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REPORT OF ENGINEERING DESIGN STUDY
ADDITIONS TO TAILINGS POND-EMBANKMENT SYSTEM
MOAB, UTAH
FOR ATLAS MINERALS

Dames & Moore Job No. 05467-018-06
Salt Lake City, Utah
February 15, 1978

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February 15, 1978

Atlas Minerals
Post Office Box 48
Moab, Utah 84532

Attention: Mr. Richard Adrian

Gentlemen:

Transmitted herewith are 10 copies of our report entitled, "Report of Engineering Design Study, Additions to Tailings Pond-Embankment System, Moab, Utah, For Atlas Minerals."

We appreciate the opportunity of performing this study for you. If you have any questions or require additional information, please contact us.

Yours very truly,

DAMES & MOORE

George C. Toland

George C. Toland
Partner

James R. Boddy

James R. Boddy
Senior Engineer

GCT/JRB/pc

Attachments

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REPORT OF ENGINEERING DESIGN STUDY
ADDITIONS TO TAILINGS POND-EMBANKMENT SYSTEM
MOAB, UTAH
FOR ATLAS MINERALS

INTRODUCTION

This report presents the results of our detailed engineering studies for the design and construction of the proposed additions to the tailings pond-embankment system near Moab, Utah, for Atlas Minerals. The location with respect to major roadways, the Colorado River, the City of Moab, and topographic features, is presented on Plate 1, Vicinity Map. A more detailed layout of the pond-embankment system including the locations of borings, test pits, ground water monitoring wells, and typical cross-sections related to this study is presented on Plate 2, Plot Plan.

The purpose of this study was to evaluate the subsurface conditions and the existing embankment system, and perform detailed design analyses for a series of three increases to the overall height of the pond and embankment system. The design study included the evaluation of the existing tailings pond embankment stability as well as the effects of the future additions on the overall stability. It is our understanding that the design study will be submitted to the Nuclear Regulatory Commission for review.

During the course of our studies, general methods and design concepts were discussed between Mr. William Badger of Atlas Minerals and Messrs. George Toland and James Boddy of Dames & Moore. Results of field exploration and laboratory testing were evaluated in order to derive subsurface conditions and engineering parameters for the detailed analyses. The results of the field exploration and laboratory testing programs performed in conjunction with this study, as well as those performed during other studies at the site, are presented in Appendix A of this report. A list of references is presented in Appendix C.

Specifically, the scope of our studies includes:

1. A review of available subsurface information and laboratory testing data obtained from previous investigations;

2. A field investigation program consisting of exploratory borings, test pits, sampling, and piezometer installation;

3. A detailed program of laboratory testing;

4. The development of refined subsurface and ground water conditions;

5. The development of refined engineering parameters and analytical procedures;

6. An evaluation of required storage capacity of the tailings pond;

7. The development of design concepts, construction methods, and configurations of the future tailings pond-embankment system;

8. An evaluation of the slope stability of the existing and the future tailings pond-embankment system;

9. The evaluation of liquefaction potential of the impoundment system;

10. The estimation of required earthwork quantities;

11. The recommendation of earthwork procedures and construction criteria; and

12. The recommendation of post-construction monitoring programs.

In the order of presentation, the following sections of this report contain topics covering:

- o the site conditions,
- o the existing tailings pond-embankment system,
- o the stability of existing embankments,
- o the design and evaluation of the proposed embankment additions,
- o the evaluation of liquefaction potential, and
- o the recommendation of post-construction monitoring.

SITE CONDITIONS

GEOLOGY

The site is situated upon the collapsed crest of the Moab Valley-Spanish Valley salt anticline within the Canyonlands section of the stable Colorado Plateau physiographic province. The northwest trending anticline is downfaulted at the crest and forms a graben structure and topographic valley. It is believed that the collapse of the anticline began in late Cretaceous or early Tertiary period as a result of solutioning and extrusion of salt upward through the fault zones.

Rock exposed in the site vicinity consists of sedimentary strata ranging from Pennsylvanian to Jurassic in age. The Moenkopi formation and the Hermosa formation outcrop directly to the west and northwest of the tailings pond. The bedrock is cut by several northwest trending, normal faults downthrown on the northeastern side. Stream-deposited sand and gravels, slope wash, and wind blown sand of Quaternary age cover the valley floor at the Atlas Minerals plant and tailings pond site.

SEISMICITY

The Colorado Plateau structural province has experienced relatively little seismic activity, and is considered a seismically stable region. Significant damaging earthquakes have occurred only at relatively large distances from the site. The nearest known earthquake epicenter is located 43 miles northwest of the site and was the location of an event of Intensity VI or less, based on the Modified Mercalli Intensity Scale. The high plateau of Utah, located 95 miles west of the site, and the Wasatch Fault zone, the nearest point located near Richfield, Utah, 137 miles to the west, have been the locations for maximum events of Intensities VI and IX, respectively. A detailed discussion of all known seismic events, both natural and manmade, which have occurred within a 200 mile radius of the site, are discussed in the Safety Analysis Report (SAR) (Dames & Moore 1975). The en echelon faults near the site are not associated with seismic activity.

Based on historic record, it has been determined in the SAR that a ground acceleration of .02g could be experienced at the site. A conservative ground acceleration of .05g is recommended for design purposes.

SURFACE CONDITIONS

The existing tailings pond system is located directly southwest of the uranium processing mill. The southeast side of the site is bordered by the Colorado River and its flood plain. The Colorado River lies approximately 800 feet east-southeast of the tailings pond embankment tce. Between the river and the embankment tow, soft and loose surface soils of silt and silty sand exist. The Moab Wash lies between the mill and the tailings pond and extends from the Colorado River northwest to the north side of the site where it parallels U. S. Highway 160 in a westerly direction. A 12-inch gas pipeline roughly parallels the same highway. To the west, southwest and northwest of the site exist roughly formed cliffs rising abruptly approximately 800-feet. The stratigraphic sequence of the Kayenta and Hermosa formation is exposed in the cliff walls. The Keyenta formation caps the Hermosa formation, the latter only sometimes is exposed at the cliff base. Utah State Highway 279 and a Utah Power and Light right-of-way lie between the base of these cliffs and the tailings pond.

The vegetation at the site consists of sparse sagebrush adjacent to the Colorado River. The red sand desert ground surface slopes downward from the base of the cliffs to the east and southeast at about three percent and flattens to about one percent near the Colorado River.

SUBSURFACE CONDITIONS

Soils

The subsurface soil conditions at the site were evaluated by the performance of several field exploration programs which consisted of drilling, sampling, and logging of exploratory borings and test pits. A description of the field exploration programs are presented in Appendix A of this report.

Subsurface soils at the site consist of a combination of

- o slope wash from nearby cliffs,
- o soils deposited by the Moab Wash,
- o wind blown sands, and
- o alluvial soils deposited by the Colorado River.

None of the borings drilled in the site vicinity during this investigation or others has penetrated the underlying bedrock formations. Further, none of the borings conducted for this report has penetrated over 20 feet into the natural soils. However, several of the borings from previous studies have extended up to 70 feet into the natural alluvial soils. Generally, the soils consist of predominantly reddish brown sand with varying amounts of silt and gravels. The gradation of the sand fraction is highly variable, and cobbles and boulders are known to be sometimes present. The natural soils are generally loose to medium dense at the surface, becoming denser with depth. Some localized cementation of the soils was noted in several of the borings. Due to the similar composition of all the subsurface soils encountered, it was not possible to distinguish the mode of deposition nor to define a layering sequence.

Ground Water

Detailed descriptions of the regional ground water conditions including present and anticipated future uses were detailed in the SAR. Briefly, the natural ground water gradient at the tailings pond site slopes southeasterly toward the Colorado

River at a rate of ten feet per mile. Ground water conditions within the tailings pond embankments are described in the following sections.

EXISTING TAILINGS POND EMBANKMENT SYSTEM

GENERAL

The layout of the tailings pond embankment system along with the location of borings and test pits conducted in relation to this and other investigations are shown on Plate 2, Plot Plan.

The existing tailings pond system was begun in 1956 with the construction of an earth starter dike along the eastern and southeastern limits of the pond. The starter dike was later extended westerly from the south and north ends. Supplemental tailings dikes were added intermittently to increase the capacity of the pond system. These starter dikes now comprise the toe of the existing embankment on the north, east, and south sides of the tailings pond. A permanent western dike has recently been completed and now encloses the embankment-pond system which has a current surface area within the uppermost dikes of approximately 83 acres. The tailings dikes now stand at a relatively uniform crest elevation of 4,040 feet and contain a total of approximately eight million tons of tailings.

The tailings discharge line, which had been in use until recently, discharged tailings from the crest of the entire pond embankment with the exception of that along the western embankment area. During January, 1978 construction of a new discharge line was completed, now enabling tailings to be discharged around the entire perimeter of the pond.

Following the settling of the tailings from the slurry, the liquor is decanted by a floating barge located near the center of the water pond and returned to the mill for treatment and reuse. Three emergency underdrain type decant lines are maintained and available for use should the present system fail.

EMBANKMENTS

Main Embankments

The existing main embankment sections are defined as those embankments on the north, east, and south sides of the tailings ponds and having limits depicted on Plate 2. The embankment material consists of a compacted starter dike of natural alluvial fill upon which a series of supplemental sand tailings dikes have been constructed by upstream methods to the present crest elevation of 4,040 feet. The upstream type of construction has resulted in an overall slope of 2.4 horizontal to 1.0 vertical. However, localized areas generally of heights less than 20 feet have slopes as steep as 1.4 horizontal to 1.0 vertical. The existing supplemental sand tailings dikes above the starter dike are constructed directly upon the hydraulically deposited beach tailings. As determined from our boring logs, this includes sand tailings and, in some areas, layers of slime tailings.

According to previous construction plans, the starter dikes were to be constructed utilizing engineered fill, having an 18-foot wide crest and sideslopes of 2.0 horizontal to 1.0 vertical. Observation of samples taken from the borings drilled near the toe of the main embankment sections show the material in the starter dikes to consist of a predominantly medium-dense to dense reddish brown silty fine sand, typical of the natural surface soils in the surrounding area. Occasionally, gravels, cobbles, and boulders obtained from nearby broken siltstone rock were also found in the starter dikes.

The supplemental dikes above the starter dam were constructed of sandy tailings pushed up from the tailings pond beach area by heavy construction equipment. A one-foot thick veneer of silt-, sand-, gravel-, and cobble-sized red siltstone fragments have been placed upon the downstream face of the embankment as an erosion-protection covering. The resultant appearance is that of an embankment constructed of a uniform red, rocky, natural material. The average width of the present embankment crest is approximately 20 feet. Sections A-A, B-B, and E-E are presented on Plates 3A, 3B, and 3D as those sections showing typical conditions along the main embankment area. The plates include the subsurface soil conditions and highest ground water levels encountered within the borings and piezometers placed along each section.

Western Embankment

Prior to the recently completed remedial construction work, the western embankment consisted of a dike constructed of on-site red silty sand material mixed with sand tailings and having an 18-foot crest width and sideslopes of approximately 1.5 horizontal to 1.0 vertical. The embankment crest elevation ranged from elevation 4,034.5 to 4,041.5 feet. With the construction of the permanent remedial measures, the western dike was raised to a uniform crest elevation of 4,040 feet (Dames & Moore, 1977). The remedial measures included the construction of a downstream embankment designed to allow water to pond against the embankment's upstream face. Such an embankment was necessary because water and unconsolidated slimes tailings exist along the upstream face of the original embankment. The present configuration along the western embankment includes a 20-foot wide embankment of compacted natural red silty sand and a sand tailings chimney filter and blanket drain with a toe drain of gravel. A layer of rip-rap gravel covers the upstream slope. Sections C-C and D-D shown on Plate 3C, along with descriptions of nearby boring soil profiles and ground water conditions, are considered typical of the newly constructed western embankment.

PONDED WATER

The present requirement for the ponded liquid within the tailings pond is to maintain the shoreline a distance of at least 150 feet behind and at least six feet below the main embankment crest on the north, east, and south sides of the ponds (Badger, 1977b). The beach is formed by uniform hydraulic deposition of tailings along the perimeter of the embankment. At the western embankment, where tailings are not discharged, the water is ponded directly against the embankment; a minimum of six feet of free board is maintained. The present plans calling for construction of a new discharge line along the western embankment will enable a beach to be formed and thus will eliminate the ponding of water directly against the embankment.

As observed in the borings and piezometers, the highest phreatic surface result from seepage of the ponded water is well below the downstream slope surface of the embankment. The highest recorded water levels for each boring are shown on Plates 3A through 3D. Generally, water in the discharged tailings seeps rapidly through the permeable beach sand tailings on a near vertical gradient to the underlying slimes tailings. The water then follows the top of the less permeable slimes layer (if present) and exits into more permeable sandy materials near the embankment toe. In two areas, one along the northern embankment just above the starter dam (near Section E-E) and one along the toe of the western embankment (near Section C-C), surface seepage had been observed. The seepage at the two locations is believed to be due to perched water exiting upon localized slimes layers known to be present at these locations. A third seepage area was detected recently at the extreme south corner of the pond at the toe of the embankment. The estimated seepage was approximately two or three gallons per minute. Prior to the detection of the seep, tailings slurry had been discharged over a period of about two weeks. After the discharge of slurry tailings ceased in the area, the amount of seepage decreased until after a week it was no longer detected.

Seepage of the ponded water through the western embankment is collected through a chimney drain and drainage blanket system (Plate 3C), which was designed and constructed in the recently completed western embankment. The collected water is pumped back to the retention pond for recycling. Seepage water that has emerged on the lower portion of the northern embankment near Section E-E is being collected through a subdrain system which has recently been installed. Where the seep was noted at the south corner of the tailings pond, it is planned to install a subdrain and collection system similar to that placed at Section E-E. The details of the north subdrain are shown on Plate 4.

