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SERGENT, HAUSKINS & BECKWITH

GEOTECHNICAL DESIGN DEVELOPMENT REPORT
Heap Leach Facility Design
Vulture Mine Project
Near Wickenburg, Arizona

Registered Professional Engineer (Civil)
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SHB Job No. E87-11

Consulting Geotechnical Engineers

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April 10, 1987

A.F. Budge (Mining) Limited
7340 East Shoeman Lane
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SHB Job No. E87-11

Attention: A. J. Fernandez
Senior Mining Engineer

Re: Heap Leach Facility Design
Vulture Mine Project
Near Wickenburg, Arizona

Gentlemen:

Presented herein is our Geotechnical Design Development Report for the leach pad and related facilities proposed for the referenced project. This report presents a description of the major elements of the heap leach facility, and a discussion of the physical constraints, design criteria, and engineering analysis utilized in development of the facility layout and design. A construction cost estimate for development of the leach pad, perimeter channel and berms and ponds is submitted under separate cover.

Should any questions arise concerning this report, please do not hesitate to contact us.

Respectfully submitted,

Sergent, Hauskins & Beckwith Engineers

Registered Professional Engineer (Civil) CERTIFICATE NO. 19677 JAMES R. FAHY Date Signed 4/10/87 ARIZONA, U.S.A.

By ______________________
James R. Fahy, P.E.

And ______________________
Nicholas J. LaFronz, E.I.T.

Registered Professional Engineer (Civil) CERTIFICATE NO. 13872 LAWRENCE A. HANSEN Date Signed 4/10/87 ARIZONA, U.S.A.

Reviewed by ______________________
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1. INTRODUCTION

This report outlines the geotechnical design development studies and engineering analyses for the proposed heap leach facilities at the A. F. Budge (Mining) Limited's (AFBL) Vulture Mine Project located in the Section 36 of Township 6 North, Range 6 West in Maricopa County, Arizona. A preliminary site characterization study was completed by Sergent, Hauskins & Beckwith Geotechnical Engineers, Inc. (SHB) in February, 1987. The preliminary report includes discussions of the geologic, hydrogeologic and seismotectonic setting. The reader is referred to the previous report for complete discussions of these areas, as this report will only briefly include specifics of the previous site study.

The objective of this phase of investigation was to characterize in detail the subsurface conditions in the leach pad and pond areas, and to perform the geotechnical and hydrological engineering evaluations to develop final design criteria for the facility.

2. PROJECT DESCRIPTION

Preliminary details of the project were provided by Mr. A. J. Fernandez, Senior Mining Engineer of AFBL.

The present plan for the project involves heap leaching about 225,000 tons of amalgamated tailings from an

FIGURE 1
PROJECT LOCATION MAP

Heap Leach Facility Design
Vulture Mine Project
Near Wickenburg, Arizona
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CONSULTING GEOTECHNICAL ENGINEERS

existing on-site disposal area followed by 100,000 tons of crushed ore from the Vulture Mine Open Pit. The ore will be crushed to a final product having a nominal maximum particle size of about 1/8 inch. Both the tailings and crushed ore will be agglomerated prior to placement on the pad. The tailings will be agglomerated to a nominal 3/8-inch size, compared to an estimated 1/4 to 3/8-inch agglomerated size for the ore. Both ore and tailings will be leached over an approximate project life of two years.

As presently planned, the leach pad will cover an area of about 225,000 square feet (5.2 acres), with a total heap height of about 75 feet. The heap will be constructed in five lifts, each of which will be 15 feet in height. The on-site tailings will be placed in the lower lifts. Approximately 25,000 square feet of ore or tailings will be under leach at any one time. The solution application rate will be about 0.004 gallons per minute per square foot (gpm/sq. ft.) with a corresponding design solution flow rate of about 100 gpm.

Discussions of the major design features of the heap leach facility are presented in Section 4 of this report.

3. INVESTIGATION

3.1 Previous Investigation

As presented in our previous report, a review of available data in the form of site plans, aerial photographs, topographic and geologic maps and trench data provided by AFBL, as well as published maps and literature, was performed.

A brief preliminary site reconnaissance and subsequent geologic reconnaissance were also performed as part of the preliminary site study.

A prior subsurface investigation consisted of 16 backhoe test pits excavated along the alignments of potential diversion channels and in the proposed leach area. Results of this investigation were also presented in our previous report.

3.2 Subsurface Investigation

A total of ten test borings were advanced in the vicinity of the proposed leach pad facility. The borings were drilled using a CME-55 truck-mounted drill rig advancing hollow stem auger. Locations of the test borings are shown on Plate 1, which is located in the map pocket at the end of this report. Plate 1 also includes the locations of Test Pits 10 through 16 (TP-10 through TP-16) completed as part of the preliminary site characterization for this project. Standard penetration testing and/or thin-wall Shelby tube sampling was performed in all of the test borings. Surficial bulk samples of both tailings and native alluvial materials were also recovered.

Soils encountered were classified using the Unified Soil Classification System (ASTM D2487), a summary of which is presented in Appendix A. Appendix A also includes a brief description of drilling and sampling equipment and procedures, a summary of terminology used to describe the relative density, consistency or firmness of soils, a summary of terminology used for the description of rock and logs of the test borings. Logs of test pits TP-10 through TP-16 are included in our previous report.

The field investigation was supervised by Andrew F. Harvey, staff geologist of this firm.

3.3 Laboratory Analysis

Grain-size analysis, Atterberg limits and moisture content tests were performed on selected samples to aid in classification. Four consolidation-collapse tests were conducted on selected tube samples in order to estimate the potential for collapse upon saturation of the existing tailings foundation materials. Two moisture-density relationship tests (Standard Proctor) were performed on bulk samples of both tailings and native alluvial materials to assess the suitability of these soils for use as structural fill.

Results of the laboratory investigation are presented in Appendix B of this report. Results of the moisture content tests are presented on the boring logs in Appendix A.

Laboratory remolded permeability and moisture-density relationship (Standard Proctor) tests on a sample of material from a potential clay borrow source are presently in progress. Results of these tests will be presented in an addendum to this report when completed.

Chemical testing of the existing tailings, in order to further define the suitability of these materials for structural fill, is also currently in progress.

3.4 Geotechnical Profile - Pad & Pond Areas

Existing tailings fill materials were encountered in all but two of the borings in the pad and pond areas to depths of 2 to 19 feet below existing grade. The tailings are generally described as fine grained silty sands, soft to firm in consistency, nonplastic and light brown in color. In general, the tailings thickness tends to increase from southwest to northeast.

As indicated by the boring logs, the tailings are underlain by alluvium generally composed of well graded sand and gravel with occasional zones of silty and clayey sand and gravel. These soils are typically medium dense to very dense in consistency, nonplastic and reddish brown to brown to light brown in color. They extended to the full depth of investigation in all but three of the borings.

Bedrock was encountered in three of the borings (Borings 7, 9 and 10) at depths of 14 to 24 feet below existing grade. The bedrock is described as Precambrian schist, strongly weathered, moderately soft to moderately hard and green to brown to reddish brown in color.

No groundwater was encountered in any of the test borings. Soil moisture contents were generally relatively low, on the order of 1 to 5 percent.

4. DESIGN FEATURES

A discussion of the major design criteria used in project development and a description of the principal features of the heap leach facility are presented in the following sections. Geotechnical and hydrological evaluations performed in developing designs for these features are summarized in Sections 5, 6 and 7. Guide specifications for subgrade preparation and earthwork construction elements are presented in Appendix D. A plan view of the leach pad and pond layout is shown on Plate 1, included in the map pocket.

4.1 Mass Grading of Pad, Perimeter Channels & Pond Areas

All vegetation and debris must be initially cleared and stripped from the entire area of the leach pad, perimeter channels and ponds within the limits of construction, as shown on Plate 1. Excavation volumes will closely balance fill volumes for the entire project.

The leach pad will be graded at a 1 percent slope from the toe of the pad to a distance of 75 feet upslope. This gently sloping corridor is provided to increase the stability at the toe of the ore pile. The pad will then be graded at 3.5 percent from the edge of the toe corridor to the upslope edge of the pad.

The solution collection channel at the toe of the pad will be graded at 0.5 percent to the pregnant pond inlet spillway at the southwest end of the pad. This inlet spillway will be graded at about 3.5 percent to the crest of the pregnant pond. A detail of the spillway is shown on Plate 2 in the map pocket.

4.2 Leach Pile Configuration

Ore placement by conveyor stacking is presently being considered. The ore will be placed in lifts of about 15 feet in height. Ultimate heap heights of 61 feet on the upslope end and 75 feet on the downslope end of the pad are planned. The heaps will be built en echelon one above the other, with a 5-foot wide bench provided every 15 feet of pile height for reasons of pile stability.

Based on our experience with similar leach facilities and published data for hard rock waste dumps, it is estimated that the lifts will slope about 1.73:1 (horizontal to vertical), or about 30 degrees, and 1.43:1 (or 34 degrees) for the tailings and crushed ore, respectively. The average slope of the ore pile, considering the 5-foot bench for each lift, will be about 1.89:1. A 10-foot wide open corridor will be provided between the toe of the heap and the perimeter of the pad to facilitate maintenance and to allow for rockfalls and minor sloughing of the pile.

A typical cross section of the leach pile configuration is shown on Plate 2.

4.3 Leach Pad Lining & Related Details

The leach pad will be lined with 30-mil polyvinyl chloride (PVC) and 36-mil industrial grade reinforced

Hypalon. Due to susceptibility to liner degradation and embrittlement from ultraviolet radiation attack, the PVC geomembrane will only be used to line the pad area that will be covered by the ore. The Hypalon liner will be connected to the PVC liner by a transition strip or a factory made dielectric weld and will cover a 15-foot wide corridor at the perimeter of the pad. Hypalon possesses excellent durability and resistance to physical degradation due to ultraviolet radiation and thermal oxidation attack.

Compression testing performed by SHB for similar projects was reviewed in evaluating potential geomembrane liners for the leach pads. This testing included geomembrane liners of varying thickness and various soil materials in order to assess the performance of the liners under service loading conditions. A summary of these investigations is presented in Section 8.

4.4 Perimeter Channel

4.4.1 Channel Geometry

A solution collection channel will be located along the toe of the leach pad, as shown on Plate 2, Cross Section A, to transport pregnant solution and storm water runoff from the leach pad to the ponds. The channel is bordered on the west side by a 5-foot high perimeter containment berm to protect against any solution discharge from the facility during an extreme storm event. The solution channel is bordered on the pad side by a 2-foot high berm that provides a protective barrier between the heap and the solution channel. The random fill berms will have 2 1/2:1 side slopes.

The V-shaped pregnant solution channel, as shown in Cross Section A on Plate 2, will be 2 feet deep with 2.5:1 side slopes. The channel has a discharge capacity equal to the design operating solution flow rate of 100 gpm with a 1.8-foot freeboard, and a discharge capacity in excess of 44,000 gpm with no freeboard.

The full channel geometry created by the safety berm and perimeter containment berm will have a capacity in excess of that required to transmit flows equal to the 6-hour probable maximum precipitation (PMP). The design channel section has an area of 10 square feet. At the minimum grade of 0.5 percent, only about 6 square feet will be needed to convey flows of 42 cfs or one-half the 6-hour PMP, which is the design storm for the facility. Hydrologic and hydraulic analyses are presented in Section 4. Typical channel sections are shown on Plate 2 in the map pocket.

4.4.2 Channel Lining

The solution channels will be fully lined with 36-mil Hypalon. The finished grading of the perimeter channels is important in providing a smooth subgrade for liner placement. It is recommended that the excavated tailings from the leach pad area be utilized as channel and berm fill. This will enable a very smooth finished surface to be constructed beneath the liner, reducing the potential for liner puncture or surface abrasion.

4.4.3 Pregnant Pond Inlet Spillway

Pregnant solution will be directed from the channel down a 3.5 percent spillway to the pregnant pond inlet. The embankment bordering the spillway varies in height from about 9 feet at the north end to about zero feet at the south end (intersection with the crest of the pond).

4.5 Barren, Pregnant & Storm Water Ponds

4.5.1 Pregnant & Barren Pond Geometry

The volume requirements for the pregnant and barren ponds were established based on the operating solution flow of 100 gpm over a 72-hour period, or about 432,000 gallons. The barren pond was sized to store this solution volume with 2 feet of freeboard, whereas the pregnant pond has a total capacity of about 480,000 gallons, with about 48,000 gallons of storm flow storage. A shallow spillway connecting the pregnant and surge ponds is also provided for transfer of excess storm water.

The barren pond will have dimensions of 105 feet by 105 feet at the crest, with 3:1 side slopes and a depth of 12 feet. The pregnant pond will have similar crest dimensions and side slopes with a depth of 15 feet. Sumps for leakage detection will be provided in both ponds.

Plan views and details of the pregnant and barren ponds are presented on Plates 1 and 2 in the map pocket.

4.5.2 Storm Water (Surge) Pond Geometry

The volume requirement of the storm water pond was established based on the runoff from one-half the six-hour PMF (Probable Maximum Flood), as determined by Hydrometeorological Report No. 49 (U.S. Department of Commerce, 1977)*. For comparison of storage requirements, total runoff volumes from the 100-year and 500-year, 24-hour floods and the six-hour PMP were computed. A summary of these analyses are included in Section 5.

*References are provided at the end of this report.

Selection of storage criteria is a complex risk analysis problem, which is essentially a matter of engineering judgment. The final design philosophy was that of a "zero discharge" facility in which all runoff would be stored by the system.

The storm water pond dimensions will be 115 feet square at the crest, with 3:1 side slopes and a 15-foot depth. A storage capacity of about 0.6 million gallons is provided. A plan view and details of the storm water pond are shown on Plates 1 and 2 included in the map pocket.

4.5.3 Grading & Embankment Construction

The ponds will be primarily below existing grade with a maximum berm height of about 3 feet on the south ends. Embankments will be constructed of random fill from the excavations compacted to at least 95 percent of maximum dry density as determined by ASTM D698. The moisture content during compaction should be maintained within the limits of 2 percent below to 3 percent above optimum moisture content as determined in accordance with ASTM D698. Minimal processing of the excavation materials will be required to discard particles larger than 6 inches nominal diameter.

4.5.4 Pregnant & Barren Pond Lining Systems

A double liner system which incorporates a central geotextile drainage blanket and leakage detection sump will be utilized for lining the pregnant and barren solution ponds. The double liner and leakage detection system will provide a second line of defense should leakage through the primary liner occur. Any leakage through the primary liner will be transmitted through the central drainage layer to a lined sump. Details of each of these features are provided in the following sections and on Plate 2.

4.5.4.1 Primary & Secondary Geomembranes

The primary geomembrane liner for the pregnant and barren ponds will be 36-mil Hypalon. A 20-mil PVC geomembrane will be placed throughout the pregnant and barren ponds as the secondary pond liner. Both liners provide good resistance to organic and inorganic acids and bases, heavy metals and salts.

PVC is not planned for use as the exposed liner due to its susceptibility to degradation and embrittlement from ultraviolet radiation attack. An extensive case history exists for Hypalon and PVC installations at mining facilities located in both hot and cold climate regions.

4.5.4.2 Drainage Blanket

A 16-ounce per square yard, nonwoven geotextile will be incorporated as the drainage layer between the primary and secondary liners. Trevira 1155, Mirafi 1160N, or Supac 16NP are acceptable geotextiles. The geotextile blanket will provide as a drainage layer to the sump and for any seepage through the primary liner, and as a separator between the Hypalon and PVC liners.

4.5.4.3 Leak Detection Sump

The pond floors will be graded at 0.5 percent to a sand-filled sump for collection of any solution in the event of a leakage through the primary geomembrane liner. The sump will be lined with 30-mil PVC and underlined with 20-mil PVC as shown in detail on Plate 2.

A 4-inch diameter PVC pipe will extend from the sump to the pond crest to provide access for monitoring potential leakage and pumping should solution fill the sump. Details of the leakage detection sump are shown on Plate 2.

4.5.4.4 Pregnant Pond Splash Pad

A shotcrete splash pad approximately 25 feet wide will extend from the pregnant channel inlet to the floor of the pregnant solution pond as shown in detail on Plate 2. The splash pad will provide scour protection at the pond entrance required to maintain the integrity of the geomembrane lining system.

4.5.5 Storm Water (Surge) Pond Lining

The storm water pond will be lined with 20-mil PVC. Due to the ultraviolet sensitivity of the PVC, a 2-foot thickness of compacted tailings will be placed over the geomembrane. A 6-inch compacted layer of tailings will underlie the PVC to provide a smooth subgrade surface.

Minor erosion of the compacted tailings overliner in the surge pond is anticipated during rainfall events. Periodic maintenance of the tailings overliner by mine personnel will be required to ensure that the PVC liner is not exposed. Due to the estimated short-term project life, the maintenance required should be minimal.

Compaction requirements for placement of the tailings are provided in Sections 3.2 and 3.3 of the Guide Specifications.

The surge pond side slope will be lined with 36-mil Hypalon at the spillway entrance from the pregnant pond. The geomembrane will provide additional erosion control should solution enter the surge pond.

5. HYDROLOGIC & HYDRAULIC ANALYSES

5.1 Diversion of Watershed Runoff

Hydrologic analyses for storm runoff from the contributing watershed were summarized in detail in our Preliminary Site Characterization Report and, therefore, will not be discussed herein. Diversion of runoff from the watershed will be provided by a trapezoidal channel, located as shown on Plate 1. The diversion channel will be 10 feet wide at the base with a height of 5 1/2 feet. Side slopes will be 2 1/2:1. The diversion has a discharge capacity of about 875 cfs or approximately the equivalent of the 100-year, 24-hour storm event for the 4.4 square mile watershed. The channel surface will be lined with shotcrete for erosion protection.

5.2 Leach Pad, Channel & Ponds

5.2.1 General

Hydrologic calculations presented herein include only runoff from the leach pad area, which was used to develop perimeter channel and storm water pond dimensions. Local on-site drainage will be collected and retained so that no precipitation that falls on the leach pads will escape the containment system. A detailed summary of the hydrologic and hydraulic analyses utilized in design of the solution channel and storm water pond are presented in the following sections.

5.2.2 Calculation Standards, Methods & Procedures

Peak runoff flow quantities for the ore pile area were calculated using the U.S. Soil Conservation Service TR-55 method (McCuen, 1982). The method is based on a computational procedure that is related to the 24-hour storm rain depth. Runoff volumes were calculated based on the total volume of storm rainfall.

For purposes of comparison, rain depth and rainfall distributions for the 100 and 500-year, 24-hour storms and the PMP and 1/2 PMP, 6-hour events were calculated using procedures outlined in The National Weather Service Hydrometeorological Report No. 49 (U.S. Department of Commerce, 1977). Storm volumes and peak flows for one half of the six-hour PMP event were used in developing the design capacity of the storm water pond. Peak runoff volumes and flows for this storm event are essentially equivalent to those of the interpolated 500-year, 24-hour storm. Although storage of the full PMP event is not provided, statistically a "zero discharge" facility is being designed, considering the estimated operation life of the project.

Rainfall depths for the 100 and 500-year storm events are based on the Precipitation Frequency Atlas for Arizona, Volume VIII, (U.S. Department of Commerce, 1973). The velocity method (McCuen, 1982) presented below was used for estimating the times of concentration.

$$tc = \frac{HL}{3600\ V}$$

where: tc = time of concentration, hours

HL = length of watershed subarea (hydraulic length), miles

V = velocity of storm runoff, fps

Leach pad hydrology calculations are presented in Appendix C.

Hydraulic calculations performed for determining channel capacities, flow depths and flow velocities are based on the Manning's Equation (Chow, 1959). A roughness coefficient of 0.01 was used in analysis of the Hypalon-lined solution channels. Computer summary sheets of the channel hydraulic calculations are included in Appendix C.

6. STABILITY ANALYSES OF LEACH PILE

Stability analyses were conducted for the leach pile in which various potential failure modes, including shallow and deep-seated failure surfaces within the ore pile, and shallow block failures along the ore-geomembrane interface were analyzed. A summary of these analyses, including the major assumptions and factors considered in the analyses, are discussed in the following sections.

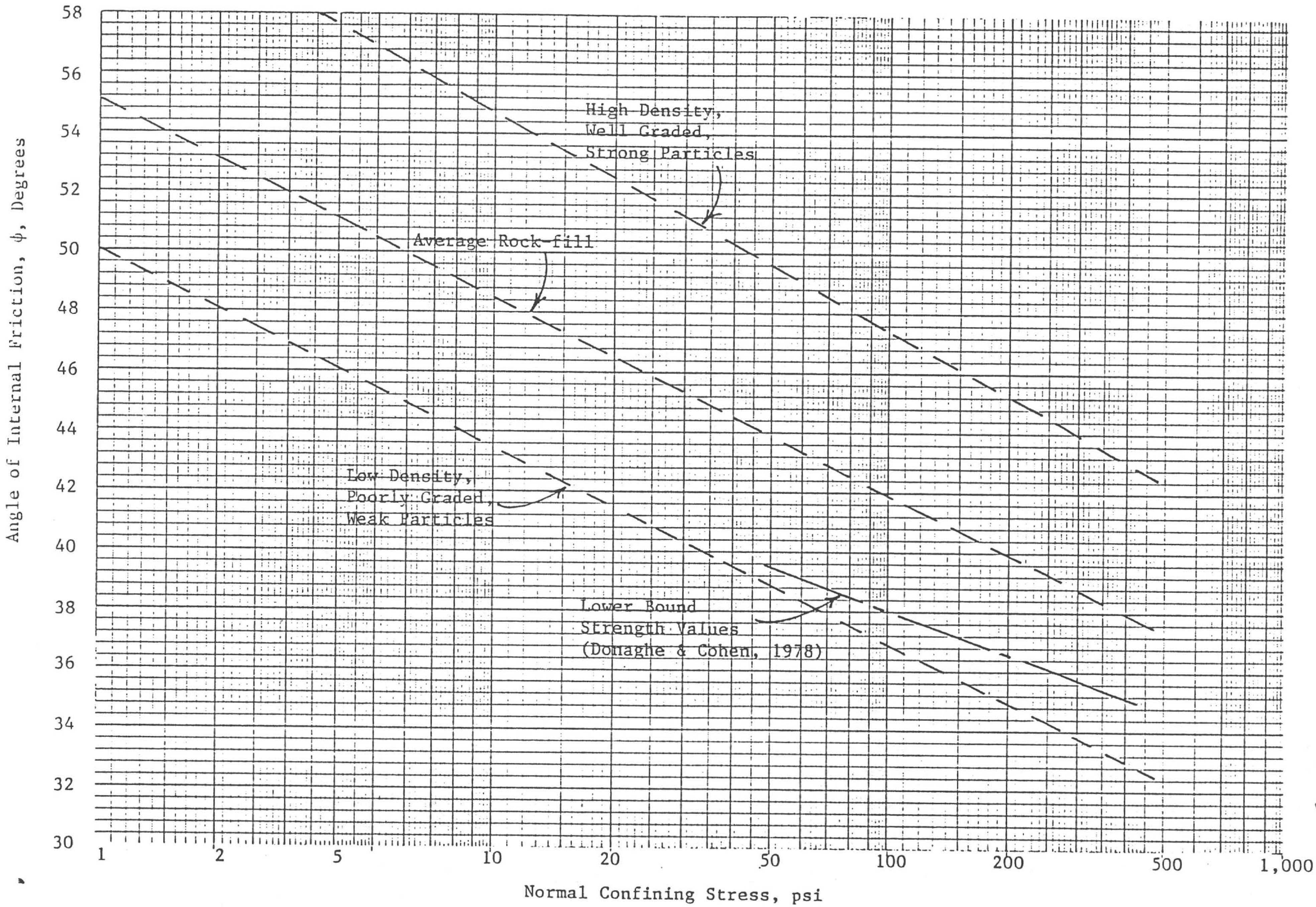
6.1 Shear Strength of Ore & Tailings

Generally, leach ore materials are difficult to test during design because they are not usually accessible for sampling and the rocks involved are highly variable in most ore bodies. Most rock involved in leach piles for "hard rock" mining of gold, silver, copper, lead, zinc and other metals fall within the range of materials which have been tested for rock-fill dam design. A widely used approach in dump design is to conservatively estimate shear strength based on this data (Call, 1981; Caldwell and others, 1981).

Figure 2 presents a summary of the large amount of triaxial shear testing done in the past 38 years on rock-fill materials as reported in the engineering literature. This chart, which has been widely

FIGURE 2

Angle of Internal Friction Versus Confining Stress



used in rock dump design, was originally based on a summary of shear strength data on quarried rock and coarse gravels by Leps (1970). Figure 2 shows the relationship between the angle of internal friction (ϕ) of broken rock and the normal stress across the shear plane (vertical confining pressure). The limits of test data available at the time and the average curve for rock-fill are given on the chart. Subsequent data on the shear strength of rock-fill (Marsal, 1974; Donaghe and Cohen, 1978; Barton and Kjaersli, 1981) are consistent with Figure 2.

