ML samples tested, which were thought to be potentially collapsible (based on the empirical relations) it does not appear to be a severe concern.

APPENDIX B - SUMMARY OF SLOPE STABILITY COMPUTER RUNS

The major portions of the following seven computer runs are presented in this report:

1 - End of construction or reservoir at freeboard storm elevation. No acceleration. Downstream slope. SLOPE-II.

2 - Reservoir at freeboard storm elevation. Acceleration (0.1g). Downstream slope. SLOPE-II.

3 - Reservoir at crest of emergency spillway. Steady state seepage. No acceleration. Downstream slope. SLOPE-II.

4 - Reservoir at crest of emergency spillway. Steady state seepage. No acceleration. Downstream slope. EBASCO.

5 - Reservoir at crest of emergency spillway. Steady state seepage. Acceleration (0.1g). Downstream slope. SLOPE-II.

6 - Reservoir at crest of emergency spillway. Steady state seepage. Acceleration (0.1g). Downstream slope. EBASCO.

7 - Sudden drawdown. No acceleration. Upstream slope. SLOPE-II.

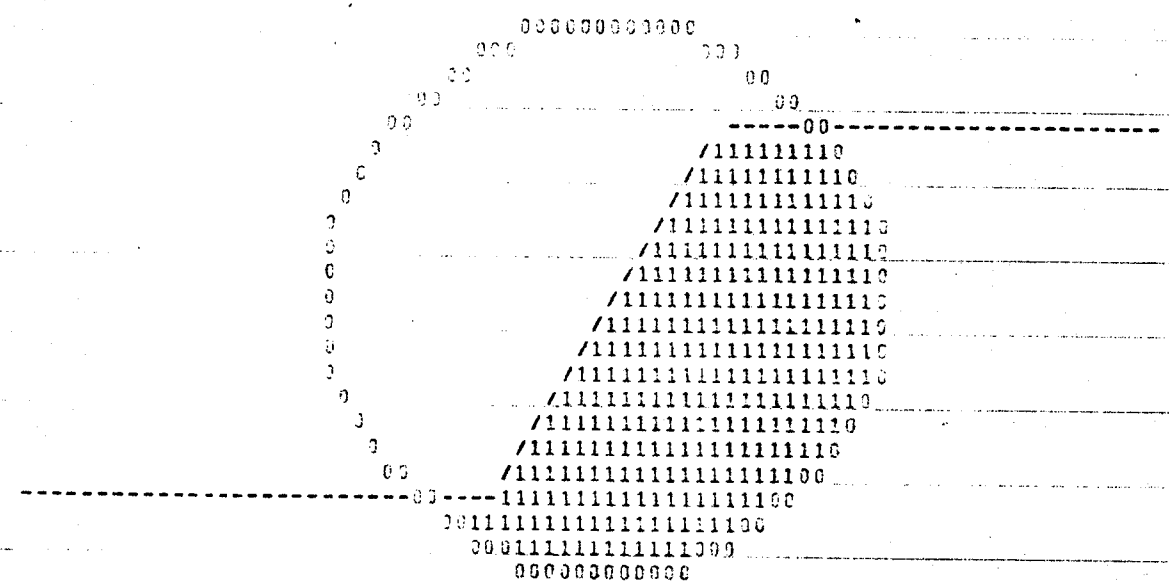
All computer printouts will be given to the SCS during the Final Design Phase.

End of Construction or Reservoir @ freeboard
Storm elevation
No acceleration
Downstream slope

```
DDDDDDDDDDD   TTTTTTTTTTTT  KK        KK  555555555555
DDDDDDDDDDDD  TTTTTTTTTTTT  KK      KK    555555555555
DD        DD       TT       KK    KK      55
DD        DD       TT       KK  KK        55
DD        DD       TT       KKKKK         55
DD        DD       TT       KKKKKK        5555555555
DD        DD       TT       KK   KK       555555555555
DD        DD       TT       KK    KK                55
DD        DD       TT       KK     KK               55
DD        DD       TT       KK      KK    55         55
DDDDDDDDDDDD       TT       KK       KK   555555555555
DDDDDDDDDDD        TT       KK        KK   5555555555

          JJ  CCCCCCCCCCCC  BBBBBBBBBBBB                555555555555   9999999999     00000000        11
          JJ  CCCCCCCCCCCC  BBBBBBBBBBBB                555555555555  999999999999   0000000000      111
          JJ  CC        CC  BB        BB                55            99        99  00        00    1111
          JJ  CC        CC  BB        BB                55            99        99  00        00      11
          JJ  CC        CC  BB        BB                55            99        99  00        00      11
          JJ  CC        CC  BBBBBBBBBB                  555555555555  999999999999  00        00      11
          JJ  CC        CC  BBBBBBBBBB                  555555555555   99999999999  00        00      11
          JJ  CC        CC  BB        BB                        55            99  00        00      11
JJ        JJ  CC        CC  BB        BB                        55            99  00        00      11
JJ        JJ  CC        CC  BB        BB                55      55  99        99  00        00      11
JJJJJJJJJJJJ  CCCCCCCCCCCC  BBBBBBBBBBBB                555555555555  999999999999   0000000000  1111111111
 JJJJJJJJJJ   CCCCCCCCCCCC  BBBBBBBBBBB                 5555555555    9999999999     00000000   1111111111

RRRRRRRRRRR    2222222222       11        2222222222   66666666666  PPPPPPPPPPP   RRRRRRRRRRR      11
RRRRRRRRRRRR  222222222222     111      222222222222  666666666666  PPPPPPPPPPPP  RRRRRRRRRRRR    111
RR        RR  22        22    1111      22        22  66            PP        PP  RR        RR   1111
RR        RR            22      11                22  66            PP        PP  RR        RR     11
RR        RR            22      11                22  66            PP        PP  RR        RR     11
RRRRRRRRRRR            22       11               22   66666666666   PPPPPPPPPPP   RRRRRRRRRRR      11
RRRRRRRRRRR          22         11             22     666666666666  PPPPPPPPPPP   RRRRRRRRRRR      11
RR     RR          22           11           22       66        66  PP            RR     RR        11
RR      RR       22             11         22         66        66  PP            RR      RR       11
RR       RR    22               11       22           66        66  PP            RR       RR      11
RR        RR   222222222222  1111111111  222222222222  666666666666  PP            RR        RR  1111111111
RR        RR   222222222222  1111111111  222222222222   6666666666   PP            RR        RR  1111111111

PPPPPPPPPPP   RRRRRRRRRRR   TTTTTTTTTTTT      11            44      00000000     3333333333
PPPPPPPPPPPP  RRRRRRRRRRRR  TTTTTTTTTTTT     111           444     0000000000   333333333333
PP        PP  RR        RR       TT        1111          4444    00        00  33        33
PP        PP  RR        RR       TT          11        44  44    00        00            33
PP        PP  RR        RR       TT          11       44   44    00        00            33
PPPPPPPPPPPP  RRRRRRRRRRRR       TT          11      44    44    00        00        3333
PPPPPPPPPPP   RRRRRRRRRRR        TT          11     44     44    00        00        3333
PP            RR     RR          TT          11   4444444444    00        00            33
PP            RR      RR         TT          11  44444444444    00        00            33
PP            RR       RR        TT          11            44    00        00  33        33
PP            RR        RR       TT     1111111111           44   0000000000  333333333333
PP            RR         RR      TT     1111111111           44     00000000   3333333333
```

RUN DATE 85/11/12.
TIME 13.42.51.

```
**************
*  SLOPE-II  *  IS A FULLY MAINTAINED, PROPRIETARY PRODUCT OF
**************

                GEO-SLOPE PROGRAMMING LTD.,
                7927 SILVER SPRINGS ROAD N.W.,
                CALGARY, ALBERTA, CANADA, T3B 4K4
```



PLEASE NOTIFY GEO-SLOPE PROGRAMMING LTD., OF ANY
PROBLEMS ENCOUNTERED WITH SLOPE-II.

DISCLAIMER:
NO WARRANTY IS EXPRESSED OR IMPLIED AS TO THE
DOCUMENTATION FUNCTION OR PERFORMANCE OF THE
PROGRAM AND THE USER OF THE PROGRAM IS EXPECTED
TO MAKE THE FINAL EVALUATION IN THE CONTEXT OF
THE PROBLEM UNDER CONSIDERATION.

VERSION 2-A RELEASE 5.0

```
*********************
*  SLOPE STABILITY  *
*********************

DATE  09-04-85                    RUN  TRIAL 01

COMMENTS        RESERVOIR AT FREEBOARD  DS NO EQ

 BISHOP'S SIMPLIFIED METHOD

          DETAILS ON THE SLIP CIRCLE GRID

   LAYOUT OF GRID FOR CENTERS OF TRIAL SLIP CIRCLES
   ************************************************

   THE X-COORDINATE OF THE LOWER LEFT CORNER OF THE GRID IS   145.0000
   THE CENTERS ARE INCREMENTED      5   TIMES IN THE X-DIRECTION WITH EACH
   INCREMENT EQUAL TO      10.0000

   THE Y-COORDINATE OF THE LOWER LEFT CORNER OF THE GRID IS  115.0000
   THE CENTERS ARE INCREMENTED      4   TIMES IN THE Y-DIRECTION WITH EACH
   INCREMENT EQUAL TO      10.0000

   THE GRID HAS A SLOPE(CHANGE IN ABSCISSA DIVIDED BY CHANGE IN ORDINATE)
   OF        .5000.

   OTHER PARTICULARS FOR THE ANALYSIS
   **********************************

   THE Y-COORDINATE OF A HORIZONTAL LINE TANGENT TO THE SLIP SURFACE OF
   MINIMUM RADIUS IS   106.0000.

   FOR EACH CENTER, RADIUS IS INCREMENTED    20 TIMES WITH EACH INCREMENT
   EQUAL TO      2.0000
   THE TOTAL NUMBER OF STABILITY ANALYSIS IS       630.

   THE GEOMETRY IS DIVIDED INTO      264 SLICES, EACH      1.0000

   THE DEPTH OF THE DEEPEST SLICE MUST BE GREATER THAN      5.0000

   THE TOLERANCE IN CALCULATING THE FACTOR OF SAFETY IS      .0100

   THE SEISMIC COEFFICIENT IS EQUAL TO      0.0000

   THE UNIT WEIGHT OF WATER IS    62.4000

   THE CONTROL QUESTIONS ANSWERED
    1.) PRINT OUT THE INPUT DATA.  IQ(1)=2
    2.) WRITE OUT ALL FACTORS OF SAFETY.  IQ(2)=1
    3.) WRITE OUT A SUMMARY OF THE MINIMUM FACTORS OF SAFETY.  IQ(3)=2
    4.) WRITE OUT A DETAILED SUMMARY OF THE COMPUTATIONS  IQ(4)=2
    7.) DISREGARD THE PREVIOUS PORE PRESSURE DATA.  IQ(7)=1
    8.) SET ALL NEGATIVE PORE PRESSURES TO ZERO.  IQ(8)=1
    9.) PRINT A DIGITAL PLOT OF THE GEOMETRY AND SLIP SURFACE.  IQ(9)=1
```

```
*********************
*  SLOPE STABILITY  *
*********************
```

DATE 09-04-85 RUN TRIAL 01

COMMENTS RESERVOIR AT FREEBOARD DS NO EQ

BISHOP'S SIMPLIFIED METHOD

SPECIAL DETAILS ON THE LAYOUT OF THE CROSS-SECTION

THE COORDINATES OF THE VERTICES OF LINES FORMING THE CONTACTS BETWEEN SOIL LAYERS ARE AS FOLLOWS

LINE NO.	COORDINATES	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
100	X	10.0	118.5	120.0	134.0	178.0	275.0	275.0	275.0	275.0	275.0	275.0	275.0	275.0	275.0	275.0
100	Y	109.4	109.4	110.0	110.0	88.0	86.0	86.0	86.0	86.0	86.0	86.0	86.0	86.0	86.0	86.0
200	X	10.0	65.0	118.5	120.0	134.0	178.0	275.0	275.0	275.0	275.0	275.0	275.0	275.0	275.0	275.0
200	Y	88.0	88.0	109.4	110.0	110.0	88.0	86.0	86.0	86.0	86.0	86.0	86.0	86.0	86.0	86.0
300	X	10.0	65.0	112.5	120.0	135.0	142.5	178.0	275.0	275.0	275.0	275.0	275.0	275.0	275.0	275.0
300	Y	88.0	88.0	88.0	83.0	83.0	88.0	86.0	86.0	86.0	86.0	86.0	86.0	86.0	86.0	86.0
400	X	10.0	275.0	275.0	275.0	275.0	275.0	275.0	275.0	275.0	275.0	275.0	275.0	275.0	275.0	275.0
400	Y	66.0	66.0	66.0	66.0	66.0	66.0	66.0	66.0	66.0	66.0	66.0	66.0	66.0	66.0	66.0

LIMITS OF THE GEOMETRY USED DURING COMPUTATIONS

THE LEFT LIMIT IS 11.00.
THE RIGHT LIMIT IS 274.00.

```
*********************
*  SLOPE STABILITY  *
*********************
```

DATE 09-04-85 RUN TRIAL 01

COMMENTS RESERVOIR AT FREEBOARD DS NO EQ

BISHOP*S SIMPLIFIED METHOD

SPECIAL DETAILS ON THE SOIL CONDITIONS

SOIL CHARACTERISTICS
****

THE GEOMETRY CONSISTS OF 4 LAYERS WITH THE FOLLOWING CHARACTERISTICS.

LAYER NO.	TOTAL UNIT WEIGHT	COHESION	ANGLE OF INTERNAL FRICTION	PORE-PRESSURE RATIO RU	SOIL DESCRIPTION
1	62.400	0.000	0.000	0.000	WATER
2	130.000	0.000	37.000	0.000	COMPACTED SM-SC MATERIAL
3	110.000	0.000	30.000	0.000	FOUNDATION SM-SC MATERIAL
4	-1.000	0.000	0.000	0.000	CALICHE

```
*********************
*  SLOPE STABILITY  *
*********************
```

DATE 09-04-85 RUN TRIAL 01

COMMENTS RESERVOIR AT FREEBOARD DS NO EQ

BISHOP*S SIMPLIFIED METHOD

SUMMARY OF MINIMUM FACTORS OF SAFETY

RADIUS INC*T	X-COORD.	Y-COORD.	RADIUS	FACTOR OF SAFETY	NO. ITER*N	MALPHA WARN*G
2	145.0	115.0	15.0	2.321	3	0
4	155.0	115.0	19.0	1.987	3	0
6	165.0	115.0	23.0	1.851	4	0
8	175.0	115.0	29.0	1.705	4	0
11	185.0	115.0	33.0	2.929	3	0
21	195.0	115.0	53.0	5.021	4	0
2	150.0	125.0	27.0	2.160	4	0
5	160.0	125.0	31.0	1.837	4	0
5	170.0	125.0	39.0	1.706	4	0
9	180.0	125.0	39.0	1.759	4	0
12	190.0	125.0	45.0	3.397	3	0
21	200.0	125.0	63.0	4.931	4	0
3	155.0	135.0	37.0	1.962	4	0
5	165.0	135.0	41.0	1.706	4	0
8	175.0	135.0	49.0	1.682	4	0
10	185.0	135.0	51.0	2.083	4	0
16	195.0	135.0	63.0	3.689	3	0
21	205.0	135.0	73.0	4.987	3	0
3	160.0	145.0	49.0	1.980	4	0
6	170.0	145.0	53.0	1.729	4	0
8	180.0	145.0	57.0	1.651	4	0
10	190.0	145.0	61.0	2.348	4	0
21	200.0	145.0	83.0	3.812	3	0
21	210.0	145.0	83.0	5.131	3	0
4	165.0	155.0	59.0	1.870	4	0
6	175.0	155.0	65.0	1.759	4	0
9	185.0	155.0	69.0	1.759	4	0
11	195.0	155.0	73.0	2.765	4	0
21	205.0	155.0	93.0	3.972	3	0
19	215.0	155.0	89.0	6.304	3	0