DISCHARGE SYSTEM

Tailings from the mill process are discharged from a pipeline located along the entire perimeter of the tailings pond. The pipeline consists of a main line which junctions at the pond to two 8-inch diameter high density polyethylene pipes each of which lies along the embankment crest for a distance of one-half the total perimeter of the pond. Spigots are placed approximately 100 feet apart and the normal discharge is such that four spigots are in operation at any one time. Up until recently, approximately 700 gallons per minute of 13 percent solids-by-weight slurry had been discharged along the pipeline, which is equivalent to about 600 tons per day (dry) of tailings. On February 9, 1978 an increase in plant operations went into effect such that 22 percent solids-by-weight slurry is being discharged, increasing the dry tailings rate to about 1200 tons per day. The location of discharge points are controlled by Atlas personnel in an effort to maintain the widest possible sand beach.

DECANT SYSTEM

The discharged slurry flows over the tailings beach where the coarse fraction (sand tailings) of the tailings grind is deposited. The fine fraction (slimes tailings) of the slurry is carried to the pond where it settles out. The clear liquid is then siphoned off via a floating decant pump and pipeline. The pond water is returned to the mill area for treatment.

In addition to the floating decant system, an underdrain decant system consisting of three decant towers and a buried pipeline is located in the pond. The additional system is maintained as an emergency backup should the main floating system fail to operate. However, the system has not been operated for some time and its present condition is not known.

PROPOSED EMBANKMENT ADDITIONS

GENERAL

The tailings impoundment area at Atlas Minerals' Moab operation will soon be completely filled. Estimates indicate that, allowing for a six-foot freeboard, the remaining usable capacity of 200 acre-feet will be depleted by July, 1978. The proposed additions to the existing tailings containment area are described below. Subsequent sections to this report, along with the appendices, present our methods of evaluation.

It is proposed that the embankments of the existing tailings pond be raised to facilitate disposal of tailings to be produced during the next twelve years of plant operation. The additional tailings storage capacity is to be achieved through three separate embankment raises of twelve feet each, for a total embankment height increase of 36 feet. The additional combined storage capacity resulting from the three raises is to be approximately 3,100 acre-feet (approximately six million tons of tailings).

It is proposed that the western embankment of the tailings pond, having recently been modified by a design allowing water to temporarily rest against the embankment's upstream slope, be raised using downstream construction methods. However, the proposed system along the western embankment is designed such that a minimum distance of 150 feet from embankment crest to the edge-of-ponded water is to be observed.

The remaining embankment, along the east, north and south borders, is to be raised by the upstream method of construction. An overall downstream slope of 3.25 horizontal to one vertical is to be used for the upstream construction.

TIME-STORAGE REQUIREMENTS

The storage capacity required for tailings disposal is a function of the amount of ore processed. In most mineral processing cases, the amount of tailings produced is equal to the amount of ore processed less the amount of concentrates obtained. However, due to the chemical reactions of the sulfuric acid used in the acid leach uranium extraction process, precipitates (predominantly calcium sulfate) add to the mass of the tailings. The result is that in some cases more tailings are produced than ore is processed. Operation data from Atlas Minerals indicate that for each ton of ore processed, from 1.1 to 1.2 tons of tailings are produced (Atlas Minerals, 1976).

The present 600-700 dry tons per day (DTPD) capacity of the mill will be increased in two stages to an ultimate capacity of 1,320 DTPD (Adrian, 1978). The schedule for the increase in plant capacity is as follows:

July, 1977	to February, 1978	600-700 DPTD
February, 1978	to June, 1978	1,100 DTPD
June, 1978	to Remaining Life	1,320 DTPD

These rates are tentative and assume 360 operating days per year. The design operational life of the mill is twelve years beyond July, 1977 (Badger 1977a). The storage requirement for the tailings produced during these next twelve years is estimated to be 3,280 acre-feet (or approximately 6.7 million dry tons of tailings). Tabular and graphic representations of the tailings storage requirements over the remaining life of the mill are presented in Plate 5 and shown in the following table:

TABLE 1

TAILINGS PRODUCTION AND STORAGE REQUIREMENTS
FOR ATLAS MINERALS, MOAB, UTAH MILL
FROM JULY, 1977

Time Period	Weight (Dry Tons)	Volume* (ac-ft)
1st year	413,000	200
2nd thru 12th year	570,000 per year	280 per year
Total for 12 years	6,683,000	3,280

*Note: The volume of water associated with the tailings is not considered. The water entrained by the tailings will occupy the pore space of the solids. Non-entrained water will be dissipated through evaporation, seepage and recycling. A steady-state volume of ponded water equal to its present size is assumed. The average dry density of in-place tailings is 94 pounds per cubic foot (Dames & Moore, 1977a).

The operation of the tailings pond calls for a minimum freeboard of six vertical feet from the dam crest to the ponded-water surface and for a minimum distance of 150 horizontal feet from the dam crest to the edge of the ponded water (Badger, 1977b). Hence, the present embankment crest elevation of 4,040 feet allows for the ponded-water surface elevation not to exceed 4,034 feet. As of July 12, 1977 (most recent survey data), the ponded-water surface elevation was 4,031.5 feet. These circumstances limit the remaining storage capacity to 170 acre-feet, as derived from Plate 6. However, the technique used for discharging the tailings into the tailings pond results in a beach wedge of tailings sands between the dam embankment and the ponded water. Since the tailings are discharged as a slurry from the dam embankment on the periphery of the tailings pond, the tailings slurry flows toward the ponded water at the center of the pond. The heavier, coarser tailings sands drop out of suspension first and the lighter, finer slimes are carried to the ponded water area where they settle out of suspension. The result of this natural sizing system is a peripheral beach wedge of tailings sands and a central area of tailings slimes and ponded water. The portion of the beach wedge which is deposited above the ponded water surface will allow for an additional 30 acre-feet of storage capacity. Thus, the remaining total capacity of the existing impoundment system as of July, 1977 is estimated at 200 acre-feet. Since the anticipated tailings production for the year July, 1977 through June, 1978 is also 200 acre-feet, the estimated present capacity of the impoundment will be depleted by July, 1978.

DESIGN EVALUATION AND RECOMMENDATIONS

STABILITY OF TAILINGS POND EMBANKMENT

General

The stability of the tailings pond embankments was analyzed and it was found that the additional height increases can be constructed by the upstream method as proposed with the embankments having factors of safety in excess of those required by the NRC Regulatory Guide 3.11. The subsequent sub-sections included in the stability section of the report discuss the method used and present the results of the analysis. Further, the evaluation of the engineering properties used in the analysis is presented in Appendix B to this report.

Critical Embankment Section

Stability analyses were performed on three embankment cross sections which were considered to be representative of the most critical sections encountered along the entire tailings pond perimeter. The sections analyzed, titled Sections A-A, C-C, and E-E, are presented on Plates 3A, 3B, and 3D; each section being chosen for special consideration. Section A-A represents the highest section; the overall slope and the location of slimes materials with the section is considered typical of the general embankment condition. Section C-C represents a section of the western embankment considered critical because of the high internal water levels and the excessive amounts of slimes material beneath the embankment. Section E-E represents a section depicting critical conditions due to high internal water levels and high amounts of slimes materials within an embankment section along the main embankment area (embankments along the north, east and south bounds of the pond area). On Plates 7, 8A through 8F, and 9A through 9E, the sections are presented in a general form as used in the actual analysis. Conservative assumptions were made in interpreting the soil and water conditions. Shown on the plates are the limits of the various soil and water conditions, the soil strength parameters assumed, and critical failure circles along with the factors of safety against failure occurring for each of the circles.

Method of Analysis

Soil parameters used in our analysis were based on the laboratory results presented in Appendix A. Conservative estimates of strength properties were made by evaluating the laboratory results in the manner as presented in Appendix B - Evaluation of Engineering Properties. The soil parameters assumed for the analysis are presented in Table 2 on the following page.

For each of the embankment sections selected for slope stability analysis, the embankment of existing configuration and of proposed configuration were evaluated under static and seismic considerations for steady-state seepage and end-of-construction loading conditions.

Slope stability was calculated by the Simplified Bishop method, using a computer and the certified Dames & Moore program EP-5A. The Dames & Moore Modified Bishop computer analysis for slope stability has received official general certification for use in USNRC projects. Hand calculation checks developed during certification work have previously been given to the USNRC.

The Simplified Bishop technique assumes a circular failure surface, and that the soil fails as a series of rigid-body segments. The effects of internal deformation within the soil mass are neglected. In considering any given circular surface, the driving and resisting forces associated with failure are calculated. The factor of safety for any circular surface is

TABLE 2. SLOPE STABILITY ANALYSIS -

CONDITIONS ANALYZED

AND

SHEAR STRENGTH PARAMETERS USED

SECTION	LOADING CONDITIONS	END-OF-CONSTRUCTION STRENGTH SLIMES	END-OF-CONSTRUCTION STRENGTH SAND TAILINGS	STEADY-STATE-SEEPAGE STRENGTH SLIMES	STEADY-STATE-SEEPAGE STRENGTH SAND TAILINGS
SECTION E-E:					
Existing Embankment-Downstream	Static	--	--	$\phi = 31^\circ$	$\phi = 37^\circ$
	Seismic	--	--	$\alpha = 21^\circ$	$\phi = 37^\circ$
First Additional Raise-Upstream	Static	α* = 21°	$\phi = 37^\circ$	--	--
	Seismic	$\alpha = 21^\circ$	$\phi = 37^\circ$	--	--
Downstream	Static	$\alpha = 21^\circ$	$\phi = 37^\circ$	$\phi = 31^\circ$	$\phi = 37^\circ$
	Seismic	$\alpha = 21^\circ$	$\phi = 37^\circ$	$\alpha = 21^\circ$	$\phi = 37^\circ$
Third Additional Raise-Upstream	Static	$\alpha = 21^\circ$	$\phi = 37^\circ$	--	--
	Seismic	$\alpha = 21^\circ$	$\phi = 37^\circ$	--	--
Downstream	Static	$\alpha = 21^\circ$	$\phi = 37^\circ$	$\phi = 31^\circ$	$\phi = 37^\circ$
	Seismic	$\alpha = 21^\circ$	$\phi = 37^\circ$	$\alpha = 21^\circ$	$\phi = 37^\circ$
SECTION A-A:					
Third Additional Raise-Downstream	Static	$\alpha = 21^\circ$	$\phi = 37^\circ$	$\phi = 31^\circ$	$\phi = 37^\circ$
	Seismic	$\alpha = 21^\circ$	$\phi = 37^\circ$	$\alpha = 21^\circ$	$\phi = 37^\circ$
SECTION C-C:					
Existing Embankment-Upstream	Static	c = 1300	--	$\phi = 31^\circ$	--
	Seismic	c = 1300	--	--	--
Downstream	Static	c = 1300	--	$\phi = 31^\circ$	--
	Seismic	c = 1300	--	c = 1300	--
First Additional Raise-Upstream	Static	--	--	--	--
	Seismic	c = 1300	--	--	--
Downstream	Static	c = 1300	$\phi = 37^\circ$	$\phi = 31^\circ$	$\phi = 37^\circ$
	Seismic	c = 1300	$\phi = 37^\circ$	c = 1300	$\phi = 37^\circ$
Third Additional Raise-Upstream	Static	c = 1300	$\phi = 37^\circ$	--	--
	Seismic	c = 1300	$\phi = 37^\circ$	--	--
Downstream	Static	c = 1300	$\phi = 37^\circ$	$\phi = 31^\circ$	$\phi = 37^\circ$
	Seismic	c = 1300	$\phi = 37^\circ$	c = 1300	$\phi = 37^\circ$

* The angle α represents the relation describing the ration $Su/\bar{\sigma}_C$ (undrained shear strength/effective overburden stress) by the following: $\alpha = \tan^{-1} Su/\bar{\sigma}_c$.

essentially the summation of the resisting forces divided by the summation of the driving forces.

Seismic forces are represented by the application of a horizontal force, equal to the assumed percent gravity acceleration, to the weight of the soil within the failure circle. As discussed in the section entitled SEISMICITY, a design horizontal acceleration of 0.05g was used in the analyses.

In addition to the Simplfied Bishop technique, evaluations were made regarding the factor of safety of the embankment material against failure by the use of the infinite slope failure theory in sands. This simple method approach may be used when considering the potential against a failure occurring in the form of sloughing within a shallow zone of near-surface materials. The formula used for the analysis is found in most soil mechanics textbooks (Lamb and Whitman, 1969). For high embankment slopes consisting primarily of cohesionless sand, infinite slope theory was applied. However, the consequences of such minor sloughing, if it were to occur, would be insignificant.

Stability Results

Summary - The geometry, soil parameters, ground water conditions used in our analysis, and resulting stability factors of safety are shown graphically on Plates 7, 8A through 8F, and 9A through 9E. The results of our analyses are summarized on Tables 3, 4 and 5. The factors of safety meet regulatory guidelines presented in USNRC Regulatory Guide 3.11, December, 1977.

Existing Embankments - The minimum factor of safety determined represents the degree of stability of the slope for each given loading condition. A brief discussion of the factor of safety for each of the three embankment sections analyzed based on existing conditions is presented as follows:

Section A-A - No independent analysis was performed for Section A-A under its existing conditions. However, the stability of the existing embankment may be inferred from the results of stability analyses performed on Section A-A after the construction of the third additional supplemental dike (Table 3 and Plate 7).