The lower limit of tests on seven rock-fill materials performed by the U.S. Army Corps of Engineers (Donaghe and Cohen, 1978) is also shown in Figure 2. This lower limit was defined by weathered, relatively soft granite. A conservative angle of internal friction equal to 34 degrees was selected for design analysis involving the crushed leach ore. This conservative value accounts for a reduction in particle angularity due to agglomeration of the ore, and any variation in rock quality and durability. The existing amalgamated tails will be agglomerated to about 3/8 inch. A conservative angle of internal friction of 30 degrees was assigned to these materials.

6.2 Possible Accumulation of Solution in the Leach Pile

Due to the effects of agglomeration, an openwork drainage condition is anticipated at the base of the pile. In addition, 3-inch diameter perforated drain pipes will be provided on the liner surface directly below the ore pile to enhance lateral drainage. Therefore, the possible development of a phreatic surface within the leach pile above 1 to 2 feet is considered remote. Saturated conditions within the ore pile to this height would not significantly affect pile stability.

It is our opinion and experience with other leach facilities that a much greater risk exists due to broken sprinkler lines and ponding of solution near the crest of the pile. This risk is reduced to a low level if proper leach pad maintenance is followed. Nonetheless, allowance in design was made for sloughing or shallow slides of the pile slopes by providing a 10-foot wide open corridor between the toe of the pile and the solution channel berm, and by the inclusion of 5-foot wide benches at the top of each 15-foot lift height.

6.3 Stability Analysis Methodology & Results

6.3.1 Shallow Sloughing & Rock Falls

The ore pile will be constructed in 15-foot lifts at slopes equal to the angle of repose of the agglomerated tailings and crushed ore. Shallow wedge-type and circular failure surfaces possess very low factors of safety for cohesionless materials placed in this manner. Just back of the slope, factors of safety approach 1.0. Minor sloughing and rock falls from the slope face are commonly caused by external driving forces such as wind, rain and ice. Although the extent of sloughing is anticipated to be very minor, the 10-foot wide corridor at the toe of the pad, the 5-foot pile benches, and the solution containment berm were, in part, designed to accommodate this occurrence.

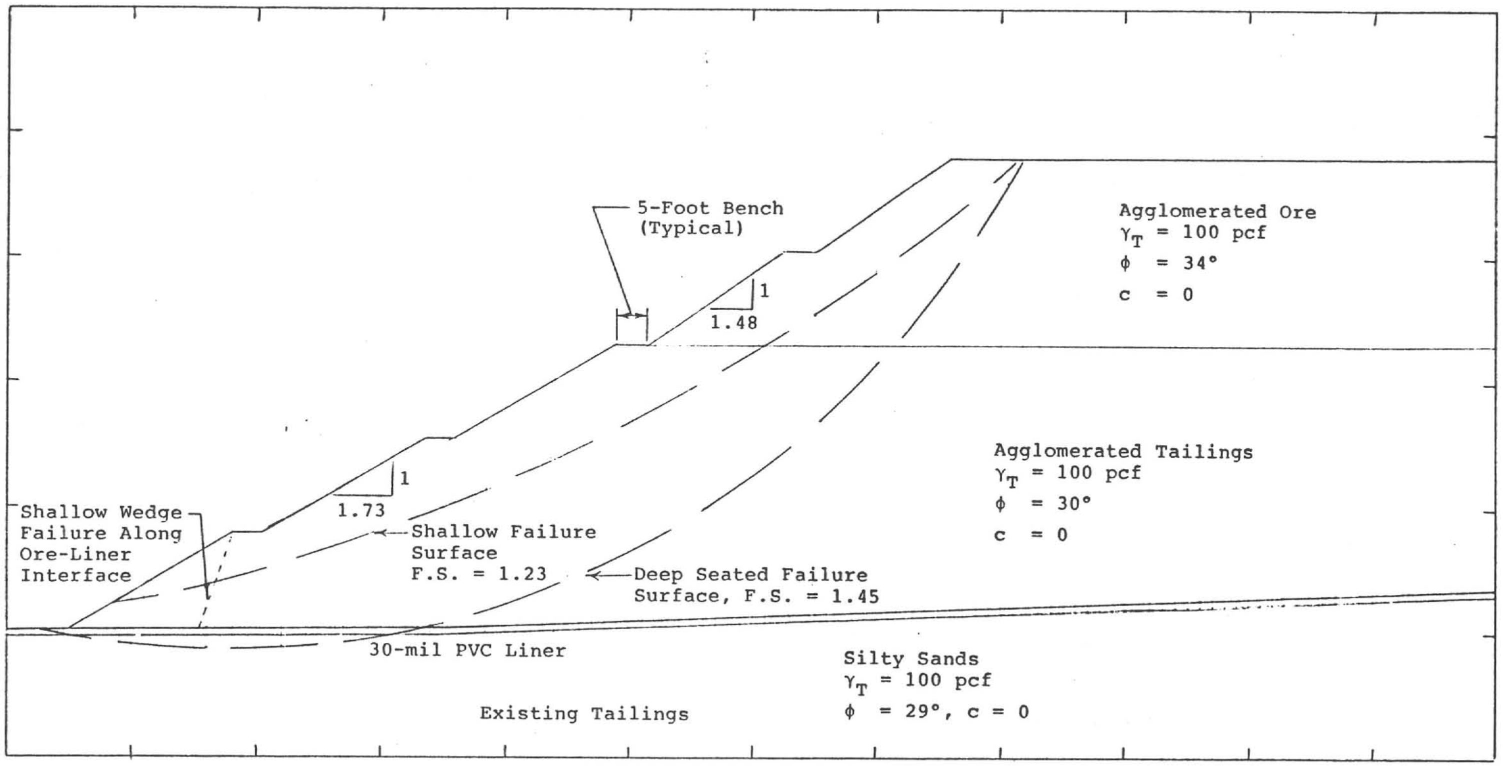
6.3.2 Potential Toe Failures

Potential toe failures along the liner-subgrade interface, as shown in Figure 3, were analyzed by a method presented by Blight (1981), which covers the case of a frictional foundation.

An important parameter required for this analysis is the estimated friction angle along the geomembrane-ore interface. Large-scale direct shear tests for various geomembrane cover material interfaces were performed by SHB as part of a special laboratory testing program for three previous leach pad designs (Deatherage, Fahy and Hansen, 1986). Resulting friction angles varied from 8.5 to 39 degrees for the PVC liner material tested, which were a function of cover soil and applied normal stress. Friction angles for mine tailings placed on PVC ranged from 12 to 13.5 degrees for the lower normal stresses.

FIGURE 3

Maximum Heap Leach Pile Cross Section
for Stability Analyses



Scale, Feet
0 10 20

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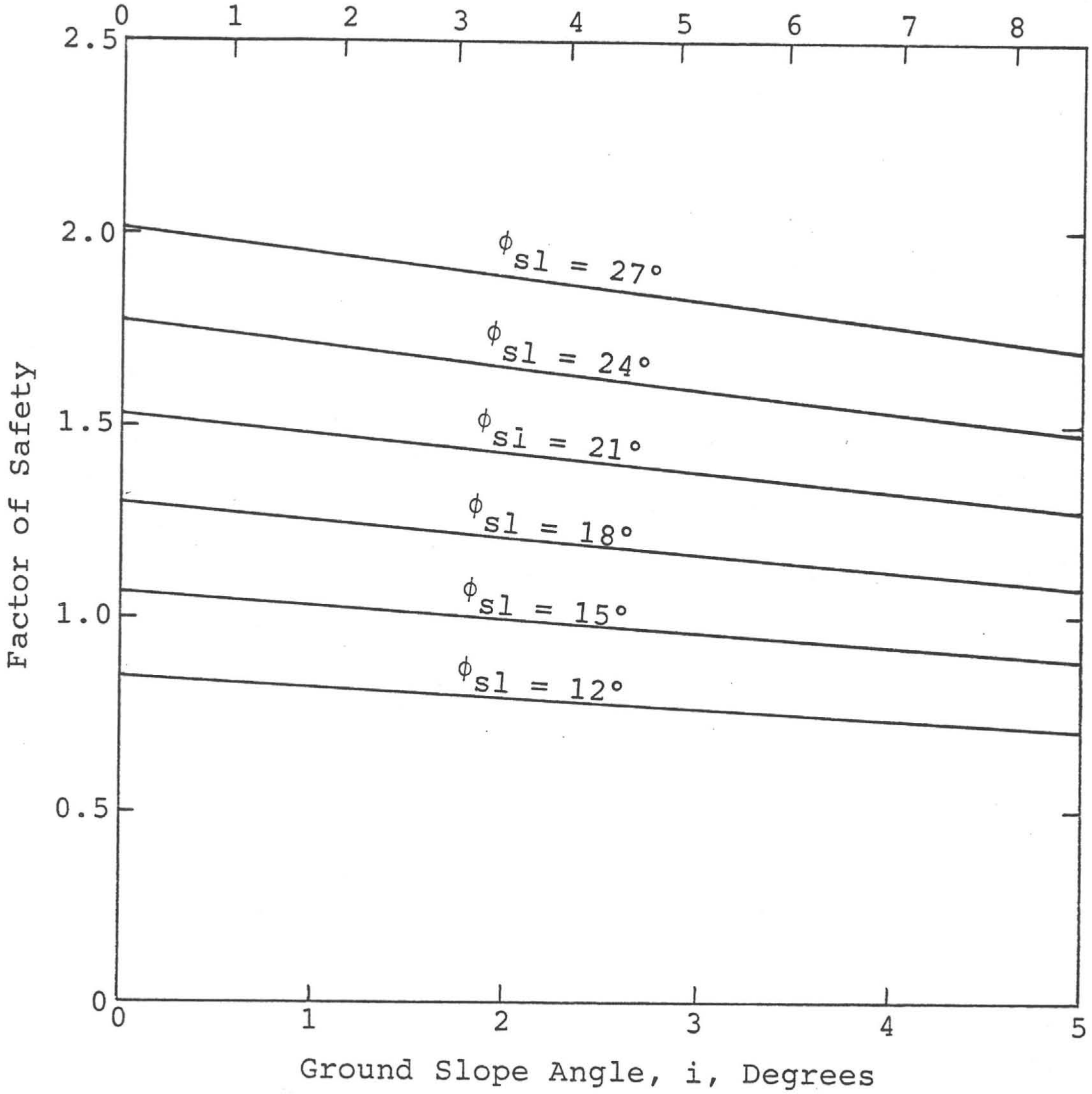
A parametric study showing the relationship between foundation slopes (i), foundation friction angle (ϕ_f), and factor of safety is presented in Figure 4. Considering a shallow wedge-type failure within the 75-foot wide pad toe corridor, which is designed at a 1 percent slope, the factor of safety for a PVC-tailings interface friction angle (ϕ_{sl}) of 12 degrees is about 0.8. To increase the friction angle at the toe of the pad and, correspondingly, the factor of safety, a 1-foot thick layer of sand and gravel will be placed along a 50-foot wide corridor at the toe of the pile. The sand and gravel layer will increase frictional response along the liner surface. The estimated soil-liner friction angle for these materials is about 17 to 18 degrees. The resulting safety factor against shallow block failure is about 1.2.

6.3.3 Generalized Failure Surface Analyses

Shallow and deep-seated failure surfaces within the ore pile were analyzed using the computer program STABL (Siegel, 1975a, 1975b). This program considers a generalized shear surface utilizing a limiting equilibrium method of slices. Noncircular trial shear surfaces are generated and analyzed to determine the critical shear surface. Both static and pseudostatic analyses are possible with this program.

FIGURE 4

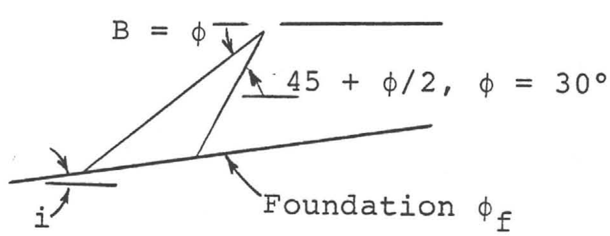
Ground Slope Angle Versus Factor of Safety for Varying Liner-Soil Friction Angles



$$F.S. = \frac{\tan \phi_{sl}}{\tan \phi_f}$$

ϕ_{sl} = Friction Angle of Soil-Liner Interface

ϕ_f = Friction Angle of Foundation for Limiting Equilibrium State



Heap Leach Facility Design
Vulture Mine Project
Near Wickenburg, Arizona
SHB Job No. E87-11

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In the analysis procedure, parallel side shear forces are assumed, vertical equilibrium of the individual slices is satisfied, and overall moment equilibrium is satisfied, but moment equilibrium of the individual slices is not satisfied. The computed factor of safety is conservative relative to more accurate methods satisfying complete equilibrium.

As shown on Figure 2, for the 1.9:1 (28°) equivalent slope and internal friction angles of 30° and 34° for the tailings and crushed ore, respectively, a factor of safety of 1.2 was determined for shallow failure surfaces. As previously stated, for ore placed at the angle of repose the factor of safety approaches 1.0 near the slope face. Deeper seated failure surfaces have higher factors of safety on the order of 1.4 to 1.5.

6.3.4 Deformation & Settlement

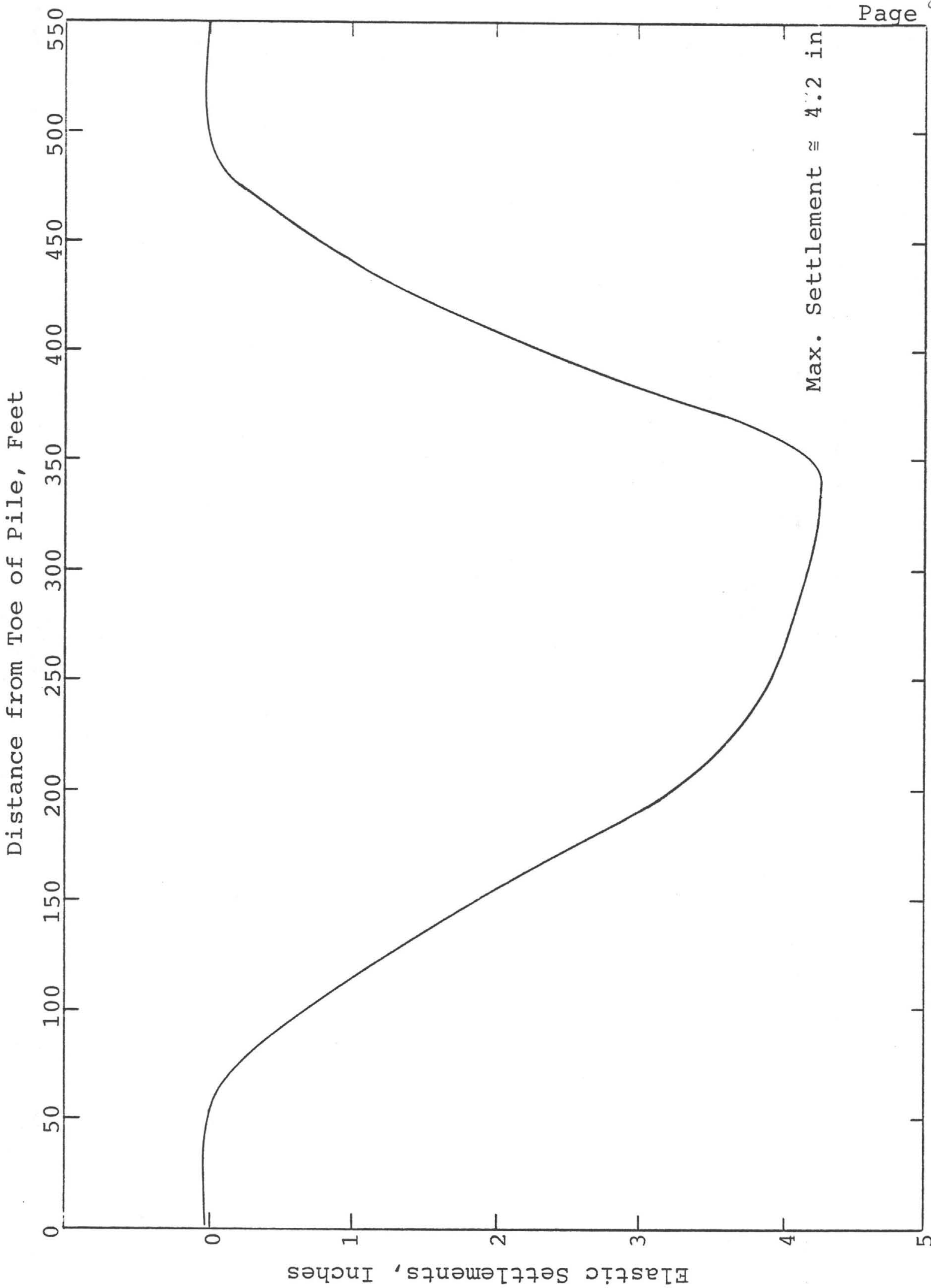
6.3.4.1 Settlement Analysis

Settlement analyses were based on elastic theory settlement relationships developed by Harr (1966) for loading on a rectangular area. The thickness of compressible foundation material was estimated to vary from 13 to 34 feet below the pile. A two-layer substrata was analyzed. A conservative equivalent deformation modulus for the two-strata profile of 2 ksi was utilized. Equivalent moduli were determined using the relationship developed by Thenn de Barros (1966). The underlying Precambrian metamorphic/igneous bedrock complex was assumed incompressible. The increase in stress on the foundation material was estimated using relationships developed by Osterberg (1957). The estimated settlement profile is presented in Figure 5. A maximum elastic settlement of about 4 inches was estimated.

Consolidation tests performed on relatively undisturbed samples of the surficial tailings deposits indicate collapse potential of between 3 and 6 percent of the stratum thickness upon saturation. Saturation or even significant moisture increase in the subgrade tailings is not anticipated provided proper liner installation techniques are followed. Should significant solution leakage into the underlying tailings occur during the project life, additional settlements of between 5 and 8 inches are possible in the pad areas underlain by the thickest deposit of tailings. The PVC liner in the pad area can withstand elongations of up to 300 percent prior to break. Liner rupture should not occur even if differential settlements of 12 inches occur across the width of the pad.

FIGURE 5

Estimated Elastic Settlement of Foundation Materials for Maximum Heap Loading



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APPENDIX A

TEST DRILLING EQUIPMENT & PROCEDURES

Drilling Equipment Truck-mounted CME-55 drill rigs powered with 4 or 6 cylinder Ford industrial engines are used in advancing test borings. The 4 cylinder and 6 cylinder engines are capable of delivering about 4,350 and 6,500 foot/pounds torque to the drill spindle, respectively. The spindle is advanced with twin hydraulic rams capable of exerting 12,000 pounds downward force. Drilling through soil or softer rock is performed with 6 1/2 O.D., 3 1/4 I.D. hollow stem auger or 4 1/2 inch continuous flight auger. Carbide insert teeth are normally used on the auger bits so they can often penetrate rock or very strongly cemented soils which require blasting or very heavy equipment for excavation. Where refusal is experienced in auger drilling, the holes are sometimes advanced with tricone gear bits and NX rods using water or air as a drilling fluid. Where auger and tricone gear bits cannot be used to advance the hole due to cobbles or caving conditions, the ODEX (overburden drilling with the eccentric method) is used. A percussion down-the-hole hammer underreams the hole and 5 inch steel casing is introduced into the hole during drilling. The drill bit is eccentric and can be removed from the center of the casing to allow sampling of the material below the bit penetration depth.

Sampling Procedures Dynamically driven tube samples are usually obtained at selected intervals in the borings by the ASTM D1586 procedure. In many cases, 2" O.D., 1 3/8" I.D. samplers are used to obtain the standard penetration resistance. "Undisturbed" samples of firmer soils are often obtained with 3" O.D. samplers lined with 2.42" I.D. brass rings. The driving energy is generally recorded as the number of blows of a 140 pound 30 inch free fall drop hammer required to advance the samplers in 6 inch increments. However, in stratified soils, driving resistance is sometimes recorded in 2 or 3 inch increments so that soil changes and the presence of scattered gravel or cemented layers can be readily detected and the realistic penetration values obtained for consideration in design. These values are expressed in blows per foot on the logs. "Undisturbed" sampling of softer soils is sometimes performed with thin walled Shelby tubes (ASTM D1587). Where samples of rock are required, they are obtained by NX diamond core drilling (ASTM D2113). Tube samples are labeled and placed in watertight containers to maintain field moisture contents for testing. When necessary for testing, larger bulk samples are taken from auger cuttings.

Continuous Penetration Tests Continuous penetration tests are performed by driving a 2" O.D. blunt nosed penetrometer adjacent to or in the bottom of borings. The penetrometer is attached to 1 5/8" O.D. drill rods to provide clearance to minimize side friction so that penetration values are as nearly as possible a measure of end resistance. Penetration values are recorded as the number of blows of a 140 pound 30 inch free fall drop hammer required to advance the penetrometer in one foot increments or less.

Boring Records Drilling operations are directed by our field engineer or geologist who examines soil recovery and prepares boring logs. Soils are visually classified in accordance with the Unified Soil Classification System (ASTM D2487) with appropriate group symbols being shown on the logs.

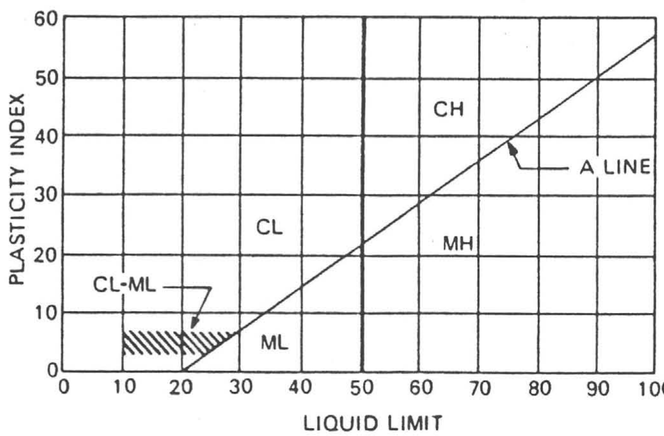
UNIFIED SOIL CLASSIFICATION SYSTEM

Soils are visually classified by the Unified Soil Classification system on the boring logs presented in this report. Grain-size analysis and Atterberg Limits Tests are often performed on selected samples to aid in classification. The classification system is briefly outlined on this chart. For a more detailed description of the system, see "The Unified Soil Classification System" Corp of Engineers, US Army Technical Memorandum No. 3-357 (Revised April 1960) or ASTM Designation: D2487-66T.

MAJOR DIVISIONS				GRAPHIC SYMBOL	GROUP SYMBOL	TYPICAL NAMES
COARSE-GRAINED SOILS (Less than 50% passes No. 200 sieve)	GRAVELS (50% or less of coarse fraction passes No. 4 sieve)	CLEAN GRAVELS (Less than 5% passes No. 200 sieve)			GW	Well graded gravels, gravel-sand mixtures, or sand-gravel-cobble mixtures.
					GP	Poorly graded gravels, gravel-sand mixtures, or sand-gravel-cobble mixtures.
		GRAVELS WITH FINES (More than 12% passes No. 200 sieve)	Limits plot below "A" line & hatched zone on plasticity chart		GM	Silty gravels, gravel-sand-silt mixtures.
			Limits plot above "A" line & hatched zone on plasticity chart		GC	Clayey gravels, gravel-sand-clay mixtures.
	SANDS (More than 50% of coarse fraction passes No. 4 sieve)	CLEAN SANDS (Less than 5% passes No. 200 seive)			SW	Well graded sands, gravelly sands.
					SP	Poorly graded sands, gravelly sands.
		SANDS WITH FINES (More than 12% passes No. 200 sieve)	Limits plot below "A" line & hatched zone on plasticity chart		SM	Silty sands, sand-silt mixtures.
			Limits plot above "A" line & hatched zone on plasticity chart		SC	Clayey sands, sand-clay mixtures.
FINE-GRAINED SOILS (50% or more passes No. 200 sieve)	SILTS Limits plot below "A" line & hatched zone on plasticity chart	SILTS OF LOW PLASTICITY (Liquid Limit Less Than 50)			ML	Inorganic silts, clayey silts with slight plasticity.
		SILTS OF HIGH PLASTICITY (Liquid Limit More Than 50)			MH	Inorganic silts, micaceous or diatomaceous silty soils, elastic silts.
	CLAYS Limits plot above "A" line & hatched zone on plasticity chart	CLAYS OF LOW PLASTICITY (Liquid Limit Less Than 50)			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.
		CLAYS OF HIGH PLASTICITY (Liquid Limit More Than 50)			CH	Inorganic clays of high plasticity, fat clays, sandy clays of high plasticity.

NOTE: Coarse grained soils with between 5% & 12% passing the No. 200 sieve and fine grained soils with limits plotting in the hatched zone on the plasticity chart to have double symbol.

PLASTICITY CHART



DEFINITIONS OF SOIL FRACTIONS

SOIL COMPONENT	PARTICLE SIZE RANGE
Cobbles	Above 3 in.
Gravel	3 in. to No. 4 sieve
Coarse gravel	3 in. to ¾ in.
Fine gravel	¾ in. to No. 4 sieve
Sand	No. 4 to No. 200
Coarse	No. 4 to No. 10
Medium	No. 10 to No. 40
Fine	No. 40 to No. 200
Fines (silt or clay)	Below No. 200 sieve

TERMINOLOGY USED TO DESCRIBE THE RELATIVE DENSITY, CONSISTENCY OR FIRMNESS OF SOILS

The terminology used on the boring logs to describe the relative density, consistency or firmness of soils relative to the standard penetration resistance is presented below. The standard penetration resistance (N) in blows per foot is obtained by the ASTM D1586 procedure using 2" O.D., 1 3/8" I.D. samplers.