THE MOST CRITICAL SLIP SURFACE

RADIUS INC*T	X-COORD.	Y-COORD.	RADIUS	FACTOR OF SAFETY	NO. ITER*N	MALPHA WARN*G
8	180.0	145.0	57.0	1.651	4	0

```
**********************
*  SLOPE STABILITY  *     DATE 09-04-85     RUNTRIAL 01     COMMENTSRESERVOIR AT FREEBOARD  DS NO EQ
**********************
BISHOP*S SIMPLIFIED METHOD
155.000 +                                  1.87      1.76      1.76      2.76      3.97      6.30
        :
        :
        :
147.000 +                          1.98      1.73      1.65      2.35      3.81      5.13
        :
        :
        :
139.000 +
        :
        :                  1.96      1.71      1.68      2.08      3.69      4.99
        :
131.000 +
        :
        :
        :            2.16      1.84      1.71      1.76      3.40      4.93
        :
123.000 +
        :
        :
        :
115.000 +      2.32      1.99      1.85      1.70      2.93      5.02
        :
        :
        :
        :
107.000 +
        +.........+.........+.........+.........+.........+.........+.........+.........+.........+.........+
            145.00    153.00    161.00    169.00    177.00    185.00    193.00    201.00    209.00    217.00
        HORIZONTAL SCALE  1 INCH =     8.00                VERTICAL SCALE  1 INCH =     8.00
```

```
*********************
*  SLOPE STABILITY  *     DATE 09-04-85     RUNTRIAL 01     COMMENTSRESERVOIR AT FREEBOARD DS NO EQ
*********************
BISHOP*S SIMPLIFIED METHOD
155.000 +
        :                                                                  *
        :
        :
        :
        :
125.000 +
        :
        :
        :        111111111111111111111111111111111222222222222S2
        :                                      2222            SS22
        :                                2222                    SS2222
 95.000 +                          2222                            SSSS222
        :        3333333333333333333333333333333333333        333333333SSSS3333333333333333333333333333333333
        :                                            3333333
        :
        :
        :        4444444444444444444444444444444444444444444444444444444444444444444444444444444444444444
 65.000 +
        +.........+.........+.........+.........+.........+.........+.........+.........+.........+.........+
              10.00     40.00     70.00    100.00    130.00    160.00    190.00    220.00    250.00    280.00
     HORIZONTAL SCALE  1 INCH =    30.00                VERTICAL SCALE  1 INCH =    30.00
   X =     180.00     Y =    145.00     RADIUS =      57.00     FACTOR OF SAFETY =      1.651
```

```
*********************
*  SLOPE STABILITY  *
*********************
```

DATE 09-04-85 RUN TRIAL 01

COMMENTS RESERVOIR AT FREEBOARD DS NO EQ

BISHOP'S SIMPLIFIED METHOD

THE INTERSECTIONS OF THE SLIP SURFACE WITH THE SURFACE LAYER ARE
LEFT INTERCEPT ..... X= 135.6756 Y= 109.1622
RIGHT INTERCEPT .... X= 177.9244 Y= 88.0378
X- LEFT AND RIGHT INTERCEPTS OF THE SLIP SURFACE AND THE BEDROCK ARE 0.0000 AND 0.0000 RESPECTIVELY
THIS IS ITERATION NUMBER 4

SLICE	X-COOR	Y-TOP	Y-BASE	ALPHA	SLICE COHESION	SLICE WEIGHT	NORMAL FORCE	PORE-WATER FORCE	SLICE MALPHA	RADIUS	NORMAL ARM	PHI ANGLE
126	136.09	108.96	108.66	50.39	0.	.3171E+02	.3209E+02	0.	.9883E+00	57.00	0.00	37.0
127	137.00	108.50	107.58	48.98	0.	.1188E+03	.1188E+03	0.	.9998E+00	57.00	0.00	37.0
128	138.00	106.00	106.46	47.47	0.	.1993E+03	.1971E+03	0.	.1011E+01	57.00	0.00	37.0
129	139.00	107.50	105.40	46.00	0.	.2725E+03	.2666E+03	0.	.1022E+01	57.00	0.00	37.0
130	140.00	107.00	104.39	44.57	0.	.3388E+03	.3283E+03	0.	.1032E+01	57.00	0.00	37.0
131	141.00	106.50	103.43	43.18	0.	.3988E+03	.3831E+03	0.	.1041E+01	57.00	0.00	37.0
132	142.00	106.00	102.51	41.81	0.	.4529E+03	.4318E+03	0.	.1049E+01	57.00	0.00	37.0
133	143.00	105.50	101.64	40.48	0.	.5014E+03	.4748E+03	0.	.1056E+01	57.00	0.00	37.0
134	144.00	105.00	100.81	39.17	0.	.5449E+03	.5126E+03	0.	.1063E+01	57.00	0.00	37.0
135	145.00	104.50	100.01	37.88	0.	.5834E+03	.5458E+03	0.	.1069E+01	57.00	0.00	37.0
136	146.00	104.00	99.25	36.62	0.	.6172E+03	.5746E+03	0.	.1074E+01	57.00	0.00	37.0
137	147.00	103.50	98.52	35.38	0.	.6467E+03	.5994E+03	0.	.1079E+01	57.00	0.00	37.0
138	148.00	103.00	97.83	34.16	0.	.6719E+03	.6204E+03	0.	.1083E+01	57.00	0.00	37.0
139	149.00	102.50	97.17	32.95	0.	.6931E+03	.6378E+03	0.	.1087E+01	57.00	0.00	37.0
140	150.00	102.00	96.53	31.76	0.	.7105E+03	.6519E+03	0.	.1090E+01	57.00	0.00	37.0
141	151.00	101.50	95.93	30.58	0.	.7241E+03	.6628E+03	0.	.1093E+01	57.00	0.00	37.0
142	152.00	101.00	95.35	29.42	0.	.7342E+03	.6707E+03	0.	.1095E+01	57.00	0.00	37.0
143	153.00	100.50	94.80	28.28	0.	.7408E+03	.6757E+03	0.	.1096E+01	57.00	0.00	37.0
144	154.00	100.00	94.28	27.14	0.	.7441E+03	.6779E+03	0.	.1098E+01	57.00	0.00	37.0
145	155.00	99.50	93.78	26.02	0.	.7441E+03	.6775E+03	0.	.1098E+01	57.00	0.00	37.0
146	156.00	99.00	93.30	24.90	0.	.7410E+03	.6744E+03	0.	.1099E+01	57.00	0.00	37.0
147	157.00	98.50	92.85	23.80	0.	.7348E+03	.6688E+03	0.	.1099E+01	57.00	0.00	37.0
148	158.00	98.00	92.42	22.70	0.	.7257E+03	.6608E+03	0.	.1098E+01	57.00	0.00	37.0
149	159.00	97.50	92.01	21.62	0.	.7137E+03	.6503E+03	0.	.1097E+01	57.00	0.00	37.0
150	160.00	97.00	91.62	20.54	0.	.6988E+03	.6375E+03	0.	.1096E+01	57.00	0.00	37.0
151	161.00	96.50	91.26	19.47	0.	.6811E+03	.6223E+03	0.	.1095E+01	57.00	0.00	37.0
152	162.00	96.00	90.92	18.41	0.	.6607E+03	.6047E+03	0.	.1093E+01	57.00	0.00	37.0
153	163.00	95.50	90.59	17.35	0.	.6377E+03	.5849E+03	0.	.1090E+01	57.00	0.00	37.0
154	164.00	95.00	90.29	16.30	0.	.6120E+03	.5627E+03	0.	.1088E+01	57.00	0.00	37.0
155	165.00	94.50	90.01	15.26	0.	.5837E+03	.5382E+03	0.	.1085E+01	57.00	0.00	37.0
156	166.00	94.00	89.75	14.22	0.	.5529E+03	.5114E+03	0.	.1081E+01	57.00	0.00	37.0
157	167.00	93.50	89.50	13.18	0.	.5196E+03	.4822E+03	0.	.1077E+01	57.00	0.00	37.0
158	168.00	93.00	89.28	12.15	0.	.4838E+03	.4507E+03	0.	.1073E+01	57.00	0.00	37.0
159	169.00	92.50	89.07	11.13	0.	.4456E+03	.4168E+03	0.	.1069E+01	57.00	0.00	37.0
160	170.00	92.00	88.88	10.10	0.	.4050E+03	.3805E+03	0.	.1064E+01	57.00	0.00	37.0
161	171.00	91.50	88.72	9.09	0.	.3619E+03	.3417E+03	0.	.1059E+01	57.00	0.00	37.0
162	172.00	91.00	88.56	8.07	0.	.3166E+03	.3003E+03	0.	.1054E+01	57.00	0.00	37.0
163	173.00	90.50	88.43	7.05	0.	.2688E+03	.2564E+03	0.	.1048E+01	57.00	0.00	37.0
164	174.00	90.00	88.32	6.04	0.	.2187E+03	.2098E+03	0.	.1042E+01	57.00	0.00	37.0

165	175.00	89.50	88.22	5.03	0.	.1663E+03	.1605E+03	0.	.1036E+01	57.00	0.00	37.0
166	176.00	89.00	88.14	4.02	0.	.1116E+03	.1084E+03	0.	.1029E+01	57.00	0.00	37.0
167	177.00	88.50	88.08	3.02	0.	.5463E+02	.5343E+02	0.	.1023E+01	57.00	0.00	37.0
168	177.71	88.14	88.05	2.50	0.	.5398E+01	.5305E+01	0.	.1017E+01	57.00	0.00	37.0

HORIZONTAL FORCES ON THE SLOPE DUE TO PARTIAL SUBMERGENCE OR TENSION CRACKS
LEFT SIDE --- FLUID FORCE = 0. AND MOMENT ARM = .3584E+02
RIGHT SIDE --- FLUID FORCE = 0. AND MOMENT ARM = .5696E+02

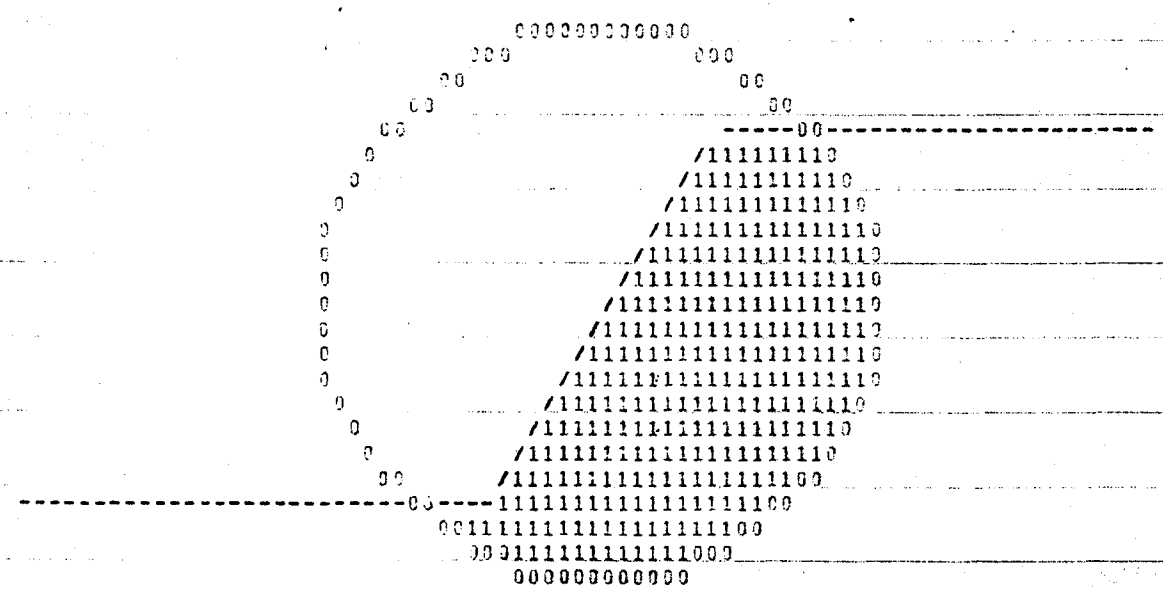
RADIUS INC*T	X-COORD.	Y-COORD.	RADIUS	FACTOR OF SAFETY	NO. ITER*N	MALPHA WARN*G
1	180.0	145.0	57.0	1.651	4	0

Reservoir @ freeboard storm elevation
Acceleration (0.18)
Downstream slope

```
DDDDDDDDDDD   TTTTTTTTTTTT  LL          NN        NN
DDDDDDDDDDD   TTTTTTTTTTTT  LL          NNN       NN
DD        DD      TT        LL          NNNN      NN
DD        DD      TT        LL          NN NN     NN
DD        DD      TT        LL          NN  NN    NN
DD        DD      TT        LL          NN   NN   NN
DD        DD      TT        LL          NN    NN  NN
DD        DD      TT        LL          NN     NN NN
DD        DD      TT        LL          NN      NNNN
DD        DD      TT        LL          NN       NNN
DDDDDDDDDDDD      TT        LLLLLLLLLLLL NN        NN
DDDDDDDDDDD       TT        LLLLLLLLLLLL NN         N

          JJ  CCCCCCCCCCC  BBBBBBBBBBB          555555555555  9999999999      11        00000000
          JJ  CCCCCCCCCCC  BBBBBBBBBBB          555555555555  999999999999    111       0000000000
          JJ  CC       CC  BB       BB          55            99       99     1111      00       00
          JJ  CC       CC  BB       BB          55            99       99       11      00       00
          JJ  CC       CC  BB       BB          55            99       99       11      00       00
          JJ  CC       CC  BBBBBBBBBBB          55555555555   999999999999     11      00       00
          JJ  CC       CC  BBBBBBBBBBB          555555555555  99999999999      11      00       00
          JJ  CC       CC  BB       BB                   55          99       11      00       00
JJ        JJ  CC       CC  BB       BB                   55          99       11      00       00
JJ        JJ  CC       CC  BB       BB          55       55  99       99      11      00       00
JJJJJJJJJJJJ  CCCCCCCCCCC  BBBBBBBBBBB          555555555555  999999999999  1111111111  0000000000
 JJJJJJJJJJ   CCCCCCCCCCC  BBBBBBBBBBB           5555555555   9999999999   1111111111  00000000

RRRRRRRRRR     2222222222      11      2222222222    66666666666  PPPPPPPPPPP   RRRRRRRRRR      11
RRRRRRRRRRR   222222222222    111     222222222222  666666666666  PPPPPPPPPPPP  RRRRRRRRRRR    111
RR        RR  22        22    1111    22        22  66            PP        PP  RR        RR   1111
RR        RR            22      11              22  66            PP        PP  RR        RR     11
RR        RR            22      11              22  66            PP        PP  RR        RR     11
RRRRRRRRRRR            22       11             22   66666666666   PPPPPPPPPPP  RRRRRRRRRRR     11
RRRRRRRRRRR          22         11           22     666666666666  PPPPPPPPPPP  RRRRRRRRRRR     11
RR     RR          22           11         22       66        66  PP           RR     RR       11
RR      RR       22             11       22         66        66  PP           RR      RR      11
RR       RR    22               11     22           66        66  PP           RR       RR     11
RR        RR   222222222222  1111111111  222222222222  666666666666  PP           RR        RR  1111111111
RR        RR   222222222222  1111111111  222222222222   6666666666  PP           RR        RR  1111111111

PPPPPPPPPPP   RRRRRRRRRRR   TTTTTTTTTTTT     11            44     00000000     3333333333
PPPPPPPPPPPP  RRRRRRRRRRRR  TTTTTTTTTTTT    111           444     0000000000   333333333333
PP        PP  RR        RR      TT        1111          4444     00       00  33        33
PP        PP  RR        RR      TT          11         44 44     00       00            33
PP        PP  RR        RR      TT          11        44  44     00       00            33
PPPPPPPPPPPP  RRRRRRRRRRRR      TT          11       44   44     00       00        3333
PPPPPPPPPPP   RRRRRRRRRRR       TT          11      44    44     00       00        3333
PP            RR     RR         TT          11     4444444444   00       00            33
PP            RR      RR        TT          11    44444444444   00       00            33
PP            RR       RR       TT          11            44    00       00  33        33
PP            RR        RR      TT        1111111111      44     0000000000   333333333333
PP            RR         RR     TT        1111111111      44      00000000     3333333333
```