The minimum factors of safety against failure in the existing portion of the slope under steady-state seepage are 1.62 and 1.40, respectively, for static and seismic conditions. The minimum factors of safety have a failure circle which is entirely in the sand tailings portion of the embankment. For potential failures occurring through the natural red sandy soils, the minimum factors of safety are 1.65 and 1.43, respectively, under static and seismic loading conditions during steady-state seepage. These factors of safety are amply adequate against the possibility of a deep-seated failure and are considered a conservative representation of the factors of safety in the existing embankment since the phreatic surface assumed was for conditions of future embankment expansion.

Section C-C - For the recently completed western embankment represented by Section C-C (Plate 3C) the absolute minimum factor of safety at the end of construction was estimated under static loading and under earthquake loading conditions and is presented on Table 4 and in Plates 8A and 8B. The critical surface corresponding to these minimum factors of safety occur through the slimes material within the downstream slope. In time, as the steady-state seepage begins to take place and as the slimes begin to consolidate under the embankment weight, the factor of safety against failure through the slimes materials will increase from those found under end-of-construction conditions.

Section E-E - In Section E-E, as shown in Table 5 and on Plate 9A, the minimum factor of safety against failure under steady-state seepage, static loading conditions is 1.60; whereas under earthquake loadings, the minimum factor of safety is 1.19. The magnitudes of these factors of safety indicate that the existing embankment, as represented by Section E-E, possesses adequate capacity against a deep-seated failure.

The above results and discussions were related to the possibility of deep-seated failures. Shallow failures of a

TABLE 3. SUMMARY OF RESULTS OF SLOPE STABILITY ANALYSIS - MINIMUM FACTOR OF SAFETY, DOWNSTREAM SLOPE, SECTION A-A[a]

SECTION A-A, DOWNSTREAM SLOPE	STATIC	SEISMIC
End-of-Construction Conditions:		
Third Additional Raise -		
Existing Portion	--	--
Total Slope	1.41	1.19
Steady-State Seepage Conditions:		
Third Additional Raise -		
Existing Portion	1.62	1.40
Total Slope	>1.82	1.17

[a] For design soil parameters see Table 2.

TABLE 4. SUMMARY OF RESULTS OF SLOPE STABILITY ANALYSIS - MINIMUM FACTOR OF SAFETY, SECTION C-C[a]

SECTION C-C	END-OF-CONSTRUCTION		STEADY-STATE-SEEPAGE	
	STATIC	SEISMIC	STATIC	SEISMIC
Existing Embankment:				
Upstream	1.45	1.26	1.60	--
Downstream	1.85	1.61	1.85	1.61
First Additional Raise:				
Upstream	1.38	1.26	--	--
Downstream	2.20	1.79	2.21	1.81
Third Additional Raise:				
Upstream	1.38	1.23	--	--
Downstream	1.55	1.37	1.55	1.37

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[a] For design soil parameters see Table 2.

TABLE 5. SUMMARY OF RESULTS OF SLOPE STABILITY ANALYSIS - MINIMUM FACTOR OF SAFETY, SECTION E-E[a]

SECTION E-E	END-OF-CONSTRUCTION STATIC	END-OF-CONSTRUCTION SEISMIC	STEADY-STATE SEEPAGE STATIC	STEADY-STATE SEEPAGE SEISMIC
Existing Embankment:				
Downstream	--	--	1.60	1.19
First Additional Raise:				
Upstream	1.93	1.70	--	--
Downstream-Existing Portion	1.39	1.18	1.60	1.18
Total Slope	1.47	1.22	1.77	1.30
Third Additional Raise:				
Upstream	1.70	1.51	--	--
Downstream-				
Existing Portion	1.34	1.15	1.58	1.13
Total Slope	1.44	1.20	1.70	1.13

[a] For design soil parameters see Table 2.

relatively thin zone of materials near the surface of the slopes were analyzed for an infinite slope failure. The materials in this zone are mainly recompacted sandy materials such as the natural-occurring red sandy soils and sand tailings. Since the phreatic surface has been and will be maintained well within the embankment slope, seepage was not considered. The factors of safety against shallow failures were calculated to be 1.69 and 1.47, respectively, for static and seismic loading conditions. Therefore, in summary, based on analyses performed on critical sections existing in the tailings pond-embankment system, the existing embankments are adequate against shallow and deep-seated failures under both the static as well as the seismic loading conditions.

Proposed Additions - Considering the most critical geometry for the upstream slopes, the embankment configuration at the end-of-construction condition was assumed to have no new formation of the beach wedge. In such cases, the stability of both the upstream and the downstream slopes were analyzed. The steady-state seepage condition was analyzed at the full impoundment capacity for that particular embankment height which offers the most critical design conditions for the downstream slope. For the condition when the beach wedge exists, the upstream embankment during steady-state seepage is more stable than the end-of-construction evaluation. The factors of safety as shown on Tables 3, 4, and 5 indicate that the embankments with proposed additions, for all loading conditions, are adequate against the possibility of failure and possess values in excess of those required by USNRC.

LIQUEFACTION STUDIES

Introduction

Static and pseudo-static stability analyses, carried out for various representative cross-sections of the tailing embankment to evaluate its static and seismic stability under a variety of conditions, indicate that the factors of safety for the various conditions analyzed are sufficiently high to preclude failure. In these previous studies, however, it has been assumed that the tailing materials would not liquefy during the postulated earthquake loading conditions. The studies presented in the report have been carried out in order to evaluate the liquefaction potential of these tailing materials.

Liquefaction in saturated cohesionless soil deposits has been observed in numerous earthquakes. Based on these field observations and on the results of laboratory and analytical studies, various procedures have been developed to allow correlation of the types of conditions where liquefaction may occur and the soils which are susceptible to liquefaction.

Initially it was felt that preliminary analyses should be performed using the blow count data and relative densities developed during the field exploration program. If the results of these studies showed that liquefaction was highly unlikely then more extensive studies would not be required. For these simplified analyses the maximum earthquake was taken as a magnitude 7 event with a maximum horizontal surface acceleration of 0.05g at the site. This has been shown to be a conservative estimate for the maximum earthquake postulated at the site based on the seismicity of the region (Dames & Moore, 1974). Based on the analyses as presented herein, it is our opinion that if a design earthquake should occur, liquefaction within any portions of the embankment that could effect the stability would be very unlikely.

The simplified Seed and Idriss (Seed and Idriss, 1971) procedure was used to calculate the cyclic stress ratio induced by the postulated design earthquake at various depths for several selected vertical sections within the embankment. The average cyclic stress ratio, $(\tau/\sigma'_o)_{avg}$, induced at any level in a soil deposit is given as:

$$\left(\frac{\tau}{\sigma'_o} \right)_{avg} = 0.65 \frac{h}{\sigma'_o} a_{max} r_d \qquad (1)$$

where,

a_{max} = maximum horizontal ground surface acceleration (g's)

γ = total unit weight

h = depth below ground surface

r_d = depth reduction factor

σ'_o = effective overburden pressure at depth of interest

The depth reduction factor r_d, was taken as the average value given by Seed and Idriss. This factor varies from a value of 1 at the ground surface to 0.68 at a depth of 60 feet. Values of cyclic stress ratio were calculated from the profiles shown on Plate 10.

Once the values of stress ratio have been calculated at various depths it is then necessary to compare these stresses with those required to produce liquefaction for the particular soils under consideration. This may be done in several ways. One approach is to use blow count data obtained from Standard Penetration Tests (SPT) and to compare these data with those measured at sites where liquefaction has or has not been observed to occur during previous earthquakes. The other procedure is based on a comparison of laboratory liquefaction test data performed on similar soils with the computed average induced cyclic stresses. Both of these methods have been used in the study presented herein.

Soil Profiles

Liquefaction analyses were performed for the four basic soil profiles shown on Plate 10, Typical Embankment Section Showing Tailings Profiles Considered in Liquefaction Analyses. As shown on the plate, the soil profiles are at various locations along a typical cross-section of the tailings pond. The density of the tailings material was estimated by averaging the density values obtained by laboratory tests on the undisturbed soil samples. The average dry density of the tailings was found to be 98 pounds per cubic foot. The average wet density above the phreatic line and the average saturated density were found to be 115 and 125 pounds per cubic foot, respectively. For each soil profile, the effective and total overburden stresses were determined and plotted against the depth of tailings material. The plots are shown on Plates 11A through 11D, Overburden Stresses Versus Depth of Tailings.

Penetration Tests

Penetration tests were performed during our field investigation to provide an estimate of the properties of the in-situ soils. The test consisted of counting the number of blows of a falling hammer (weight: 140 pounds; fall: 30 inches) to advance a Dames & Moore U-Type sampler (see Appendix A - Plate A-1) into the undisturbed soil at the bottom of a bore hole. The resulting blow count (number of hammer blows per foot of penetration, N_1) is a measure of the penetration resistance of the soil. Interpretation of blow count data is dependent upon appropriate identification of the types of soil being penetrated. Because the penetration test is performed in the field at the bottom of a drill hole and often under adverse conditions, it is subjected to many possible errors too numerous to discuss in this section. Nevertheless, the penetration test is widely used and offers many advantages to other methods of estimating in-situ properties of soil. However, N-values from penetration tests should be used with caution.

The most widely used penetration test is the Standard Penetration Test performed in accordance with ASTM Standard D1568. This test is similar to the test described above with the exception that the sampler used is a standard 2.0-inch outside diameter split-barrel sampler. The blow counts obtained from the penetration test using the Type-U sampler, N_1, may be converted to standard blow counts, N, by the following approximate correlation (Lacroix and Horn, 1973):

$$N = N_1 * \left(\frac{2\ \text{in}}{D_1}\right)^2 * \frac{12\ \text{in}}{L_1} * \frac{W_1}{140\ \text{lb}} * \frac{H_1}{30\ \text{in}} = \frac{2*N_1*W_1*H_1}{175*D_1^2*L_1}$$

where D_1 = outside diameter of the split spoon or conical point in inches (3.25 inches)

L_1 = depth of penetration in inches (12 inches)

W_1 = weight of hammer in pounds (140 pounds)

H_1 = height of free fall of hammer in inches (30 inches).

The influence of confining pressure on the strength of cohesionless soils results in increasing blow counts with depth below the ground surface for a deposit of constant relative density. The standard N-values should therefore be interpreted in terms of state of stress in the ground. In order to take the aforementioned influence of confining pressure into account, a correction factor should be applied to the standard measured N-value.

The blow counts from the penetration test using the Dames & Moore Type-U sampler, N_1, as measured during our field exploration programs, are shown on Plate 12, Blow Counts - Type-U Sampler Penetration Test Versus Depth. Representative Standard Penetration Test Values, N, as computed from the blow counts obtained from the Type-U sampler penetration tests are shown on Plate 13, Standard Penetration Test Values Versus Depth. Also shown on Plate 13 is a curve showing representative corrected N-values, N', versus depth. The calculated SPT blow counts were corrected using the correction factors proposed by Marcuson and Bieganousky (1977) for materials with relative densities in the range of 40 to 60 percent (see bottom of Plate 13).

Liquefaction Analysis - Empirical Method

Empirical methods involve the comparison of field conditions where liquefaction did or did not occur during previous earthquakes with the actual site conditions. Past practice has been to compare the relative density of sand deposits with field values of the ratio γ avg/σ_o' computed using the Seed and Idriss simplified procedure and the estimated ground surface accelerations for the various sites. While relative density is suitable for laboratory studies of the liquefaction behavior of a uniform soil at various densities, computation of this parameter for natural soil deposits is difficult and may result in significant errors. Since relative density was usually determined from

Standard Penetration Tests, recent studies have used the Standard Penetration Test blow count data directly to evaluate the liquefaction potential at a site (Seed et al, 1975).

The most recent correlation between field liquefaction behavior of sands for level ground conditions and corrected penetration resistance based on field data and large scale laboratory test data is shown on Plate 14. Values of the cyclic stress ratio known to be associated with liquefaction or no liquefaction in the field are plotted in this figure as a function of the corrected average penetration resistance of the sand deposit involved. This modified penetration resistance corresponds to the measured penetration resistance of the sand corrected to an effective overburden pressure of 1 ton per square foot (0.98 kg/cm^2) based on the results of Gibbs and Holtz (1957).

With the aid of Plate 14 the liquefaction potential of any soil deposit can be estimated. The steps involved are the following:

1. Establish the magnitude of the earthquake to be considered.

2. Correct the actual measured Standard Penetration Test blow count data obtained from the field investigation.

3. Enter Plate 14 with the corrected values of penetration resistance and compute the cyclic stress ratios which will have caused initial liquefaction for the earthquake magnitude under consideration.

4. Compare the values of cyclic stress ratio computed in Step 3 with those calculated using the simplified procedure (Equation 1) or obtained from a one-dimensional ground response analysis.

A summary of the liquefaction analysis made on the basis of the empirical data presented on Plate 14 is given on Plate 15. The righthand column lists the factor of safety calculated on the basis of comparing the average induced cyclic stress ratios with those obtained from Plate 14 for the relationship corresponding to an M=7 event. These are plotted as a function of depth for the various soil profiles considered on Plate 16.

Liquefaction Analysis - Laboratory Method

The second procedure used to evaluate the liquefaction potential of the tailings deposit is based on a comparison of the cyclic stress ratios induced at various depths within the deposit (Equation 1) with the cyclic shaking stress ratios required to cause liquefaction of similar materials at similar densities in the laboratory.