1. Relative Density. Terms for description of relative density of cohesionless, uncemented sands and sand-gravel mixtures.

N	Relative Density
0-4	Very loose
5-10	Loose
11-30	Medium dense
31-50	Dense
50+	Very dense

2. Relative Consistency. Terms for description of clays which are saturated or near saturation.

N	Relative Consistency	Remarks
0-2	Very soft	Easily penetrated several inches with fist.
3-4	Soft	Easily penetrated several inches with thumb.
5-8	Medium stiff	Can be penetrated several inches with thumb with moderate effort.
9-15	Stiff	Readily indented with thumb, but penetrated only with great effort.
16-30	Very stiff	Readily indented with thumbnail.
30+	Hard	Indented only with difficulty by thumbnail.

3. Relative Firmness. Terms for description of partially saturated and/or cemented soils which commonly occur in the Southwest including clays, cemented granular materials, silts and silty and clayey granular soils.

N	Relative Firmness
0-4	Very soft
5-8	Soft
9-15	Moderately firm
16-30	Firm
31-50	Very firm
50+	Hard

EXPLANATION OF CORE LOG PRESENTATION
& TERMINOLOGY FOR THE DESCRIPTION OF ROCK

I. ROCK QUALITY DESIGNATION (RQD). Percentage of rock core per core run which is relatively sound and unfractured and which is longer than 0.33 feet in length. Rock which is soft or weathered, closely jointed, or rock from which the core recovery is low, will have poor to fair RQD.

II. DISCONTINUITIES

A. Spacing of Joints

Code	Spacing of Joints	Descriptive Term
1	Greater than 10 ft.	Very wide
2	3 ft. - 10 ft.	Wide
3	1 ft. - 3 ft.	Moderately close
4	0.2 ft. - 1 ft.	Close
5	Less than 0.2 ft.	Very close

B. Orientation of Joints

Measurements presented represent dip angles from horizontal.

Symbol	Description
Rdm	Random - preferred orientation cannot be determined.

C. Condition of Joints

1. Roughness

Symbol	Descriptive Term	Properties
Smth	Smooth	Appears smooth and is essentially smooth to the touch. May be slickensided.
SRgh	Slightly rough	Asperities on the fracture surfaces are visible and can be distinctly felt.
MRgh	Medium rough	Asperities are clearly visible and fracture surface feels abrasive.
Rgh	Rough	Large angular asperities can be seen. Some ridge and high side angle steps evident.

Symbol	Descriptive Term	Properties
VRgh	Very rough	Near vertical steps and ridges occur on the fracture surface.

2. Presence or Absence of Fracture Filling Material

Symbol	Descriptive Term	Definition
Cln	Clean	No fracture filling material.
Stn	Stained	Coloration of rock only. No recognizable filling material.
Fld	Filled	Fracture filled with recognizable filling material.

III. BEDDING

Symbol	Descriptive Term	Definition
TL	Thinly laminated	Less than 0.01 ft.
L	Laminated	0.01 ft. to 0.04 ft.
ThB	Thinly bedded	0.04 ft. to 0.20 ft.
MB	Medium bedded	0.20 ft. to 2.00 ft.
TkB	Thickly bedded	More than 2.00 ft.

IV. DEGREE OF WEATHERING

Symbol	Descriptive Term / Properties
Dec	Decomposed, generally soil-like, can be crumbled by hand pressure.
HiW	Highly weathered, generally rock-like, can be broken easily, but crumbles with difficulty by hand.
MdW	Moderately weathered, fabric stained rusty brown, can be indented by steel nail, breaks only with difficulty.
SlW	Slightly weathered, open discontinuities are weathered, coated, but only slight weathering of rock mass, generally not indented by steel nail.
UnW Ex Jts	Unweathered except joints, weathering limited to the surface of discontinuities; fabric is fresh throughout but most joints show rusty stain and/or soil filling material.
UnW Inc Jts	Unweathered including joints, rock mass and discontinuities are unweathered; only occasional joints show rusty stain, practically no soil filling.

V. HARDNESS

Descriptive Term	Properties
Very hard	Cannot be scratched with knife or sharp pick. Breaking of hand specimens requires several hard blows of geologist's pick.
Hard	Can be scratched with knife or pick only with difficulty. Hard blow of hammer required to detach hand specimen.
Moderately hard	Can be scratched with knife or pick. Gouges or grooves to ¼ inch deep can be excavated by hard blow of point of a geologist's pick. Hand specimens can be detached by moderate blow.
Moderately soft	Can be grooved or gouged 1/16 inch deep by firm pressure on knife or pick point. Can be excavated in small chips to pieces about 1 inch maximum size by hard blows of the point of a geologist's pick.
Soft	Can be gouged or grooved readily with knife or pick point. Can be excavated in chips to pieces several inches in size by moderate blows of a pick point. Small thin pieces can be broken by finger pressure.
Very soft	Can be carved with knife. Can be excavated readily with point of pick. Pieces 1 inch or more in thickness can be broken with finger pressure. Can be scratched readily by fingernail.

VI. MISCELLANEOUS ABBREVIATIONS

Symbol	Description	Symbol	Description
Bkn	Broken	Incl	Inclusions
Brc	Brecciated	Qtz	Quartz
Calc	Calcite	Slicks	Slickensides
Cem	Cemented	SZ	Shear Zone
Frct	Fractured		
Gog	Gouge		

PROJECT Heap Leach Facility Design Vulture Mine Project

JOB NO. E87-11 DATE 3-4-87

LOG OF TEST BORING NO. 1

RIG TYPE CME-55

BORING TYPE 6½" Hollow Stem Auger

SURFACE ELEV. 2047'

DATUM *VMP

Depth in Feet	Continuous Penetration Resistance	Graphical Log	Sample	Sample Type	Blows per foot 140 lb. 30" free-fall drop hammer	Dry Density Lbs. per cu. ft.	Moisture Content Per Cent of Dry Wt.	Unified Soil Classification	REMARKS	VISUAL CLASSIFICATION
0			☒	S	9			SM		FILL - TAILINGS
									slightly moist moderately firm	SILTY SAND, fine grained, subangular, non-plastic to low plasticity, light brown note: considerable lime content; uncemented
			☒	S	71			GW		
5			☒	S	45					
10									slightly moist dense to very dense	SANDY GRAVEL, considerable silt & clay, fine to medium grained sand & gravel, angular to subrounded, strongly lime cemented, low plasticity to nonplastic, reddish brown
										Stopped auger at 4'6" Stopped sampler at 6' *Vulture Mine Project Composite Surface Map

GROUND WATER

DEPTH	HOUR	DATE
	none	

SAMPLE TYPE

A – Auger cuttings. B – Block sample

S – 2" O.D. 1.38" I.D. tube sample.

U – 3" O.D. 2.42" I.D. tube sample.

T – 3" O.D. thin-walled Shelby tube.

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PROJECT Heap Leach Facility Design Vulture Mine Project

JOB NO. E87-11 DATE 3-4-87

LOG OF TEST BORING NO. 2

RIG TYPE CME-55
BORING TYPE 6½" Hollow Stem Auger
SURFACE ELEV. 2043'
DATUM *VMP

Depth in Feet	Continuous Penetration Resistance	Graphical Log	Sample	Sample Type	Blows per foot 140 lb. 30" free-fall drop hammer	Dry Density Lbs. per cu. ft.	Moisture Content Per Cent of Dry Wt.	Unified Soil Classification
0		o o o	X	S	95		5	
		o o o	X	S	50/5"			
5		o o o	X	S	38		2	
10		o o o	X	S	42			SW-SM
15		o o o	X	S	51		3	
20		o o	X	S	25			
25								

REMARKS

slightly moist

medium dense to very dense

VISUAL CLASSIFICATION

SILTY, GRAVELLY SAND, occasional cobble, subangular to angular, medium to coarse grained sand, subangular to angular, medium to coarse grained gravel, moderately lime cemented, nonplastic, light brown

note: numerous sand lenses throughout

note: trace to small amount of clay below 8'

note: gravel amount decreases with depth below 10'

Stopped auger at 19'6"
Stoppe sampler at 21'

*Vulture Mine Project Composite Surface Map

GROUND WATER

DEPTH	HOUR	DATE
	none	

SAMPLE TYPE

A – Auger cuttings. B – Block sample
S – 2" O.D. 1.38" I.D. tube sample.
U – 3" O.D. 2.42" I.D. tube sample.
T – 3" O.D. thin-walled Shelby tube.

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JOB NO. E87-11 DATE 3-4-87

LOG OF TEST BORING NO. 3

RIG TYPE CME-55

BORING TYPE 6½ Hollow Stem Auger

SURFACE ELEV. 2053'

DATUM *VMP

Depth in Feet	Continuous Penetration Resistance	Graphical Log	Sample	Sample Type	Blows per foot 140 lb. 30" free-fall drop hammer	Dry Density Lbs. per cu. ft.	Moisture Content Per Cent of Dry Wt.	Unified Soil Classification	REMARKS	VISUAL CLASSIFICATION
0			X							FILL - TAILINGS
			X						slightly moist	SILTY SAND, fine grained, subangular, nonplastic to low plasticity, light brown
5			X				1		soft to firm	note: considerable lime content; uncemented
10			X					SM		
15			X							
20			X						slightly moist	SANDY GRAVEL, occasional cobble, medium to coarse grained, subangular to angular sand, fine to medium grained, subangular to angular gravel, moderately lime cemented, nonplastic, light brown
								GW	medium dense to dense	
25			X							
30										Stopped auger at 24'6" Stopped sampler at 26' *Vulture Mine Project Composite Surface Map

GROUND WATER

DEPTH	HOUR	DATE
	none	

SAMPLE TYPE

A – Auger cuttings. B – Block sample

S – 2" O.D. 1.38" I.D. tube sample.

U – 3" O.D. 2.42" I.D. tube sample.

T – 3" O.D. thin-walled Shelby tube.

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PROJECT Heap Leach Facility Design Vulture Mine Project

JOB NO. E87-11 DATE 3-4-87

LOG OF TEST BORING NO. 4

RIG TYPE CME-55

BORING TYPE 6½" Hollow Stem Auger

SURFACE ELEV. 2064'

DATUM *VMP

Depth in Feet	Continuous Penetration Resistance	Graphical Log	Sample	Sample Type	Blows per foot 140 lb. 30" free-fall drop hammer	Dry Density Lbs. per cu. ft.	Moisture Content Per Cent of Dry Wt.	Unified Soil Classification	REMARKS	VISUAL CLASSIFICATION
0			X	S	7					FILL- TAILINGS
			X	S	6				slightly moist	SILTY SAND, fine grained, subangular, nonplastic, light brown
5			X	S	10				moderately firm	note: considerable lime content; uncemented
10			X	S	13			SM		
15			X	T	(15'-16'10")2					
20				S	50/5"				slightly moist	SANDY GRAVEL, occasional cobble, medium to coarse grained, subangular to angular sand, fine to medium grained, subangular to angular gravel, strongly lime cemented, nonplastic, light brown
								GW	very dense	
25				S	50/2"					
										Stopped auger at 24'6" Sampler refused at 24'8"
										*Vulture Mine Project Composite Surface Map

GROUND WATER

DEPTH	HOUR	DATE
	none	

SAMPLE TYPE

A – Auger cuttings. B – Block sample

S – 2" O.D. 1.38" I.D. tube sample.

U – 3" O.D. 2.42" I.D. tube sample.

T – 3" O.D. thin-walled Shelby tube.

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Vulture Mine Project

JOB NO. E87-11 DATE 3-4-87

LOG OF TEST BORING NO. 5

RIG TYPE CME-55
BORING TYPE 6½" Hollow Stem Auger
SURFACE ELEV. 2050'
DATUM *VMP

Depth in Feet	Continuous Penetration Resistance	Graphical Log	Sample	Sample Type	Blows per foot 140 lb. 30" free-fall drop hammer	Dry Density Lbs. per cu. ft.	Moisture Content Per Cent of Dry Wt.	Unified Soil Classification
0			☒	S	12		4	
			☒	S	6			
5			☒	T	(5'-7'4")		2	
10								SM
			☒	T	(10'-12'2")			
15			☒	S	51		1	
								GW
20			☒	S	50/3"			
25								

Depth (ft)	REMARKS	VISUAL CLASSIFICATION
0–15	slightly moist soft to moderately firm	FILL - TAILINGS SILTY SAND, fine grained, subangular, nonplastic to low plasticity, reddish brown to light brown note: considerable lime content; uncemented
15–20	slightly moist dense to very dense	SANDY GRAVEL, occasional cobble, medium to coarse grained, subangular to angular sand, fine to medium grained, subangular to angular gravel, strongly lime cemented, nonplastic, reddish brown
	note: installed 2" schedule 40 PVC in adjacent boring for isolated interval permeability test; slotted interval from 5' to 10'	Stopped auger at 19'6" Sampler refused at 20'3" *Vulture Mine Project Composite Surface Map

GROUND WATER

DEPTH	HOUR	DATE
	none	

SAMPLE TYPE

A – Auger cuttings. B – Block sample
S – 2" O.D. 1.38" I.D. tube sample.
U – 3" O.D. 2.42" I.D. tube sample.
T – 3" O.D. thin-walled Shelby tube.

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PROJECT Heap Leach Facility Design Vulture Mine Project

JOB NO. E87-11 DATE 3-5-87

LOG OF TEST BORING NO. 6

RIG TYPE CME-55
BORING TYPE 6½" Hollow Stem Auger
SURFACE ELEV. 2029'
DATUM *VMP

Depth in Feet	Continuous Penetration Resistance	Graphical Log	Sample	Sample Type	Blows per foot 140 lb. 30" free-fall drop hammer	Dry Density Lbs. per cu. ft.	Moisture Content Per Cent of Dry Wt.	Unified Soil Classification	REMARKS	VISUAL CLASSIFICATION
0–4.5			☒	S	12			SM	slightly moist; moderately firm to firm	FILL - TAILINGS. SILTY SAND, fine grained, subangular, non-plastic to low plasticity, reddish brown to light brown. note: considerable lime content; uncemented
			☒	S	20					
4.5–19.5			☒	S	21			GW	slightly moist; medium dense to dense	SANDY GRAVEL, occasional cobble, medium to coarse grained, subangular to angular sand, fine to medium grained, subangular to angular gravel, strongly lime cemented, nonplastic, reddish brown to brown
			☒	S	48					
			☒	S	48		2			
19.5–21			☒	S	33		2	SW	slightly moist; dense	GRAVELLY SAND, medium to coarse grained, subangular to angular sand, fine grained, subangular to angular gravel, weakly lime cemented, nonplastic, light brown

Stopped auger at 19'6"
Stopped sampler at 21'

*Vulture Mine Project Composite Surface Map

GROUND WATER

DEPTH	HOUR	DATE
	none	

SAMPLE TYPE
A – Auger cuttings. B – Block sample
S – 2" O.D. 1.38" I.D. tube sample.
U – 3" O.D. 2.42" I.D. tube sample.
T – 3" O.D. thin-walled Shelby tube.

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PROJECT Heap Leach Facility Design Vulture Mine Project

LOG OF TEST BORING NO. 7

JOB NO. E87-11 DATE 3-5-87

RIG TYPE CME-55

BORING TYPE 6½" Hollow Stem Auger

SURFACE ELEV. 2035'

DATUM *VMP

Depth in Feet	Continuous Penetration Resistance	Graphical Log	Sample	Sample Type	Blows per foot 140 lb. 30" free-fall drop hammer	Dry Density Lbs. per cu. ft.	Moisture Content Per Cent of Dry Wt.	Unified Soil Classification
0			X	S	10			
			X	T	(2'6"-4'8") 2			SM
5			X	S	22			
10			X	S	50/5"			GW
				S	50/3"			
15								

REMARKS	VISUAL CLASSIFICATION
	FILL - TAILINGS
slightly moist moderately firm to firm	SILTY SAND, fine grained, subangular, low plasticity to non-plastic, medium brown to yellowish brown note: considerable lime content, uncemented
slightly moist very dense	SANDY GRAVEL, occasional cobble, medium to coarse grained, subangular to angular sand, fine to medium grained, subangular to angular gravel, strongly lime cemented, nonplastic, reddish brown
	BEDROCK, undifferentiated Precambrian Schist, highly weathered, moderately soft to moderately hard, green to brown
	Stopped auger at 13' Sampler refused at 13'3" *Vulture Mine Project Composite Surface Map

GROUND WATER

DEPTH	HOUR	DATE
	none	

SAMPLE TYPE

A – Auger cuttings. B – Block sample

S – 2" O.D. 1.38" I.D. tube sample.

U – 3" O.D. 2.42" I.D. tube sample.

T – 3" O.D. thin-walled Shelby tube.

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LOG OF TEST BORING NO. 8

JOB NO. E87-11 DATE 3-5-87

RIG TYPE CME-55
BORING TYPE 6½" Hollow Stem Auger
SURFACE ELEV. 2040'
DATUM *VMP

Depth in Feet	Continuous Penetration Resistance	Graphical Log	Sample	Sample Type	Blows per foot 140 lb. 30" free-fall drop hammer	Dry Density Lbs. per cu. ft.	Moisture Content Per Cent of Dry Wt.	Unified Soil Classification
0			☒	S	13			
			☒	T	(2'6"-4'7")			SM
5			☒	T	(5'-7'2")			
10			☒	S	24			
								GW
15			—	S	50/2"			

REMARKS	VISUAL CLASSIFICATION
slightly moist moderately firm to firm	FILL - TAILINGS SILTY SAND, fine grained, subangular, nonplastic to low plasticity, light brown note: considerable lime content; uncemented
slightly moist medium dense to very dense	SANDY GRAVEL, occasional cobble, medium to coarse grained, subangular to angular sand, fine to medium grained, subangular to angular gravel, moderately lime cemented, nonplastic, medium brown

Stopped auger at 14'6"
Sampler refused at 14'8"

note: installed 2" Schedule 40 PVC in boring for isolated interval permeability test; slotted interval from 10' to 15'

*Vulture Mine Project Composite Surface Map

GROUND WATER

DEPTH	HOUR	DATE
	none	

SAMPLE TYPE

A – Auger cuttings. B – Block sample
S – 2" O.D. 1.38" I.D. tube sample.
U – 3" O.D. 2.42" I.D. tube sample.
T – 3" O.D. thin-walled Shelby tube.

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PROJECT Heap Leach Facility Design Vulture Mine Project

JOB NO. E87-11 DATE 3-5-87

LOG OF TEST BORING NO. 9

RIG TYPE CME-55

BORING TYPE 6½" Hollow Stem Auger

SURFACE ELEV. 2028'

DATUM *VMP

Depth in Feet	Continuous Penetration Resistance	Graphical Log	Sample	Sample Type	Blows per foot 140 lb. 30" free-fall drop hammer	Dry Density Lbs. per cu. ft.	Moisture Content Per Cent of Dry Wt.	Unified Soil Classification	REMARKS	VISUAL CLASSIFICATION
0–23			☒	S	18			SM-GM	slightly moist firm to hard	SILTY, SANDY GRAVEL, occasional cobbles, medium to coarse grained, subangular to angular sand, fine to coarse grained, subangular to angular gravel, moderately to strongly lime cemented, nonplastic, to low plasticity, light brown to tan note: silt decrease below 12'
			☒	S	53					
5			☒	S	50/5.5"					
10			☒	S	26					
15			☒	S	59					
20			☒	S	105					
23–40					50/3" (no recovery)					BEDROCK, undifferentiated Precambrian Schist, highly weathered, moderately soft to moderately hard, green to reddish brown
30				S	50/3"					
35				S	50/2"					
40				S	50/1"					
40–50										Stopped auger at 39'6" Sampler refused at 39'7" *Vulture Mine Project Composite Surface Map

GROUND WATER

DEPTH	HOUR	DATE
	none	

SAMPLE TYPE

A – Auger cuttings. B – Block sample

S – 2" O.D. 1.38" I.D. tube sample.

U – 3" O.D. 2.42" I.D. tube sample.

T – 3" O.D. thin-walled Shelby tube.

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PROJECT: Heap Leach Facility Design — Vulture Mine Project

JOB NO. E87-11 DATE 3-5-87

LOG OF TEST BORING NO. 10

RIG TYPE: CME-55
BORING TYPE: 6½" Hollow Stem Auger
SURFACE ELEV.: 2034'
DATUM: *VMP

Depth in Feet	Continuous Penetration Resistance	Graphical Log	Sample	Sample Type	Blows per foot 140 lb. 30" free-fall drop hammer	Dry Density Lbs. per cu. ft.	Moisture Content Per Cent of Dry Wt.	Unified Soil Classification	REMARKS	VISUAL CLASSIFICATION
0			☒	S	24			SM		FILL- TAILINGS
			☒	S	84				slightly moist firm	SILTY SAND, some clay, fine grained, subangular, low plasticity, light brown. note: considerable lime content; uncemented
5			☒	S	56					
10				S	50/3"			GM-GC	slightly moist hard	CLAYEY TO SILTY GRAVEL, considerable sand, occasional cobble, fine to coarse grained, subangular to angular sand, fine to coarse grained, subangular to angular gravel, weakly to moderately lime cemented, low plasticity, gray to tan
15				S	50/1"					BEDROCK, undifferentiated Precambrian Schist, highly weathered, moderately soft to moderately hard, greenish gray to brown

Stopped auger at 14'6"
Sampler refused at 14'7" on bedrock

*Vulture Mine Project Composite Surface Map

GROUND WATER

DEPTH	HOUR	DATE
	none	

SAMPLE TYPE
A – Auger cuttings. B – Block sample
S – 2" O.D. 1.38" I.D. tube sample.
U – 3" O.D. 2.42" I.D. tube sample.
T – 3" O.D. thin-walled Shelby tube.

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APPENDIX B

LABORATORY TESTING PROCEDURES

Consolidation Tests Soiltest or Clockhouse apparatus of the "floating-ring" type are employed for the one-dimensional consolidation tests. They are designed to receive one inch high 2.5 inch O.D. brass liner rings with soil specimens as secured in the field. Procedures for the tests generally are those outlined in ASTM D2435. Loads are applied in several increments to the upper surface of the test specimen and the resulting deformations are recorded at selected time intervals for each increment. For soils which are essentially saturated, each increment of load is maintained until the deformation versus log of time curve indicates completion of primary consolidation. For partially saturated soils, each increment of load is maintained until the rate of deformation is equal or less than 1/10,000 inch per hour. Applied loads are such that each new increment is equal to the total previously applied loading. Porous stones are placed in contact with the top and bottom of the specimens to permit free addition or expulsion of water. For partially saturated soils, the tests are normally performed at in situ moisture conditions until consolidation is complete under stresses approximately equal to those which will be imposed by the combined overburden and foundation loads. The samples are then submerged to show the effect of moisture increase and the tests continued under higher loadings. Generally, the tests are continued to about twice the anticipated curve due to overburden and structural loads with a rebound curve then being established by releasing loads.

Expansion Tests The same type of consolidometer apparatus described above is used in expansion testing. Undisturbed samples contained in brass liner rings are placed in the consolidometers, subjected to appropriate surcharge loads and submerged. The loads are maintained until the expansion versus log of time curve indicates the completion of "primary swell".

Direct Shear Tests Direct shear tests are run using a Clockhouse or Soiltest apparatus of the strain-control of approximately 0.05 inches per minute. The machine is designed to receive one of the one inch high 2.42 inch diameter specimens obtained by tube sampling. Generally, each sample is sheared under a normal load equivalent to the effective overburden pressure at the point of sampling. In some instances, samples are sheared at several normal loads to obtain the cohesion and angle of internal friction. When necessary, samples are saturated and/or consolidated before shearing in order to approximate the anticipated controlling field loading conditions.