RUN DATE 85/11/12.
TIME 13.42.52.

```
**************
*  SLOPE-II  *  IS A FULLY MAINTAINED, PROPRIETARY PRODUCT OF
**************

                GEO-SLOPE PROGRAMMING LTD.,
                7927 SILVER SPRINGS ROAD N.W.,
                CALGARY, ALBERTA, CANADA, T3B 4K4
```



```
                PLEASE NOTIFY GEO-SLOPE PROGRAMMING LTD., OF ANY
                PROBLEMS ENCOUNTERED WITH SLOPE-II.

                DISCLAIMER:
                NO WARRANTY IS EXPRESSED OR IMPLIED AS TO THE
                DOCUMENTATION FUNCTION OR PERFORMANCE OF THE
                PROGRAM AND THE USER OF THE PROGRAM IS EXPECTED
                TO MAKE THE FINAL EVALUATION IN THE CONTEXT OF
                THE PROBLEM UNDER CONSIDERATION.

                VERSION 2-A   RELEASE 5.0
```

```
*********************
*  SLOPE STABILITY  *
*********************

DATE  09-04-85                    RUN  TRIAL 02

COMMENTS        RESERVOIR AT FREEBOARD  DS EARTHQUAKE 0.1 G

 BISHOP*S SIMPLIFIED METHOD

        * DETAILS ON THE SLIP CIRCLE GRID

   LAYOUT OF GRID FOR CENTERS OF TRIAL SLIP CIRCLES
   ************************************************

   THE X-COORDINATE OF THE LOWER LEFT CORNER OF THE GRID IS   145.0000
   THE CENTERS ARE INCREMENTED    5   TIMES IN THE X-DIRECTION WITH EACH
   INCREMENT EQUAL TO      10.0000

   THE Y-COORDINATE OF THE LOWER LEFT CORNER OF THE GRID IS   115.0000
   THE CENTERS ARE INCREMENTED     4    TIMES IN THE Y-DIRECTION WITH EACH
   INCREMENT EQUAL TO      10.0000

   THE GRID HAS A SLOPE(CHANGE IN ABSCISSA DIVIDED BY CHANGE IN ORDINATE)
   OF        .5000.


   OTHER PARTICULARS FOR THE ANALYSIS
   **********************************

   THE Y-COORDINATE OF A HORIZONTAL LINE TANGENT TO THE SLIP SURFACE OF
   MINIMUM RADIUS IS   106.0000.

   FOR EACH CENTER, RADIUS IS INCREMENTED    20 TIMES WITH EACH INCREMENT
   EQUAL TO     2.0000
   THE TOTAL NUMBER OF STABILITY ANALYSIS IS      630.

   THE GEOMETRY IS DIVIDED INTO      264 SLICES, EACH     1.0000

   THE DEPTH OF THE DEEPEST SLICE MUST BE GREATER THAN     5.0000

   THE TOLERANCE IN CALCULATING THE FACTOR OF SAFETY IS     .0100

   THE SEISMIC COEFFICIENT IS EQUAL TO        .1000

   THE UNIT WEIGHT OF WATER IS    62.4000

   THE CONTROL QUESTIONS ANSWERED
    1.) PRINT OUT THE INPUT DATA.  IQ(1)=2
    2.) WRITE OUT ALL FACTORS OF SAFETY.  IQ(2)=1
    3.) WRITE OUT A SUMMARY OF THE MINIMUM FACTORS OF SAFETY.  IQ(3)=2
    4.) WRITE OUT A DETAILED SUMMARY OF THE COMPUTATIONS  IQ(4)=2
    7.) DISREGARD THE PREVIOUS PORE PRESSURE DATA.  IQ(7)=1
    8.) SET ALL NEGATIVE PORE PRESSURES TO ZERO.  IQ(8)=1
    9.) PRINT A DIGITAL PLOT OF THE GEOMETRY AND SLIP SURFACE.  IQ(9)=1
```

```
*********************
*  SLOPE STABILITY  *
*********************
```

DATE 09-04-85 RUN TRIAL 02

COMMENTS RESERVOIR AT FREEBOARD DS EARTHQUAKE 0.1 G

BISHOP'S SIMPLIFIED METHOD

SPECIAL DETAILS ON THE LAYOUT OF THE CROSS-SECTION

THE COORDINATES OF THE VERTICES OF LINES FORMING THE CONTACTS BETWEEN SOIL LAYERS ARE AS FOLLOWS

LINE NO.	COORDINATES	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
100	X	10.0	118.5	120.0	134.0	178.0	275.0	275.0	275.0	275.0	275.0	275.0	275.0	275.0	275.0	275.0
100	Y	109.4	109.4	110.0	110.0	88.0	86.0	86.0	86.0	86.0	86.0	86.0	86.0	86.0	86.0	86.0
200	X	10.0	65.0	118.5	120.0	134.0	178.0	275.0	275.0	275.0	275.0	275.0	275.0	275.0	275.0	275.0
200	Y	88.0	88.0	109.4	110.0	110.0	88.0	86.0	86.0	86.0	86.0	86.0	86.0	86.0	86.0	86.0
300	X	10.0	65.0	112.5	120.0	135.0	142.5	176.0	275.0	275.0	275.0	275.0	275.0	275.0	275.0	275.0
300	Y	88.0	88.0	88.0	83.0	83.0	88.0	88.0	86.0	86.0	86.0	86.0	86.0	86.0	86.0	86.0
400	X	10.0	275.0	275.0	275.0	275.0	275.0	275.0	275.0	275.0	275.0	275.0	275.0	275.0	275.0	275.0
400	Y	66.0	66.0	66.0	66.0	66.0	66.0	66.0	66.0	66.0	66.0	66.0	66.0	66.0	66.0	66.0

LIMITS OF THE GEOMETRY USED DURING COMPUTATIONS

THE LEFT LIMIT IS 11.00.
THE RIGHT LIMIT IS 274.00.

```
*********************
*  SLOPE STABILITY  *
*********************
```

DATE 09-04-85 RUN TRIAL 02

COMMENTS RESERVOIR AT FREEBOARD DS EARTHQUAKE 0.1 G

BISHOP'S SIMPLIFIED METHOD

SPECIAL DETAILS ON THE SOIL CONDITIONS

SOIL CHARACTERISTICS
****
THE GEOMETRY CONSISTS OF 4 LAYERS WITH THE FOLLOWING CHARACTERISTICS.

LAYER NO.	TOTAL UNIT WEIGHT	COHESION	ANGLE OF INTERNAL FRICTION	PORE-PRESSURE RATIO RU	SOIL DESCRIPTION
1	62.400	0.000	0.000	0.000	WATER
2	130.000	0.000	37.000	0.000	COMPACTED SM-SC MATERIAL
3	110.000	0.000	30.000	0.000	FOUNDATION SM-SC MATERIAL
4	-1.000	0.000	0.000	0.000	CALICHE

```
*********************
*  SLOPE STABILITY  *
*********************
```

DATE 09-04-85 RUN TRIAL 02

COMMENTS RESERVOIR AT FREEBOARD DS EARTHQUAKE 0.1 G

BISHOP*S SIMPLIFIED METHOD

SUMMARY OF MINIMUM FACTORS OF SAFETY

RADIUS INC*T	X-COORD.	Y-COORD.	RADIUS	FACTOR OF SAFETY	NO. ITER*N	MALPHA WARN*G
2	145.0	115.0	15.0	1.860	3	0
4	155.0	115.0	19.0	1.600	3	0
6	165.0	115.0	23.0	1.486	4	0
8	175.0	115.0	29.0	1.359	3	0
11	185.0	115.0	33.0	2.080	3	0
21	195.0	115.0	53.0	3.310	4	0
2	150.0	125.0	27.0	1.699	4	0
5	160.0	125.0	31.0	1.473	4	0
9	170.0	125.0	39.0	1.368	3	0
9	180.0	125.0	39.0	1.378	4	0
12	190.0	125.0	45.0	2.298	3	0
21	200.0	125.0	63.0	3.166	4	0
3	155.0	135.0	37.0	1.538	4	0
5	165.0	135.0	41.0	1.362	4	0
8	175.0	135.0	49.0	1.337	4	0
10	185.0	135.0	51.0	1.578	4	0
14	195.0	135.0	59.0	2.460	3	0
21	205.0	135.0	73.0	3.112	3	0
3	160.0	145.0	49.0	1.543	4	0
6	170.0	145.0	53.0	1.373	4	0
8	180.0	145.0	57.0	1.316	4	0
10	190.0	145.0	61.0	1.724	4	0
16	200.0	145.0	73.0	2.561	3	0
21	210.0	145.0	83.0	3.107	3	0
4	165.0	155.0	59.0	1.460	4	0
6	175.0	155.0	65.0	1.391	4	0
9	185.0	155.0	69.0	1.376	4	0
11	195.0	155.0	73.0	1.948	3	0
21	205.0	155.0	93.0	2.622	3	0
19	215.0	155.0	89.0	3.405	3	0

THE MOST CRITICAL SLIP SURFACE

RADIUS INC*T	X-COORD.	Y-COORD.	RADIUS	FACTOR OF SAFETY	NO. ITER*N	MALPHA WARN*G
8	180.0	145.0	57.0	1.316	4	0

```
*********************
* SLOPE STABILITY   *     DATE 09-04-85     RUNTRIAL 02     COMMENTSRESERVOIR AT FREEBOARD  DS EARTHQUAKE 0.1 G
*********************
BISHOP*S SIMPLIFIED METHOD
155.000 +                                   1.46        1.39        1.38        1.95        2.62        3.41
        .
        .
        .
147.000 +                           1.54        1.37        1.32        1.72        2.56        3.11
        .
        .
        .
139.000 +
        .
        .                   1.54        1.36        1.34        1.58        2.46        3.11
        .
131.000 +
        .
        .           1.70        1.47        1.37        1.38        2.30        3.17
        .
123.000 +
        .
        .
        .
115.000 +       1.86        1.60        1.49        1.36        2.08        3.31
        .
        .
        .
107.000 +
        +.........+.........+.........+.........+.........+.........+.........+.........+.........+.........+
            145.00    153.00    161.00    169.00    177.00    185.00    193.00    201.00    209.00    217.00
        HORIZONTAL SCALE  1 INCH =     8.00                VERTICAL SCALE  1 INCH =     8.00
```

```
*********************
*  SLOPE STABILITY  *     DATE 09-04-85     RUNTRIAL 02     COMMENTSRESERVOIR AT FREEBOARD  DS EARTHQUAKE 0.1 G
*********************
BISHOP'S SIMPLIFIED METHOD
155.000 +
        .                                                                                  *
        .
        .
        .
        .
125.000 +
        .
        .
        .        1111111111111111111111111111111111222222222222S2
        .                                      2222                SS22
        .                                2222                        SS2222
 95.000 +                          2222                                SSSS222
        .        3333333333333333333333333333333333333        333333333SSSS3333333333333333333333333333333333
        .                                            3333333
        .
        .
        .        44444444444444444444444444444444444444444444444444444444444444444444444444444444444444444444
 65.000 +
        +.........+.........+.........+.........+.........+.........+.........+.........+.........+.........+
              10.00     40.00     70.00    100.00    130.00    160.00    190.00    220.00    250.00    280.00
          HORIZONTAL SCALE  1 INCH =    30.00                    VERTICAL SCALE  1 INCH =    30.00
       X =      180.00     Y =      145.00     RADIUS =      57.00     FACTOR OF SAFETY =      1.316
```

```
*********************
*  SLOPE STABILITY  *
*********************
```

DATE 09-04-85 RUN TRIAL 02

COMMENTS RESERVOIR AT FREEBOARD DS EARTHQUAKE 0.1 G

BISHOP'S SIMPLIFIED METHOD

THE INTERSECTIONS OF THE SLIP SURFACE WITH THE SURFACE LAYER ARE
LEFT INTERCEPT ..... X= 135.6756 Y= 109.1622
RIGHT INTERCEPT .... X= 177.9244 Y= 88.0378
X- LEFT AND RIGHT INTERCEPTS OF THE SLIP SURFACE AND THE BEDROCK ARE 0.0000 AND 0.0000 RESPECTIVELY
THIS IS ITERATION NUMBER 4

SLICE	X-COOR	Y-TOP	Y-BASE	ALPHA	SLICE COHESION	SLICE WEIGHT	NORMAL FORCE	PORE-WATER FORCE	SLICE MALPHA	RADIUS	NORMAL ARM	PHI ANGLE
126	136.09	108.96	108.66	50.39	0.	.3171E+02	.2944E+02	0.	.1077E+01	57.00	0.00	37.0
127	137.00	108.50	107.58	48.98	0.	.1188E+03	.1093E+03	0.	.1087E+01	57.00	0.00	37.0
128	138.00	108.00	106.46	47.47	0.	.1993E+03	.1818E+03	0.	.1096E+01	57.00	0.00	37.0
129	139.00	107.50	105.40	46.00	0.	.2725E+03	.2466E+03	0.	.1105E+01	57.00	0.00	37.0
130	140.00	107.00	104.39	44.57	0.	.3388E+03	.3044E+03	0.	.1113E+01	57.00	0.00	37.0
131	141.00	106.50	103.43	43.18	0.	.3988E+03	.3562E+03	0.	.1120E+01	57.00	0.00	37.0
132	142.00	106.00	102.51	41.81	0.	.4529E+03	.4023E+03	0.	.1126E+01	57.00	0.00	37.0
133	143.00	105.50	101.64	40.48	0.	.5014E+03	.4434E+03	0.	.1131E+01	57.00	0.00	37.0
134	144.00	105.00	100.81	39.17	0.	.5449E+03	.4798E+03	0.	.1136E+01	57.00	0.00	37.0
135	145.00	104.50	100.01	37.88	0.	.5834E+03	.5119E+03	0.	.1140E+01	57.00	0.00	37.0
136	146.00	104.00	99.25	36.62	0.	.6172E+03	.5401E+03	0.	.1143E+01	57.00	0.00	37.0
137	147.00	103.50	98.52	35.38	0.	.6467E+03	.5645E+03	0.	.1146E+01	57.00	0.00	37.0
138	148.00	103.00	97.83	34.16	0.	.6719E+03	.5854E+03	0.	.1148E+01	57.00	0.00	37.0
139	149.00	102.50	97.17	32.95	0.	.6931E+03	.6030E+03	0.	.1149E+01	57.00	0.00	37.0
140	150.00	102.00	96.53	31.76	0.	.7105E+03	.6175E+03	0.	.1151E+01	57.00	0.00	37.0
141	151.00	101.50	95.93	30.58	0.	.7241E+03	.6291E+03	0.	.1151E+01	57.00	0.00	37.0
142	152.00	101.00	95.35	29.42	0.	.7342E+03	.6377E+03	0.	.1151E+01	57.00	0.00	37.0
143	153.00	100.50	94.80	28.28	0.	.7408E+03	.6437E+03	0.	.1151E+01	57.00	0.00	37.0
144	154.00	100.00	94.28	27.14	0.	.7441E+03	.6470E+03	0.	.1150E+01	57.00	0.00	37.0
145	155.00	99.50	93.78	26.02	0.	.7441E+03	.6477E+03	0.	.1149E+01	57.00	0.00	37.0
146	156.00	99.00	93.30	24.90	0.	.7410E+03	.6459E+03	0.	.1147E+01	57.00	0.00	37.0
147	157.00	98.50	92.85	23.80	0.	.7348E+03	.6417E+03	0.	.1145E+01	57.00	0.00	37.0
148	158.00	98.00	92.42	22.70	0.	.7257E+03	.6351E+03	0.	.1143E+01	57.00	0.00	37.0
149	159.00	97.50	92.01	21.62	0.	.7137E+03	.6261E+03	0.	.1140E+01	57.00	0.00	37.0
150	160.00	97.00	91.62	20.54	0.	.6988E+03	.6148E+03	0.	.1137E+01	57.00	0.00	37.0
151	161.00	96.50	91.26	19.47	0.	.6811E+03	.6012E+03	0.	.1133E+01	57.00	0.00	37.0
152	162.00	96.00	90.92	18.41	0.	.6607E+03	.5852E+03	0.	.1129E+01	57.00	0.00	37.0
153	163.00	95.50	90.59	17.35	0.	.6377E+03	.5670E+03	0.	.1125E+01	57.00	0.00	37.0
154	164.00	95.00	90.29	16.30	0.	.6120E+03	.5464E+03	0.	.1120E+01	57.00	0.00	37.0
155	165.00	94.50	90.01	15.26	0.	.5837E+03	.5236E+03	0.	.1115E+01	57.00	0.00	37.0
156	166.00	94.00	89.75	14.22	0.	.5529E+03	.4984E+03	0.	.1109E+01	57.00	0.00	37.0
157	167.00	93.50	89.50	13.18	0.	.5196E+03	.4708E+03	0.	.1104E+01	57.00	0.00	37.0
158	168.00	93.00	89.28	12.15	0.	.4838E+03	.4408E+03	0.	.1098E+01	57.00	0.00	37.0
159	169.00	92.50	89.07	11.13	0.	.4456E+03	.4083E+03	0.	.1091E+01	57.00	0.00	37.0
160	170.00	92.00	88.88	10.10	0.	.4050E+03	.3734E+03	0.	.1085E+01	57.00	0.00	37.0
161	171.00	91.50	88.72	9.09	0.	.3619E+03	.3359E+03	0.	.1078E+01	57.00	0.00	37.0
162	172.00	91.00	88.56	8.07	0.	.3166E+03	.2958E+03	0.	.1070E+01	57.00	0.00	37.0
163	173.00	90.50	88.43	7.05	0.	.2688E+03	.2530E+03	0.	.1062E+01	57.00	0.00	37.0
164	174.00	90.00	88.32	6.04	0.	.2187E+03	.2074E+03	0.	.1054E+01	57.00	0.00	37.0