The relative density of the in-situ sand tailings were based on two sets of data. First, the relationship shown on Plate 17, Relationship Between Standard Penetration Test Resistance, Relative Density, and Effective Overburden Pressure (Gibbs and Holtz, 1957) were used to estimate relative densities from the computed Standard Penetration Test blow count data (Plate 13). In addition, dry densities calculated for the samples obtained during our field investigation were used to obtain values of average relative density. Plate 18, Relative Density Versus Depth of In-Situ Sand Tailings, shows the relative densities calculated using these two approaches. In addition, a curve which represents a conservative estimate of relative density versus depth is shown. Values of the stress ratio required to cause liquefaction in the laboratory at similar relative densities were obtained from the shaking table test data for Monterey No. 0 sand shown on Plate 19, Comparison of Shaking Table and Triaxial Test Results (De Alba, Chan and Seed, 1975). For this study, it was considered appropriate to use 10 equivalent stress cycles as representative of the number of cycles corresponding to the postulated M=7 event. The results of the liquefaction analysis are presented on Plate 20, Comparison of Induced Cyclic Stress Ratio and that Required to Produce Initial Liquefaction in Shaking Table Tests. The corresponding factors of safety for the various soil profiles considered are plotted graphically on Plate 21, Factor of Safety Against Liquefaction Based on Laboratory Data and Relative Density.

Results

Of the four soil profiles considered in this study, the one shown most susceptible to liquefaction under the postulated seismic event is Profile A which corresponds to the condition where the sand tailings are completely saturated with the free water level at the surface. This profile is located some 150 feet from the edge of the embankment crest. For this condition the lowest factor of safety calculated was slightly greater than one and this was obtained on the basis of empirical data. On the basis of laboratory data the lowest factor of safety was almost two. Under conditions where the free water surface is 20 feet or greater below the surface, the lowest calculated factor of safety against liquefaction occurring by either method of analysis is 1.40. Plate 22, Cyclic Stress Ratios Versus Depth, shows the induced cyclic stress ratios for the four different soil profiles as well as the stress ratios required to cause liquefaction based on the empirical approach and on the shaking table test data of Monterey No. 0 sand.

Even should the conditions favorable to liquefaction develop within the embankment and an earthquake of a level possible for this region of very low seismicity occur, the occurrence of liquefaction throughout most of the embankment would be very unlikely. It is possible, however, that some local liquefaction might occur in isolated portions of the embankment during the postulated seismic event. Based on the analyses presented in this report it is our opinion that even if this were to occur the overall stability of the tailings embankment would not be in jeopardy.

RECOMMENDED EMBANKMENT RAISES

General

Beyond the remaining capacity of the existing impoundment, an additional storage capacity of about 3,100 acre-feet is required for the remaining life of the operation. To provide this capacity, a total height increase of the embankment of 36 feet will be necessary. The increase can best be accomplished by raising the crest in three separate stages of 12 feet each. The western embankment of the impoundment will be raised by the use of downstream construction; the remaining embankments will be raised through the upstream method of construction. A minimum freeboard of six feet and a minimum crest-to-ponded-water distance of 150 feet will be maintained for all embankments, including the western embankment, for all three raises. In order that the uranium milling operations can continue uninterrupted, it is imperative that the first raise be completed by July, 1978. Further, based on future production predictions, raises for the second and third stages are recommended for no later than April, 1982 and January, 1986, respectively.

Stability analyses were performed to evaluate the effects of construction of the additional supplemental dikes on the tailings pond-embankment system. Result of these analyses have shown that the factor of safety of the embankments against failure is adequate and the proposed embankment additions may be made without endangering the stability of the tailings pond-embankment system.

Western Embankment

The western embankment is the linear embankment section as defined on Plate 2, Plot Plan. Cross-sections C-C and D-D on Plate 3C show the configurations of the three embankment raises to be constructed using the downstream construction method. The first raise will have upstream slopes of 2:1 and downstream slopes of 3:1. The second and third raises will have the same 2:1 (horizontal to vertical) upstream slopes and downstream slopes of 2.5:1 and 2:1, respectively. The crest widths of the three raises will be 30, 25 and 20 feet, respectively for Raises 1, 2 and 3. All three raises will be 12 feet in height. It should be noted that before placement of the first embankment raise and continuing thereafter, a sand beach must be formed and maintained through tailings discharge control along the western embankment. The minimum crest-to-ponded-water distance of 150 feet must be maintained at all times.

To incorporate the use of the western embankment's existing filter system, it will be necessary to construct a trench along the western embankment's existing downstream toe. The trench constructed parallel to the toe should extend in a southeastern direction along the entire length of the western embankment connecting to a sump constructed at the depression at the extreme south corner of the tailings pond area (see Plate 2). The trench should be filled with gravel. In the center of the trench a perforated pipe should be placed to collect and transport to the sump any seepage which might occur within the embankment. The sump should be equipped such that any accumulated water can be pumped to the ponded water area within the tailings impoundment area. The filter system will serve to ensure that any seepage will be controlled and thus eliminate any possibilities of piping occurring along the future embankment downstream toe.

Accidents have occurred to tailings pond areas in the past whereby a ruptured or improperly operated discharge pipe spilled tailings slurry onto the embankment crest thus causing erosion channels within the embankment's crest and downstream slope. Such erosional gullies then became channelways for the tailings that were otherwise confined within the ponded area. The erosion had increased as more tailings flowed through the gullies to such a degree that a major-type progressive failure occurred. To elimate the possibility of such a failure occurring at Atlas' tailings pond area, it is recommended that the embankment crest surface be constructed to tilt toward the pond area. The configuration of the crest should be such that the elevation of the downstream-most point of the crest is 2 to 3 feet higher than that at the upstream-most point. Thus, if any slurry should flow onto the crest area, it will drain into the pond area.

Main Embankments

The main embankments comprise all other tailings pond embankments other than the existing western embankment, and for these embankments the required 36-foot raises may be accomplished by means of the upstream construction method. Typical cross-sections of the main embankments including the proposed raises are shown on Plates 3A, 3B and 3D. The three raises will have similar geometry in that each height increase will be 12 feet, the crest width will be 15 feet, and both the upstream and downstream slopes of each raise will be two horizontal to one vertical. Further, the downstream face of each raise will intersect the existing crest 15 feet from the downstream edge of the preceding dike's crest. This arrangement provides for an overall downstream slope for the three raises of 3.25 horizontal to one vertical.

The embankment crest along the main embankment area should be constructed to tilt downward toward the direction of the ponded area in a manner similar to the configuration of the western embankment crest.

STORAGE CAPACITIES AND REQUIRED FILL QUANTITIES

Storage capacity and additional embankment fill requirements are dependent on the operational constraints placed on the tailings pond freeboard (6-foot minimum) and ponded-water configuration (150-foot minimum distance from crest-to-ponded-water). Since the production rate of tailings does not permit simultaneous discharge of tailings around the entire periphery, discharge must be restricted to specific segments of the periphery at any one time. By periodically changing the location of the discharge along the entire periphery in an organized manner, a uniform beach wedge may be constructed at the outer edge of the pond and the ponded water would then be contained within the central area of the tailings pond. Thus, the 150 foot minimum crest-to-ponded-water distances would be maintained at all locations along the tailing area's perimeter. Further, data taken from a recent topographic map of the tailings pond show that the slope of the beach wedge is about 2.4 percent. To simultaneously achieve a six-foot freeboard and a 150-foot crest-to-ponded-water distance, the deposited beach wedge should begin on the upstream slope of the embankment at a vertical distance of two and one-half feet below the embankment crest.

By definition, the base of the beach wedge is the horizontal plane corresponding to the elevation of the ponded water surface. Thus, the beach wedge allows for storage capacity above the required six-foot minimum freeboard and also provides a source of sandy fill material which can be used in the construction of the next raise. However, dike construction must be performed in such a manner that the borrow limits of beach tailings do not extend below the six-foot freeboard elevation and the surface of the tailings remaining after construction always be smooth and sloping towards the ponded water.

The volume of sand contained in a beach wedge will not be sufficient to construct an entire embankment raise, and borrow fill from outside sources will be required for each of the three raises. The raises for the main embankment sections (that is, the north, east, and south embankments) will be constructed upstream and consequently will be built directly on top of a major portion of the sand tailings beach wedge material. However, the downstream construction of the western embankment will allow full use of the bench wedge material for fill since all construction is downstream and there will be no construction on top of the beach wedge.

To reduce radiation emanation from the beach sand used as construction material and to reduce erosion of the beach sand, a minimum four-foot thick protection will be placed over the beach sand dike material. The covering will be of the on-site or near-site reddish brown sandy surface soils and will be a part of the structural section of the dike.

Of the 724,000 cubic yards of construction fill required for the total of the three staged raises, 108,000 cubic yards would be tailings beach sand and the remainder would be borrowed from outside sources. Table 6, Fill Requirements for Proposed Embankment Raises, lists the storage capacity, maximum water surface area, and detailed fill requirements for each of the three proposed embankments.

EMBANKMENT MATERIALS AND PLACEMENT

General

It is recommended that the embankment additions be constructed with the materials and configuration as described in the

TABLE 6

FILL REQUIREMENTS FOR PROPOSED EMBANKMENT RAISES

Raise No.	Additional Storage Capacity (ac-ft)	Maximum Water Surface (acres)	Fill Requirements: Western Embankment (10^3 yd^3)	Fill Requirements: Non-Western Embankments (10^3 yd^3)	Fill Requirements: Total (10^3 yd^3)	Fill Sources: Beach Sand (10^3 yd^3)	Fill Sources: Outside Sources (10^3 yd^3)
1	1,070	65	92	111	203	39	164
2	1,040	63	142	127	269	35	234
3	1,010	61	126	126	252	34	218
Total	3,120		360	364	724	108	616

section entitled RECOMMENDED EMBANKMENT RAISES and as present on Plates 2, 3A, 3B, and 3C.

Site Preparation

Before fill placement, the area along the downstream toe of the western embankment slope should be stripped of all vegetation, organic materials, and topsoil (to an estimated three to six inches) and removed from the construction area.

Materials

Three materials will be required to complete the proposed construction: (1) on-site or near-site natural reddish brown fine sand or silty sand; (2) medium- and fine-grained sand tailings containing about 5 to 25 percent silt; and (3) imported well-graded gravel material (2-inch maximum-sized pieces). No complete gradation ranges need be specified for either the natural reddish brown sands or the sand tailings since nearly all the locally available natural or tailing beach materials fit adequately into the design criteria used. In order to protect the western embankment downstream toe drainage trench against contaminating and plugging the drain by surrounding soil, it is recommended that a well-graded sand/gravel mix be used as trench-fill material with transition filter of well-graded fine to course sand placed at eight inches thick around the entire gravel-filled trench.

Material Placement and Compaction

The embankment construction material should be placed in lifts not exceeding 12 inches in loose thickness and compacted to 95 percent of the maximum dry density determined by the standard American Society of Testing Materials (ASTM) D-698 criteria. The moisture content of the natural silty sand will generally be on the dry side of optimum moisture. If it is necessary to add moisture to obtain the desired compaction, the loose lifts should be wetted and scarified lightly so as to obtain an even distribution of moisture. The fill (natural silty sand and beach sand tailings) will probably be most effectively compacted with a heavy, smooth-wheeled, vibratory roller. A minimum of two passes with such equipment should be sufficient to compact a 12-inch lift as specified.

We recommend that a Dames & Moore soils engineer be present during all site preparation, excavation, placement and compaction of earth fills. The purpose of the Dames & Moore engineer would be to inspect the new embankment foundations and control the quality of the fill materials and their compaction. It is recommended that an inspection report containing all field density test results and descriptions of the earthwork be performed to provide proper documentation and assure the quality of all construction.

OPERATIONAL PROCEDURES AND POST-CONSTRUCTION INSPECTIONS

Tailings Discharge and Pond Configuration

The most variable and many times the most difficult parameter to control within a tailings pond is the phreatic surface within the embankment section. In order to control the phreatic surface, a minimum distance between the crest of the embankment and the nearest point of the ponded water must be maintained. It is recommended and assumed in our design that a minimum beach distance of 150 feet be maintained at all times along the entire perimeter of the tailing pond. Further, a well-maintained beach will protect the upstream face of the embankment from the possible effects of wave erosion. To insure against the possibility of such erosion occurring, it is recommended that an elevation difference of six feet between the ponded water surface and the embankment crest be maintained at all times.

In order to maintain a proper beach area and satisfy the above conditions of the minimum required distance to the ponded water, the deposition of tailings must be performed by a properly regulated discharge operation. Correct maintenance of the tailings discharge will result in a sloped beach on the tailings pond surface. With continual regulated discharge the overall slope from the edge of embankment to the water pool area should be expected to approach an overall gradient of slightly above two percent.

Future Monitoring

The phreatic surface within an embankment section of any tailings pond is perhaps the single most important factor in the stability of the embankment. In order to maintain surveillance of the phreatic surface we recommend continued observation of the existing piezometers placed at the location of each of the exploration borings drilled in conjunction with this study and shown on Plate 2, Plot Plan. Should any of these piezometers be damaged or covered by construction and/or maintenance operations, they should be replaced.

Since the time the piezometers were placed along the various sections of the embankment (March, 1977), readings have been taken on a monthly basis. It is recommended that readings be continued at the monthly intervals. Further, we recommend the data be transmitted to a qualified soils engineer for the purpose of review. Such review should be undertaken continuously in a cursory manner in order to detect any gross changes which might have occurred. Then, quarterly, a comparative review should be made to detect any general trends.

Embankment Surveillance

We recommend that the tailings pond be subjected to a regular program of continuing visual surveillance during the complete life of the pond to that time when the deposited tailings reach the ultimate embankment elevation. We recommend that such surveillance be carried out by an experienced soils engineer at least once per year. During such surveillance, attention should be paid to any signs of tension cracking, slumping, erosion, and seepage, and records should be maintained of the observances made for each surveillance. Immediate remedial measures should be undertaken to repair any points of distress noted. At the time of the annual detailed inspection, the reviewing soils engineer should review the design assumptions and perform comparisons of the assumptions with the field observation data received during the time of inspection including a review of the available piezometer data.