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TABULATION OF TEST RESULTS

Job No. E87-11
W/O 2

HOLE NO	DEPTH	UNIFIED CLASS	L.L.	P.I.	#200	#100	#50	#40	#30	#16	#10	#8	#4	.25"	.375"	.5"	.75"	1"	1.5"	2"	2.5"	3"	3.5"	4"	6"	8"	10"	12"	LAB NO
					SIEVE ANALYSIS-ACCUM % PASSING																								
2	4.5'-6'	SW-SM	NV	NP	9.0	13	20	26	33	46	59	65	78	85	93	97	100												7-11-9
3	4.5'-6'	SM	NV	NP	41	61	86	96	100																				7-11-14
4	15'-16.8'	ML	NV	NP	65	86	98																						7-11-23
5	5'-7.3'	ML	NV	NP	51	70	88	96	99	100																			7-11-28
6	14.5'-16'		NA	NA	12	18	26	32	38	48	57	61	71	76	86	90	100												7-11-36
6	19.5'-21'		NA	NA	8.1	12	18	24	32	54	70	76	88	91	97	99	100												7-11-37
7	2.5'-4.7'		NA	NA	58	82	97	99																					7-11-39
TAILINGS	---	SM	NV	NP	23	47	76	90	98	100																			7-11-61
ALLUVIUM	---	SW-SM	NV	NP	7.9	11	14	17	21	32	43	48	67	76	88	93	97	99	100										7-11-62

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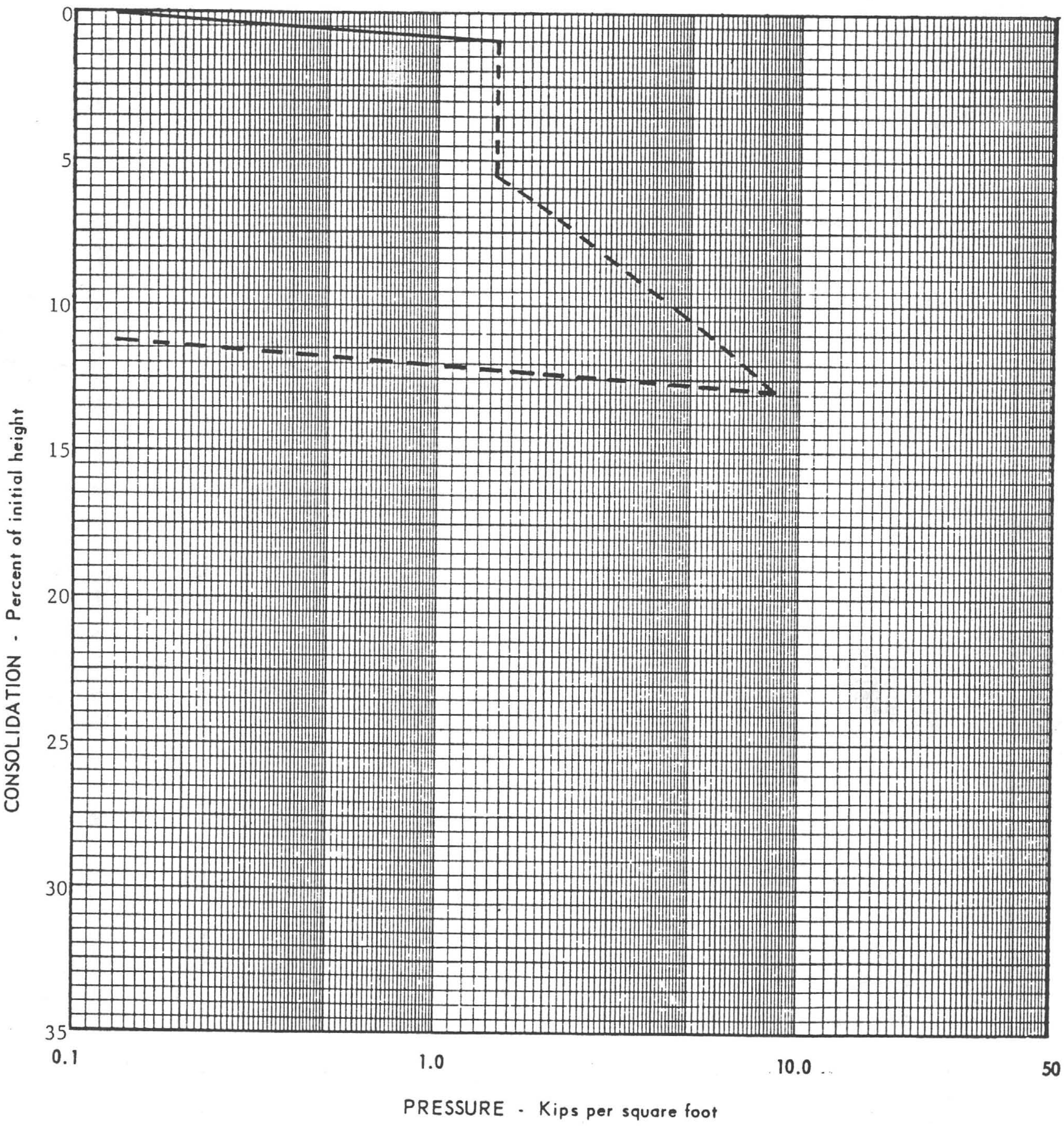
TABULATION OF TEST RESULTS

Job No. E87-11
W/O 2

HOLE NO	DEPTH	MOISTURE CONTENT	LAB NO
2	4.5'-6'	1.9 %	7-11-9
3	4.5'-6'	1.1 %	7-11-14
4	15'-16.8'	1.8 %	7-11-23
5	5'-7.3'	1.8 %	7-11-28
6	14.5'-16'	2.1 %	7-11-36
6	19.5'-21'	2.3 %	7-11-37
7	2.5'-4.7'	2.2 %	7-11-39
TAILINGS	---	5 %	7-11-61
ALLUVIUM	---	1.7 %	7-11-62

SUMMARY OF CONSOLIDATION TESTS

PROJECT Heap Leach Facility Design
Vulture Mine Project **JOB NO.** E87-11



CURVE	SAMPLE	INITIAL DRY DENSITY LBS./CU. FT.	MOISTURE CONTENT % DRY WEIGHT INITIAL	MOISTURE CONTENT % DRY WEIGHT FINAL	UNIFIED SOIL CLASSIFICATION
23	Boring 4 @ 15'-16.8'	94.7	2.3	18.6	SM

SOIL MOISTURE CONDITION

———	INSITU
– – – –	SUBMERGED

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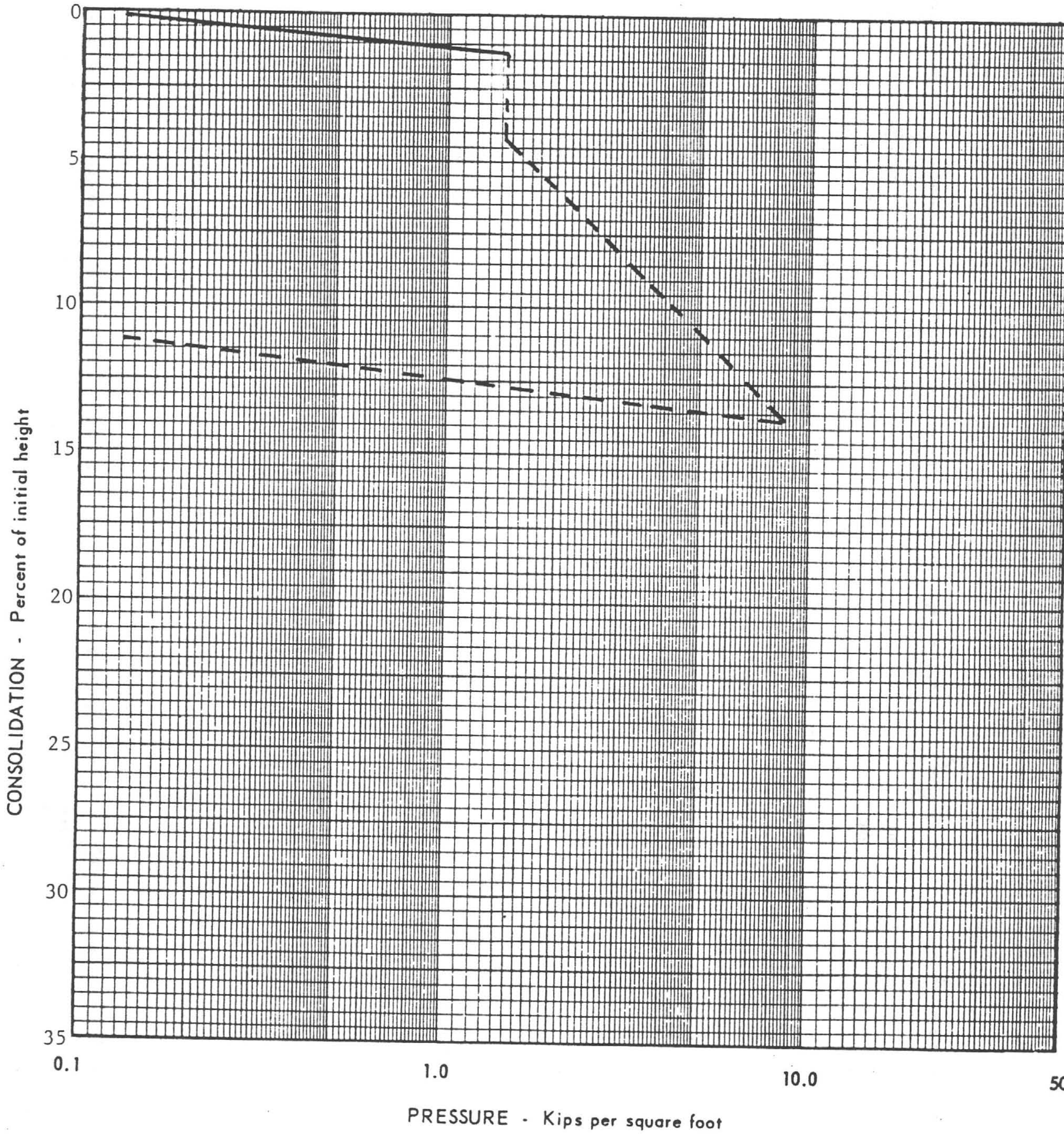
SUMMARY OF CONSOLIDATION TESTS

PROJECT Heap Leach Facility Design Vulture Mine Project JOB NO. E87-11



CURVE	SAMPLE	INITIAL DRY DENSITY LBS./CU. FT.	MOISTURE CONTENT % DRY WEIGHT		UNIFIED SOIL CLASSIFICATION
			INITIAL	FINAL	
28	Boring 5 @ 5'-7.3'	90.6	3.2	22.4	SM

SOIL MOISTURE CONDITION

- ——— INSITU
- - - - - SUBMERGED

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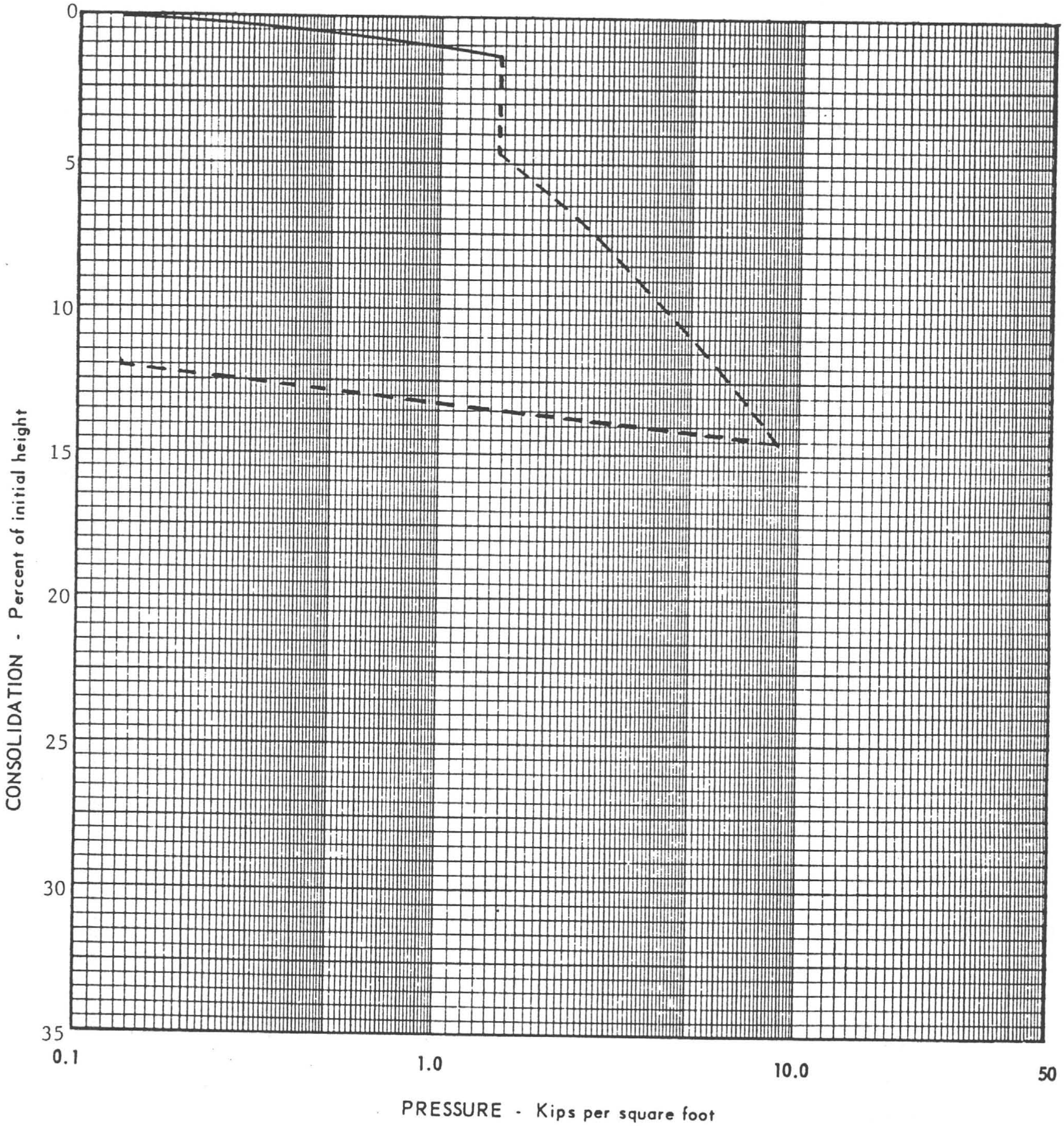
SUMMARY OF CONSOLIDATION TESTS

PROJECT Heap Leach Facility Design Vulture Mine Project JOB NO. E87-11



CURVE	SAMPLE	INITIAL DRY DENSITY LBS./CU. FT.	MOISTURE CONTENT % DRY WEIGHT		UNIFIED SOIL CLASSIFICATION
			INITIAL	FINAL	
29	Boring 5 @ 10'-12.2'	90.7	4.5	21.8	SM

SOIL MOISTURE CONDITION

——— INSITU

– – – – SUBMERGED

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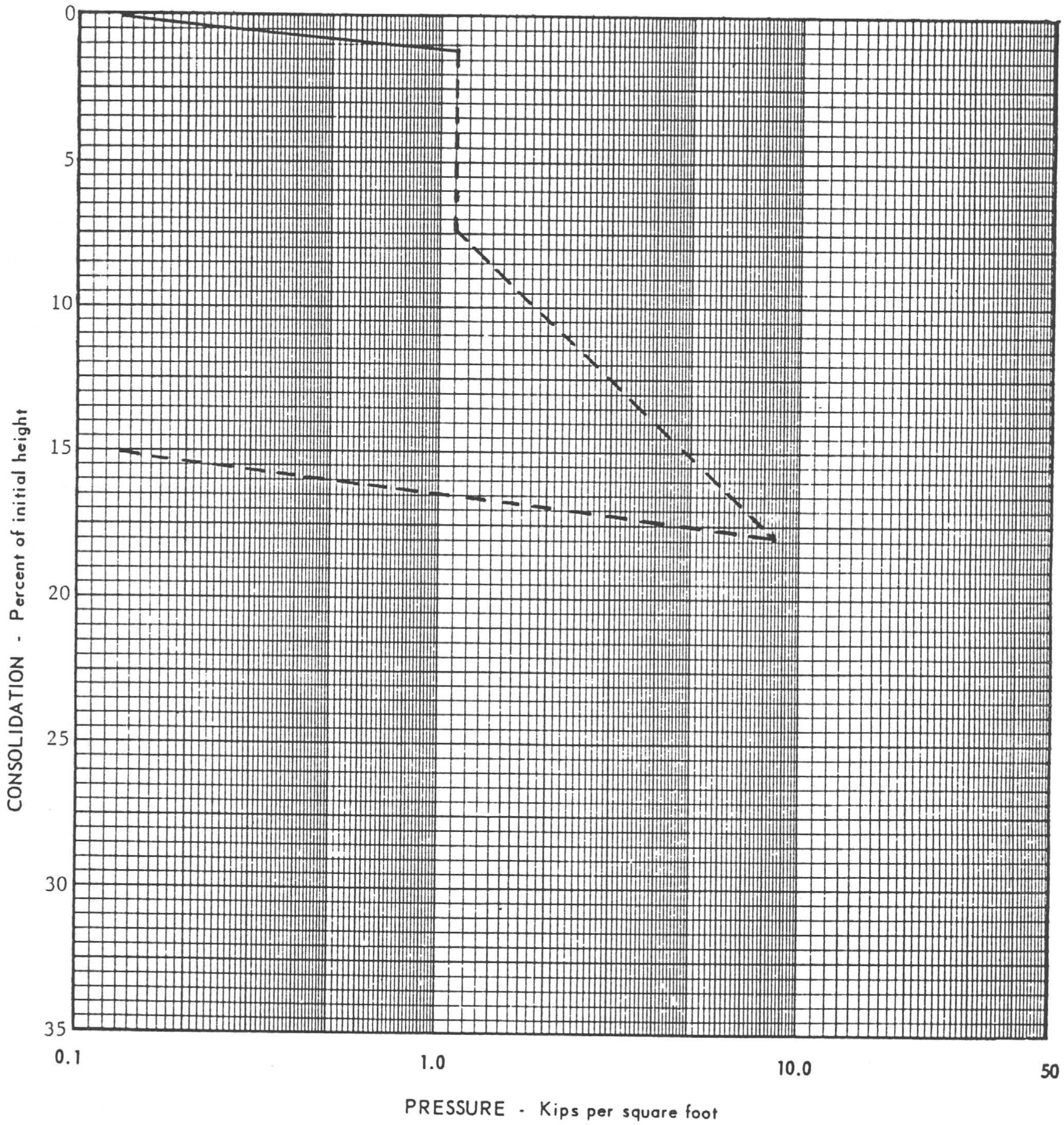
SUMMARY OF CONSOLIDATION TESTS

PROJECT Heap Leach Facility Design Vulture Mine Project JOB NO. E87-11



CURVE	SAMPLE	INITIAL DRY DENSITY LBS./CU. FT.	MOISTURE CONTENT % DRY WEIGHT		UNIFIED SOIL CLASSIFICATION
			INITIAL	FINAL	
45	Boring 8 @ 5'-7.2'	94.0	2.5	19.2	SM

SOIL MOISTURE CONDITION

———	INSITU
– – – –	SUBMERGED

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REPORT OF LABORATORY TESTS — DATE 4/8/87

PROJECT: VULTURE MINE — JOB NO. E87-11

LOCATION: #4 @ 2.5'-4' & 4.5'-6' — W.O.NO. 2

LAB NO. 20 & 21

DIRECT SHEAR TEST(SATURATED)

POINT NO. 1 (NORMAL STRESS 0.995 KSF)

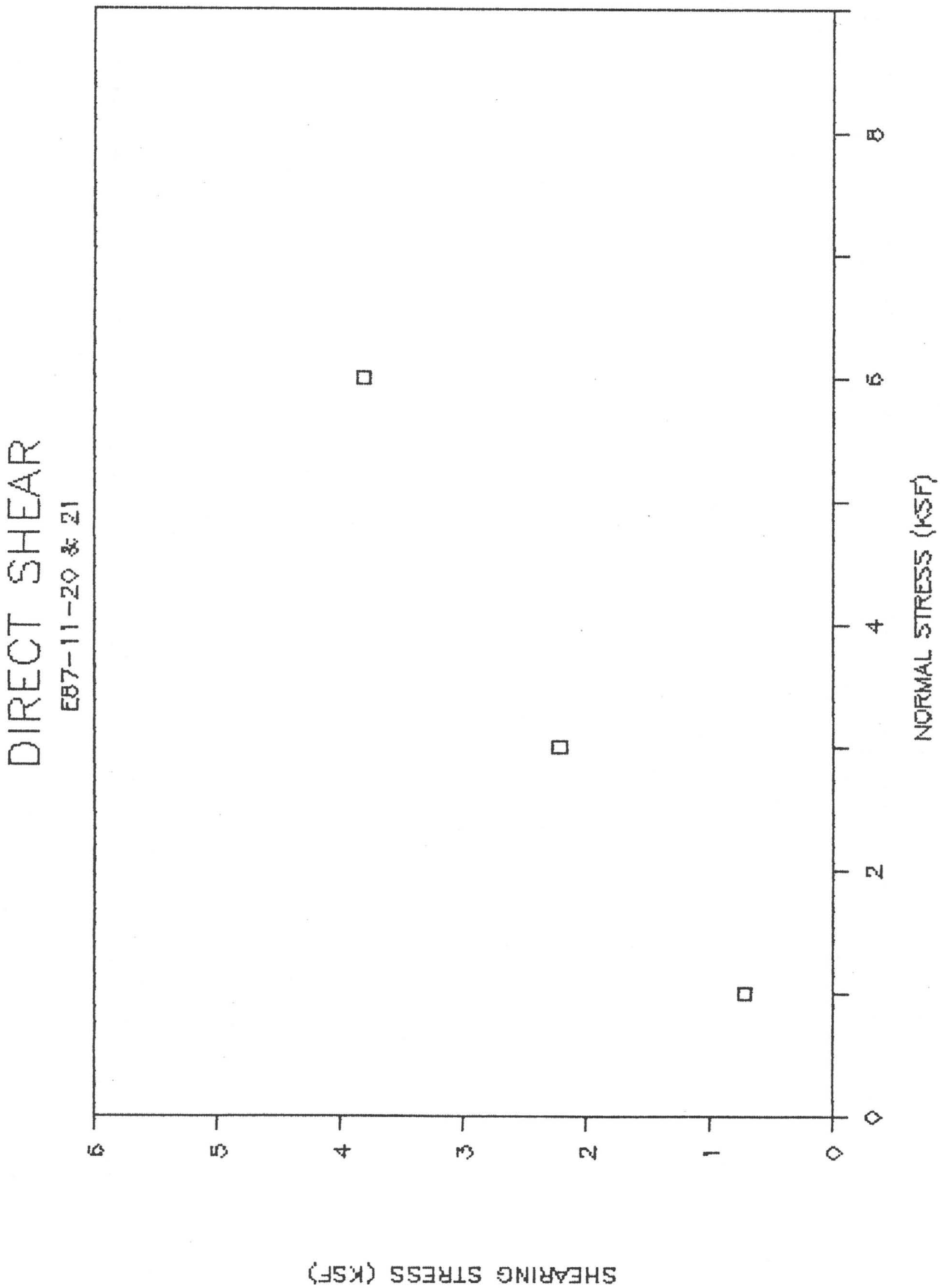
Initial Moisture Content	13.4%
Dry Density	105.0 LB/CU FT
Moisture at Saturation	22.3%
Maximum Vertical Deformation @ T max.	-0.03 IN
Shearing Stress, T max.	0.7 KSF

POINT NO. 2 (NORMAL STRESS 2.998 KSF)

Initial Moisture Content	13.7%
Dry Density	104.8 LB/CU FT
Moisture at Saturation	22.6%
Maximum Vertical Deformation @ T max.	-0.04 IN
Shearing Stress, T max.	2.2 KSF

POINT NO. 3 (NORMAL STRESS 6.003 KSF)

Initial Moisture Content	13.5%
Dry Density	104.9 LB/CU FT
Moisture at Saturation	22.6%
Maximum Vertical Deformation @ T max.	-0.029 IN
Shearing Stress, T max.	3.8 KSF


DIRECT SHEAR
E87-11-20 & 21
NORMAL STRESS (KSF)
SHEARING STRESS (KSF)
0
2
4
6
8
0
1
2
3
4
5
6

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REPORT OF LABORATORY TESTS	DATE 3/31/87
PROJECT: VULTURE MINE	JOB NO. E87-11
LOCATION: #4 @ 15' TO 16.8'	W.O.NO. 2
	LAB NO. 23

DIRECT SHEAR TEST(INSITU)

POINT NO. 1 (NORMAL STRESS 0.995 KSF)