165	175.03	89.50	88.22	5.03	0.	.1663E+03	.1590E+03	0.	.1046E+01	57.00	0.00	37.0
166	176.35	89.00	88.14	4.02	0.	.1116E+03	.1076E+03	0.	.1038E+01	57.00	0.00	37.0
167	177.73	88.50	88.08	3.02	0.	.5463E+02	.5311E+02	0.	.1029E+01	57.00	0.00	37.0
168	177.71	88.14	88.05	2.30	0.	.5358E+01	.5281E+01	0.	.1022E+01	57.00	0.00	37.0

HORIZONTAL FORCES ON THE SLOPE DUE TO PARTIAL SUBMERGENCE OR TENSION CRACKS
LEFT SIDE --- FLUID FORCE = 0. AND MOMENT ARM = .3584E+02
RIGHT SIDE --- FLUID FORCE = 0. AND MOMENT ARM = .5696E+02

RADIUS INC*T	X-COORD.	Y-COORD.	RADIUS	FACTOR OF SAFETY	NO. ITER*N	MALPHA WARN*G
1	180.0	145.0	57.0	1.316	4	0

Reservoir @ crest of emergency spillway
Steady State Seepage
No Acceleration
Downstream Slope

```
DDDDDDDDDDD   TTTTTTTTTTTT  HH        HH   9999999999
DDDDDDDDDDDD  TTTTTTTTTTTT  HH        HH  999999999999
DD        DD        TT      HH        HH  99        99
DD        DD        TT      HH        HH  99        99
DD        DD        TT      HH        HH  99        99
DD        DD        TT      HHHHHHHHHHHH  999999999999
DD        DD        TT      HHHHHHHHHHHH   99999999999
DD        DD        TT      HH        HH            99
DD        DD        TT      HH        HH            99
DD        DD        TT      HH        HH  99        99
DDDDDDDDDDDD        TT      HH        HH  999999999999
DDDDDDDDDDD         TT      HH        HH   9999999999

          JJ  CCCCCCCCCCCC  BBBBBBBBBBBB                555555555555  8888888888          44    777777777777
          JJ  CCCCCCCCCCCC  BBBBBBBBBBBB                555555555555  888888888888        444    77777777777
          JJ  CC        CC  BB        BB                55            88        88      4444    77        77
          JJ  CC        CC  BB        BB                55            88        88    44 44           77
          JJ  CC        CC  BB        BB                55            88        88   44  44          77
          JJ  CC        CC  BBBBBBBBBBB                 55555555555    8888888888   44   44         77
          JJ  CC        CC  BBBBBBBBBBB                 555555555555   8888888888  44    44         77
          JJ  CC        CC  BB        BB                        55  88        88  4444444444         77
JJ        JJ  CC        CC  BB        BB                        55  88        88  444444444444       77
JJ        JJ  CC        CC  BB        BB                55      55  88        88          44         77
JJJJJJJJJJJJ  CCCCCCCCCCCC  BBBBBBBBBBBB                555555555555  888888888888        44         77
 JJJJJJJJJJ   CCCCCCCCCCCC  BBBBBBBBBBB                 5555555555    8888888888          44         77

RRRRRRRRRRR    2222222222      11      2222222222   6666666666666  PPPPPPPPPPP   RRRRRRRRRRR      11
RRRRRRRRRRRR  222222222222    111    222222222222  666666666666   PPPPPPPPPPPP  RRRRRRRRRRRR    111
RR        RR  22        22   1111    22        22  66            PP        PP  RR        RR   1111
RR        RR            22     11                22  66            PP        PP  RR        RR     11
RR        RR            22     11                22  66            PP        PP  RR        RR     11
RRRRRRRRRRRR           22      11               22   66666666666   PPPPPPPPPPPP  RRRRRRRRRRRR     11
RRRRRRRRRRR          22        11             22     666666666666  PPPPPPPPPPP   RRRRRRRRRRR      11
RR      RR         22          11           22       66        66  PP            RR      RR       11
RR       RR      22            11         22         66        66  PP            RR       RR      11
RR        RR   22              11       22           66        66  PP            RR        RR     11
RR         RR  222222222222  1111111111  222222222222  666666666666  PP            RR         RR  1111111111
RR         RP  222222222222  1111111111  222222222222   6666666666   PP            RR         RR  1111111111

PPPPPPPPPPP   RRRRRRRRRRR   TTTTTTTTTTTT      11            44     00000000     3333333333
PPPPPPPPPPPP  RRRRRRRRRRRR  TTTTTTTTTTTT     111           444     0000000000   333333333333
PP        PP  RR        RR        TT        1111          4444   00        00  33        33
PP        PP  RR        RR        TT          11        44 44    00        00            33
PP        PP  RR        RR        TT          11       44  44    00        00            33
PPPPPPPPPPPP  RRRRRRRRRRRR        TT          11      44   44    00        00        3333
PPPPPPPPPPP   RRRRRRRRRRR         TT          11     44    44    00        00        3333
PP            RR      RR          TT          11   44444444444 00        00            33
PP            RR       RR         TT          11  444444444444 00        00            33
PP            RR        RR        TT          11            44 00        00  33        33
PP            RR         RR       TT        1111111111      44  0000000000  333333333333
PP            RR          RR      TT        1111111111      44   00000000   3333333333
```

```
*********************
*  SLOPE STABILITY  *
*********************

DATE  09-04-85                    RUN  TRIAL 04

COMMENTS          STEADY STATE SEEPAGE  DS 00 EG

 BISHOP*S SIMPLIFIED METHOD

          DETAILS ON THE SLIP CIRCLE GRID

   LAYOUT OF GRID FOR CENTERS OF TRIAL SLIP CIRCLES
   ************************************************

   THE X-COORDINATE OF THE LOWER LEFT CORNER OF THE GRID IS   145.0000
   THE CENTERS ARE INCREMENTED    5   TIMES IN THE X-DIRECTION WITH EACH
   INCREMENT EQUAL TO      10.0000

   THE Y-COORDINATE OF THE LOWER LEFT CORNER OF THE GRID IS  115.0000
   THE CENTERS ARE INCREMENTED     4    TIMES IN THE Y-DIRECTION WITH EACH
   INCREMENT EQUAL TO      10.0000

   THE GRID HAS A SLOPE(CHANGE IN ABSCISSA DIVIDED BY CHANGE IN ORDINATE)
   OF        .5000.

   OTHER PARTICULARS FOR THE ANALYSIS
   **********************************

   THE Y-COORDINATE OF A HORIZONTAL LINE TANGENT TO THE SLIP SURFACE OF
   MINIMUM RADIUS IS   106.0000.

   FOR EACH CENTER, RADIUS IS INCREMENTED    20 TIMES WITH EACH INCREMENT
   EQUAL TO      2.0000
   THE TOTAL NUMBER OF STABILITY ANALYSIS IS      630.

   THE GEOMETRY IS DIVIDED INTO     264 SLICES, EACH     1.0000

   THE DEPTH OF THE DEEPEST SLICE MUST BE GREATER THAN     5.0000

   THE TOLERANCE IN CALCULATING THE FACTOR OF SAFETY IS     .0100

   THE SEISMIC COEFFICIENT IS EQUAL TO      0.0000

   THE UNIT WEIGHT OF WATER IS    62.4000

   THE CONTROL QUESTIONS ANSWERED
    1.) PRINT OUT THE INPUT DATA.  IQ(1)=2
    2.) WRITE OUT ALL FACTORS OF SAFETY.  IQ(2)=1
    3.) WRITE OUT A SUMMARY OF THE MINIMUM FACTORS OF SAFETY.  IQ(3)=2
    4.) WRITE OUT A DETAILED SUMMARY OF THE COMPUTATIONS  IQ(4)=2
    7.) DISREGARD THE PREVIOUS PORE PRESSURE DATA.  IQ(7)=1
    8.) SET ALL NEGATIVE PORE PRESSURES TO ZERO.  IQ(8)=1
    9.) PRINT A DIGITAL PLOT OF THE GEOMETRY AND SLIP SURFACE.  IQ(9)=1
```

```
*********************
*  SLOPE STABILITY  *
*********************
```

DATE 09-04-85 RUN TRIAL 04

COMMENTS STEADY STATE SEEPAGE DS NO EQ

BISHOP'S SIMPLIFIED METHOD

SPECIAL DETAILS ON THE LAYOUT OF THE CROSS-SECTION

THE COORDINATES OF THE VERTICES OF LINES FORMING THE CONTACTS BETWEEN SOIL LAYERS ARE AS FOLLOWS

LINE NO.	COORDINATES	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
100	X	10.0	94.2	120.0	134.0	178.0	275.0	275.0	275.0	275.0	275.0	275.0	275.0	275.0	275.0	275.0
100	Y	99.7	99.7	110.0	110.0	88.0	86.0	86.0	86.0	86.0	86.0	86.0	86.0	86.0	86.0	86.0
200	X	10.0	65.0	94.2	120.0	134.0	178.0	275.0	275.0	275.0	275.0	275.0	275.0	275.0	275.0	275.0
200	Y	88.0	88.0	99.7	110.0	110.0	88.0	86.0	86.0	86.0	86.0	86.0	86.0	86.0	86.0	86.0
300	X	10.0	65.0	112.5	120.0	135.0	142.5	178.0	275.0	275.0	275.0	275.0	275.0	275.0	275.0	275.0
300	Y	88.0	88.0	88.0	83.0	63.0	88.0	88.0	86.0	86.0	86.0	86.0	86.0	86.0	86.0	86.0
400	X	10.0	275.0	275.0	275.0	275.0	275.0	275.0	275.0	275.0	275.0	275.0	275.0	275.0	275.0	275.0
400	Y	66.0	66.0	66.0	66.0	66.0	66.0	66.0	66.0	66.0	66.0	66.0	66.0	66.0	66.0	66.0

LIMITS OF THE GEOMETRY USED DURING COMPUTATIONS

THE LEFT LIMIT IS 11.00.
THE RIGHT LIMIT IS 274.00.

```
*********************
*  SLOPE STABILITY  *
*********************
```

DATE 09-04-85 RUN TRIAL 04

COMMENTS STEADY STATE SEEPAGE DS NO EG

BISHOP'S SIMPLIFIED METHOD

SPECIAL DETAILS ON THE SOIL CONDITIONS

SOIL CHARACTERISTICS
****
THE GEOMETRY CONSISTS OF 4 LAYERS WITH THE FOLLOWING CHARACTERISTICS.

LAYER NO.	TOTAL UNIT WEIGHT	COHESION	ANGLE OF INTERNAL FRICTION	PORE-PRESSURE RATIO RU	SOIL DESCRIPTION
1	62.400	0.000	0.000	0.000	WATER
2	130.000	0.000	37.000	0.000	COMPACTED SM-SC MATERIAL
3	110.000	0.000	30.000	0.000	FOUNDATION SM-SC MATERIAL
4	-1.000	0.000	0.000	0.000	CALICHE

```
*********************
*  SLOPE STABILITY  *
*********************
```

DATE 09-04-85 RUN TRIAL 04

COMMENTS STEADY STATE SEEPAGE OS NO EQ

BISHOP'S SIMPLIFIED METHOD
SPECIAL DETAILS ON THE PORE PRESSURE CONDITIONS

THE COORDINATES OF THE VERTICES OF LINES FORMING THE PIEZOMETRIC LINES ARE AS FOLLOWS,

LINE NO.	COORDINATES	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
1	X	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
1	Y	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
2	X	0.0	94.2	170.0	178.0	275.0	275.0	275.0	275.0	275.0	275.0	275.0	275.0	275.0	275.0	275.0
2	Y	99.7	99.7	92.0	88.3	85.0	85.0	85.0	85.0	85.0	85.0	85.0	85.0	85.0	85.0	85.0
3	X	0.0	65.0	178.0	275.0	275.0	275.0	275.0	275.0	275.0	275.0	275.0	275.0	275.0	275.0	275.0
3	Y	99.7	99.7	88.0	85.0	85.0	85.0	85.0	85.0	85.0	85.0	85.0	85.0	85.0	85.0	85.0
4	X	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
4	Y	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0

```
*********************
*  SLOPE STABILITY  *
*********************
```

DATE 09-04-85 RUN TRIAL 34

COMMENTS STEADY STATE SEEPAGE US NO EQ

BISHOP*S SIMPLIFIED METHOD

SUMMARY OF MINIMUM FACTORS OF SAFETY

RADIUS INC*T	X-COORD.	Y-COORD.	RADIUS	FACTOR OF SAFETY	NO. ITER*N	MALPHA WARN*G
2	145.0	115.0	15.0	2.321	3	0
14	155.0	115.0	39.0	1.900	4	0
12	165.0	115.0	35.0	1.485	4	0
8	175.0	115.0	29.0	1.323	3	0
11	185.0	115.0	33.0	1.699	4	0
21	195.0	115.0	53.0	2.636	4	0
2	150.0	125.0	27.0	2.160	4	0
12	160.0	125.0	45.0	1.680	3	0
11	170.0	125.0	43.0	1.399	3	0
9	180.0	125.0	39.0	1.338	4	0
13	190.0	125.0	47.0	1.942	4	0
21	200.0	125.0	63.0	2.598	4	0
3	155.0	135.0	37.0	1.962	4	0
11	165.0	135.0	53.0	1.588	3	0
9	175.0	135.0	51.0	1.396	4	0
10	185.0	135.0	51.0	1.470	4	0
21	195.0	135.0	73.0	2.071	4	0
21	205.0	135.0	73.0	2.634	4	0
10	160.0	145.0	63.0	1.921	4	0
11	170.0	145.0	63.0	1.564	4	0
8	180.0	145.0	57.0	1.406	4	0
10	190.0	145.0	61.0	1.611	4	0
21	200.0	145.0	83.0	2.122	4	0
21	210.0	145.0	83.0	2.713	4	0
11	165.0	155.0	73.0	1.848	4	0
9	175.0	155.0	71.0	1.577	4	0
9	185.0	155.0	69.0	1.454	4	0
13	195.0	155.0	77.0	1.775	3	0
21	205.0	155.0	93.0	2.205	4	0
19	215.0	155.0	89.0	3.260	4	0