RECLAMATION

Reclamation measures to be undertaken during the operational life of the tailings pond-embankment system and after abandonment have been described in an earlier report (Dames & Moore, 1977).

oOo

In addition to the tables and plates shown in the List of Tables and List of Plates, the following Appendices are attached and complete this report:

Appendix A - Field Exploration and Laboratory Testing

Appendix B - Evaluation of Engineering Properties

Appendix C - References

Respectfully submitted,

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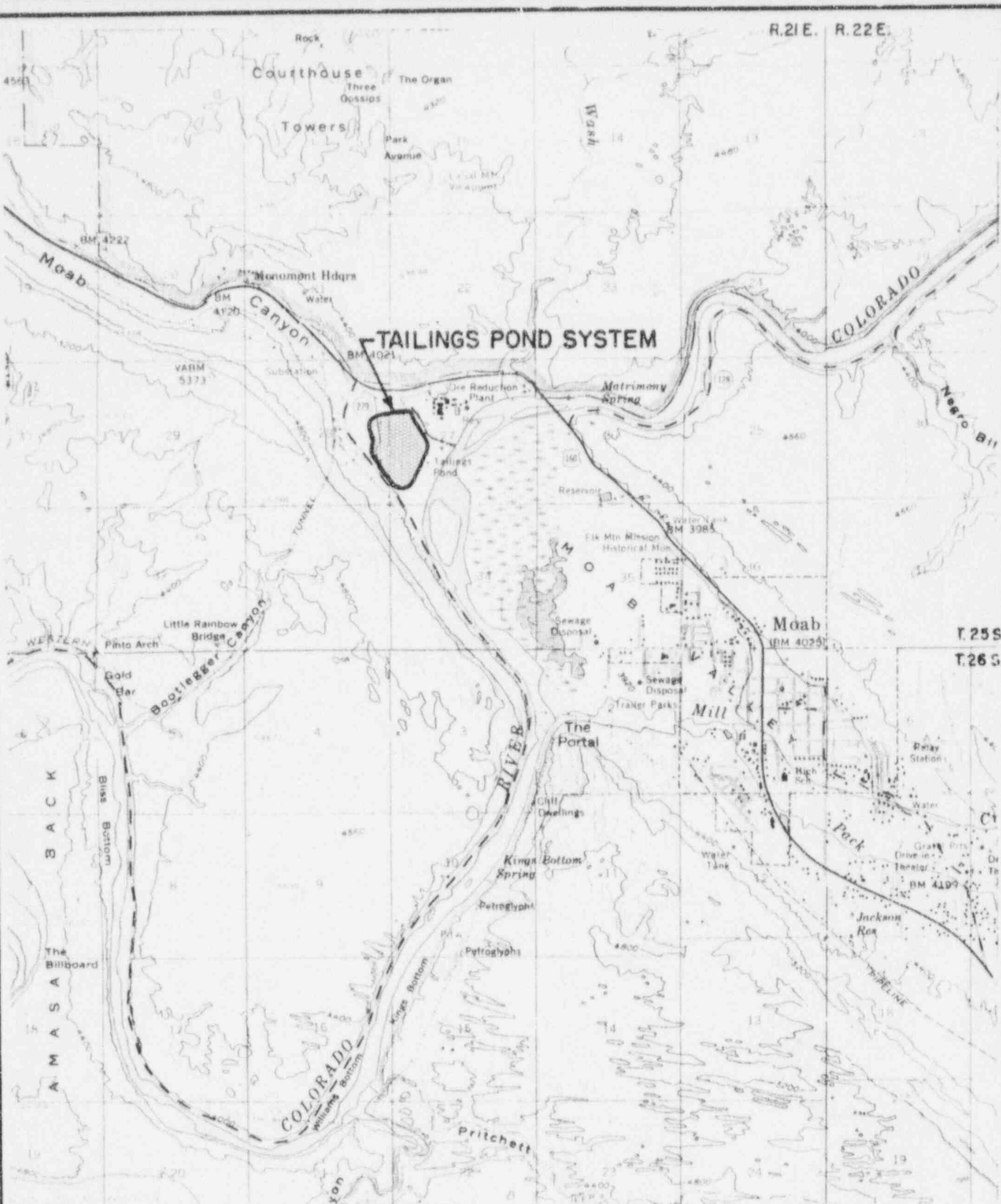

REFERENCE:
QUADRANGLE SHEET TITLED "MOAB, UTAH, AND DATED 1959.

VICINITY MAP

SCALE IN MILES

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DAMES & MOORE

PLATE 1

BY A.B.D. DATE 1-4-73 FILE 5467-006 ATLAS MINERALS - MOAB, UT. REVISIONS BY ____ DATE ____

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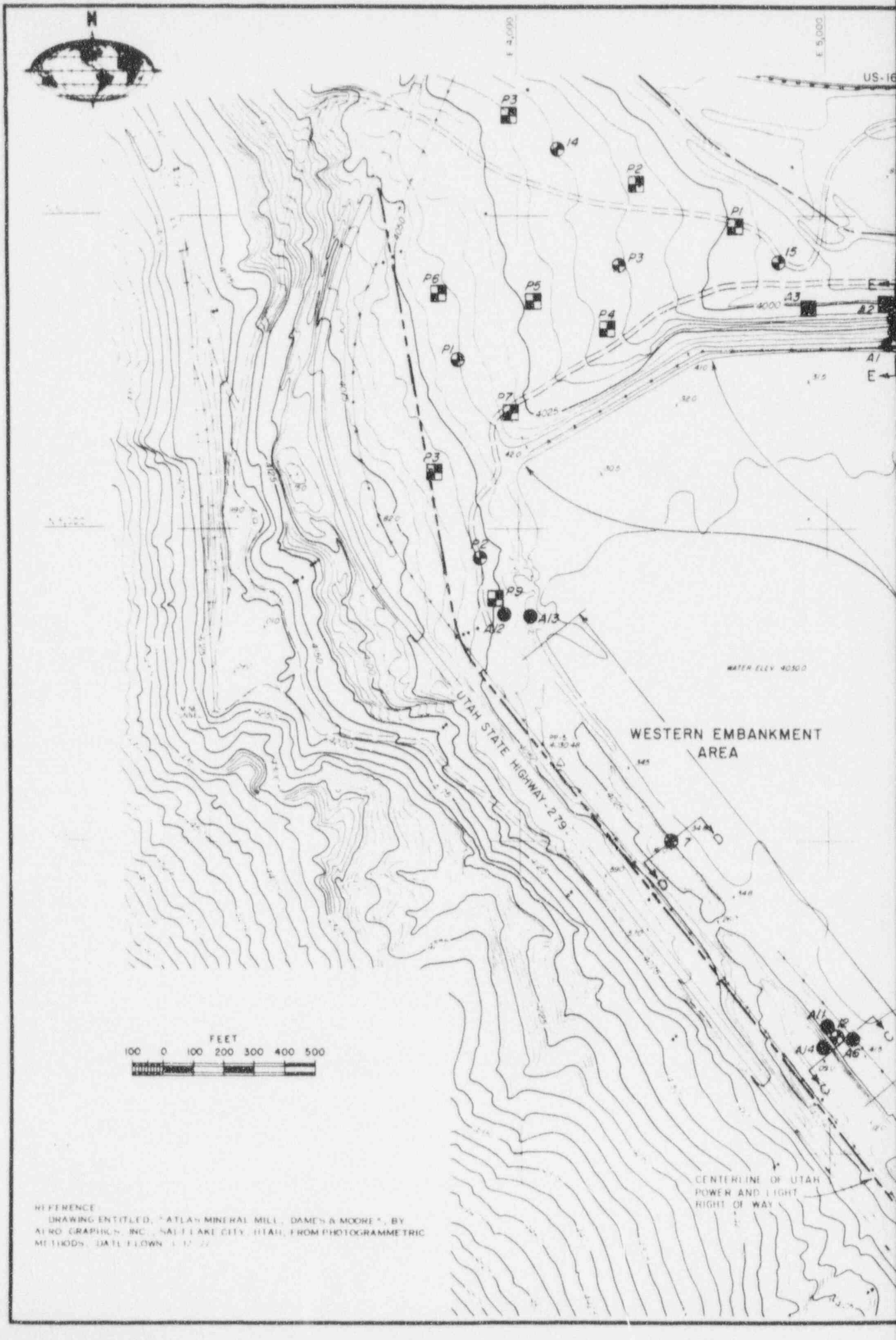

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A12
A13
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WESTERN EMBANKMENT AREA
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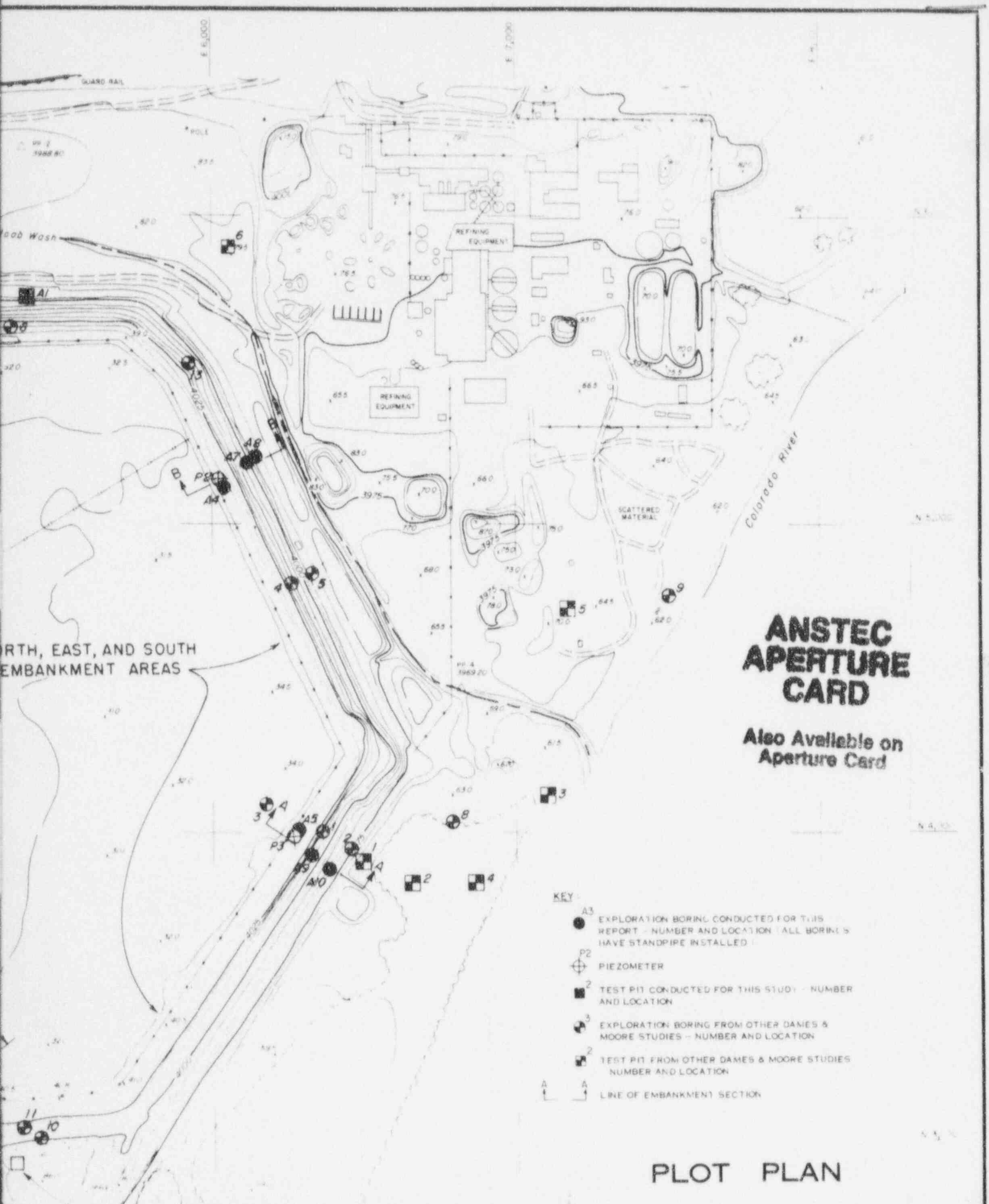
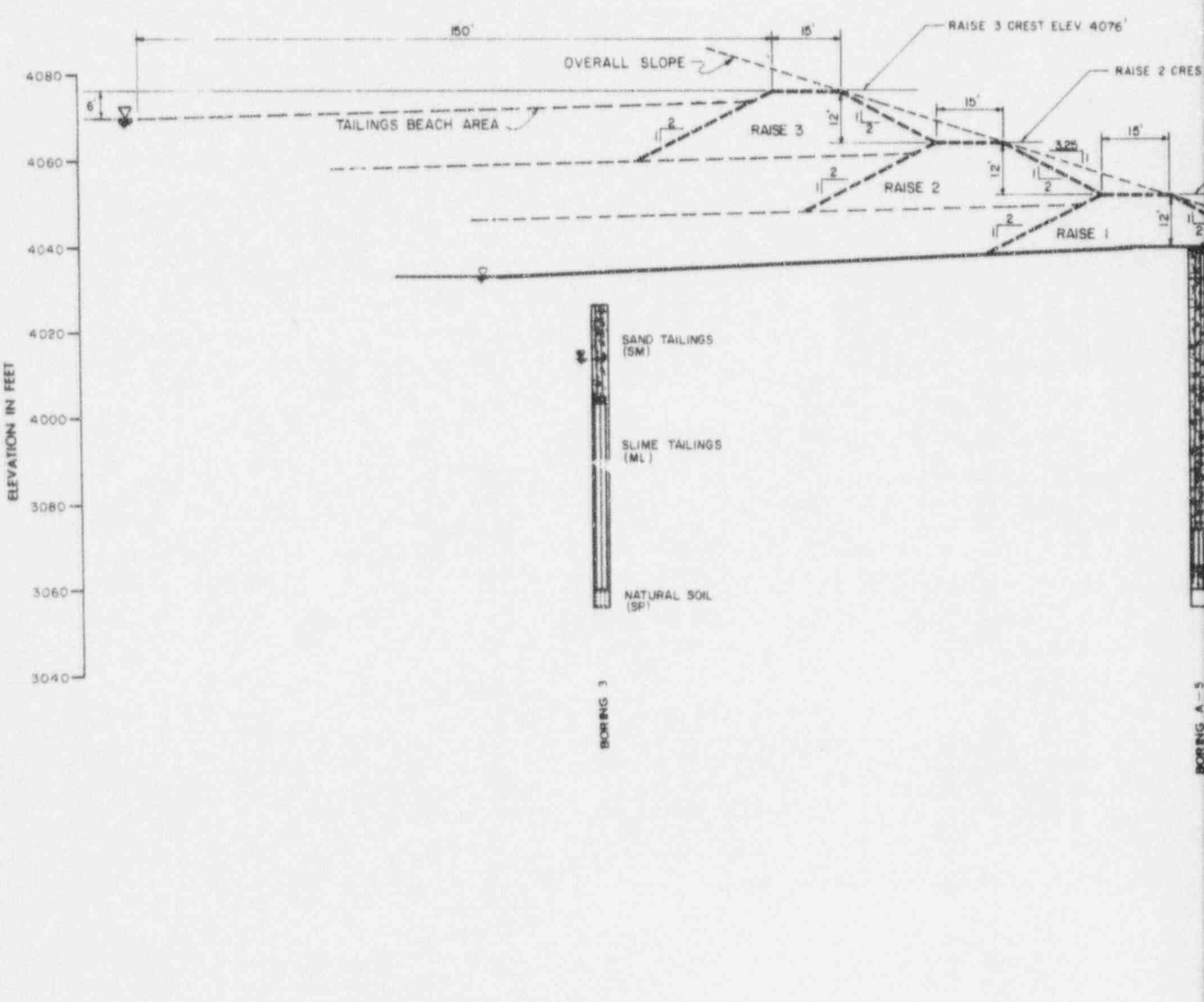
D
A11
A14
A6
C
FEET
100 0 100 200 300 400 500
CENTERLINE OF UTAH POWER AND LIGHT RIGHT OF WAY
REFERENCE:
DRAWING ENTITLED, "ATLAS MINERAL MILL, DAMES & MOORE", BY AERO GRAPHICS, INC., SALT LAKE CITY, UTAH, FROM PHOTOGRAMMETRIC METHODS. DATE FLOWN