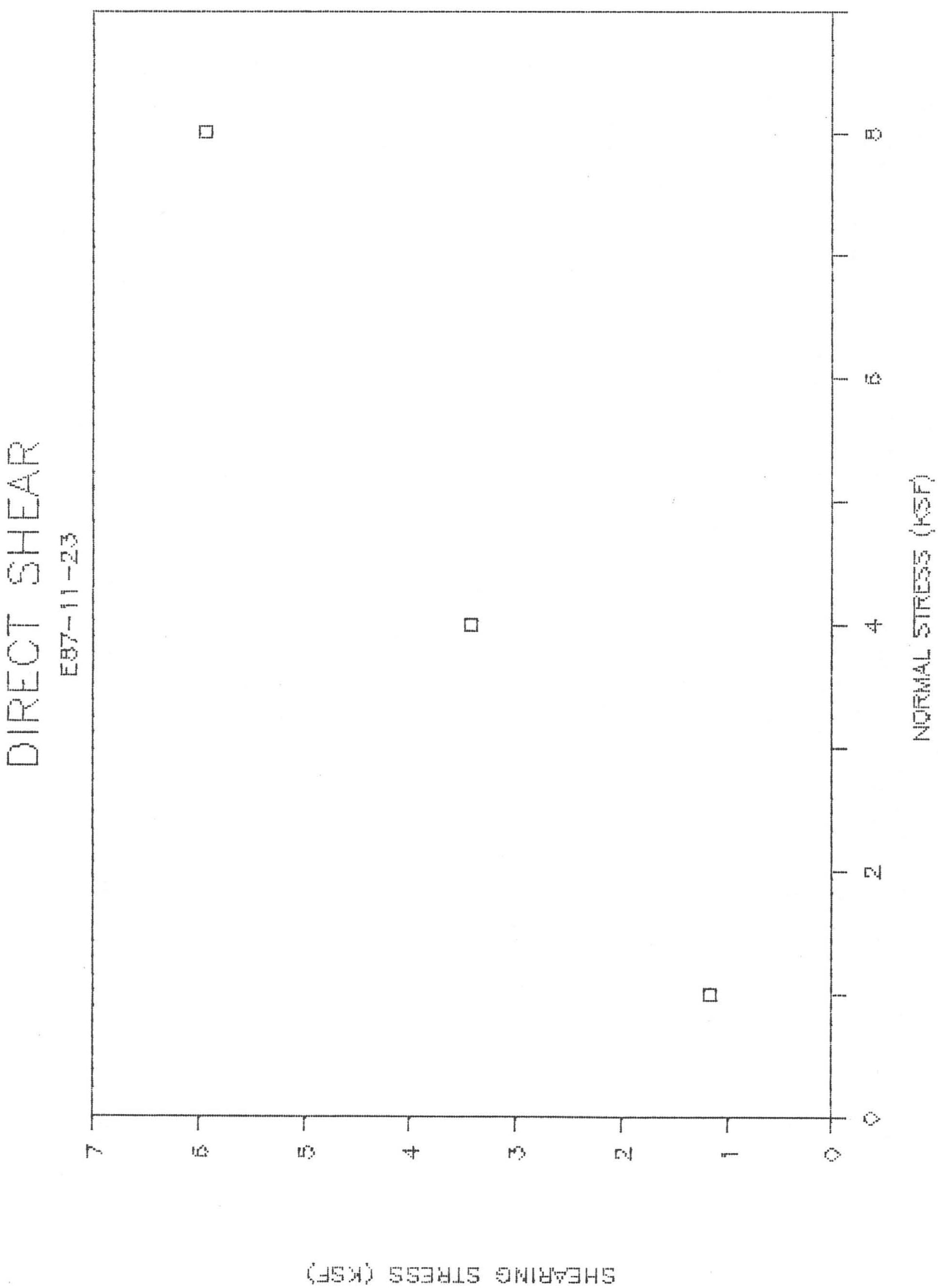
Initial Moisture Content	2.1%
Dry Density	96.7 LB/CU FT
Maximum Vertical Deformation @ T max.	-0.001 IN
Shearing Stress, T max.	1.2 KSF

POINT NO. 2 (NORMAL STRESS 4 KSF)

Initial Moisture Content	1.9%
Dry Density	98.1 LB/CU FT
Maximum Vertical Deformation @ T max.	-0.015 IN
Shearing Stress, T max.	3.4 KSF

POINT NO. 3 (NORMAL STRESS 8 KSF)

Initial Moisture Content	2.4%
Dry Density	95.2 LB/CU FT
Maximum Vertical Deformation @ T max.	-0.041 IN
Shearing Stress, T max.	5.9 KSF



DIRECT SHEAR
E87-11-23
NORMAL STRESS (KSF)
SHEARING STRESS (KSF)
0
2
4
6
8
0
1
2
3
4
5
6
7

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REPORT OF LABORATORY TESTS	DATE 3/31/87
PROJECT: VULTURE MINE	JOB NO. E87-11
LOCATION: #5 @ 5' TO 7.3'	W.O.NO. 2
	LAB NO. 28

DIRECT SHEAR TEST(INSITU)

POINT NO. 1 (NORMAL STRESS 0.995 KSF)

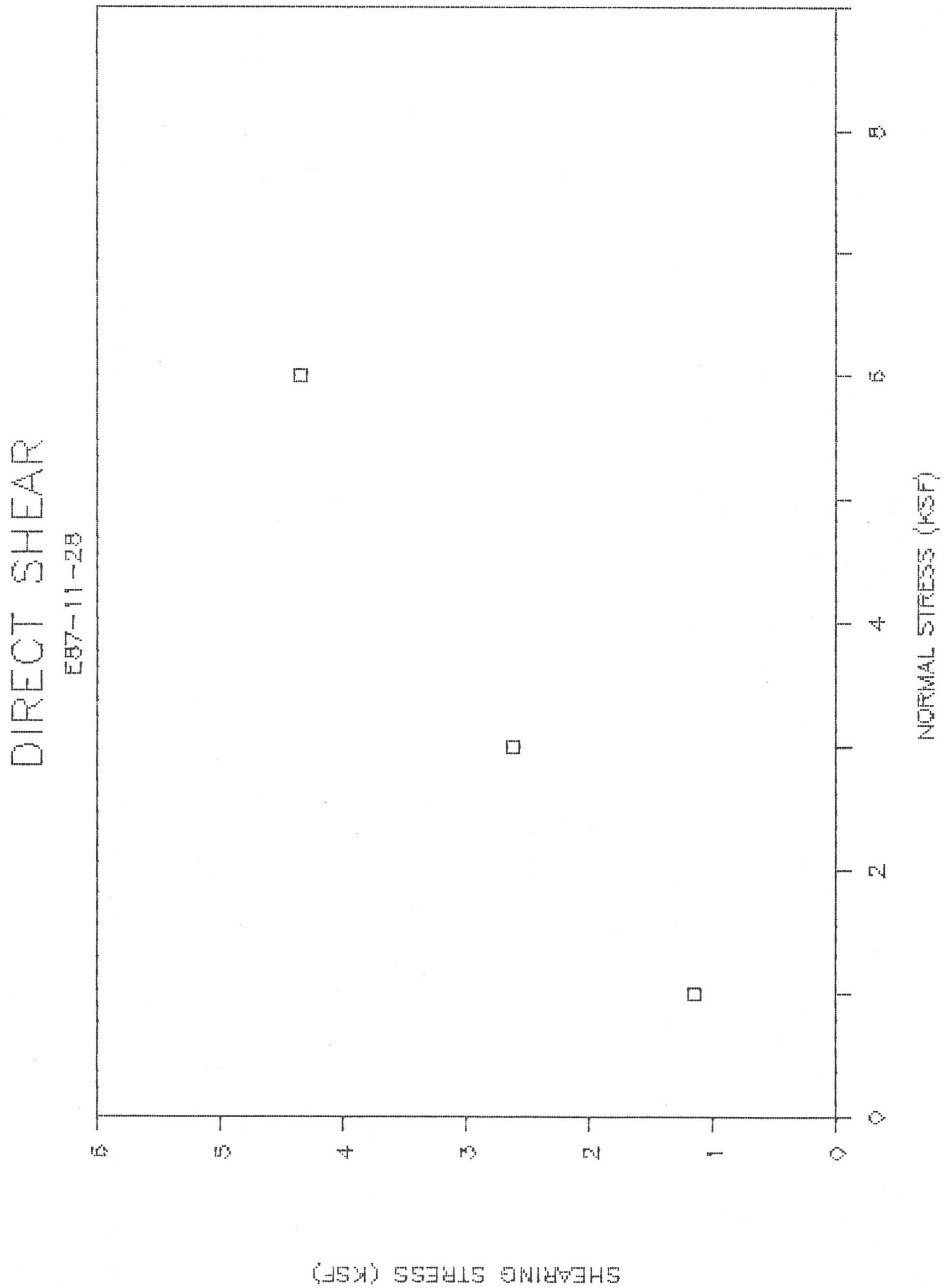
Initial Moisture Content	2.9%
Dry Density	95.6 LB/CU FT
Maximum Vertical Deformation @ T max.	0.003 IN
Shearing Stress, T max.	1.2 KSF

POINT NO. 2 (NORMAL STRESS 2.998 KSF)

Initial Moisture Content	1.5%
Dry Density	95.0 LB/CU FT
Maximum Vertical Deformation @ T max.	0.007 IN
Shearing Stress, T max.	2.6 KSF

POINT NO. 3 (NORMAL STRESS 6 KSF)

Initial Moisture Content	2.1%
Dry Density	94.9 LB/CU FT
Maximum Vertical Deformation @ T max.	-0.026 IN
Shearing Stress, T max.	4.4 KSF



DIRECT SHEAR
E87-11-28
NORMAL STRESS (KSF)
SHEARING STRESS (KSF)

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REPORT OF LABORATORY TESTS	DATE 3/31/87
PROJECT: VULTURE MINE	JOB NO. E87-11
LOCATION:#5 @ 10' TO 12.2'	W.O.NO. 2
	LAB NO. 29

DIRECT SHEAR TEST(SATURATED)

POINT NO. 1 (NORMAL STRESS 0.995 KSF)

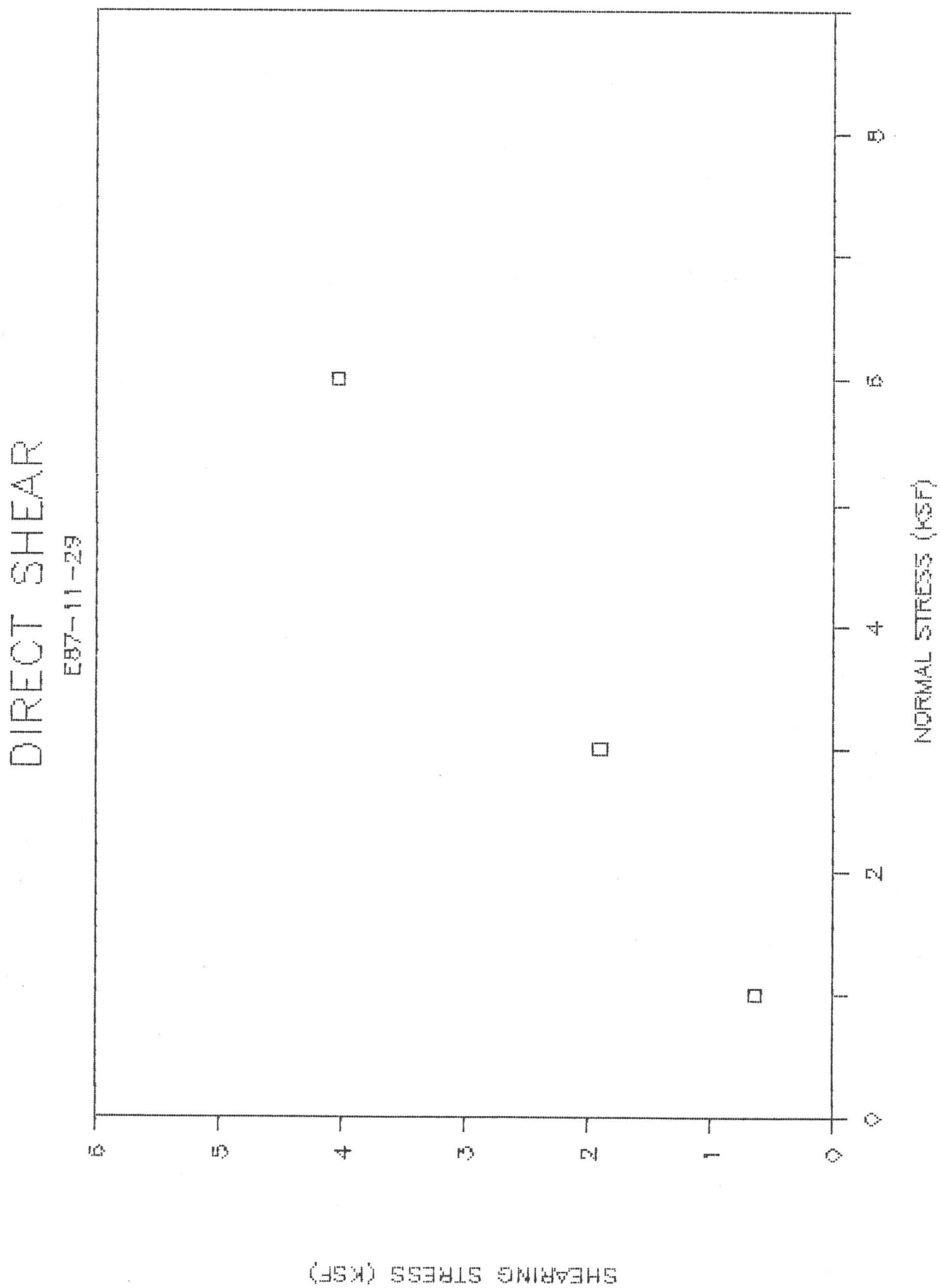
Initial Moisture Content	3.9%
Dry Density	93.9 LB/CU FT
Moisture at Saturation	29.9%
Maximum Vertical Deformation @ T max.	0.058 IN
Shearing Stress, T max.	0.7 KSF

POINT NO. 2 (NORMAL STRESS 2.998 KSF)

Initial Moisture Content	3.5%
Dry Density	94.2 LB/CU FT
Moisture at Saturation	29.2%
Maximum Vertical Deformation @ T max.	-0.036 IN
Shearing Stress, T max.	1.9 KSF

POINT NO. 3 (NORMAL STRESS 6.003 KSF)

Initial Moisture Content	4.1%
Dry Density	94.5 LB/CU FT
Moisture at Saturation	29.2%
Maximum Vertical Deformation @ T max.	-0.042 IN
Shearing Stress, T max.	4.0 KSF


DIRECT SHEAR
E87-11-29
NORMAL STRESS (KSF)
SHEARING STRESS (KSF)
0
2
4
6
8
0
1
2
3
4
5
6

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REPORT OF LABORATORY TESTS	DATE 4/8/87
PROJECT: VULTURE MINE	JOB NO. E87-11
LOCATION: #8 @ 5'-7.2'	W.O.NO. 2
	LAB NO. 45

DIRECT SHEAR TEST(SATURATED)

POINT NO. 1 (NORMAL STRESS 0.995 KSF)

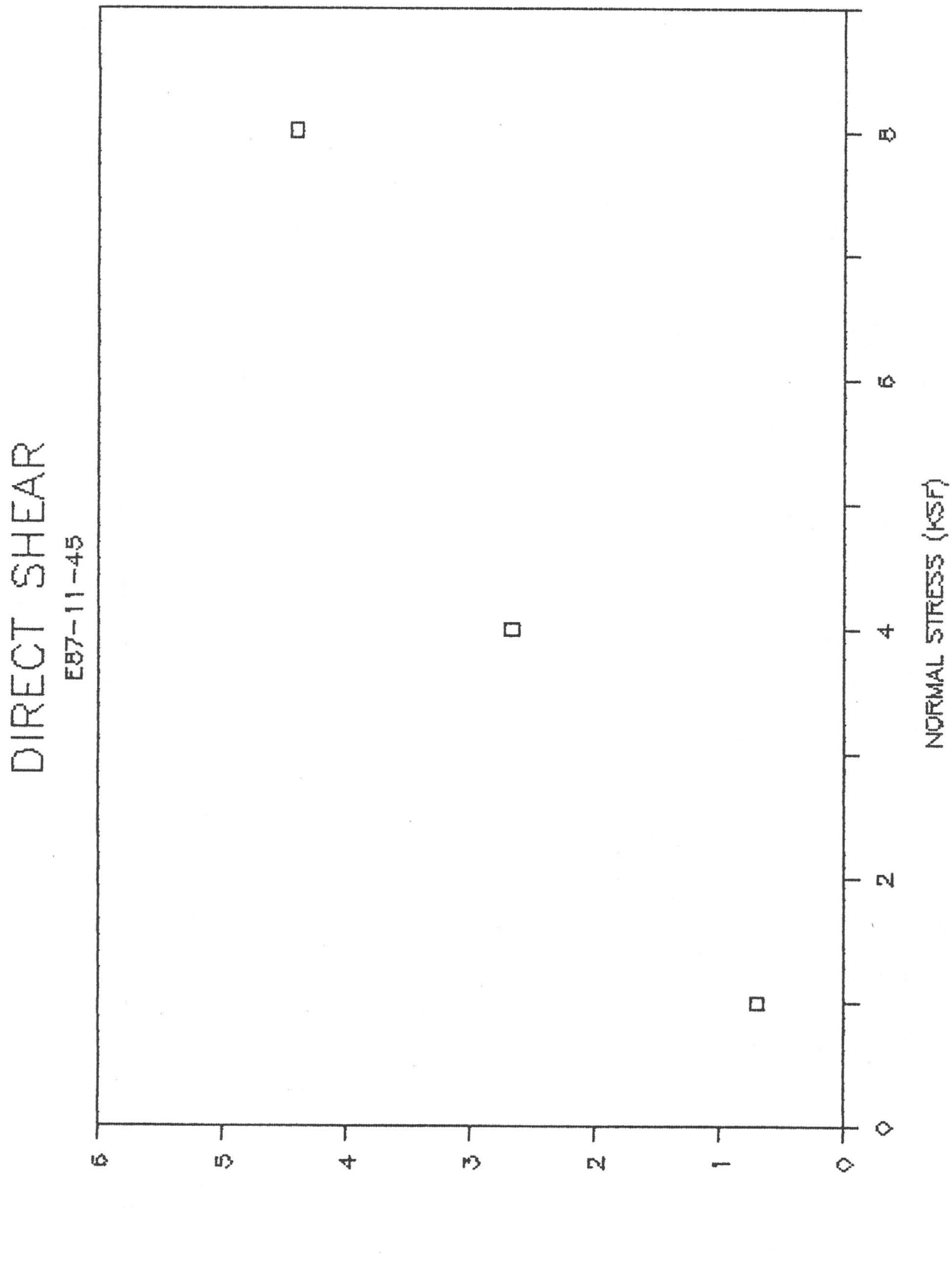
Initial Moisture Content	2.8%
Dry Density	94.4 LB/CU FT
Moisture at Saturation	29.6%
Maximum Vertical Deformation @ T max.	-0.032 IN
Shearing Stress, T max.	0.7 KSF

POINT NO. 2 (NORMAL STRESS 4.003 KSF)

Initial Moisture Content	2.9%
Dry Density	94.3 LB/CU FT
Moisture at Saturation	29.7%
Maximum Vertical Deformation @ T max.	-0.04 IN
Shearing Stress, T max.	2.7 KSF

POINT NO. 3 (NORMAL STRESS 8.01 KSF)

Initial Moisture Content	3.4%
Dry Density	91.9 LB/CU FT
Moisture at Saturation	31.4%
Maximum Vertical Deformation @ T max.	0.03 IN
Shearing Stress, T max.	4.4 KSF


DIRECT SHEAR
E87-11-45
NORMAL STRESS (KSF)
SHEARING STRESS (KSF)
0
2
4
6
8
0
1
2
3
4
5
6

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REPORT OF LABORATORY TESTS

PROJECT: VULTURE MINE

LOCATION: TAILINGS

DATE 3/25/87

JOB NO. E87-11

W.O.NO. 2

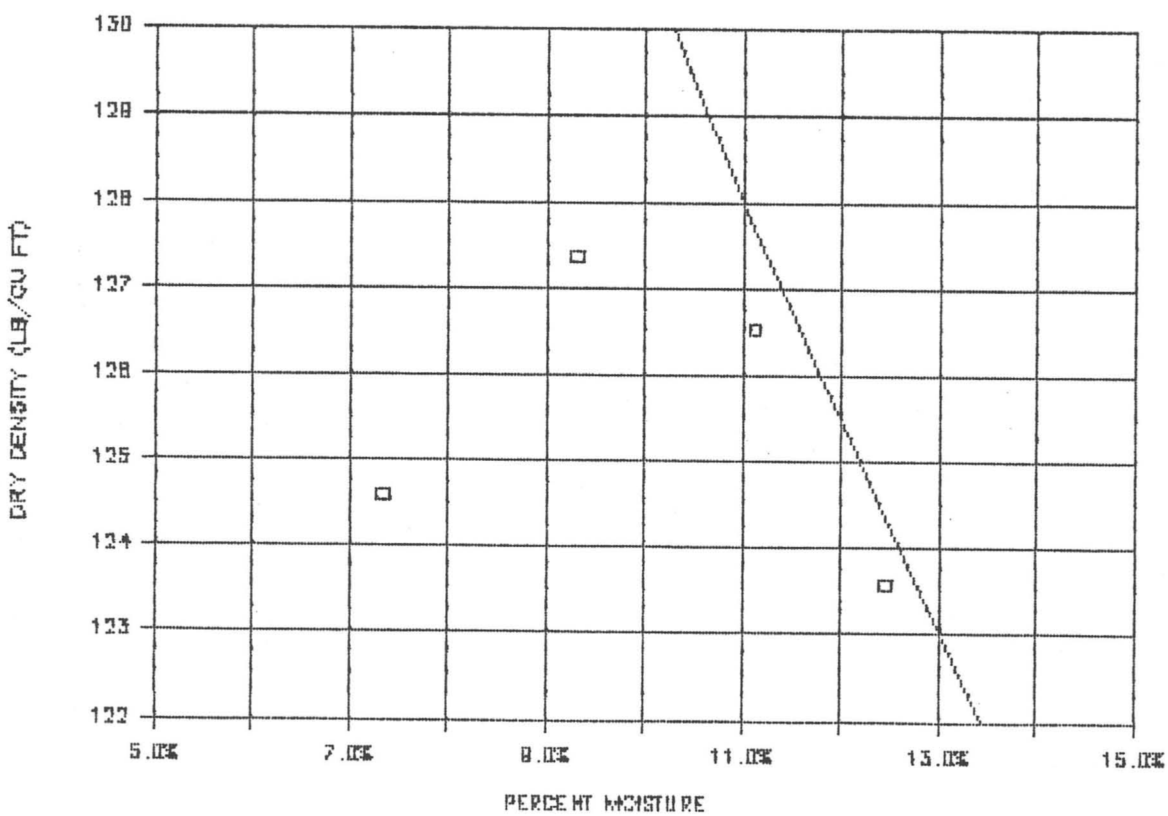
LAB NO. 62

==

TEST DESIGNATION ASTM D698
METHOD A
CURVE

MAXIMUM DRY DENSITY 127.8 LB/CU FT
OPTIMUM MOISTURE CONTENT 10.2%

MOISTURE — DENSITY RELATIONSHIP



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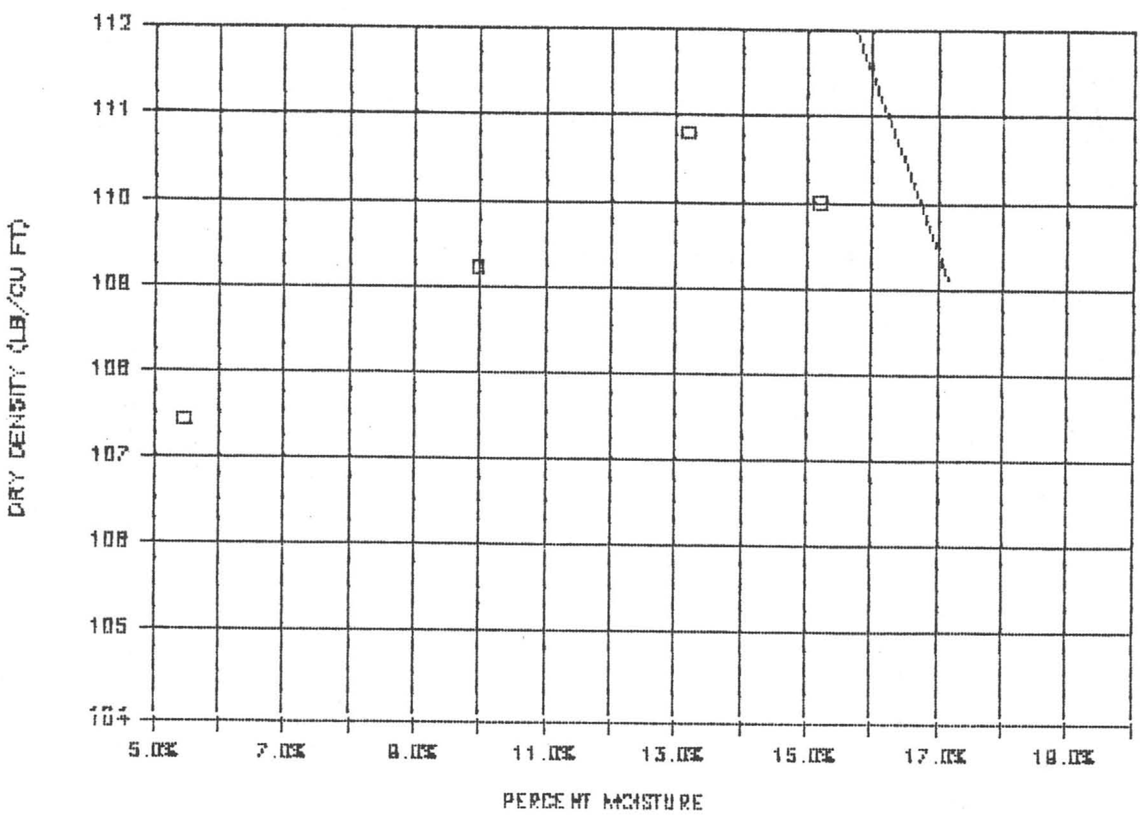
REPORT OF LABORATORY TESTS	DATE 3/25/87
PROJECT: VULTURE MINE	JOB NO. E87-11
LOCATION: TAILINGS	W.O.NO. 2
	LAB NO. 61

==

TEST DESIGNATION ASTM D698
METHOD A
CURVE

MAXIMUM DRY DENSITY 110.9 LB/CU FT
OPTIMUM MOISTURE CONTENT 13.4%

MOISTURE — DENSITY RELATIONSHIP



APPENDIX C

SERGENT, HAUSKINS & BECKWITH

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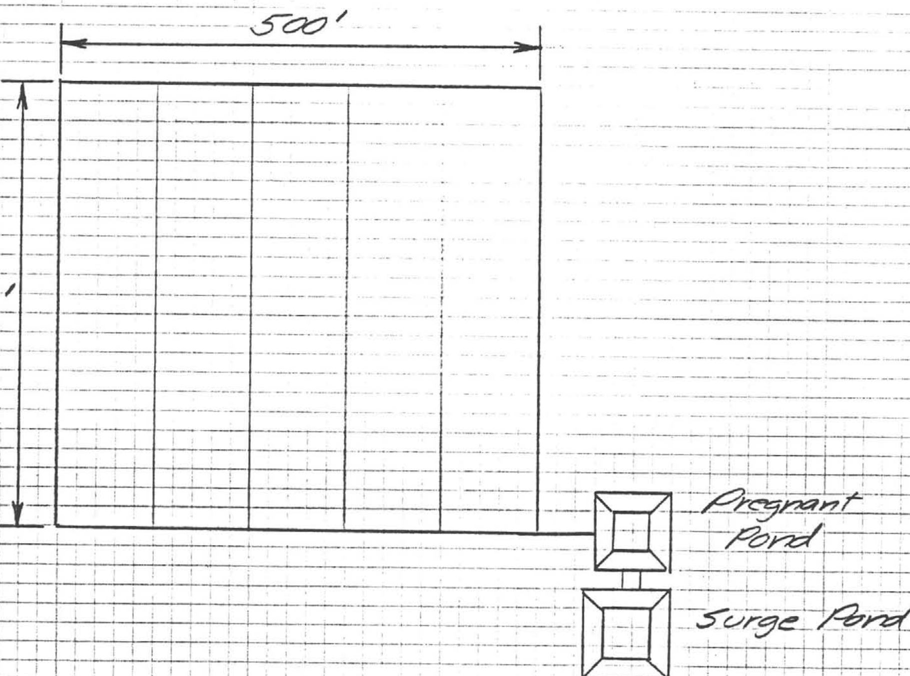
APPENDIX C

STORMWATER (SURGE) POND SIZING:

Calculate Runoff Volume From Leach Pad For Worst Case Condition In Which All Five Pad Segments Are Lined And Covered With Single Lift Of Tailings.