THE MOST CRITICAL SLIP SURFACE

RADIUS INC*T	X-COORD.	Y-COORD.	RADIUS	FACTOR OF SAFETY	NO. ITER*N	MALPHA WARN*G
8	175.0	115.0	29.0	1.323	3	0

```
*********************
*  SLOPE STABILITY  *     DATE 09-04-85     RUNTRIAL 04     COMMENTS  STEADY STATE SEEPAGE  DS NO EQ
*********************
BISHOP'S SIMPLIFIED METHOD
155.000 +                                 1.85      1.58      1.45      1.78      2.21      3.26
        .
        .
        .
147.000 +                         1.92      1.56      1.41      1.61      2.12      2.71
        .
        .
        .
139.000 +
        .
        .             1.96      1.59      1.40      1.47      2.07      2.63
        .
131.000 +
        .
        .
        .         2.16      1.68      1.40      1.34      1.94      2.60
        .
123.000 +
        .
        .
        .
115.000 +     2.32      1.90      1.48      1.32      1.70      2.64
        .
        .
        .
107.000 +
        +.........+.........+.........+.........+.........+.........+.........+.........+.........+.........+
            145.00    153.00    161.00    169.00    177.00    185.00    193.00    201.00    209.00    217.00
      HORIZONTAL SCALE  1 INCH =     8.00                VERTICAL SCALE  1 INCH =     8.00
```

```
*********************
* SLOPE STABILITY   *     DATE 09-04-85     RUNTRIAL 04     COMMENTS  STEADY STATE SEEPAGE  DS NO EQ
*********************
BISHOP*S SIMPLIFIED METHOD
155.000 +
        :
        :
        :
        :
        :
125.000 +
        :                                                                  *
        :                                        222222222222
        :                                  2222              222
        :   1111111111111111111111112222                      S2222
 95.000 +                        22222                         SSS 222
        :   3333333333333333333333333333333333333333333333SSSSSSSS3333333333333333333333333333333333
        :                                        3333333
        :
        :
        :   4444444444444444444444444444444444444444444444444444444444444444444444444444444444444444
 65.000 +
        +.........+.........+.........+.........+.........+.........+.........+.........+.........+.........+
             10.00     40.00     70.00    100.00    130.00    160.00    190.00    220.00    250.00    280.00
      HORIZONTAL SCALE  1 INCH =   30.00                VERTICAL SCALE  1 INCH =   30.00
   X =     175.00     Y =     115.00     RADIUS =      29.00     FACTOR OF SAFETY =      1.323
```

```
*********************
*  SLOPE STABILITY  *
*********************
```

DATE 09-04-85 RUN TRIAL 04

COMMENTS STEADY STATE SEEPAGE DS NO EG

BISHOP'S SIMPLIFIED METHOD

THE INTERSECTIONS OF THE SLIP SURFACE WITH THE SURFACE LAYER ARE

LEFT INTERCEPT ..... X= 148.7800 Y= 102.6100

RIGHT INTERCEPT .... X= 185.1963 Y= 87.8516

X- LEFT AND RIGHT INTERCEPTS OF THE SLIP SURFACE AND THE BEDROCK ARE 0.0000 AND 0.0000 RESPECTIVELY

THIS IS ITERATION NUMBER 3

SLICE	X-COOR	Y-TOP'	Y-BASE	ALPHA	SLICE COHESION	SLICE WEIGHT	NORMAL FORCE	PORE-WATER FORCE	SLICE MALPHA	RADIUS	NORMAL ARM	PHI ANGLE
139	149.14	102.43	101.88	63.13	0.	.5117E+02	.5351E+02	0.	.9563E+00	29.00	0.00	37.0
140	150.00	102.00	100.33	59.61	0.	.2192E+03	.2206E+03	0.	.9936E+00	29.00	0.00	37.0
141	151.00	101.50	98.72	55.89	0.	.3602E+03	.3501E+03	0.	.1029E+01	29.00	0.00	37.0
142	152.00	101.00	97.34	52.51	0.	.4754E+03	.4497E+03	0.	.1057E+01	29.00	0.00	37.0
143	153.00	100.50	96.11	49.37	0.	.5706E+03	.5282E+03	0.	.1080E+01	29.00	0.00	37.0
144	154.00	100.00	95.00	46.42	0.	.6494E+03	.5910E+03	0.	.1099E+01	29.00	0.00	37.0
145	155.00	99.50	94.00	43.62	0.	.7145E+03	.6414E+03	0.	.1114E+01	29.00	0.00	37.0
146	156.00	99.00	93.09	40.95	0.	.7677E+03	.6809E+03	.2735E+02	.1126E+01	29.00	0.00	37.0
147	157.00	98.50	92.26	38.38	0.	.8105E+03	.7402E+03	.8424E+02	.1135E+01	29.00	0.00	37.0
148	158.00	98.00	91.51	35.90	0.	.8440E+03	.7776E+03	.1320E+03	.1142E+01	29.00	0.00	37.0
149	159.00	97.50	90.81	33.49	0.	.8690E+03	.8052E+03	.1724E+03	.1146E+01	29.00	0.00	37.0
150	160.00	97.00	90.18	31.15	0.	.8862E+03	.8244E+03	.2067E+03	.1148E+01	29.00	0.00	37.0
151	161.00	96.50	89.60	28.87	0.	.8963E+03	.8364E+03	.2360E+03	.1149E+01	29.00	0.00	37.0
152	162.00	96.00	89.08	26.64	0.	.8997E+03	.8418E+03	.2608E+03	.1147E+01	29.00	0.00	37.0
153	163.00	95.50	88.60	24.45	0.	.8969E+03	.8414E+03	.2819E+03	.1144E+01	29.00	0.00	37.0
154	164.00	95.00	88.17	22.30	0.	.8880E+03	.8355E+03	.2996E+03	.1140E+01	29.00	0.00	37.0
155	165.00	94.50	87.78	20.17	0.	.8692E+03	.8131E+03	.1042E+03	.1088E+01	29.00	0.00	30.0
156	166.00	94.00	87.43	18.08	0.	.8423E+03	.7910E+03	.1189E+03	.1085E+01	29.00	0.00	30.0
157	167.00	93.50	87.13	16.02	0.	.8110E+03	.7649E+03	.1307E+03	.1081E+01	29.00	0.00	30.0
158	168.00	93.00	86.86	13.97	0.	.7755E+03	.7350E+03	.1401E+03	.1075E+01	29.00	0.00	30.0
159	169.00	92.50	86.63	11.94	0.	.7358E+03	.7013E+03	.1470E+03	.1068E+01	29.00	0.00	30.0
160	170.00	92.00	86.43	9.93	0.	.6921E+03	.6638E+03	.1517E+03	.1060E+01	29.00	0.00	30.0
161	171.00	91.50	86.28	7.93	0.	.6443E+03	.6223E+03	.1542E+03	.1050E+01	29.00	0.00	30.0
162	172.00	91.00	86.16	5.94	0.	.5927E+03	.5769E+03	.1547E+03	.1039E+01	29.00	0.00	30.0
163	173.00	90.50	86.07	3.96	0.	.5372E+03	.5273E+03	.1532E+03	.1027E+01	29.00	0.00	30.0
164	174.00	90.00	86.02	1.98	0.	.4779E+03	.4734E+03	.1497E+03	.1014E+01	29.00	0.00	30.0
165	175.00	89.50	86.00	0.00	0.	.4148E+03	.4148E+03	.1442E+03	.1000E+01	29.00	0.00	30.0
166	176.00	89.00	86.02	-1.98	0.	.3479E+03	.3514E+03	.1367E+03	.9845E+00	29.00	0.00	30.0
167	177.00	88.50	86.07	-3.96	0.	.2772E+03	.2826E+03	.1273E+03	.9677E+00	29.00	0.00	30.0
168	178.00	88.00	86.16	-5.94	0.	.2027E+03	.2080E+03	.1157E+03	.9498E+00	29.00	0.00	30.0
169	179.00	87.98	86.28	-7.93	0.	.1871E+03	.1942E+03	.1066E+03	.9307E+00	29.00	0.00	30.0
170	180.00	87.96	86.43	-9.93	0.	.1675E+03	.1762E+03	.9527E+02	.9103E+00	29.00	0.00	30.0
171	181.00	87.94	86.63	-11.94	0.	.1440E+03	.1538E+03	.8163E+02	.8887E+00	29.00	0.00	30.0
172	182.00	87.92	86.86	-13.97	0.	.1164E+03	.1266E+03	.6551E+02	.8658E+00	29.00	0.00	30.0
173	183.00	87.90	87.13	-16.02	0.	.8470E+02	.9400E+02	.4675E+02	.8417E+00	29.00	0.00	30.0
174	184.00	87.88	87.43	-18.08	0.	.4870E+02	.5553E+02	.2511E+02	.8162E+00	29.00	0.00	30.0
175	184.85	87.86	87.72	-19.85	0.	.1031E+02	.1248E+02	.2777E+01	.7935E+00	29.00	0.00	30.0

LEFT SIDE --- FLUID FORCE = 0. AND MOMENT ARM = .1239E+02
RIGHT SIDE --- FLUID FORCE = 0. AND MOMENT ARM = .2715E+02

RADIUS INC*T	X-COORD.	Y-COORD.	RADIUS	FACTOR OF SAFETY	NO. ITER*N	ALPHA WARN*G
1	175.0	115.0	29.0	1.323	3	0

Reservoir @ crest of emergency spillway
Steady State Seepage
No Acceleration
Downstream Slope

```
*************************************
*                                   *
*   SLIP CIRCLE STABILITY ANALYSIS  *
*                                   *
*          VERSION NO. 1.5          *
*                                   *
*************************************
```

APACHE JUNCTION FRS
(PROJECT)

USDA-SCS
(CLIENT)

STEADY STATE SEEPAGE (NO ACC)
(TASK)

3037.100
(CHARGE NUMBER)

AP1
(DATA FILE NAME)

11-08-85
(DATE)

BY V BOLANO
(NAME) (SIGNATURE)

CHECKED BY S GOYAL
(NAME) (SIGNATURE)

PAGE 1
APACHE JUNCTION FRS
USDA-SCS
STEADY STATE SEEPAGE (NO ACC)
11-08-85

CROSS SECTION POINTS

NO. OF POINTS FOR OUTLINE OF THE CROSS SECTION = 12

POINT NO.	X-COORDINATE (FT)	Y-COORDINATE (FT)
1	0.00	88.00
2	65.00	88.00
3	120.00	110.00
4	134.00	110.00
5	178.00	88.00
6	275.00	86.00
7	112.50	88.00
8	120.00	83.00
9	135.00	83.00
10	142.50	88.00
11	0.00	66.00
12	275.00	66.00

CROSS SECTION LINES

NO. OF LINES = 11

LINE NO.	FIRST POINT OF THE LINE	SECOND POINT OF THE LINE	ID. NO. OF THE UNDERLYING SOIL
1	1	2	2
2	2	3	1
3	3	4	1
4	4	5	1
5	5	6	2
6	2	7	2
7	7	8	2
8	8	9	2
9	9	10	2
10	10	5	2
11	11	12	3

APACHE JUNCTION FRS
USDA-SCS
STEADY STATE SEEPAGE (NO ACC)
11-08-85

SOIL TYPES

NO. OF SOIL TYPES = 3

ID.NO. OF THE UNDERLYING SOIL	UNIT WEIGHT OF SOIL (LBS/FT^3)	COHESION FACTOR (PSF)	FRICTION ANGLE (DEG)	ID. NO. OF ASSOC. PIEZ. SURFACE
1	130.00	0.00	37.00	1
2	110.00	0.00	30.00	2
3	125.00	1000.00	40.00	1

PIEZOMETRIC SURFACES

THERE ARE 9 POINTS FOR THE PIEZOMETRIC LINES:

ID. NO. OF THE PIEZ. SURFACE	ID. NO. OF PIEZ. PTS.	X-COORDINATE OF THE POINT	Y-COORDINATE OF THE POINT
1	1	0.00	99.70
1	2	94.20	99.70
1	3	170.00	92.00
1	4	178.00	88.00
1	5	275.00	85.00
2	1	0.00	99.70
2	2	65.00	99.70
2	3	178.00	88.00
2	4	275.00	85.00

APACHE JUNCTION FRS
USDA-SCS
STEADY STATE SEEPAGE (NO ACC)
11-08-85

SEARCHING FACTOR OF SAFETY OF THE CIRCLE:

X-COORDINATE OF THE INITIAL SEARCH POINT (FT) = 180
Y-COORDINATE OF THE INITIAL SEARCH POINT (FT) = 125
INCREMENT OF X VALUES (FT) = 0
INCREMENT OF Y VALUES (FT) = 0

LIMITATION OF THE SIZE OF THE RADIUS USED IN A SEARCH:

COORDINATE PERMITTED AT THE LOWEST POINT ON ANY TRIAL CIRCLE:

MAXIMUM Y-COORDINATE (FT) = 93
MINIMUM Y-COORDINATE (FT) = 88

HEIGHT ALLOWED FOR TALLEST SLICE IN ANY CIRCLE:

MAXIMUM HEIGHT (FT) = 0
MINIMUM HEIGHT (FT) = 0
FRACTIONAL INCREMENT OF RADIUS IN A SEARCH = .2

THE SLIP CIRCLE WILL BE ANALYZED WITH 45 SLICES.

PAGE 4
APACHE JUNCTION FRS
USDA-SCS
STEADY STATE SEEPAGE (NO ACC)
11-08-85

CENTER OF TRIAL CIRCLE:
X = 180 Y = 125

RADIUS	RESISTING	DRIVING	SLICES	FS
37.000	4032.61	3247.01	7	1.2419
36.000	2249.61	1664.13	6	1.3518
36.750	3239.78	2441.35	6	1.3270
36.500	2932.06	2184.80	6	1.3420
36.250	2606.09	1925.57	6	1.3534

* TOO FEW SLICES AT RADIUS 35 --- CIRCLE IGNORED *

AT X = 180 Y = 125 LOWEST FS = 1.24195 FOR R = 37

APACHE JUNCTION FRS
USDA-SCS
STEADY STATE SEEPAGE (NO ACC)
11-08-85

CENTER OF TRIAL CIRCLE:
X = 180 Y = 125

RADIUS	RESISTING	DRIVING	SLICES	FS
37.000	4032.61	3247.01	7	1.2419
36.000	2249.61	1664.13	6	1.3518
36.750	3239.78	2441.35	6	1.3270
36.500	2932.06	2184.80	6	1.3420
36.250	2606.09	1925.57	6	1.3534