9705230158-01

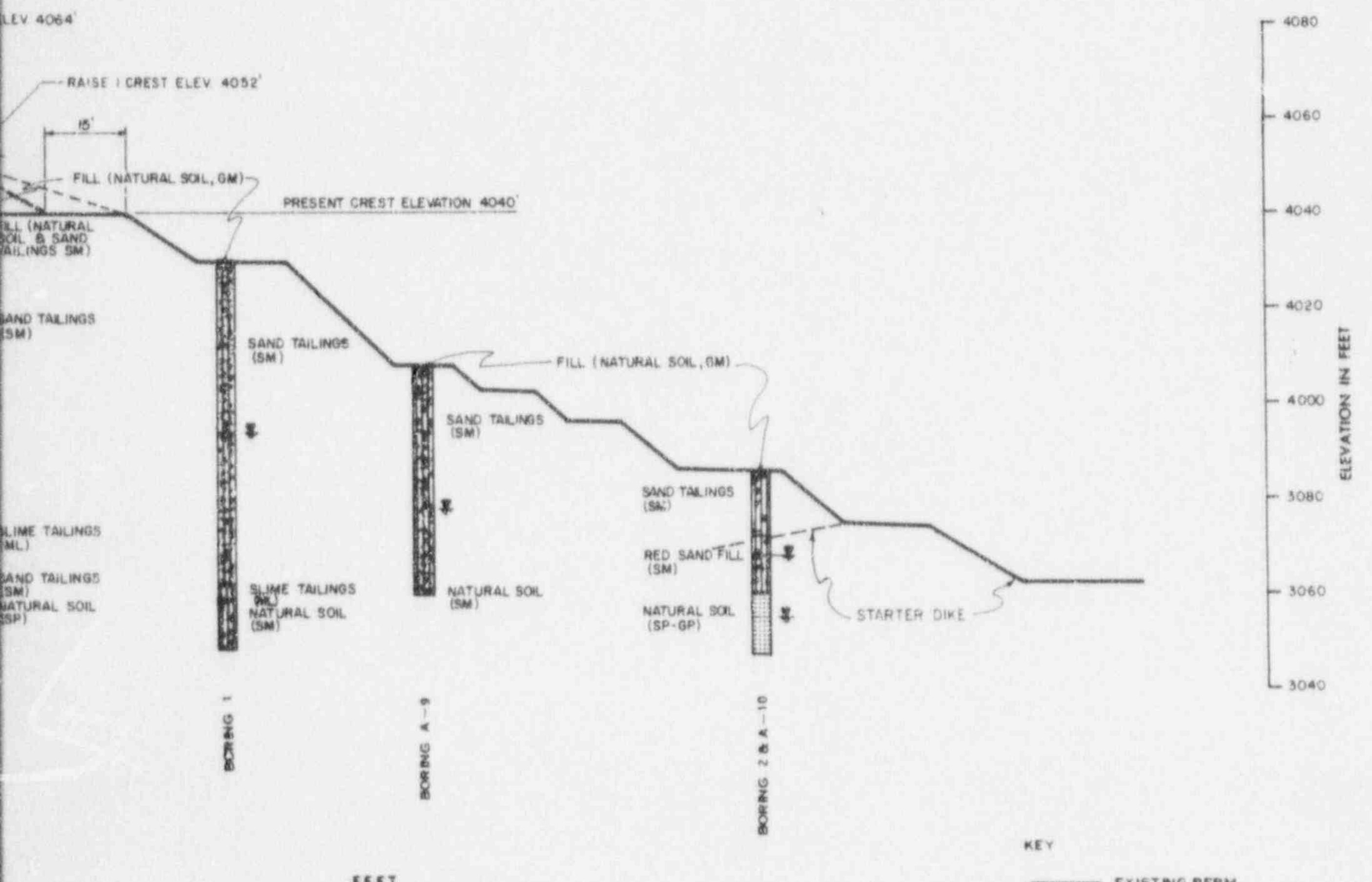
PLATE 2



150'
15'
RAISE 3 CREST ELEV 4076'
RAISE 2 CRES
OVERALL SLOPE
6'
TAILINGS BEACH AREA
RAISE 3
RAISE 2
RAISE 1
15'
15'
3.25
12'
4080
4060
4040
4020
4000
3080
3060
3040
ELEVATION IN FEET
SAND TAILINGS (SM)
SLIME TAILINGS (ML)
NATURAL SOIL (SP)
BORING 3
BORING A-3

ANSTEC APERTURE CARD

Also Available on Aperture Card



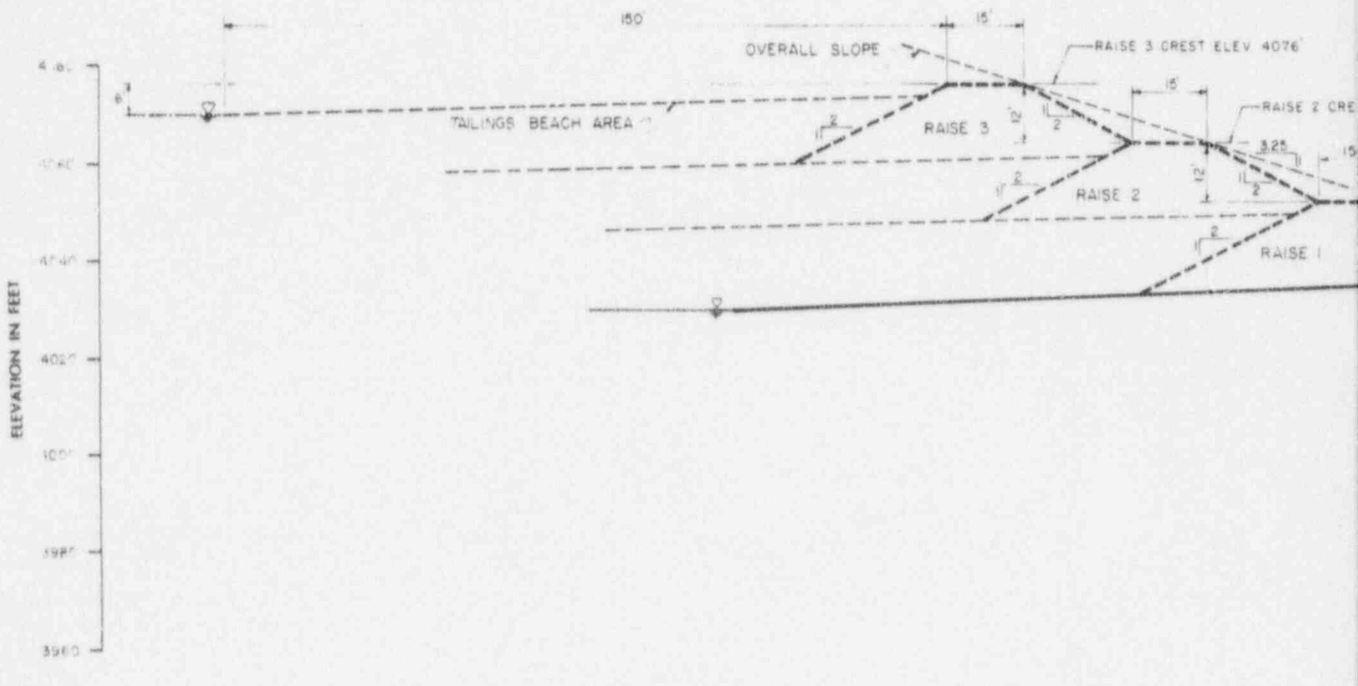
SECTION A-A

EMBANKMENT SECTION

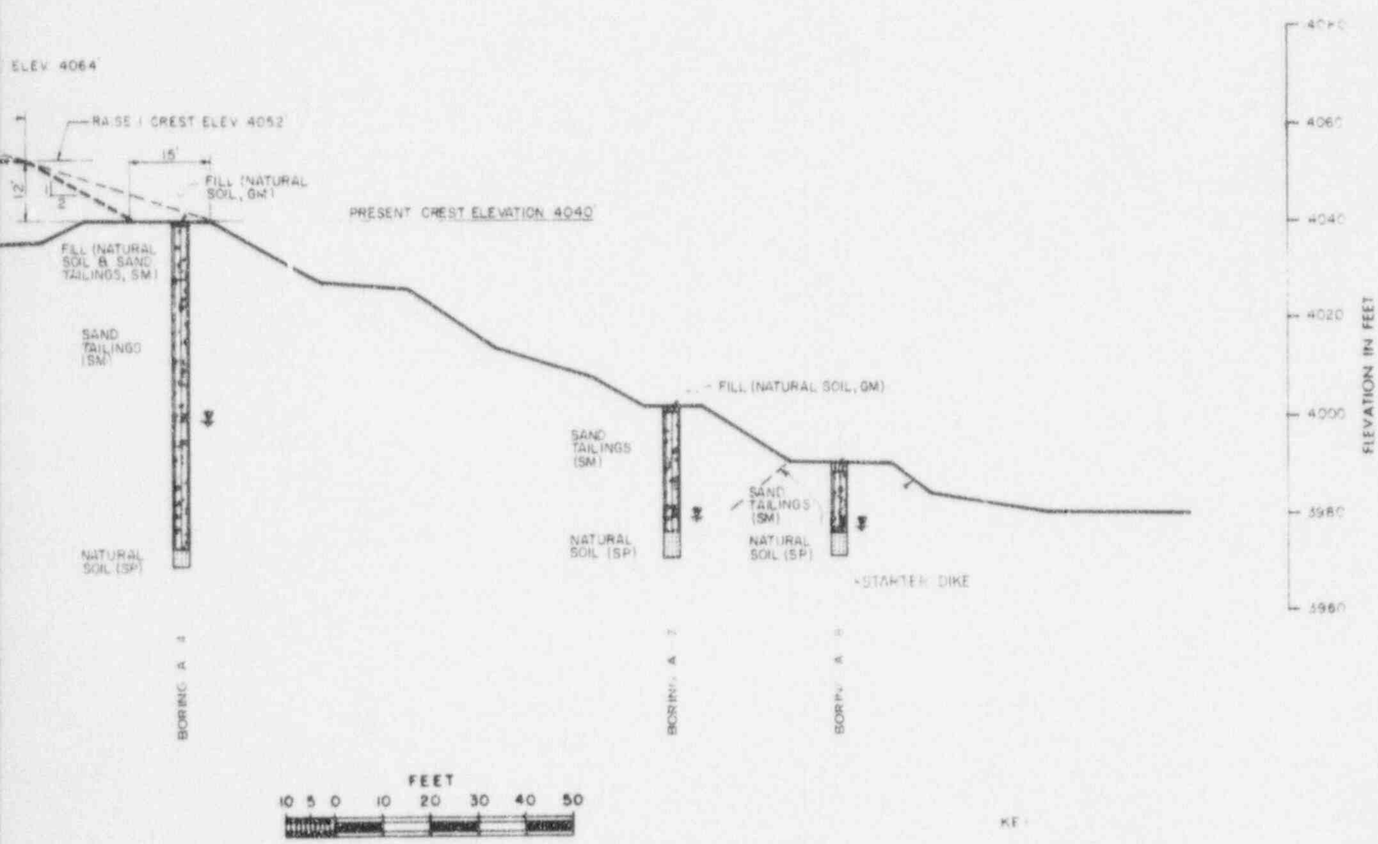
9705230158 - 02

DAMES & MOORE

PLATE 3A



ELEVATION IN FEET
4020
3980
3960
150'
15'
OVERALL SLOPE
RAISE 3 CREST ELEV. 4076'
15'
RAISE 2 CRE
6'
TAILINGS BEACH AREA
RAISE 3
12'
RAISE 2
12'
3.25
RAISE 1