Assume: Curve Number (CN) = 98 For Geomembrane Covered Pad w/wo Single Lift Of Tailings. (conservative)

Runoff Volume = (Surface Area Of Pad)(Rainfall)(CN)

Case	Peak Rainfall, in.	Runoff Volume, ft^3 (gal)
100 Yr., 24 Hr.	4.1	75,338 (563,525)
500 Yr., 24 Hr.	5.1	93,713 (700,970)
6 Hr., ½ PMP	4.7	86,363 (645,992)
6 Hr., PMP	9.3	170,888 (1,278,239)

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Project: Vulture Mine Project
Job No: E87-11
Computed by: NJL Ckd. by: JRF
Date: 4/87 Page 1 of 14

100 yr. + 500 yr. Events Are From NOAA Precipitation - Frequency Atlas Of The United States, Volume VIII - Arizona, 1973. (See Attached Figure)

1/2 PMP and PMP Are Computed From U.S. Department of Commerce, Probable Maximum Precipitation Estimates, Colorado River and Great Basin Drainages, HMR No. 49, September, 1977. (See Attached Computation Sheet).

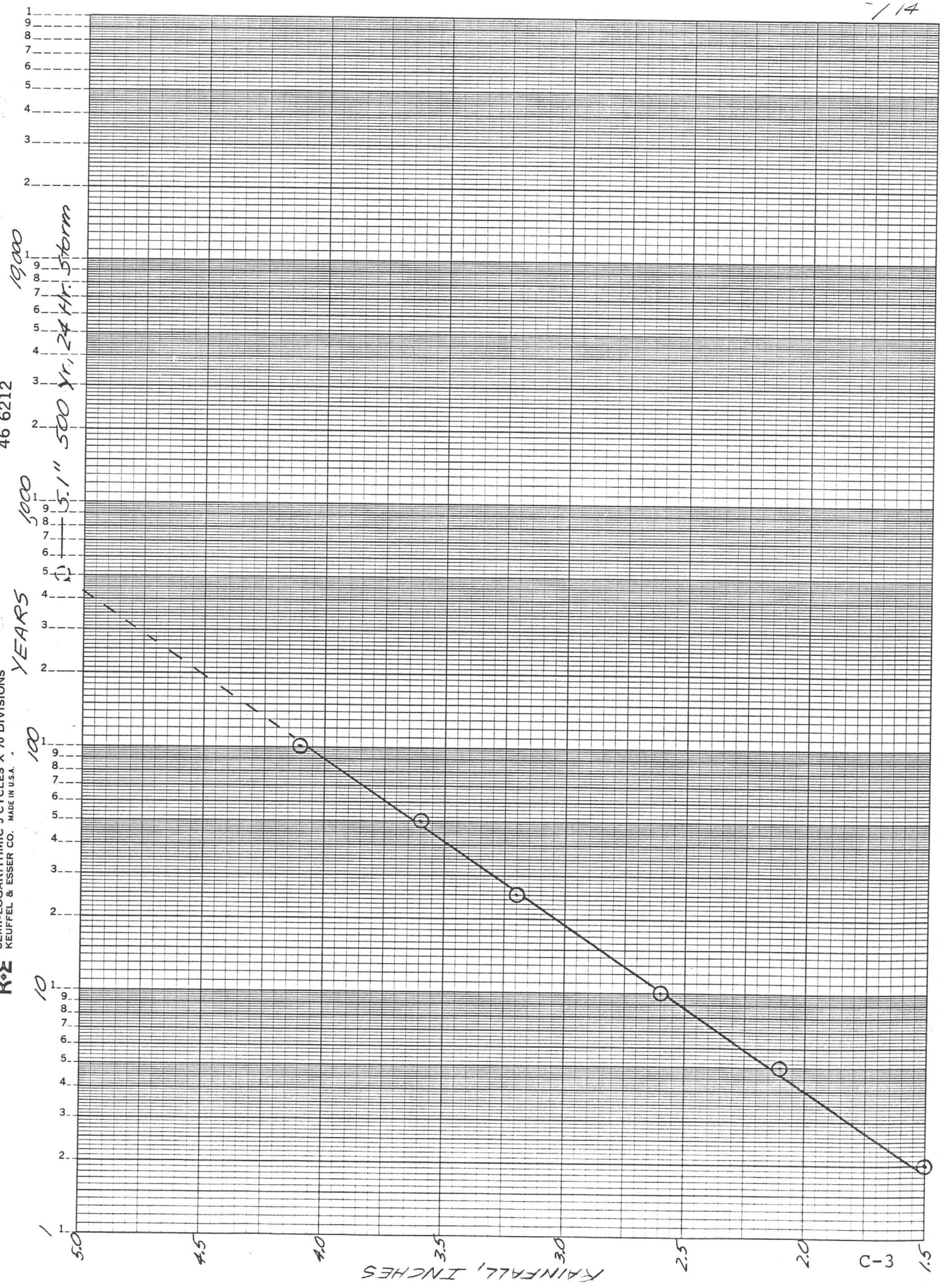
10,000
1000
100
10
1
YEARS
5.1" 500 Yr., 24 Hr. Storm
5.0
4.5
4.0
3.5
3.0
2.5
2.0
1.5
RAINFALL, INCHES

Table 6.1.--General-storm PMP computations for the Colorado River and Great basin

450' x 500' = 225,000 ft^2

Drainage Vulture Mine Leach Pad Area < 10 mi^2

Latitude 33° 50', Longitude 112° 50' of basin center

Month All Season (Highest Value Over All Months)

Step		Duration (hrs) 6	12	18	24	48	72	
A.	**Convergence PMP**							
1.	Drainage average value from one of figures 2.5 to 2.16 13.6 in. (August)							
2.	Reduction for barrier-elevation [fig. 2.18] 80 %							
3.	Barrier-elevation reduced PMP [step 1 X step 2] 10.9 in.							
4.	Durational variation [figs. 2.25 to 2.27 and table 2.7].	74	89	95	100	112	117	% (August)
5.	Convergence PMP for indicated durations [steps 3 X 4]	8.1	9.7	10.4	10.9	12.2	12.8	in.
6.	Incremental 10 mi^2 (26 km^2) PMP [successive subtraction in step 5]	8.1	1.6	0.7	0.5	1.3	0.6	in.
7.	Areal reduction [select from figs. 2.28 and 2.29]	100	100	100	100	100	100	% (August)
8.	Areally reduced PMP [step 6 X step 7]	8.1	1.6	0.7	0.5	1.3	0.6	in.
9.	Drainage average PMP [accumulated values of step 8]	8.1	9.7	10.4	10.9	12.2	12.8	in.
B.	**Orographic PMP**							
1.	Drainage average orographic index from figure 3.11a to d. 3.5 in. (3.11d)							
2.	Areal reduction [figure 3.20] 100%							
3.	Adjustment for month [one of figs. 3.12 to 3.17] 100% (August)							
4.	Areally and seasonally adjusted PMP [steps 1 X 2 X 3] 3.5 in.							
5.	Durational variation [table 3.9]	35	62	83	100	143	162	%
6.	Orographic PMP for given durations [steps 4 X 5]	1.2	2.2	2.9	3.5	5.0	5.7	in.
C.	**Total PMP**							
1.	Add steps A9 and B6	9.3	11.9	13.3	14.4	17.2	20.5	in.
2.	PMP for other durations from smooth curve fitted to plot of computed data.							
3.	Comparison with local-storm PMP (see sec. 6.3).							

∴ Use 6-Hr. General Storm PMP Rainfall of 9.3 in.

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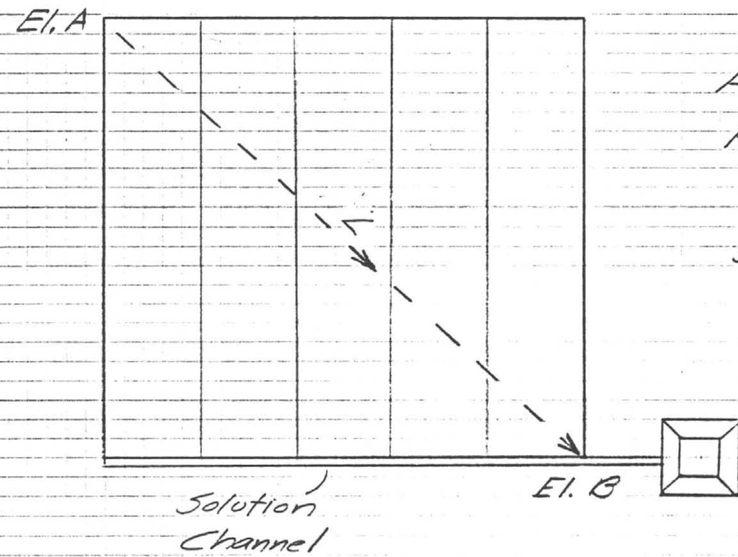
PEAK STORM FLOWS

$CN_{Liner} = 98$



Area ≅ 5.2 Acres

$H = El.\ A - El.\ B$

$L = 673$ Ft.

Slope, $Y = \frac{HL}{L}$

Calculate Storm Flows Using Soil Conservation Service TR-55 Method (McCuen, 1982)

Compute And Compare Times of Concentration Using Lag Method, Velocity Method TR-55 Method and Kirpich Equation.

Lag Method

$t_c = \frac{5}{3} L'$ L' From Figure 7 In McCuen (1982),

@ Slope $= \frac{H}{L} = \frac{16.4'}{673'} = 2.4\%$

$L = 673$ Ft.

$\therefore L' \cong 0.075$ hr.

$t_c = \frac{5}{3}(0.075) \cong 0.125$ Hr.

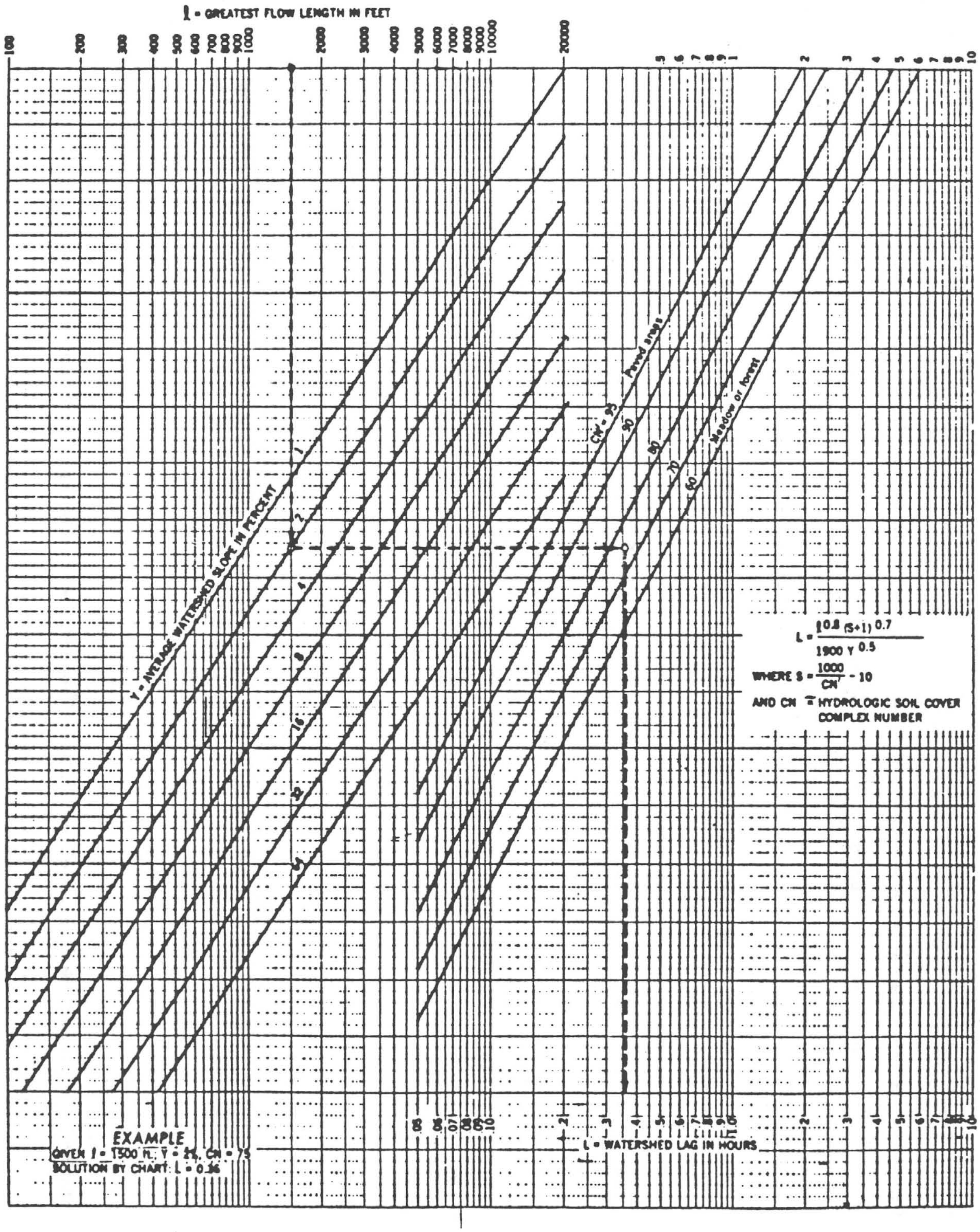


Figure 7.--Curve number method for estimating lag (L)

$$L = \frac{l^{0.8}(S+1)^{0.7}}{1900\,Y^{0.5}}$$

WHERE $S = \frac{1000}{CN} - 10$

AND CN = HYDROLOGIC SOIL COVER COMPLEX NUMBER

Reference: McCuen, R.H., A Guide To Hydrologic Analysis Using SCS Methods, 1982.

Velocity Method

$$t_c = \frac{HL}{3600V}$$

V = Velocity From Figure 8 In McCuen @ Slope = 2.4%

HL = 673 Ft.

∴ V ≙ 3.2 Ft./s

$$t_c = \frac{673}{3600(3.2)} = 0.058 \text{ Hr.}$$

TR-55 Method

$$t_c = \frac{5}{3}\,\frac{L^{0.8}(S+1)^{0.7}}{1900\ Y^{0.5}}$$

Y = Slope % = 2.4

$$S = \frac{1000}{CN} - 10$$

$$= \frac{1000}{98} - 10$$

$$= 0.20$$

L = 673'

$$t_c = \frac{5}{3}\,\frac{(673)^{0.8}(0.20+1)^{0.7}}{1900\ (2.4)^{0.5}} = 0.35 \text{ Hr.}$$

Kirpich Equation

$$t_c = \left(\frac{11.9L^3}{H}\right)^{0.385}$$

L = Length, Miles

$$t_c = \left[\frac{11.9\left(\frac{673}{5280}\right)^3}{16.4}\right]^3 = 0.082 \text{ Hr.}$$

∴ Use Conservative Value For t_c Of 0.058 Hr. (From Velocity Method).

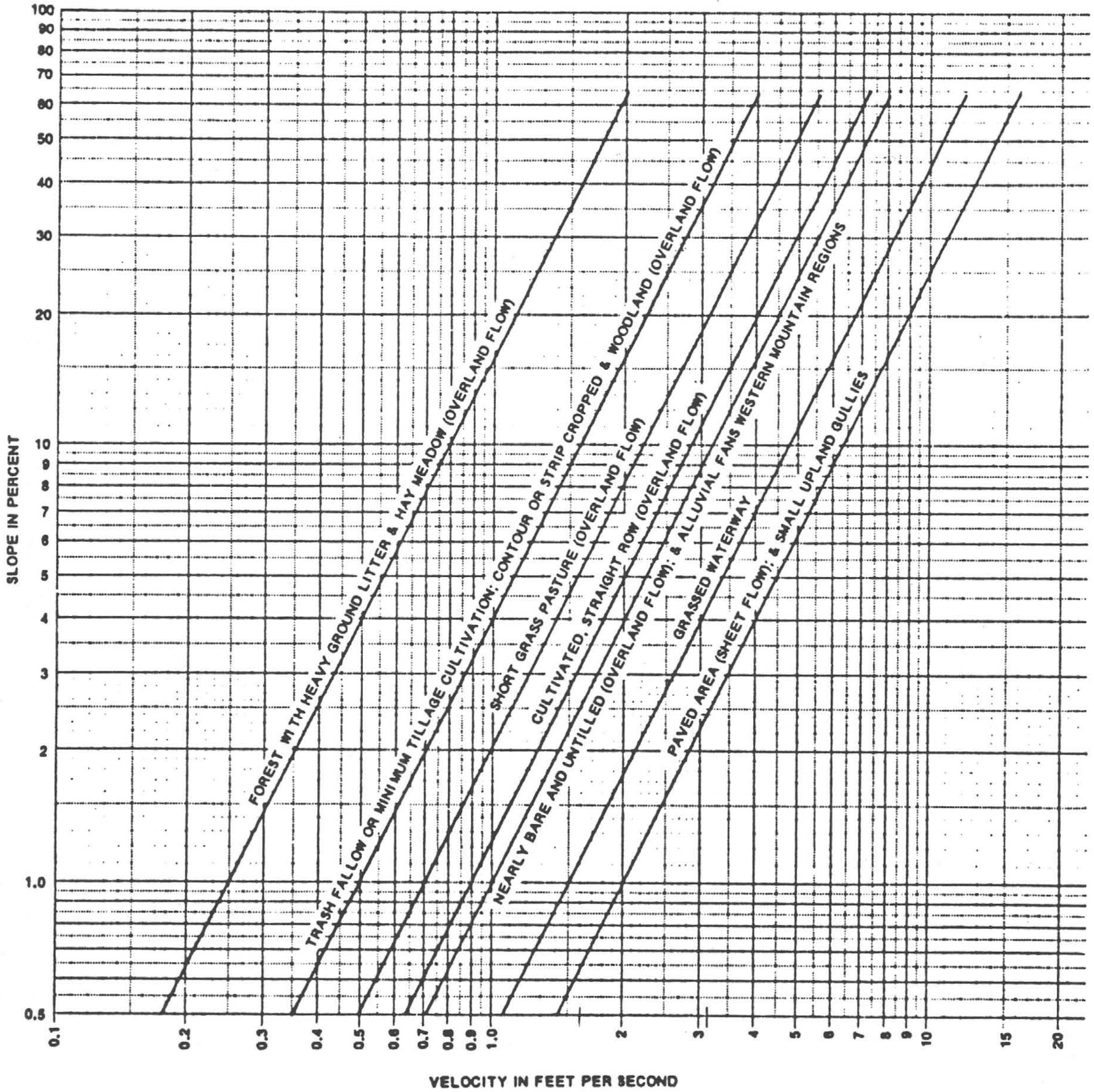
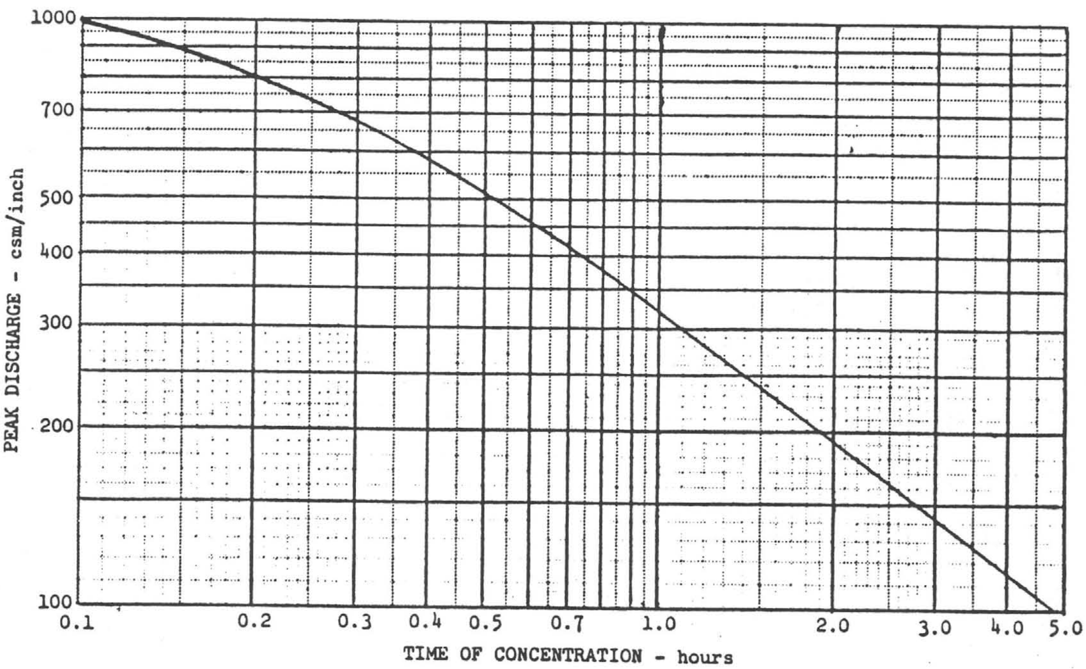


Figure 8. Velocities for upland method of estimating T_c

Reference: McCuen, 1982.