* TOO FEW SLICES AT RADIUS 35 --- CIRCLE IGNORED *

AT X = 180 Y = 125 LOWEST FS = 1.24195 FOR R = 37

Reservoir @ crest of emergency spillway
Steady State Seepage
Accelerator (0.18)
Downstream Slope

```
DDDDDDDDDDD   TTTTTTTTTTTT            JJ  KK        KK
DDDDDDDDDDDD  TTTTTTTTTTTT            JJ  KK      KK
DD        DD      TT                  JJ  KK    KK
DD        DD      TT                  JJ  KK  KK
DD        DD      TT                  JJ  KKKKK
DD        DD      TT                  JJ  KKKKKK
DD        DD      TT                  JJ  KK   KK
DD        DD      TT                  JJ  KK    KK
DD        DD      TT        JJ        JJ  KK     KK
DD        DD      TT        JJ        JJ  KK      KK
DDDDDDDDDDDD      TT        JJJJJJJJJJJJ  KK       KK
DDDDDDDDDDD       TT         JJJJJJJJJJ   KK        KK

          JJ  CCOOOOOOOOOO  BBBBBBBBBBB                 555555555555   8888888888   555555555555   777777777777
          JJ  CCOOOOOOOOOO  BBBBBBBBBBB                 555555555555   8888888888   555555555555   777777777777
          JJ  CC        CO  BB        BB                55            88        88 55            77      77
          JJ  CC        OO  BB        BB                55            88        88 55                 77
          JJ  OO        OO  BB        BB                55            88        88 55                 77
          JJ  CC        CO  BBBBBBBBBBB                 55555555555    8888888888  55555555555        77
          JJ  CC        CO  BBBBBBBBBBB                 555555555555   8888888888  555555555555       77
          JJ  CC        CO  BB        BB                        55  88        88          55        77
JJ        JJ  CC        CO  BB        BB                        55  88        88          55        77
JJ        JJ  CO        OO  BB        BB                55      55  88        88 55       55        77
JJJJJJJJJJJJ  CCOOOOOOOOOO  BBBBBBBBBBB                 555555555555 8888888888888 555555555555     77
 JJJJJJJJJJ   CCOOOOOOOOOO  BBBBBBBBBBB                  5555555555   8888888888   5555555555       77

RRRRRRRRRR     2222222222       11        2222222222   66666666666  PPPPPPPPPPP   RRRRRRRRRR        11
RRRRRRRRRRR   222222222222     111       222222222222  666666666666 PPPPPPPPPPPP  RRRRRRRRRRR      111
RR        RR  22        22    1111       22        22  66           PP        PP  RR        RR    1111
RR        RR            22      11                 22  66           PP        PP  RR        RR      11
RR        RR            22      11                 22  66           PP        PP  RR        RR      11
RRRRRRRRRRR            22       11                22   66666666666  PPPPPPPPPPP   RRRRRRRRRRR      11
RRRRRRRRRRR          22         11              22     666666666666 PPPPPPPPPPP   RRRRRRRRRRR      11
RR     RR          22           11            22       66        66 PP            RR     RR        11
RR      RR       22              11          22        66        66 PP            RR      RR       11
RR       RR    22                11        22          66        66 PP            RR       RR      11
RR        RR   222222222222  1111111111  222222222222  666666666666 PP            RR        RR  1111111111
RR        RR   222222222222  1111111111  222222222222   6666666666  PP            RR        RR  1111111111

PPPPPPPPPPP   RRRRRRRRRRR   TTTTTTTTTTTT     11              44      00000000     3333333333
PPPPPPPPPPPP  RRRRRRRRRRRR  TTTTTTTTTTTT    111             444     0000000000   333333333333
PP        PP  RR        RR       TT       1111            4444    00        00 33        33
PP        PP  RR        RR       TT         11           44 44    00        00           33
PP        PP  RR        RR       TT         11          44  44    00        00           33
PPPPPPPPPPP   RRRRRRRRRRR       TT         11         44    44    00        00        3333
PPPPPPPPPPP   RRRRRRRRRRR       TT         11        44     44    00        00        3333
PP            RR     RR        TT         11       4444444444    00        00           33
PP            RR      RR       TT         11      444444444444   00        00           33
PP            RR       RR      TT         11              44     00        00 33        33
PP            RR        RR     TT       1111111111            44  0000000000   333333333333
PP            RR        RR     TT       1111111111            44   00000000     3333333333
```

```
**********************
*  SLOPE STABILITY   *
**********************

DATE  09-04-85                    RUN  TRIAL 05

COMMENTS          STEADY STATE SEEPAGE  DS EARTHQUAKE 0.1 G

BISHOP'S SIMPLIFIED METHOD

          DETAILS ON THE SLIP CIRCLE GRID

   LAYOUT OF GRID FOR CENTERS OF TRIAL SLIP CIRCLES
   ************************************************

   THE X-COORDINATE OF THE LOWER LEFT CORNER OF THE GRID IS   145.0000
   THE CENTERS ARE INCREMENTED      5   TIMES IN THE X-DIRECTION WITH EACH
   INCREMENT EQUAL TO      10.0000

   THE Y-COORDINATE OF THE LOWER LEFT CORNER OF THE GRID IS  115.0000
   THE CENTERS ARE INCREMENTED      4   TIMES IN THE Y-DIRECTION WITH EACH
   INCREMENT EQUAL TO      10.0000

   THE GRID HAS A SLOPE(CHANGE IN ABSCISSA DIVIDED BY CHANGE IN ORDINATE)
   OF        .5000.

   OTHER PARTICULARS FOR THE ANALYSIS
   **********************************

   THE Y-COORDINATE OF A HORIZONTAL LINE TANGENT TO THE SLIP SURFACE OF
   MINIMUM RADIUS IS   106.0000.

   FOR EACH CENTER, RADIUS IS INCREMENTED    20 TIMES WITH EACH INCREMENT
   EQUAL TO      2.0000
   THE TOTAL NUMBER OF STABILITY ANALYSIS IS       630.

   THE GEOMETRY IS DIVIDED INTO       264 SLICES, EACH      1.0000

   THE DEPTH OF THE DEEPEST SLICE MUST BE GREATER THAN      5.0000

   THE TOLERANCE IN CALCULATING THE FACTOR OF SAFETY IS      .0100

   THE SEISMIC COEFFICIENT IS EQUAL TO        .1000

   THE UNIT WEIGHT OF WATER IS    62.4000

   THE CONTROL QUESTIONS ANSWERED
    1.) PRINT OUT THE INPUT DATA.  IQ(1)=2
    2.) WRITE OUT ALL FACTORS OF SAFETY.  IQ(2)=1
    3.) WRITE OUT A SUMMARY OF THE MINIMUM FACTORS OF SAFETY.  IQ(3)=2
    4.) WRITE OUT A DETAILED SUMMARY OF THE COMPUTATIONS  IQ(4)=2
    7.) DISREGARD THE PREVIOUS PORE PRESSURE DATA.  IQ(7)=1
    8.) SET ALL NEGATIVE PORE PRESSURES TO ZERO.  IQ(8)=1
    9.) PRINT A DIGITAL PLOT OF THE GEOMETRY AND SLIP SURFACE.  IQ(9)=1
```

```
*********************
*  SLOPE STABILITY  *
*********************
```

DATE 09-04-85 RUN TRIAL 05

COMMENTS STEADY STATE SEEPAGE DS EARTHQUAKE 0.1 G

BISHOP'S SIMPLIFIED METHOD

SPECIAL DETAILS ON THE LAYOUT OF THE CROSS-SECTION

THE COORDINATES OF THE VERTICES OF LINES FORMING THE CONTACTS BETWEEN SOIL LAYERS ARE AS FOLLOWS

LINE NO.	COORDINATES	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
100	X	10.0	94.2	120.0	134.0	178.0	275.0	275.0	275.0	275.0	275.0	275.0	275.0	275.0	275.0	275.0
100	Y	99.7	99.7	110.0	110.0	88.0	86.0	86.0	86.0	86.0	86.0	86.0	86.0	86.0	86.0	86.0
200	X	10.0	65.0	94.2	120.0	134.0	178.0	275.0	275.0	275.0	275.0	275.0	275.0	275.0	275.0	275.0
200	Y	88.0	88.0	99.7	110.0	110.0	88.0	86.0	86.0	86.0	86.0	86.0	86.0	86.0	86.0	86.0
300	X	10.0	65.0	112.5	120.0	135.0	142.5	178.0	275.0	275.0	275.0	275.0	275.0	275.0	275.0	275.0
300	Y	88.0	88.0	88.0	83.0	83.0	88.0	88.0	86.0	86.0	86.0	86.0	86.0	86.0	86.0	86.0
400	X	10.0	275.0	275.0	275.0	275.0	275.0	275.0	275.0	275.0	275.0	275.0	275.0	275.0	275.0	275.0
400	Y	66.0	66.0	66.0	66.0	66.0	66.0	66.0	66.0	66.0	66.0	66.0	66.0	66.0	66.0	66.0

LIMITS OF THE GEOMETRY USED DURING COMPUTATIONS

THE LEFT LIMIT IS 11.00.
THE RIGHT LIMIT IS 274.00.

```
**********************
*  SLOPE STABILITY   *
**********************
```

DATE 09-04-85 RUN TRIAL 05

COMMENTS STEADY STATE SEEPAGE DS EARTHQUAKE 0.1 G

BISHOP'S SIMPLIFIED METHOD

SPECIAL DETAILS ON THE SOIL CONDITIONS

SOIL CHARACTERISTICS
****

THE GEOMETRY CONSISTS OF 4 LAYERS WITH THE FOLLOWING CHARACTERISTICS.

LAYER NO.	TOTAL UNIT WEIGHT	COHESION	ANGLE OF INTERNAL FRICTION	PORE-PRESSURE RATIO RU	SOIL DESCRIPTION
1	62.400	0.000	0.000	0.000	WATER
2	130.000	0.000	37.000	0.000	COMPACTED SM-SC MATERIAL
3	110.000	0.000	30.000	0.000	FOUNDATION SM-SC MATERIAL
4	-1.000	0.000	0.000	0.000	CALICHE

```
*********************
*  SLOPE STABILITY  *
*********************
```

DATE 09-04-85 RUN TRIAL 05

COMMENTS STEADY STATE SEEPAGE DS EARTHQUAKE 0.1 G

BISHOP*S SIMPLIFIED METHOD
SPECIAL DETAILS ON THE PORE PRESSURE CONDITIONS

THE COORDINATES OF THE VERTICES OF LINES FORMING THE PIEZOMETRIC LINES ARE AS FOLLOWS,

****

LINE NO.	COORDINATES	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
1	X	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
1	Y	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
2	X	0.0	94.2	170.0	178.0	275.0	275.0	275.0	275.0	275.0	275.0	275.0	275.0	275.0	275.0	275.0
2	Y	99.7	99.7	92.0	88.0	85.0	85.0	85.0	85.0	85.0	85.0	85.0	85.0	85.0	85.0	85.0
3	X	0.0	65.0	178.0	275.0	275.0	275.0	275.0	275.0	275.0	275.0	275.0	275.0	275.0	275.0	275.0
3	Y	99.7	99.7	88.0	85.0	85.0	85.0	85.0	85.0	85.0	85.0	85.0	85.0	85.0	85.0	85.0
4	X	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
4	Y	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0

```
*********************
*  SLOPE STABILITY  *
*********************
```

DATE 09-04-85 RUN TRIAL 05

COMMENTS STEADY STATE SEEPAGE DS EARTHQUAKE 0.1 G

BISHOP'S SIMPLIFIED METHOD

SUMMARY OF MINIMUM FACTORS OF SAFETY

RADIUS INC*T	X-COORD.	Y-COORD.	RADIUS	FACTOR OF SAFETY	NO. ITER*N	MALPHA WARN*G
2	145.0	115.0	15.0	1.860	3	0
14	155.0	115.0	39.0	1.494	4	0
12	165.0	115.0	35.0	1.183	4	0
9	175.0	115.0	31.0	1.036	3	0
11	185.0	115.0	33.0	1.178	4	0
21	195.0	115.0	53.0	1.718	4	0
15	150.0	125.0	53.0	1.654	4	0
12	160.0	125.0	45.0	1.307	3	0
11	170.0	125.0	43.0	1.096	3	0
9	180.0	125.0	39.0	1.028	4	0
11	190.0	125.0	43.0	1.278	3	0
21	200.0	125.0	63.0	1.643	4	0
16	155.0	135.0	63.0	1.496	3	0
11	165.0	135.0	53.0	1.226	3	0
10	175.0	135.0	53.0	1.083	3	0
10	185.0	135.0	51.0	1.087	4	0
14	195.0	135.0	59.0	1.360	4	0
21	205.0	135.0	73.0	1.616	4	0
13	160.0	145.0	69.0	1.411	3	0
11	170.0	145.0	63.0	1.194	4	0
10	180.0	145.0	61.0	1.098	4	0
10	190.0	145.0	61.0	1.154	4	0
17	200.0	145.0	75.0	1.405	4	0
21	210.0	145.0	83.0	1.613	4	0
12	165.0	155.0	75.0	1.365	4	0
11	175.0	155.0	75.0	1.189	4	0
9	185.0	155.0	69.0	1.122	4	0
13	195.0	155.0	77.0	1.224	3	0
21	205.0	155.0	93.0	1.425	4	0
19	215.0	155.0	89.0	1.730	4	0

THE MOST CRITICAL SLIP SURFACE

RADIUS INC*T	X-COORD.	Y-COORD.	RADIUS	FACTOR OF SAFETY	NO. ITER*N	MALPHA WARN*G
9	180.0	125.0	39.0	1.028	4	0

```
*********************
*  SLOPE STABILITY  *      DATE 09-04-85      RUNTRIAL 05      COMMENTS  STEADY STATE SEEPAGE  DS EARTHQUAKE 0.1 G
*********************
BISHOP'S SIMPLIFIED METHOD
155.000 +                                   1.36      1.19      1.12      1.22      1.43      1.73
        :
        :
147.000 +                         1.41      1.19      1.10      1.15      1.40      1.61
        :
        :
139.000 +
        :
                    1.50      1.23      1.08      1.09      1.36      1.62
        :
131.000 +
        :
            1.65      1.31      1.10      1.03      1.28      1.64
        :
123.000 +
        :
        :
115.000 +       1.86      1.49      1.18      1.04      1.18      1.72
        :
        :
107.000 +
        +.........+.........+.........+.........+.........+.........+.........+.........+.........+.........+
            145.00    153.00    161.00    169.00    177.00    185.00    193.00    201.00    209.00    217.00
        HORIZONTAL SCALE  1 INCH =     8.00                VERTICAL SCALE  1 INCH =     8.00
```

```
*********************
*  SLOPE STABILITY  *     DATE 09-04-85     RUNTRIAL 05     COMMENTS  STEADY STATE SEEPAGE  DS EARTHQUAKE 0.1 G
*********************
BISHOP'S SIMPLIFIED METHOD
135.000 +
        :
        :
        :
        :
        :
        :
125.000 +                                                                                   *
        :
        :
        :                                                  222222222222
        :                                          2222                222
        :          11111111111111111111111112222                      SS222
 95.000 +                                 22222                          SS 222
        :          3333333333333333333333333333333333333       33333333SSSSSSSSSS3333333333333333333333333333
        :                                                 3333333
        :
        :
        :          44444444444444444444444444444444444444444444444444444444444444444444444444444444444444
 65.000 +
        +.........+.........+.........+.........+.........+.........+.........+.........+.........+.........+
             10.00     40.00     70.00    100.00    130.00    160.00    190.00    220.00    250.00    280.00
      HORIZONTAL SCALE  1 INCH =    30.00                    VERTICAL SCALE  1 INCH =    30.00
   X =      180.00     Y =      125.00     RADIUS =       39.00     FACTOR OF SAFETY =       1.028
```

```
*********************
*  SLOPE STABILITY  *
*********************
```

DATE 09-04-85 RUN TRIAL 05

COMMENTS STEADY STATE SEEPAGE DS EARTHQUAKE 0.1 G

BISHOP'S SIMPLIFIED METHOD

THE INTERSECTIONS OF THE SLIP SURFACE WITH THE SURFACE LAYER ARE
LEFT INTERCEPT ..... X= 147.6933 Y= 103.1534
RIGHT INTERCEPT .... X= 191.4622 Y= 67.7224
X- LEFT AND RIGHT INTERCEPTS OF THE SLIP SURFACE AND THE BEDROCK ARE 0.0000 AND 0.0000 RESPECTIVELY
THIS IS ITERATION NUMBER 4