SECTION B-B

KEY

EXISTING BERM
PROPOSED RAISE

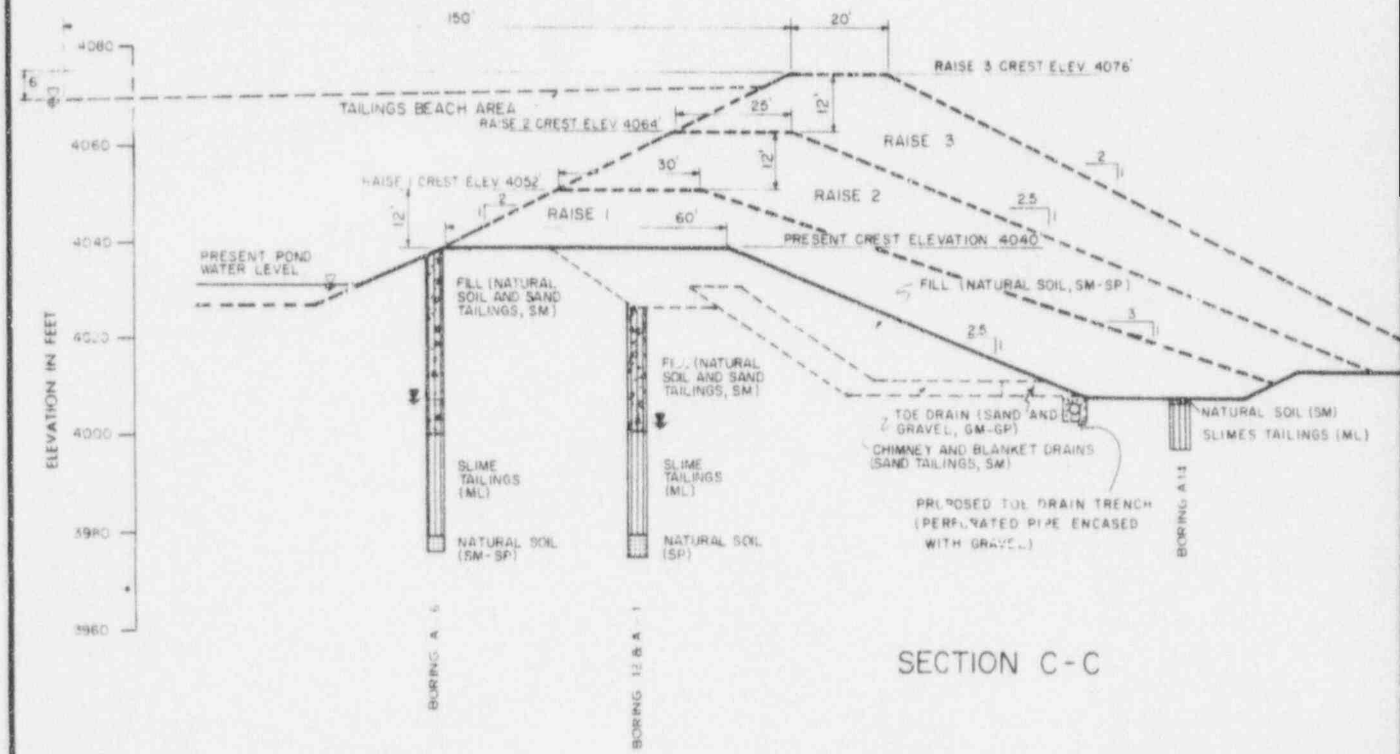
EMBANKMENT SECTION

9705230158 - 03

DAMES & MOORE

PLATE 3B

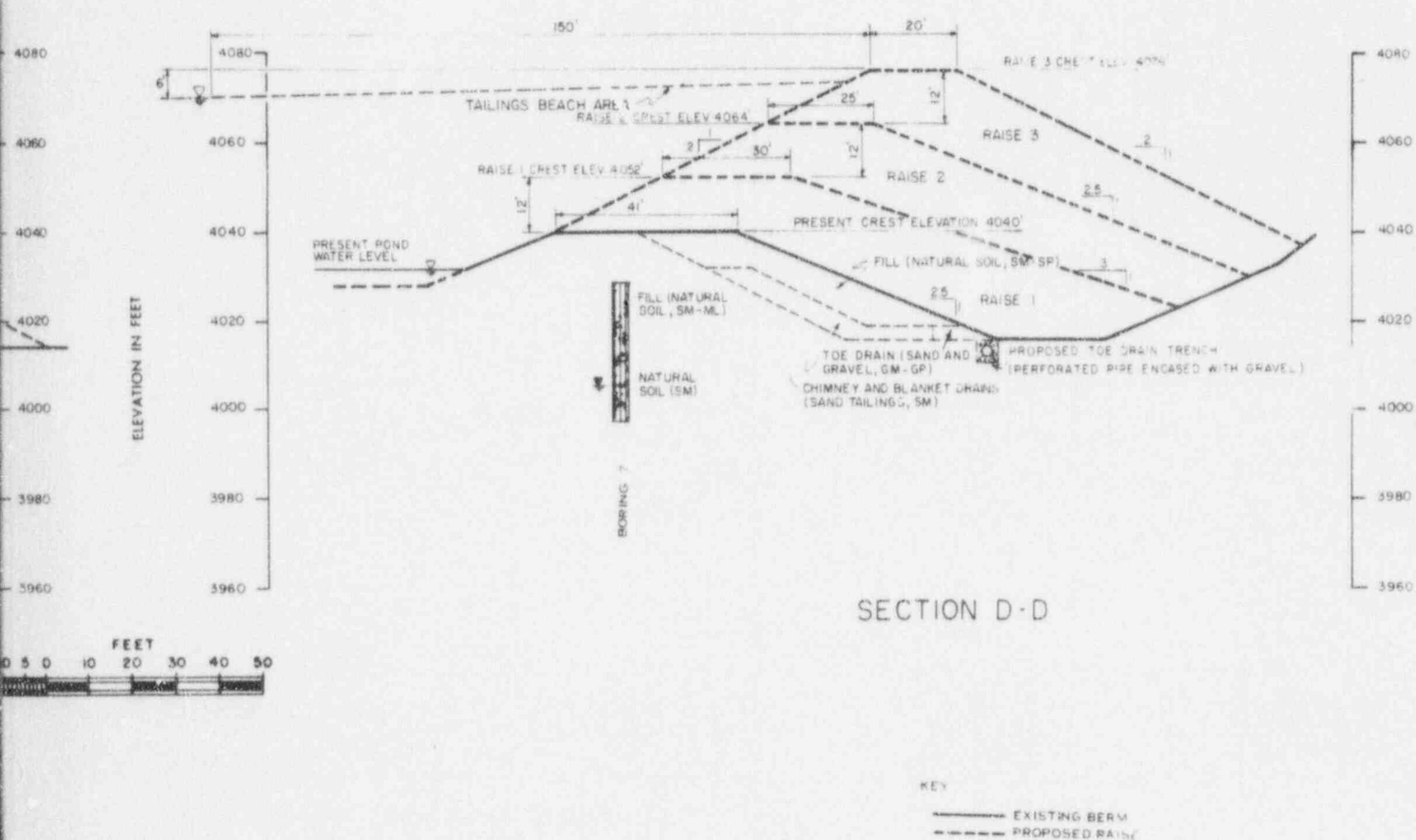
Atlas Minerals



SECTION C-C

ANSTEC
APERTURE
CARD

Also Available on
Aperture Card



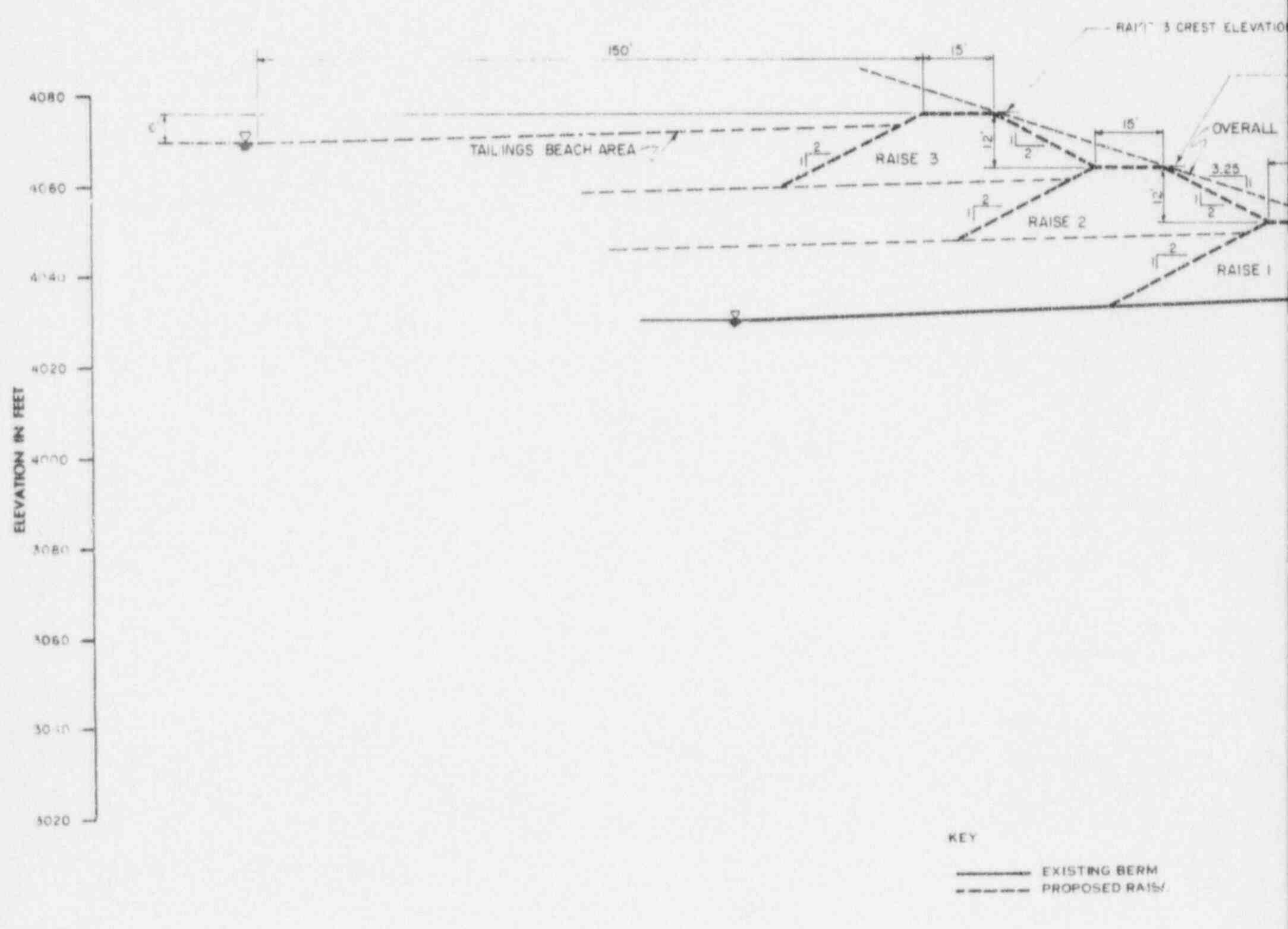
SECTION D-D

EMBANKMENT
SECTIONS

9705230158 - 04

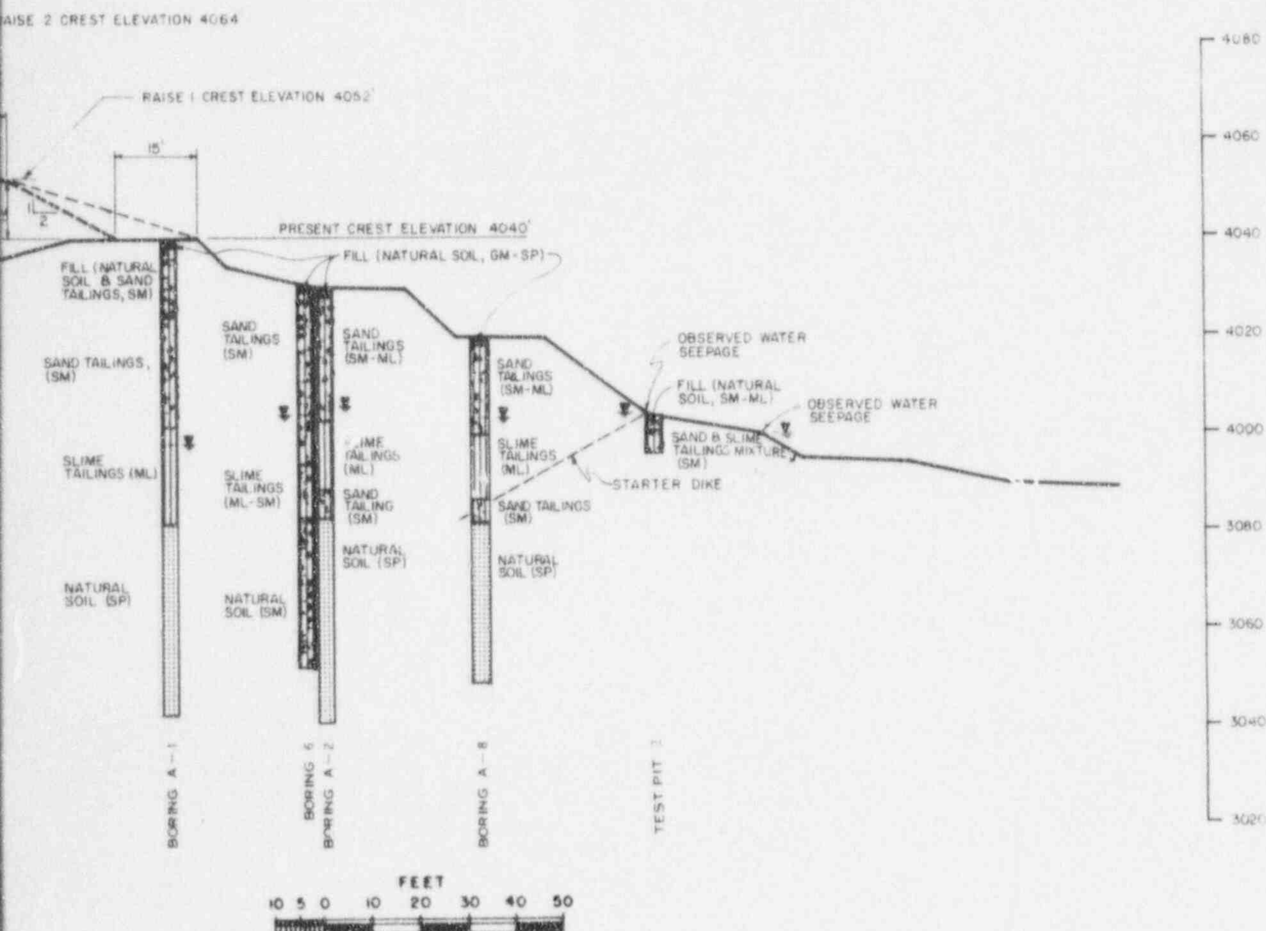
DAMES & MOORE

PLATE 30



BY DATE CHECKED BY DATE

BY DATE PLATE OF



SECTION E-E

EMBANKMENT SECTION

9705230158 - 05

DAMES & MOORE