Peak Flows:

Condition	(P) Rainfall in.	CN	Q in.	A mi^2	t_c hr.	(From Fig. 9, McCuen) q'_p cfs/mi^2/in.
6 Hr. PMP	9.3	98	9.06	0.008	0.058	1150
6 Hr. ½ PMP	4.7	↓	4.46	↓	↓	↓
500 yr., 24 Hr.	5.1	↓	4.86	↓	↓	↓
100 yr., 24 Hr.	4.1	↓	3.86	↓	↓	↓



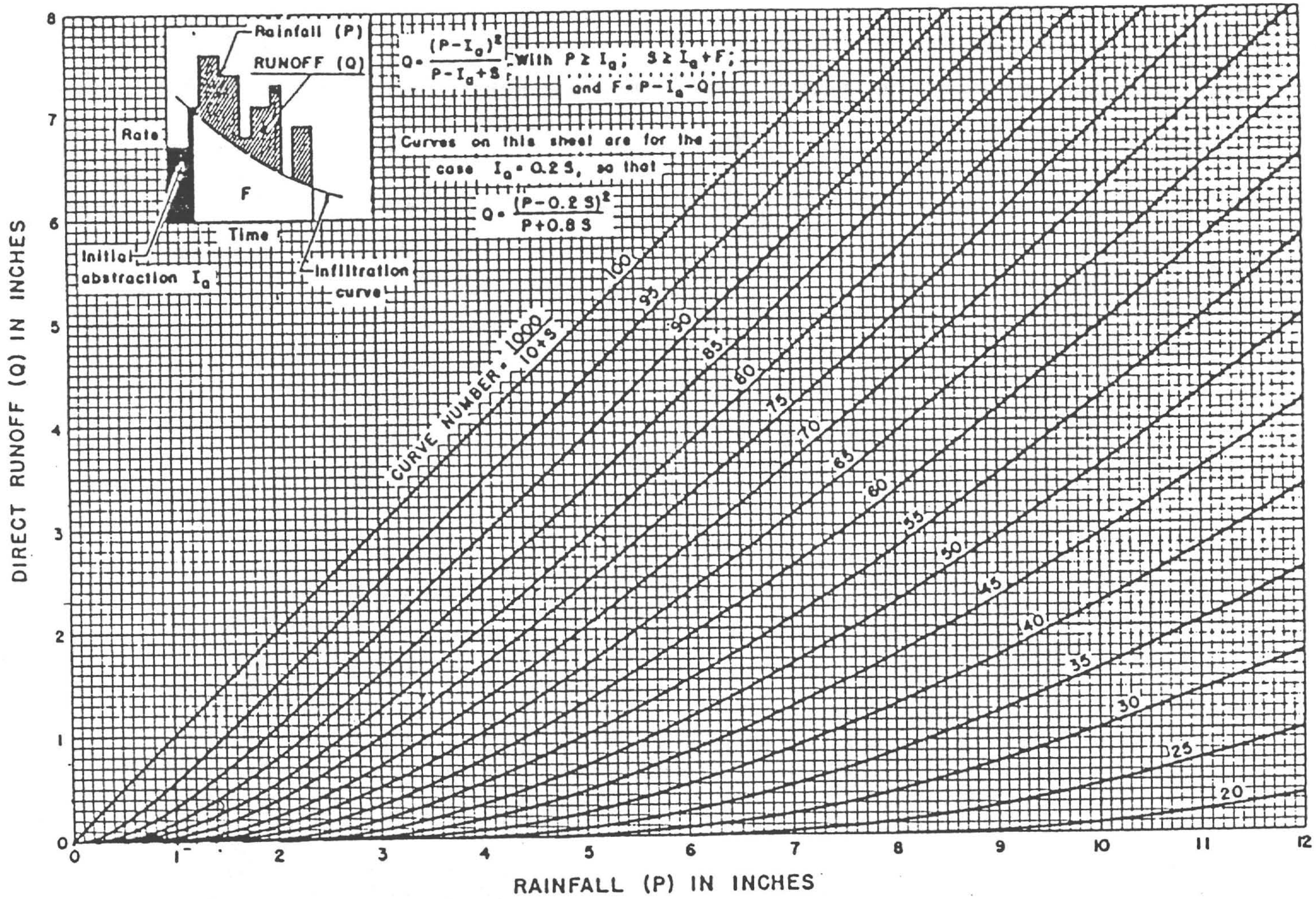
Where $Q = \frac{(P-0.2S)^2}{P-0.8S}$ Or From Fig. 9.1 (Mockus + Victor)

For P = 0-12 inches
Q = 0-8 inches

Or $Q = \frac{(P-0.2S)^2}{P+0.8S}$ For P = 8-40 inches
Q = 0.40 inches

HYDROLOGY: SOLUTION OF RUNOFF EQUATION $Q = \frac{(P-0.2S)^2}{P-0.8S}$

P=0 to 12 inches
Q=0 to 8 inches



REFERENCE Mockus, Victor: Estimating direct runoff amounts from storm rainfall: Central Technical Unit, October, 1955

FIGURE 9.1

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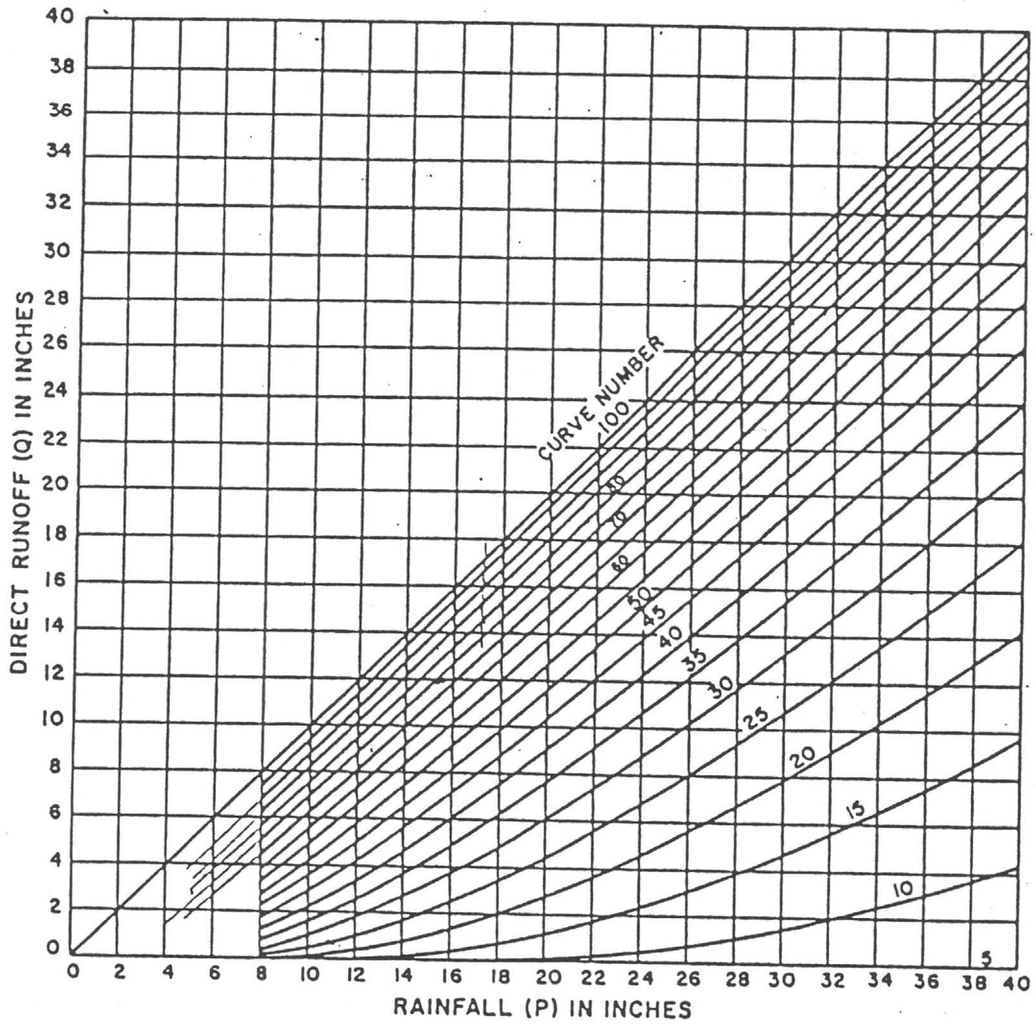
HYDROLOGY: SOLUTION OF RUNOFF EQUATION $Q = \frac{(P-0.2S)^2}{(P+0.8S)}$ P=8 to 40 inches Q=0 to 40 inches



REFERENCES Mockus, Victor: Estimating direct runoff amouts from storm rainfall: Central Technical Unit, October, 1955

FIGURE 9.2

SIZE CHANNEL FOR STORM RUNOFF

Pad Area = 5.2 Acres = 0.008 Mi^2

Hydraulic Length = 673'

H = 16.4'

Use Velocity Method For Conservatism

$$t_c = \frac{673}{3600(3.2)} = 0.058 \text{ Hr.}$$

Design Conditions

All Five Pad Segments Lined w/ Geomembrane.

CN = 98

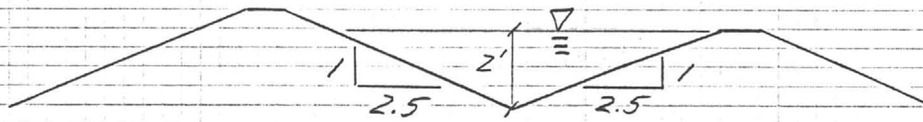
$$S = \frac{1000}{98} - 10 = 0.20$$

Condition	P in.	Q in	q'_p csm/in.	A mi^2	Qp cfs
6 Hr. PMP	9.3	9.06	1150	0.008	84.4
6 Hr. ½ PMP	4.7	4.46	↓	↓	41.6
500 Yr. 24 Hr.	5.1	4.86	↓	↓	45.3
100 Yr. 24 Hr.	4.1	3.86	↓	↓	36.0

$$Q_p = q'_p Q A = (1150)(0.008) Q = 9.32 Q$$

CHANNEL HYDRAULICS

Typical Channel Section



Width, B = 0'

Sides, Z = 2.5 : 1

Slope, S = 0.005

Roughness, n = 0.010

Manning Equation $Q = \frac{1.486}{n} A R^{2/3} S^{1/2}$

Where A = Channel Cross Sectional Area

R = Hydraulic Radius = $\frac{A}{\text{Wetted Perimeter}}$

S = Channel Slope

Condition	Q_p CFS	H_N Ft	V_N Ft/s	H_{CR} Ft.	V_{CR} Ft/s	S_{CR}
½ PMP	41.6	1.44	8.03	1.77	5.33	0.00168
500 Yr.	45.3	1.49	8.20	1.83	5.42	0.00166
100 Yr.	36.0	1.36	7.74	1.67	5.18	0.00171
Preg. Flow	0.2	0.20	2.10	0.21	1.82	0.0034

H_N = Normal Height Of Flow

V_N = Normal Velocity

H_{CR} = Critical Height

V_{CR} = Critical Velocity

S_{CR} = Critical Slope

See Computer Summary Sheets For Summary Of Channel Hydraulic Analysis.

SERGENT, HAUSKINS & BECKWITH

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Project Vulture Mine Project
Job No: E87-11
Computed by: NJL Ckd. by: JRF
Date 4/87 Page 13 of 14

VULTURE MINE - SOLUTION/STORMWATER CHANNEL DESIGN

TRAPEZOIDAL SECTION

BOTTOM WIDTH .... B = 0.0 FEET
SIDE SLOPE .... Z = 2.50
MAXIMUM HEIGHT .... H = 2.0 FEET

CHANNEL SLOPE .... S = 0.00500
MANNING'S NUMBER .... n = 0.01000

HEIGHT (FT)	AREA (SQ FT)	WET P. (FT)	R (FT)	VELOCITY (FT/SEC)	DISCHARGE (CFS)
0.00	0.0	0.0	0.001	0.070	0.0
0.10	0.0	0.5	0.047	1.359	0.0
0.20	0.1	1.1	0.093	2.156	0.2
0.30	0.2	1.6	0.139	2.825	0.6
0.40	0.4	2.2	0.186	3.421	1.4
0.50	0.6	2.7	0.232	3.970	2.5
0.60	0.9	3.2	0.279	4.483	4.0
0.70	1.2	3.8	0.325	4.968	6.1
0.80	1.6	4.3	0.371	5.430	8.7
0.90	2.0	4.8	0.418	5.873	11.9
1.00	2.5	5.4	0.464	6.301	15.8
1.10	3.0	5.9	0.511	6.714	20.3
1.20	3.6	6.5	0.557	7.115	25.6
1.30	4.2	7.0	0.604	7.505	31.7
1.40	4.9	7.5	0.650	7.885	38.6
1.50	5.6	8.1	0.696	8.256	46.4
1.60	6.4	8.6	0.743	8.619	55.2
1.70	7.2	9.2	0.789	8.974	64.8
1.80	8.1	9.7	0.836	9.323	75.5
1.90	9.0	10.2	0.882	9.665	87.2
2.00	10.0	10.8	0.929	10.001	100.0

DISCHARGE (CFS)	NORMAL HEIGHT (FT)	NORMAL VELOCITY (FT/SEC)	CRITICAL HEIGHT (FT)	CRITICAL VELOCITY (FT/SEC)	CRITICAL SLOPE
41.6	1.440	8.03	1.767	5.33	0.001677
45.3	1.486	8.20	1.828	5.42	0.001658
36.0	1.364	7.74	1.667	5.18	0.001710
0.2	0.195	2.10	0.209	1.82	0.003366

APPENDIX D

SERGENT, HAUSKINS & BECKWITH

CONSULTING GEOTECHNICAL ENGINEERS
PHOENIX • ALBUQUERQUE • SANTA FE

Heap Leach Facility Design
Vulture Mine Project
Near Wickenburg, Arizona
SHB Job No. E87-11

GUIDE SPECIFICATIONS FOR SITE GRADING

1. Observations & Tests

The geotechnical engineer shall act as the engineer's representative and shall make observations of site grading operations and tests as considered necessary for quality control.

Where foundations or other critical elements are to be supported on structural fill, continuous observations and tests of grading operations shall be made by representatives of the geotechnical engineer. All tests shall be performed in accordance with procedures set forth in Section 4, Volumes 04, 02, 04.03 and 04.08, of the 1984 Book of ASTM Standards.

2. Foundation Preparation

2.1 Clearing & Grubbing

All vegetation, debris and other deleterious material shall be removed from areas of cut and fill. This includes the leach pad, perimeter channel and pond areas as shown on the plans.

Materials to be removed include surface boulders, organic matter including trees, stumps and roots, or other objectional material as determined by the Engineer.

2.2 Stockpiling Topsoil

All topsoil removed during the clearing and grubbing operation may be stockpiled in areas designated by the Construction Manager.

2.3 Surface Compaction

The upper 6 inches of native soils beneath cut surfaces and areas to receive fill in the pond areas shall be scarified and compacted to a minimum of 95 percent of maximum density as determined in accordance with ASTM D698. Moisture content during compaction shall be maintained within the limits of 2 percent below to 3 percent above optimum moisture content as determined by ASTM D698.

Where tailings are present as foundation materials beneath the pad, perimeter channel and berms, the upper 1 foot shall be overexcavated and recompacted in accordance with the aforementioned standards.

3. Embankment Construction

3.1 Random Fill Material Quality

All embankment materials shall be free of excessive vegetation, debris, organic matter and other deleterious materials. Soils classified as random fill may be obtained from overexcavated areas of the pad and perimeter channel, pregnant, barren and surge pond excavations, or other areas on-site approved by the Geotechnical Engineer. All excavation from the area of the pad and ponds shall be considered suitable for use on embankments provided they contain no particles larger than 6 inches nominal diameter.

3.2 Compaction Requirements

Compaction of embankments shall be to a minimum of 95 percent of ASTM D698 maximum dry density. The moisture content during compaction shall be maintained within the limits of 2 percent below to 3 percent above optimum moisture content as determined in accordance with ASTM D698.

3.3 Placement & Compaction

Embankment and fill materials shall be placed to the lines and grades shown on the plans.

The materials shall be placed in lifts not to exceed 8 inches in thickness, unless otherwise authorized by the Geotechnical Engineer. Where the Contractor demonstrates the equipment being used effectively compacts lifts thicker than 8 inches, thicker lifts may be authorized by the geotechnical engineer.

4. Finished Subgrade in Ponds

A 6-inch thick compacted tailings layer shall be placed as the finished subgrade surface for liner placement in the three ponds. The tailings shall be placed and compacted in accordance with the specifications of Sections 3.2 and 3.3.

5. Overliner in Surge Pond

A 2-foot thick tailings cover shall be placed over the 20-mil PVC in the surge pond. The upper 1 foot shall be compacted in accordance with the specifications of Sections 3.2. and 3.3 to provide a surface less susceptible to erosion.

6. Shotcrete Channel Protection

6.1 Areas of Placement

Shotcrete shall be placed beneath the geomembrane in the pregnant pond spillway as shown on the design plans. Additionally, shotcrete shall be placed as erosion protection on the floor and side slopes of the storm water diversion channel.

6.2 Material Quality & Placement

Shotcrete shall meet ACI Recommended Practice 506-66 (Revised 1983) for wet mix process pneumatically placed mortar. Additional requirements to these specifications are listed below.

A. Fine aggregate shall consist of washed sand and shall be hard, dense, durable, clean, and well graded from fine to coarse, with no particles larger than 3/8 inch in diameter. It shall be free from organic matter and shall not contain more than 5 percent by weight passing the no. 200 sieve.

B. The sand to be used shall contain not less than 3 percent nor more than 6 percent of moisture.

C. Before placing mixture in the hopper of the gun, all lumps too large for proper handling by the gun shall be removed by passing the mixture through a screen of suitable size.

D. The air pressure shall be maintained uniform and shall be sufficient to maintain uniform and satisfactory nozzle operation.

E. At any construction joint, shotcrete shall be sloped to a thin edge. Before shooting adjacent section, sloped portion shall be thoroughly cleaned and wetted. No square joints will be allowed.

F. Shotcrete shall be membrane cured as described in MAG (525.7). No shotcrete shall be placed during freezing weather, except when proper protective measures are taken as with ordinary concrete work. Shotcrete shall not be placed against frosted surfaces.

G. Succeeding layers shall be placed less than one hour apart.

7. Field Density Tests

For purposes of acceptance, the in-place density of random and tailings fills shall be defined as that determined by the sand cone method (ASTM D1556) or by other test procedures acceptable to the geotechnical engineer.

7.1 Compaction Methods & Equipment

Mechanical compaction equipment shall be used in all grading operations. In no case shall water settling or "jetting" be employed.

7.2 Thickness of Lifts

Fill lifts shall have a loose thickness of no more than 10 inches unless otherwise authorized by the geotechnical engineer.

8. Weather Limitations

Unless approved in the field by the geotechnical engineer, controlled fill shall not be constructed when the atmospheric temperature is at 35°F and falling. When the temperature falls below 35°F, it shall be the responsibility of the contractor to protect all areas of completed surface against any detrimental effects by methods approved by the geotechnical engineer. Any areas damaged by freezing shall be reconditioned, reshaped, and recompacted by the contractor in conformance with the requirements of this specification.

TABLE OF CONTENTS

Registered Professional Engineer (Civil) CERTIFICATE NO. 13872 LAWRENCE A. HANSEN Date Signed 4/10/87 ARIZONA, U.S.A.

SHB Job No. E87-11

SERGENT, HAUSKINS & BECKWITH
Geotechnical Engineers, Inc.

3232 West Virginia Avenue
Phoenix, Arizona 85009
(602) 272-6848

DMEA LTD.
JUN 3 1987
RECEIVED

TRANSMITTAL

DATE 6/2

TO A.F. BUDGE (MINING) LTD.
7340 E. SHOEMAN LANE
SUITE 111 "B" (E) SCOTTSDALE AZ 85251-3335

ATTENTION JOE FERNANDEZ

PROJECT VULTURE MINE HEAP LEACH FACILITY

JOB/PROPOSAL NO. E87-11

WE ARE SENDING YOU:

- [x] Attached
- [ ] Under separate cover the following:
- [ ] Boring Logs
- [ ] Calculations
- [ ] Design Charts
- [ ] Progress Reports
- [x] Laboratory Results
- [ ] Plans
- [ ] Specifications
- [ ] ______________________

DELIVERY BY:

- [ ] Hand Delivery
- [x] First Class Mail
- [ ] Registered Mail
- [ ] Express Mail
- [ ] Federal Express
- [ ] Other
- [ ] Return Receipt Requested

TRANSMITTED FOR:

- [ ] Review & Comment
- [ ] Approval
- [x] Your Files/Information
- [ ] As Requested

DESCRIPTION TEST RESULTS FOR EP TOXICITY - METALS, REACTIVITY, CORROSIVITY & GROSS ALPHA/BETA POTENTIAL FROM SAMPLES WITHIN THE EXISTING TAILS AT THE PAD SITE.

REMARKS JOE - ONLY CONCERN IS Pb IN TEST PIT SAMPLE No. 15. EP TOXICITY LEVEL IS 5 mg/l - WE HAVE 42.1 mg/l.

COPY TO E87-11 FILE

SIGNED Jim [signature]

INDICATORS
PROJECT: VULTURE MINE E 87-11

SAMPLE DATE: 04/22/87
LAB RECEIPT DATE: 04/22/87
MATRIX: SOIL

SAMPLE ID: TP-10
LOCATION:
LAB SAMPLE #: 9597-1
UNITS: mg/kg

ANALYSIS DATE	CONSTITUENT NAME	DILUTION FACTOR	DETECTION LIMIT	RESULT
4/23/87	CYANIDE (CN)		0.5	ND
4/27/87	PHENOLS		2.0	ND
4/23/87	CORROSIVITY (pH)		----	8.46
5/06/87	S=		20	ND
	REACTIVITY			NONE

NOTE: ND = NOT DETECTED
NA = NOT ANALYZED

SUPERVISORY REVIEW: R. GERRY MCCULLOUGH RH DATE: 15 MAY 1987

INDICATORS
PROJECT: VULTURE MINE E 87-11

SAMPLE DATE: 04/22/87
LAB RECEIPT DATE: 04/22/87
MATRIX: SOIL

SAMPLE ID: TP-15
LOCATION:
LAB SAMPLE #: 9597-2
UNITS: mg/kg

ANALYSIS DATE	CONSTITUENT NAME	DILUTION FACTOR	DETECTION LIMIT	RESULT
4/23/87	CYANIDE (CN)		0.5	ND
4/27/87	PHENOLS		2.0	ND
4/23/87	CORROSIVITY (pH)		----	8.30
5/06/87	S=		20	ND
	REACTIVITY			NONE

NOTE: ND = NOT DETECTED
NA = NOT ANALYZED

SUPERVISORY REVIEW: R. GERRY MCCULLOUGH DATE: 15 MAY 1987

AnalyticalTechnologies,Inc. 2113 S. 48th Street Suites 107-110 Tempe, AZ 85282 (602) 438-1530

METALS
PROJECT: VULTURE MINE E 87-11

SAMPLE DATE: 04/22/87
LAB RECEIPT DATE: 04/22/87
MATRIX: SOIL

SAMPLE ID: TP-10
LOCATION:
LAB SAMPLE #: 9597-1
UNITS: mg/l as EPTOX

ANALYSIS DATE	CONSTITUENT NAME	DILUTION FACTOR	DETECTION LIMIT	RESULT
4/27/87	ARSENIC (As)		0.010	ND
5/01/87	BARIUM (Ba)		0.07	0.09
4/27/87	CADMIUM (Cd)		0.003	0.154
4/27/87	TOTAL CHROMIUM (Cr Tot)		0.010	ND
4/30/87	LEAD (Pb)		0.02	2.39
5/01/87	MERCURY (Hg)		0.0005	ND
4/27/87	SELENIUM (Se)		0.010	ND
4/30/87	SILVER (Ag)		0.010	ND

NOTE: ND = NOT DETECTED
NA = NOT ANALYZED

SUPERVISORY REVIEW: R. GERRY MCCULLOUGH DATE: 15 MAY 1987

METALS
PROJECT: VULTURE MINE E 87-11

SAMPLE DATE: 04/22/87
LAB RECEIPT DATE: 04/22/87
MATRIX: SOIL

SAMPLE ID: TP-15
LOCATION:
LAB SAMPLE #: 9597-2
UNITS: mg/l as EPTOX

ANALYSIS DATE	CONSTITUENT NAME	DILUTION FACTOR	DETECTION LIMIT	RESULT
4/27/87	ARSENIC (As)		0.010	ND
5/01/87	BARIUM (Ba)		0.07	ND
4/27/87	CADMIUM (Cd)		0.003	0.254
4/27/87	TOTAL CHROMIUM (Cr Tot)		0.010	0.010
4/30/87	LEAD (Pb)		0.02	42.1
5/01/87	MERCURY (Hg)		0.0005	ND
4/27/87	SELENIUM (Se)		0.010	ND
4/30/87	SILVER (Ag)		0.010	ND

NOTE: ND = NOT DETECTED
NA = NOT ANALYZED

SUPERVISORY REVIEW: R. GERRY MCCULLOUGH DATE: 15 MAY 1987

TMA
Thermo Analytical Inc.

TMA/Norcal
2030 Wright Avenue
Richmond, CA 94804-0040
(415) 235-2633

SOIL ANALYSIS REPORT

TMA/Norcal CN No.: 4227
Customer P.O. No.: ----
Date Received: 4/27/87
Date Reported: 5/19/87
No. of Samples: 2

Kathy Meinders
Analytical Technology
2113 South 48th. St., Ste. 110
Tempe, AZ 85282

Sample Number	Collection Date	Results pCi/g ± 2 σ	
		Gross Alpha	Gross Beta
9597-1	4/22/87	6 ± 3	52 ± 6
9597-2	4/22/87	15 ± 3	68 ± 7

Marvin P. Hunt
Program Manager
TMA/Norcal

MPH/ss

Enclosure: Chain of Custody

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MAP POCKET

Registered Professional Engineer (Civil) CERTIFICATE NO. 13872 LAWRENCE A. HANSEN Date Signed 4/10/87 ARIZONA, U.S.A.

SHB Job No. E87-11

develop perimeter channel and storm water pond dimensions. Local on-site drainage will be collected and retained so that no precipitation that falls on the leach pads will escape the containment system. A detailed summary of the hydrologic and hydraulic analyses utilized in design of the solution channel and storm water pond are presented in the following sections.