SLICE	X-COOR	Y-TOP	Y-BASE	ALPHA	SLICE COHESION	SLICE WEIGHT	NORMAL FORCE	PORE-WATER FORCE	SLICE MALPHA	RADIUS	NORMAL ARM	PHI ANGLE
138	148.10	102.95	102.57	54.90	0.	.3981E+02	.3396E+02	0.	.1172E+01	39.00	0.00	37.0
139	149.00	102.50	101.34	52.66	0.	.1507E+03	.1270E+03	0.	.1187E+01	39.00	0.00	37.0
140	150.00	102.00	100.08	50.30	0.	.2491E+03	.2075E+03	0.	.1200E+01	39.00	0.00	37.0
141	151.00	101.50	98.92	48.05	0.	.3345E+03	.2761E+03	0.	.1211E+01	39.00	0.00	37.0
142	152.00	101.00	97.85	45.90	0.	.4088E+03	.3350E+03	0.	.1220E+01	39.00	0.00	37.0
143	153.00	100.50	96.86	43.82	0.	.4732E+03	.3856E+03	0.	.1227E+01	39.00	0.00	37.0
144	154.00	100.00	95.93	41.82	0.	.5286E+03	.4291E+03	0.	.1232E+01	39.00	0.00	37.0
145	155.00	99.50	95.07	39.88	0.	.5760E+03	.4662E+03	0.	.1235E+01	39.00	0.00	37.0
146	156.00	99.00	94.26	37.99	0.	.6160E+03	.4978E+03	0.	.1237E+01	39.00	0.00	37.0
147	157.00	98.50	93.50	36.14	0.	.6492E+03	.5244E+03	0.	.1238E+01	39.00	0.00	37.0
148	158.00	98.00	92.80	34.34	0.	.6761E+03	.5569E+03	.3186E+02	.1237E+01	39.00	0.00	37.0
149	159.00	97.50	92.14	32.58	0.	.6970E+03	.5872E+03	.7263E+02	.1236E+01	39.00	0.00	37.0
150	160.00	97.00	91.52	30.86	0.	.7124E+03	.6109E+03	.1088E+03	.1233E+01	39.00	0.00	37.0
151	161.00	96.50	90.94	29.16	0.	.7224E+03	.6287E+03	.1410E+03	.1229E+01	39.00	0.00	37.0
152	162.00	96.00	90.40	27.49	0.	.7275E+03	.6410E+03	.1696E+03	.1224E+01	39.00	0.00	37.0
153	163.00	95.50	89.90	25.85	0.	.7278E+03	.6483E+03	.1949E+03	.1218E+01	39.00	0.00	37.0
154	164.00	95.00	89.43	24.22	0.	.7235E+03	.6510E+03	.2173E+03	.1211E+01	39.00	0.00	37.0
155	165.00	94.50	89.00	22.62	0.	.7148E+03	.6491E+03	.2371E+03	.1204E+01	39.00	0.00	37.0
156	166.00	94.00	88.60	21.04	0.	.7019E+03	.6430E+03	.2545E+03	.1195E+01	39.00	0.00	37.0
157	167.00	93.50	88.23	19.47	0.	.6849E+03	.6327E+03	.2697E+03	.1186E+01	39.00	0.00	37.0
158	168.00	93.00	87.89	17.92	0.	.6617E+03	.6004E+03	.7499E+02	.1124E+01	39.00	0.00	30.0
159	169.00	92.50	87.58	16.38	0.	.6307E+03	.5769E+03	.8771E+02	.1117E+01	39.00	0.00	30.0
160	170.00	92.00	87.30	14.86	0.	.5964E+03	.5501E+03	.9842E+02	.1110E+01	39.00	0.00	30.0
161	171.00	91.50	87.05	13.34	0.	.5591E+03	.5199E+03	.1072E+03	.1102E+01	39.00	0.00	30.0
162	172.00	91.00	86.83	11.84	0.	.5186E+03	.4863E+03	.1142E+03	.1093E+01	39.00	0.00	30.0
163	173.00	90.50	86.63	10.34	0.	.4752E+03	.4494E+03	.1195E+03	.1084E+01	39.00	0.00	30.0
164	174.00	90.00	86.46	8.85	0.	.4288E+03	.4091E+03	.1231E+03	.1074E+01	39.00	0.00	30.0
165	175.00	89.50	86.32	7.37	0.	.3795E+03	.3653E+03	.1251E+03	.1063E+01	39.00	0.00	30.0
166	176.00	89.00	86.21	5.89	0.	.3273E+03	.3179E+03	.1256E+03	.1052E+01	39.00	0.00	30.0
167	177.00	88.50	86.12	4.41	0.	.2722E+03	.2668E+03	.1244E+03	.1040E+01	39.00	0.00	30.0
168	178.00	88.00	86.05	2.94	0.	.2142E+03	.2119E+03	.1218E+03	.1027E+01	39.00	0.00	30.0
169	179.00	87.98	86.01	1.47	0.	.2162E+03	.2149E+03	.1221E+03	.1014E+01	39.00	0.00	30.0
170	180.00	87.96	86.00	0.00	0.	.2153E+03	.2153E+03	.1209E+03	.1000E+01	39.00	0.00	30.0
171	181.00	87.94	86.01	-1.47	0.	.2117E+03	.2131E+03	.1183E+03	.9853E+00	39.00	0.00	30.0
172	182.00	87.92	86.05	-2.94	0.	.2052E+03	.2081E+03	.1140E+03	.9700E+00	39.00	0.00	30.0
173	183.00	87.90	86.12	-4.41	0.	.1958E+03	.2004E+03	.1083E+03	.9540E+00	39.00	0.00	30.0
174	184.00	87.88	86.21	-5.89	0.	.1836E+03	.1897E+03	.1009E+03	.9374E+00	39.00	0.00	30.0
175	185.00	87.86	86.32	-7.37	0.	.1686E+03	.1761E+03	.9197E+02	.9200E+00	39.00	0.00	30.0
176	186.00	87.84	86.46	-8.85	0.	.1507E+03	.1593E+03	.8136E+02	.9023E+00	39.00	0.00	30.0

177	187.00	87.81	86.63	-10.34	0.	.1298E+03	.1391E+03	.6903E+02	.8834E+00	39.00	0.00	30.0
178	188.00	87.79	86.83	-11.84	0.	.1060E+03	.1154E+03	.5492E+02	.8640E+00	39.00	0.00	30.0
179	189.00	87.77	87.05	-13.34	0.	.7913E+02	.8781E+02	.3894E+02	.8439E+00	39.00	0.00	30.0
180	190.00	87.75	87.30	-14.86	0.	.4923E+02	.5615E+02	.2098E+02	.8231E+00	39.00	0.00	30.0
181	190.98	87.73	87.58	-16.36	0.	.1623E+02	.1999E+02	.1255E+01	.8020E+00	39.00	0.00	30.0

HORIZONTAL FORCES ON THE SLOPE DUE TO PARTIAL SUBMERGENCE OR TENSION CRACKS
LEFT SIDE --- FLUID FORCE = 0. AND MOMENT ARM = .2185E+02
RIGHT SIDE --- FLUID FORCE = 0. AND MOMENT ARM = .3728E+02

RADIUS INC*T	X- COORD.	Y- COORD.	RADIUS	FACTOR OF SAFETY	NO. ITER*N	MALPHA WARN*G
1	100.0	125.0	39.0	1.028	4	0

Reservoir @ crest of emergency spillway
Steady State Seepage
Acceleration (0.1g)
Downstream Slope

```
*************************************
*                                   *
*   SLIP CIRCLE STABILITY ANALYSIS  *
*                                   *
*          VERSION NO. 1.5          *
*                                   *
*************************************
```

APACHE JUNCTION FRS
(PROJECT)

USDA-SCS
(CLIENT)

STEADY STATE SEEPAGE (ACC=0.1G)
(TASK)

3037.100
(CHARGE NUMBER)

AP1
(DATA FILE NAME)

11-08-85
(DATE)

BY V BOLANO
(NAME) (SIGNATURE)

CHECKED BY S GOYAL
(NAME) (SIGNATURE)

PAGE 1
APACHE JUNCTION FRS
USDA-SCS
STEADY STATE SEEPAGE (ACC=0.1G)
11-08-85

CROSS SECTION POINTS

NO. OF POINTS FOR OUTLINE OF THE CROSS SECTION = 12

POINT NO.	X-COORDINATE (FT)	Y-COORDINATE (FT)
1	0.00	88.00
2	65.00	88.00
3	120.00	110.00
4	134.00	110.00
5	178.00	88.00
6	275.00	86.00
7	112.50	88.00
8	120.00	83.00
9	135.00	83.00
10	142.50	88.00
11	0.00	66.00
12	275.00	66.00

CROSS SECTION LINES

NO. OF LINES = 11

LINE NO.	FIRST POINT OF THE LINE	SECOND POINT OF THE LINE	ID. NO. OF THE UNDERLYING SOIL
1	1	2	2
2	2	3	1
3	3	4	1
4	4	5	1
5	5	6	2
6	2	7	2
7	7	8	2
8	8	9	2
9	9	10	2
10	10	5	2
11	11	12	3

APACHE JUNCTION FRS
USDA-SCS
STEADY STATE SEEPAGE (ACC=0.1G)
11-08-85

SOIL TYPES

NO. OF SOIL TYPES = 3

ID.NO. OF THE UNDERLYING SOIL	UNIT WEIGHT OF SOIL (LBS/FT^3)	COHESION FACTOR (PSF)	FRICTION ANGLE (DEG)	ID. NO. OF ASSOC. PIEZ. SURFACE
1	130.00	0.00	37.00	1
2	110.00	0.00	30.00	2
3	125.00	1000.00	40.00	1

PIEZOMETRIC SURFACES

THERE ARE 9 POINTS FOR THE PIEZOMETRIC LINES:

ID. NO. OF THE PIEZ. SURFACE	ID. NO. OF PIEZ. PTS.	X-COORDINATE OF THE POINT	Y-COORDINATE OF THE POINT
1	1	0.00	99.70
1	2	94.20	99.70
1	3	170.00	92.00
1	4	178.00	88.00
1	5	275.00	85.00
2	1	0.00	99.70
2	2	65.00	99.70
2	3	178.00	88.00
2	4	275.00	85.00

SEISMIC LOADS

HORIZONTAL GROUND ACCELERATION (IN G'S) = .1
VERTICAL GROUND ACCELERATION (IN G'S) = 0
THE RATIO OF EFFECTIVE VERTICAL ACCELERATION
OF THE FREESTANDING WATER TO THE VERTICAL
GROUND ACCELERATION:

SOIL TYPE	RATIO
1	1
2	1
3	1

APACHE JUNCTION FRS
USDA-SCS
STEADY STATE SEEPAGE (ACC=0.1G)
11-08-85

SEARCHING FACTOR OF SAFETY OF THE CIRCLE:

X-COORDINATE OF THE INITIAL SEARCH POINT (FT) = 180
Y-COORDINATE OF THE INITIAL SEARCH POINT (FT) = 125
INCREMENT OF X VALUES (FT) = 0
INCREMENT OF Y VALUES (FT) = 0

LIMITATION OF THE SIZE OF THE RADIUS USED IN A SEARCH:

COORDINATE PERMITTED AT THE LOWEST POINT ON ANY TRIAL CIRCLE:

MAXIMUM Y-COORDINATE (FT) = 93
MINIMUM Y-COORDINATE (FT) = 88

HEIGHT ALLOWED FOR TALLEST SLICE IN ANY CIRCLE:

MAXIMUM HEIGHT (FT) = 0
MINIMUM HEIGHT (FT) = 0
FRACTIONAL INCREMENT OF RADIUS IN A SEARCH = .2

THE SLIP CIRCLE WILL BE ANALYZED WITH 45 SLICES.

APACHE JUNCTION FRS
USDA-SCS
STEADY STATE SEEPAGE (ACC=0.1G)
11-08-85

CENTER OF TRIAL CIRCLE:
X = 180 Y = 125

RADIUS	RESISTING	DRIVING	SLICES	FS
37.000	3781.41	3895.80	7	0.9706
36.000	2120.40	2007.56	6	1.0562
36.750	3049.72	2969.56	6	1.0270
36.500	2761.93	2649.65	6	1.0424
36.250	2456.25	2328.81	6	1.0547

* TOO FEW SLICES AT RADIUS 35 --- CIRCLE IGNORED *

AT X = 180 Y = 125 LOWEST FS = .970637 FOR R = 37

```
BB        BB  GG        GG      4444     XX      XX
BB        BB  GG        GG     44 44    XX    XX
BB        BB  GG        GG    44  44   XX  XX
BBBBBBBBBBB   GG        GG   44   44     XXXX
BBBBBBBBBBB   GG        GG  44    44     XXXX
BB        BB  GG    GG  GG  44444444444  XX  XX
BB        BB  GG    GG  GG  44444444444  XX  XX
BB        BB  GG     GGGG        44    XX    XX
BBBBBBBBBBBB  GGGGGGGGGGG       44   XX      XX
BBBBBBBBBBBB   GGGGGGGG GG      44   XX      XX
```

```
          JJ  CCCCCCCCCCCC  BBBBBBBBBBBB                555555555555        44     2222222222   777777777777
          JJ  CCCCCCCCCCCC  BBBBBBBBBBBB                555555555555       444     2222222222   777777777777
          JJ  CC        CC  BB        BB                55               4444     22        22  77        77
          JJ  CC        CC  BB        BB                55              44 44               22        77
          JJ  CC        CC  BB        BB                55             44  44               22        77
          JJ  CC        CC  BBBBBBBBBBBB                55555555555   44    44               22        77
          JJ  CC        CC  BBBBBBBBBBBB                555555555555 44     44            22          77
          JJ  CC        CC  BB        BB                        55 44444444444        22            77
JJ        JJ  CC        CC  BB        BB                        55 444444444444     22              77
JJ        JJ  CC        CC  BB        BB                55      55       44     22                  77
JJJJJJJJJJJJ  CCCCCCCCCCCC  BBBBBBBBBBBB                555555555555       44     2222222222222       77
 JJJJJJJJJJ   CCCCCCCCCCCC  BBBBBBBBBBBB                 5555555555        44     222222222222        77
```

```
RRRRRRRRRRR    2222222222      11      2222222222   777777777777  PPPPPPPPPPP   RRRRRRRRRRR      11
RRRRRRRRRRRR   2222222222     111     2222222222    777777777777  PPPPPPPPPPPP  RRRRRRRRRRRR    111
RR        RR   22        22  1111    22        22  77        77  PP        PP  RR        RR   1111
RR        RR        22      11                22        77      PP        PP  RR        RR     11
RR        RR        22      11                22      77        PP        PP  RR        RR     11
RRRRRRRRRRRR          22    11              22      77          PPPPPPPPPPPP  RRRRRRRRRRRR     11
RRRRRRRRRRR         22      11            22        77          PPPPPPPPPPP   RRRRRRRRRRR      11
RR      RR        22        11          22          77          PP            RR      RR       11
RR      RR      22          11        22            77          PP            RR      RR       11
RR      RR    2222222222    1111111111  2222222222    77          PP            RR      RR   1111111111
RR      RR    2222222222    1111111111  2222222222    77          PP            RR      RR   1111111111
```

```
PPPPPPPPPPP   RRRRRRRRRRR   TTTTTTTTTTTT      11            44     00000000    3333333333
PPPPPPPPPPPP  RRRRRRRRRRRR  TTTTTTTTTTTT     111           444     00000000    3333333333
PP        PP  RR        RR        TT        1111          4444    00      00  33      33
PP        PP  RR        RR        TT          11         44 44    00      00          33
PP        PP  RR        RR        TT          11        44  44    00      00          33
PPPPPPPPPPPP  RRRRRRRRRRRR        TT          11       44    44    00      00        3333
PPPPPPPPPPP   RRRRRRRRRRR         TT          11      44     44    00      00        3333
PP            RR      RR          TT          11     44444444444 00      00          33
PP            RR      RR          TT          11    444444444444 00      00          33
PP            RR      RR          TT          11            44   00      00  33      33
PP            RR      RR          TT     1111111111         44    00000000    3333333333
PP            RR        RR        TT     1111111111         44     00000000   3333333333
```