5.2.2 Calculation Standards, Methods & Procedures

Peak runoff flow quantities for the ore pile area were calculated using the U.S. Soil Conservation Service TR-55 method (McCuen, 1982). The method is based on a computational procedure that is related to the 24-hour storm rain depth. Runoff volumes were calculated based on the total volume of storm rainfall.

For purposes of comparison, rain depth and rainfall distributions for the 100 and 500-year, 24-hour storms and the PMP and 1/2 PMP, 6-hour events were calculated using procedures outlined in The National Weather Service Hydrometeorological Report No. 49 (U.S. Department of Commerce, 1977). Storm volumes and peak flows for one half of the six-hour PMP event were used in developing the design capacity of the storm water pond. Peak runoff volumes and flows for this storm event are essentially equivalent to those of the interpolated 500-year, 24-hour storm. Although storage of the full PMP event is not provided, statistically a "zero discharge" facility is being designed, considering the estimated operation life of the project.

Rainfall depths for the 100 and 500-year storm events are based on the Precipitation Frequency Atlas for Arizona, Volume VIII, (U.S. Department of Commerce, 1973). The velocity method (McCuen, 1982) presented below was used for estimating the times of concentration.

$$tc = \frac{HL}{3600\ V}$$

where: tc = time of concentration, hours

HL = length of watershed subarea (hydraulic length), miles

V = velocity of storm runoff, fps

Leach pad hydrology calculations are presented in Appendix C.

Hydraulic calculations performed for determining channel capacities, flow depths and flow velocities are based on the Manning's Equation (Chow, 1959). A roughness coefficient of 0.01 was used in analysis of the Hypalon-lined solution channels. Computer summary sheets of the channel hydraulic calculations are included in Appendix C.

6. STABILITY ANALYSES OF LEACH PILE

Stability analyses were conducted for the leach pile in which various potential failure modes, including shallow and deep-seated failure surfaces within the ore pile, and shallow block failures along the ore-geomembrane interface were analyzed. A summary of these analyses, including the major assumptions and factors considered in the analyses, are discussed in the following sections.

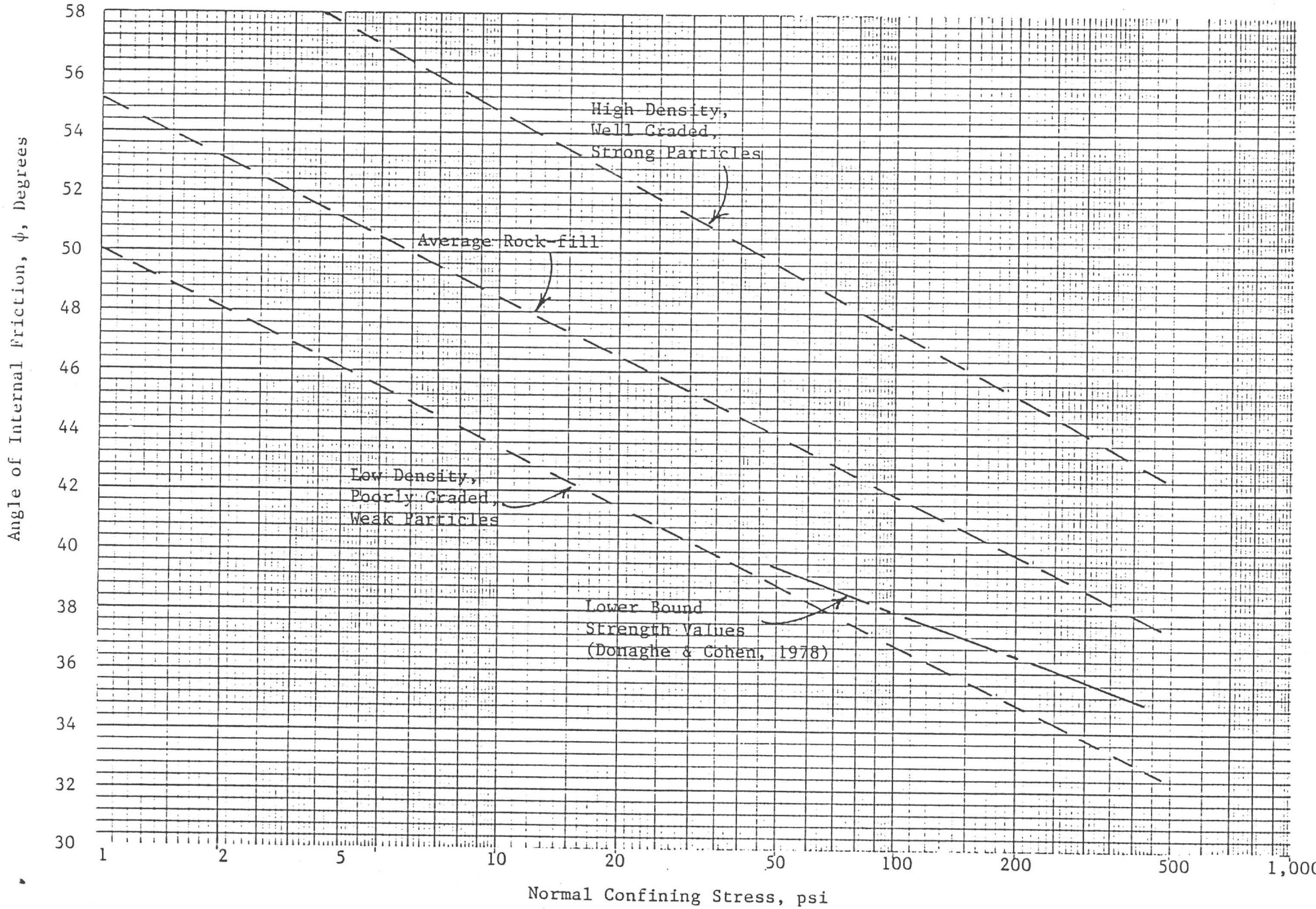
6.1 Shear Strength of Ore & Tailings

Generally, leach ore materials are difficult to test during design because they are not usually accessible for sampling and the rocks involved are highly variable in most ore bodies. Most rock involved in leach piles for "hard rock" mining of gold, silver, copper, lead, zinc and other metals fall within the range of materials which have been tested for rock-fill dam design. A widely used approach in dump design is to conservatively estimate shear strength based on this data (Call, 1981; Caldwell and others, 1981).

Figure 2 presents a summary of the large amount of triaxial shear testing done in the past 38 years on rock-fill materials as reported in the engineering literature. This chart, which has been widely

FIGURE 2

Angle of Internal Friction Versus Confining Stress



used in rock dump design, was originally based on a summary of shear strength data on quarried rock and coarse gravels by Leps (1970). Figure 2 shows the relationship between the angle of internal friction (ϕ) of broken rock and the normal stress across the shear plane (vertical confining pressure). The limits of test data available at the time and the average curve for rock-fill are given on the chart. Subsequent data on the shear strength of rock-fill (Marsal, 1974; Donaghe and Cohen, 1978; Barton and Kjaersli, 1981) are consistent with Figure 2.

The lower limit of tests on seven rock-fill materials performed by the U.S. Army Corps of Engineers (Donaghe and Cohen, 1978) is also shown in Figure 2. This lower limit was defined by weathered, relatively soft granite. A conservative angle of internal friction equal to 34 degrees was selected for design analysis involving the crushed leach ore. This conservative value accounts for a reduction in particle angularity due to agglomeration of the ore, and any variation in rock quality and durability. The existing amalgamated tails will be agglomerated to about 3/8 inch. A conservative angle of internal friction of 30 degrees was assigned to these materials.

6.2 Possible Accumulation of Solution in the Leach Pile

Due to the effects of agglomeration, an openwork drainage condition is anticipated at the base of the pile. In addition, 3-inch diameter perforated drain pipes will be provided on the liner surface directly below the ore pile to enhance lateral drainage. Therefore, the possible development of a phreatic surface within the leach pile above 1 to 2 feet is considered remote. Saturated conditions within the ore pile to this height would not significantly affect pile stability.

It is our opinion and experience with other leach facilities that a much greater risk exists due to broken sprinkler lines and ponding of solution near the crest of the pile. This risk is reduced to a low level if proper leach pad maintenance is followed. Nonetheless, allowance in design was made for sloughing or shallow slides of the pile slopes by providing a 10-foot wide open corridor between the toe of the pile and the solution channel berm, and by the inclusion of 5-foot wide benches at the top of each 15-foot lift height.

6.3 Stability Analysis Methodology & Results

6.3.1 Shallow Sloughing & Rock Falls

The ore pile will be constructed in 15-foot lifts at slopes equal to the angle of repose of the agglomerated tailings and crushed ore. Shallow wedge-type and circular failure surfaces possess very low factors of safety for cohesionless materials placed in this manner. Just back of the slope, factors of safety approach 1.0. Minor sloughing and rock falls from the slope face are commonly caused by external driving forces such as wind, rain and ice. Although the extent of sloughing is anticipated to be very minor, the 10-foot wide corridor at the toe of the pad, the 5-foot pile benches, and the solution containment berm were, in part, designed to accommodate this occurrence.

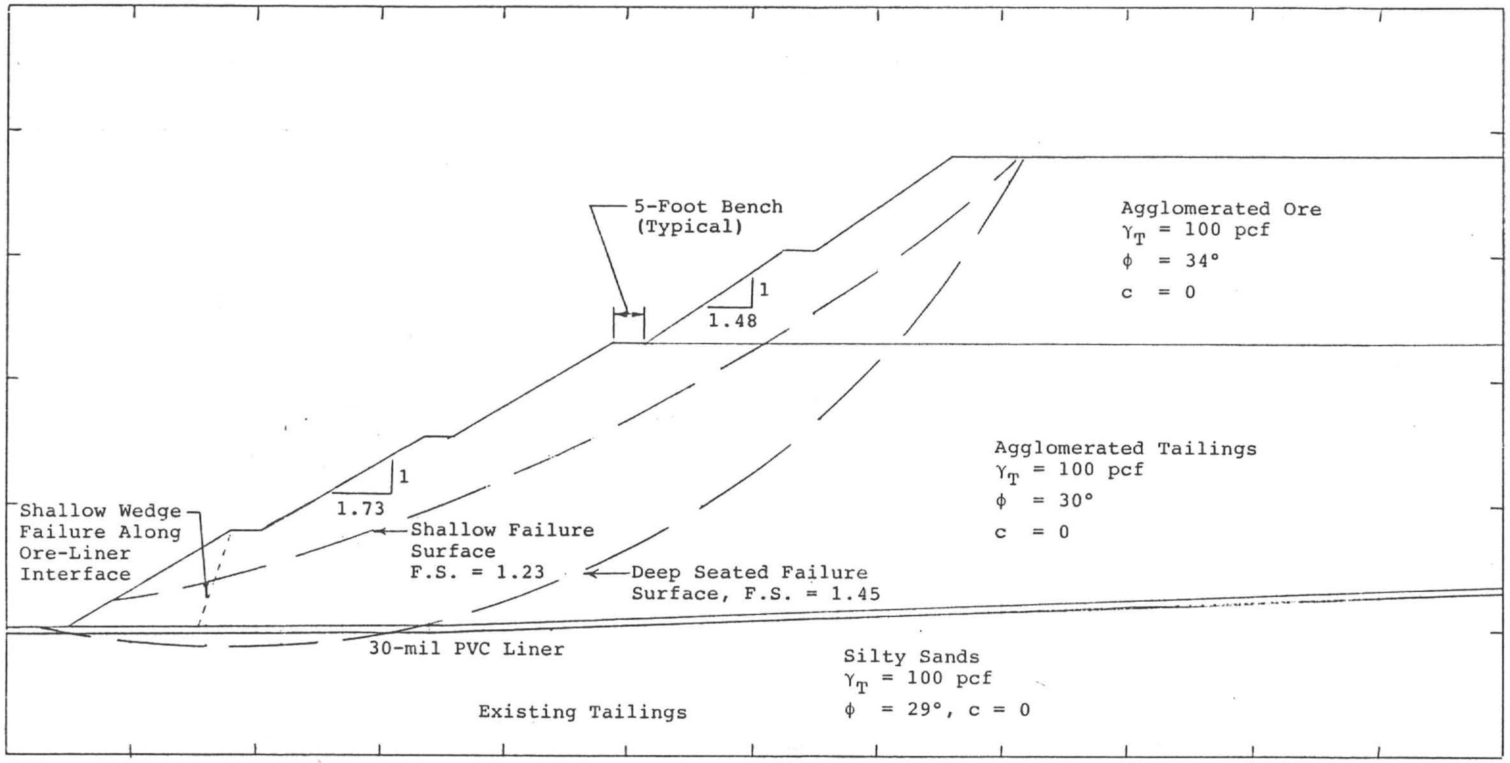
6.3.2 Potential Toe Failures

Potential toe failures along the liner-subgrade interface, as shown in Figure 3, were analyzed by a method presented by Blight (1981), which covers the case of a frictional foundation.

An important parameter required for this analysis is the estimated friction angle along the geomembrane-ore interface. Large-scale direct shear tests for various geomembrane cover material interfaces were performed by SHB as part of a special laboratory testing program for three previous leach pad designs (Deatherage, Fahy and Hansen, 1986). Resulting friction angles varied from 8.5 to 39 degrees for the PVC liner material tested, which were a function of cover soil and applied normal stress. Friction angles for mine tailings placed on PVC ranged from 12 to 13.5 degrees for the lower normal stresses.

FIGURE 3

Maximum Heap Leach Pile Cross Section for Stability Analyses



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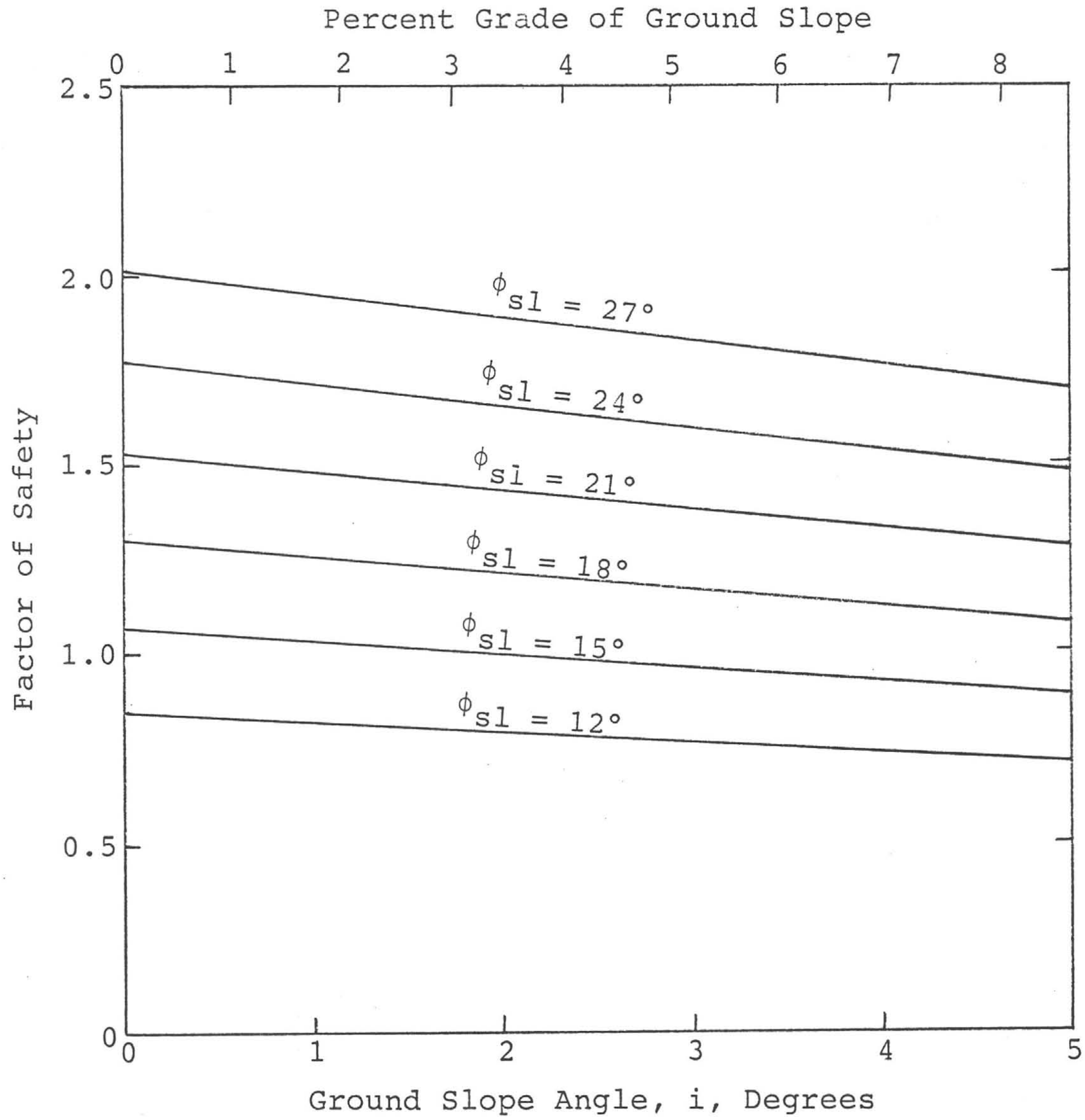
A parametric study showing the relationship between foundation slopes (i), foundation friction angle (ϕ_f), and factor of safety is presented in Figure 4. Considering a shallow wedge-type failure within the 75-foot wide pad toe corridor, which is designed at a 1 percent slope, the factor of safety for a PVC-tailings interface friction angle (ϕ_{sl}) of 12 degrees is about 0.8. To increase the friction angle at the toe of the pad and, correspondingly, the factor of safety, a 1-foot thick layer of sand and gravel will be placed along a 50-foot wide corridor at the toe of the pile. The sand and gravel layer will increase frictional response along the liner surface. The estimated soil-liner friction angle for these materials is about 17 to 18 degrees. The resulting safety factor against shallow block failure is about 1.2.

6.3.3 Generalized Failure Surface Analyses

Shallow and deep-seated failure surfaces within the ore pile were analyzed using the computer program STABL (Siegel, 1975a, 1975b). This program considers a generalized shear surface utilizing a limiting equilibrium method of slices. Noncircular trial shear surfaces are generated and analyzed to determine the critical shear surface. Both static and pseudostatic analyses are possible with this program.

FIGURE 4

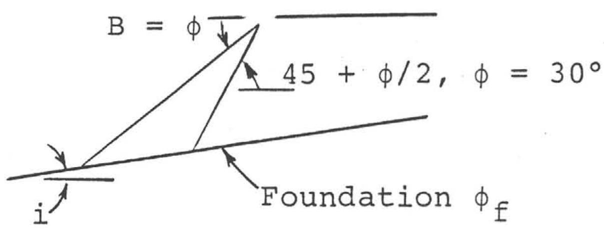
Ground Slope Angle Versus Factor of Safety for Varying Liner-Soil Friction Angles



$$F.S. = \frac{\tan \phi_{sl}}{\tan \phi_f}$$

ϕ_{sl} = Friction Angle of Soil-Liner Interface

ϕ_f = Friction Angle of Foundation for Limiting Equilibrium State



Heap Leach Facility Design
Vulture Mine Project
Near Wickenburg, Arizona
SHB Job No. E87-11

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In the analysis procedure, parallel side shear forces are assumed, vertical equilibrium of the individual slices is satisfied, and overall moment equilibrium is satisfied, but moment equilibrium of the individual slices is not satisfied. The computed factor of safety is conservative relative to more accurate methods satisfying complete equilibrium.

As shown on Figure 2, for the 1.9:1 (28°) equivalent slope and internal friction angles of 30° and 34° for the tailings and crushed ore, respectively, a factor of safety of 1.2 was determined for shallow failure surfaces. As previously stated, for ore placed at the angle of repose the factor of safety approaches 1.0 near the slope face. Deeper seated failure surfaces have higher factors of safety on the order of 1.4 to 1.5.

6.3.4 Deformation & Settlement

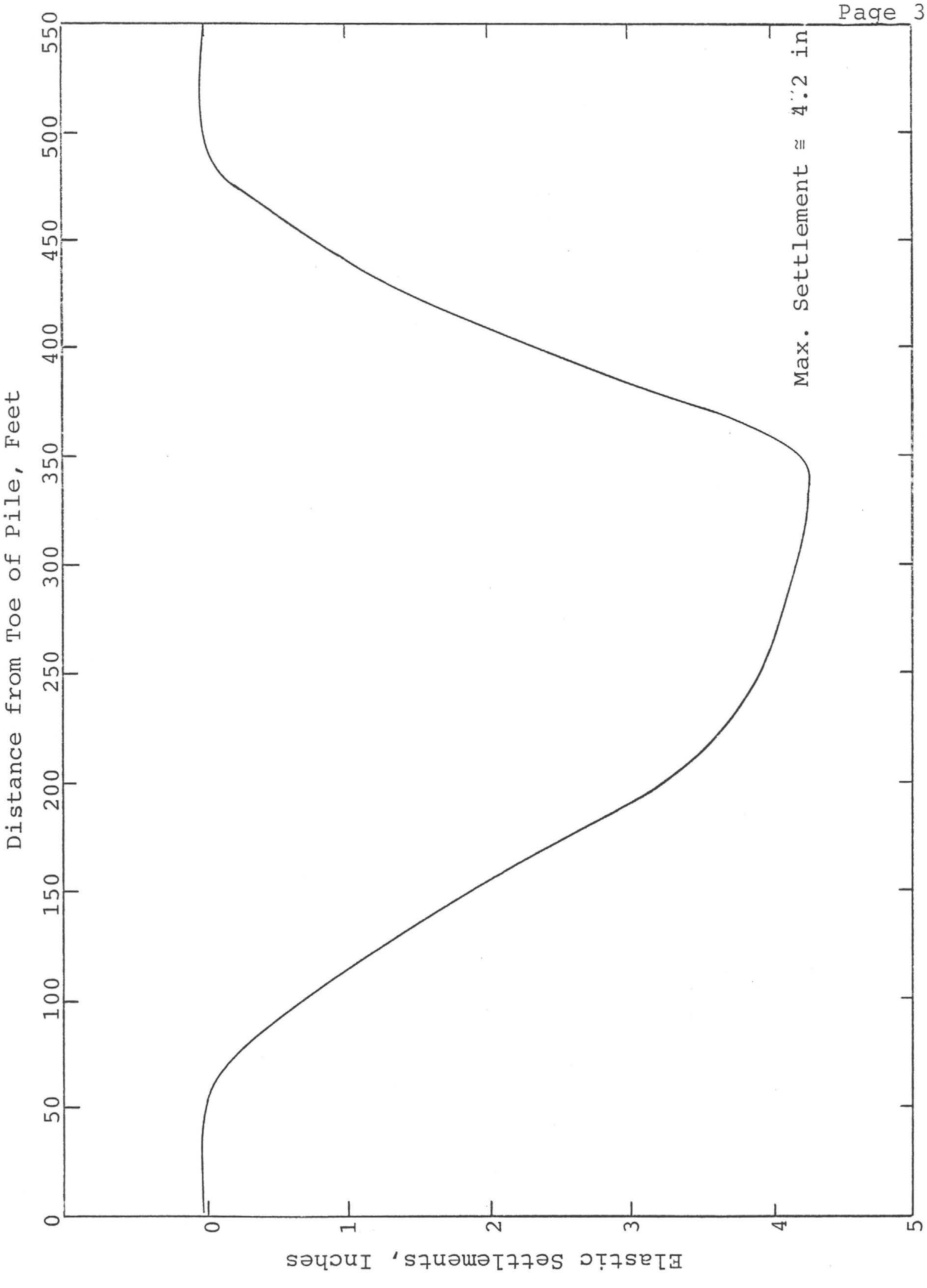
6.3.4.1 Settlement Analysis

Settlement analyses were based on elastic theory settlement relationships developed by Harr (1966) for loading on a rectangular area. The thickness of compressible foundation material was estimated to vary from 13 to 34 feet below the pile. A two-layer substrata was analyzed. A conservative equivalent deformation modulus for the two-strata profile of 2 ksi was utilized. Equivalent moduli were determined using the relationship developed by Thenn de Barros (1966). The underlying Precambrian metamorphic/igneous bedrock complex was assumed incompressible. The increase in stress on the foundation material was estimated using relationships developed by Osterberg (1957). The estimated settlement profile is presented in Figure 5. A maximum elastic settlement of about 4 inches was estimated.

Consolidation tests performed on relatively undisturbed samples of the surficial tailings deposits indicate collapse potential of between 3 and 6 percent of the stratum thickness upon saturation. Saturation or even significant moisture increase in the subgrade tailings is not anticipated provided proper liner installation techniques are followed. Should significant solution leakage into the underlying tailings occur during the project life, additional settlements of between 5 and 8 inches are possible in the pad areas underlain by the thickest deposit of tailings. The PVC liner in the pad area can withstand elongations of up to 300 percent prior to break. Liner rupture should not occur even if differential settlements of 12 inches occur across the width of the pad.

FIGURE 5

Estimated Elastic Settlement of Foundation Materials for Maximum Heap Loading



Heap Leach Facility Design
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