SUDDEN DRAWDOWN - US

No acceleration

```
**********************
*  SLOPE STABILITY   *
**********************

DATE  09-04-85                    RUN  TRIAL 07

COMMENTS        APACHE JCT FRS DRAWDOWN CONDITION US

BISHOP*S SIMPLIFIED METHOD

          DETAILS ON THE SLIP CIRCLE GRID

   LAYOUT OF GRID FOR CENTERS OF TRIAL SLIP CIRCLES
   ************************************************

   THE X-COORDINATE OF THE LOWER LEFT CORNER OF THE GRID IS     60.0000
   THE CENTERS ARE INCREMENTED      2    TIMES IN THE X-DIRECTION WITH EACH
   INCREMENT EQUAL TO      10.0000

   THE Y-COORDINATE OF THE LOWER LEFT CORNER OF THE GRID IS  115.0000
   THE CENTERS ARE INCREMENTED      5    TIMES IN THE Y-DIRECTION WITH EACH
   INCREMENT EQUAL TO      10.0000

   THE GRID HAS A SLOPE(CHANGE IN ABSCISSA DIVIDED BY CHANGE IN ORDINATE)
   OF       -.5000.

   OTHER PARTICULARS FOR THE ANALYSIS
   **********************************

   THE Y-COORDINATE OF A HORIZONTAL LINE TANGENT TO THE SLIP SURFACE OF
   MINIMUM RADIUS IS    106.0000.

   FOR EACH CENTER, RADIUS IS INCREMENTED     20 TIMES WITH EACH INCREMENT
   EQUAL TO      2.0000
   THE TOTAL NUMBER OF STABILITY ANALYSIS IS       378.

   THE GEOMETRY IS DIVIDED INTO       264 SLICES, EACH      1.0000

   THE DEPTH OF THE DEEPEST SLICE MUST BE GREATER THAN      5.0000

   THE TOLERANCE IN CALCULATING THE FACTOR OF SAFETY IS      .0100

   THE SEISMIC COEFFICIENT IS EQUAL TO       0.0000

   THE UNIT WEIGHT OF WATER IS     62.4000

   THE CONTROL QUESTIONS ANSWERED
    1.) PRINT OUT THE INPUT DATA.  IQ(1)=2
    2.) WRITE OUT ALL FACTORS OF SAFETY.  IQ(2)=1
    3.) WRITE OUT A SUMMARY OF THE MINIMUM FACTORS OF SAFETY.  IQ(3)=2
    4.) WRITE OUT A DETAILED SUMMARY OF THE COMPUTATIONS  IQ(4)=2
    7.) DISREGARD THE PREVIOUS PORE PRESSURE DATA.  IQ(7)=1
    8.) SET ALL NEGATIVE PORE PRESSURES TO ZERO.  IQ(8)=1
    9.) PRINT A DIGITAL PLOT OF THE GEOMETRY AND SLIP SURFACE.  IQ(9)=1
```

```
*********************
* SLOPE STABILITY   *
*********************
```

DATE 09-04-85 RUN TRIAL 07

COMMENTS APACHE JCT FRS DRAWDOWN CONDITION US

BISHOP*S SIMPLIFIED METHOD

SPECIAL DETAILS ON THE LAYOUT OF THE CROSS-SECTION

THE COORDINATES OF THE VERTICES OF LINES FORMING THE CONTACTS BETWEEN SOIL LAYERS ARE AS FOLLOWS

LINE NO.	COORDINATES	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
100	X	10.0	65.0	120.0	134.0	178.0	275.0	275.0	275.0	275.0	275.0	275.0	275.0	275.0	275.0	275.0
100	Y	88.0	88.0	110.0	110.0	88.0	86.0	86.0	86.0	86.0	86.0	86.0	86.0	86.0	86.0	86.0
200	X	10.0	65.0	112.5	120.0	135.0	142.5	178.0	275.0	275.0	275.0	275.0	275.0	275.0	275.0	275.0
200	Y	88.0	88.0	88.0	83.0	83.0	86.0	88.0	86.0	86.0	86.0	86.0	86.0	86.0	86.0	86.0
300	X	10.0	275.0	275.0	275.0	275.0	275.0	275.0	275.0	275.0	275.0	275.0	275.0	275.0	275.0	275.0
300	Y	66.0	66.0	66.0	66.0	66.0	66.0	66.0	66.0	66.0	66.0	66.0	66.0	66.0	66.0	66.0

LIMITS OF THE GEOMETRY USED DURING COMPUTATIONS

THE LEFT LIMIT IS 11.00.
THE RIGHT LIMIT IS 274.00.

```
*********************
*  SLOPE STABILITY  *
*********************
```

DATE 05-04-85 RUN TRIAL 07

COMMENTS APACHE JCT FRS DRAWDOWN CONDITION US

BISHOP'S SIMPLIFIED METHOD

SPECIAL DETAILS ON THE SOIL CONDITIONS

SOIL CHARACTERISTICS
****
THE GEOMETRY CONSISTS OF 3 LAYERS WITH THE FOLLOWING CHARACTERISTICS.

LAYER NO.	TOTAL UNIT WEIGHT	COHESION	ANGLE OF INTERNAL FRICTION	PORE-PRESSURE RATIO RU	SOIL DESCRIPTION
1	130.000	0.000	37.000	0.000	COMPACTED SM-SC MATERIAL
2	110.000	0.000	30.000	0.000	FOUNDATION SM-SC MATERIAL
3	-1.000	0.000	0.000	0.000	CALICHE

```
*********************
*  SLOPE STABILITY  *
*********************
```

DATE 09-04-85 RUN TRIAL 07

COMMENTS APACHE JCT FRS DRAWDOWN CONDITION US

BISHOP'S SIMPLIFIED METHOD
SPECIAL DETAILS ON THE PORE PRESSURE CONDITIONS

THE COORDINATES OF THE VERTICES OF LINES FORMING THE PIEZOMETRIC LINES ARE AS FOLLOWS.

LINE NO.	COORDINATES	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
1	X	10.0	65.0	94.2	170.0	178.0	275.0	275.0	275.0	275.0	275.0	275.0	275.0	275.0	275.0	275.0
1	Y	88.0	88.0	99.7	92.0	88.0	85.0	85.0	85.0	85.0	85.0	85.0	85.0	85.0	85.0	85.0
2	X	10.0	65.0	94.2	178.0	275.0	275.0	275.0	275.0	275.0	275.0	275.0	275.0	275.0	275.0	275.0
2	Y	88.0	88.0	99.7	88.0	85.0	85.0	85.0	85.0	85.0	85.0	85.0	85.0	85.0	85.0	85.0
3	X	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
3	Y	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0

```
*********************
*  SLOPE STABILITY  *
*********************
```

DATE 09-04-85 RUN TRIAL 07

COMMENTS APACHE JCT FRS DRAWDOWN CONDITION US

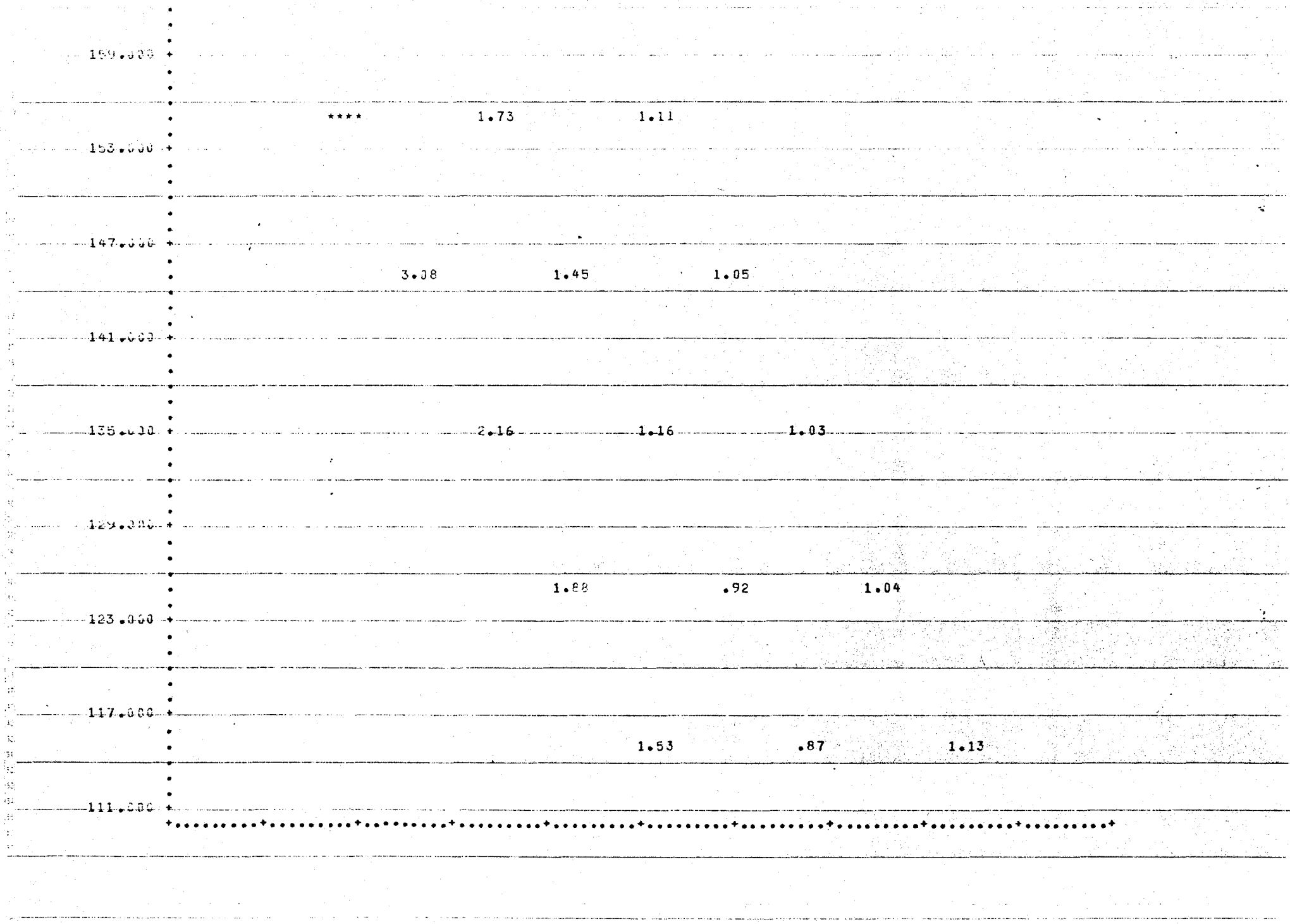
BISHOP'S SIMPLIFIED METHOD

SUMMARY OF MINIMUM FACTORS OF SAFETY

RADIUS INC*T	X-COORD.	Y-COORD.	RADIUS	FACTOR OF SAFETY	NO. ITER*N	MALPHA WARN*G
11	60.0	115.0	33.0	1.533	4	0
9	70.0	115.0	29.0	.871	4	0
10	80.0	115.0	31.0	1.134	4	0
13	35.0	125.0	47.0	1.881	4	0
9	65.0	125.0	39.0	.921	3	0
8	75.0	125.0	39.0	1.037	3	0
14	50.0	135.0	59.0	2.161	4	0
10	60.0	135.0	51.0	1.161	3	0
9	70.0	135.0	49.0	1.027	4	0
12	45.0	145.0	65.0	3.078	3	0
10	55.0	145.0	63.0	1.447	3	0
8	65.0	145.0	59.0	1.051	4	0
1	46.0	155.0	49.0	999.000	0	0
11	50.0	155.0	73.0	1.732	3	0
9	60.0	155.0	69.0	1.110	4	0
1	35.0	165.0	59.0	999.000	0	0
11	45.0	165.0	83.0	2.193	3	0
10	55.0	165.0	81.0	1.317	3	0

THE MOST CRITICAL SLIP SURFACE

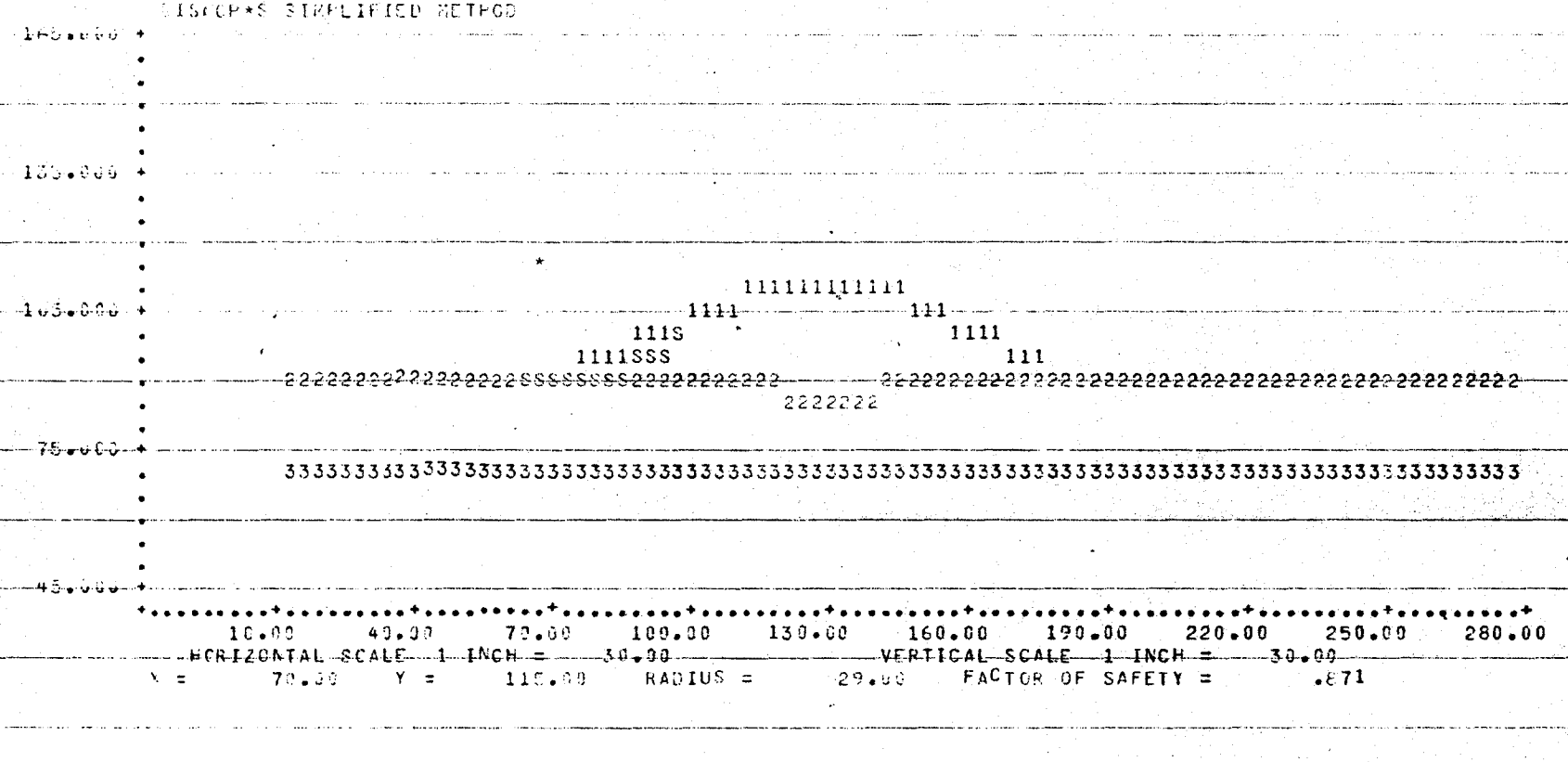
RADIUS INC*T	X-COORD.	Y-COORD.	RADIUS	FACTOR OF SAFETY	NO. ITER*N	MALPHA WARN*G
9	70.0	115.0	29.0	.871	4	0


159.000
153.000
147.000
141.000
135.000
129.000
123.000
117.000
111.000
****
1.73
1.11
3.08
1.45
1.05
2.16
1.16
1.03
1.88
.92
1.04
1.53
.87
1.13
35.00
41.00
47.00
53.00
59.00
65.00
71.00
77.00
83.00
89.00
VERTICAL SCALE 1 INCH =

[redacted] [redacted] [redacted] [redacted] [redacted] [redacted] [redacted] [redacted] [redacted] [redacted] [redacted] [redacted] [redacted] [redacted] [redacted] [redacted] [redacted]

****

ISOCP*S SIMPLIFIED METHOD



HORIZONTAL SCALE 1 INCH = 30.00 VERTICAL SCALE 1 INCH = 30.00

X = 70.00 Y = 110.00 RADIUS = 29.00 FACTOR OF SAFETY = .871