PLATE 3D

946.7 (REV. 6-61)

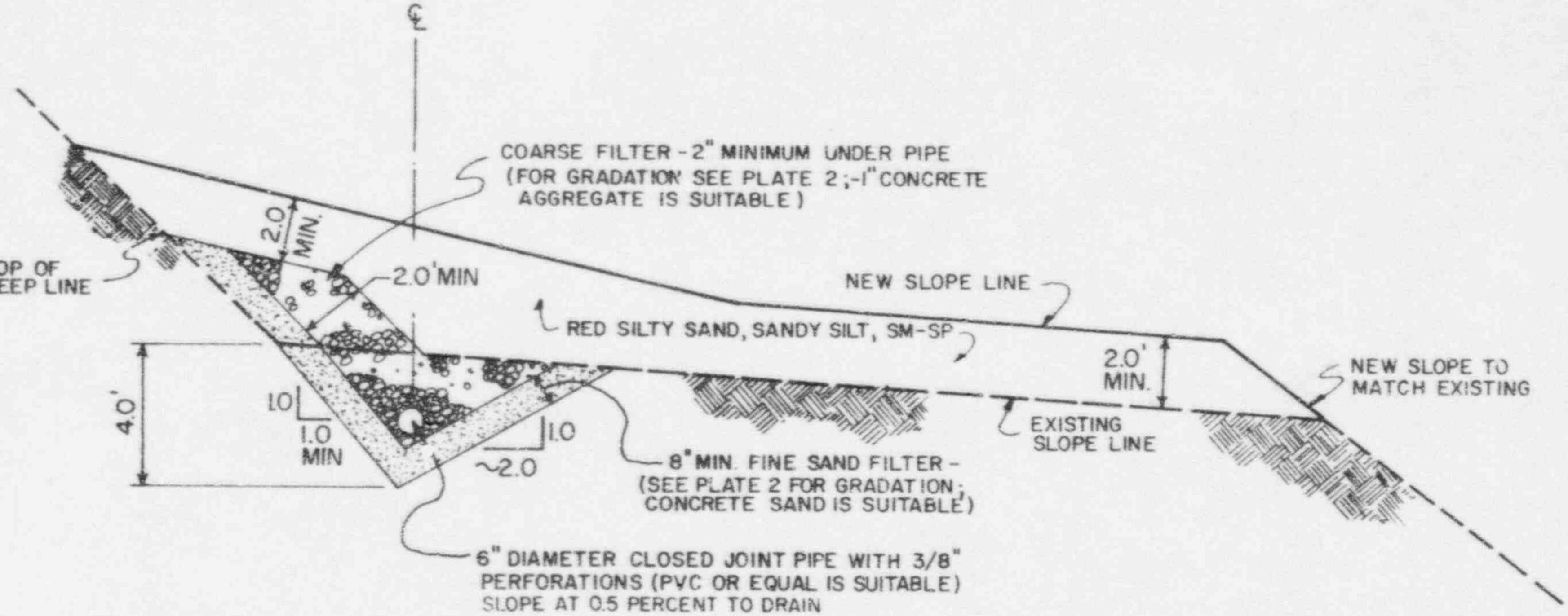
BY ____________ DATE 9-2-77

CHECKED BY R. Rogers

FILE 09467-0?8 Atlas Minerals

REVISIONS -

BY ________ DATE ______



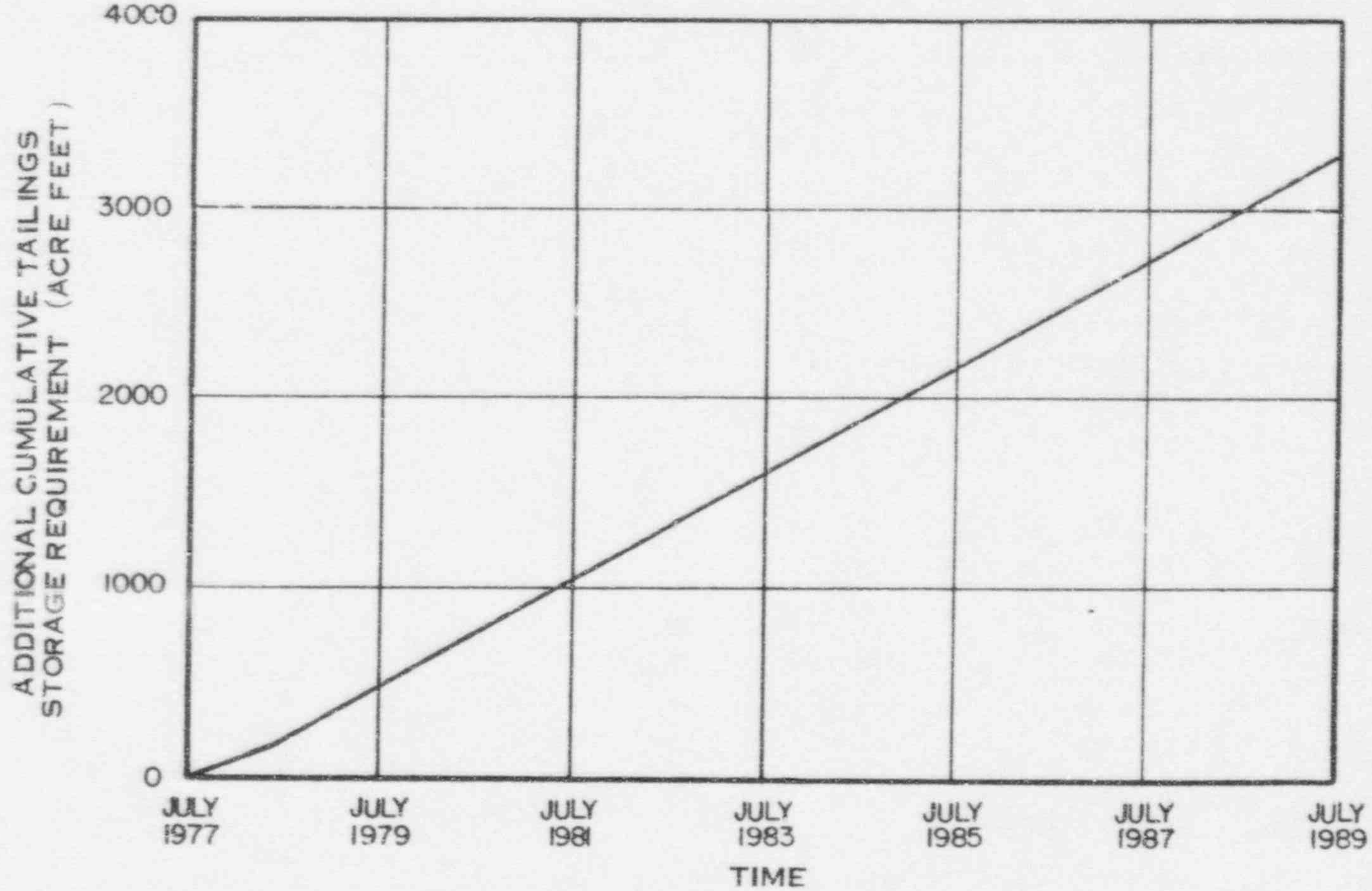
EMBANKMENT SEEPAGE
SUB-DRAIN - TYPICAL

DAMES & MOORE

PLATE 4

946.7 (REV. 6-[illegible])

BY R Nattle DATE ______
CHECKED BY ______ 7-7
FILE 05467-018 Atlas Minerals
REVISIONS
BY ______ DATE ______

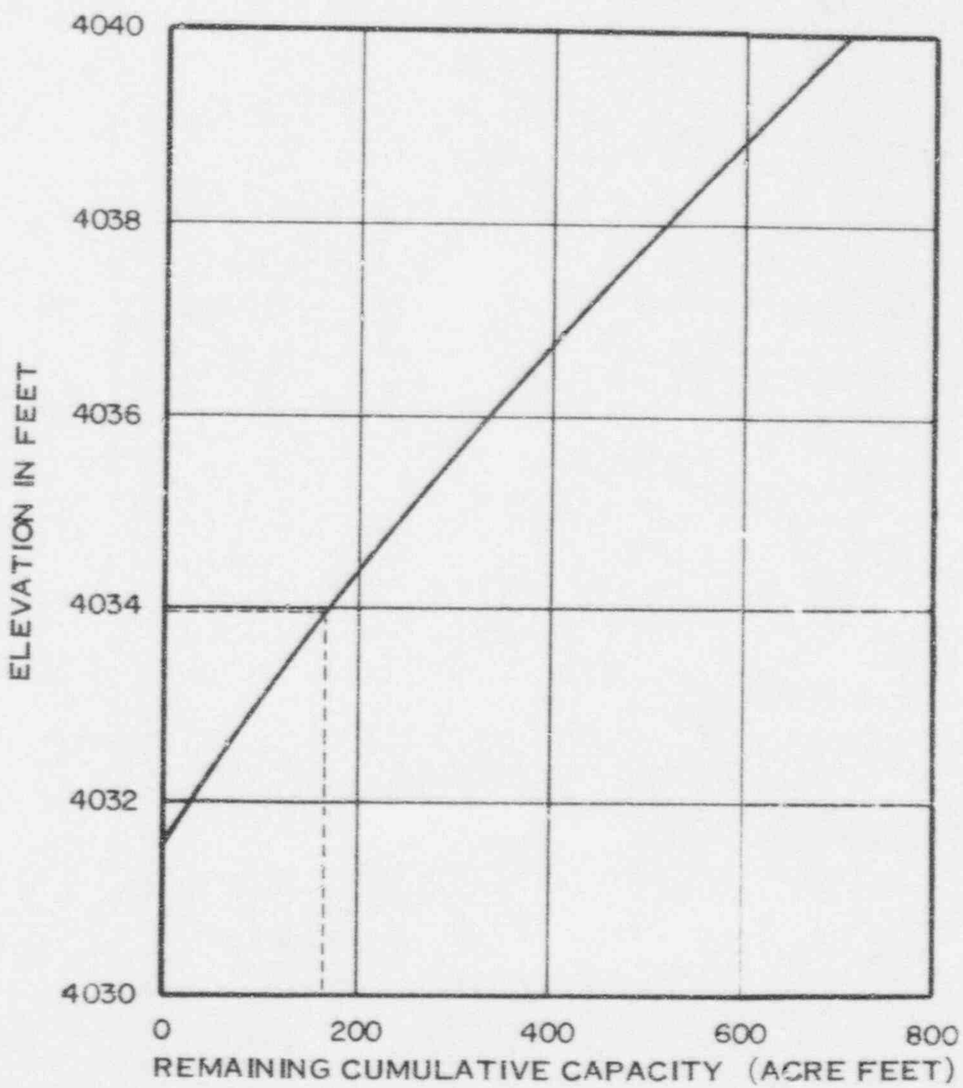


FUTURE TAILINGS STORAGE REQUIREMENTS
FOR ATLAS MINERALS - MOAB, UTAB MILL

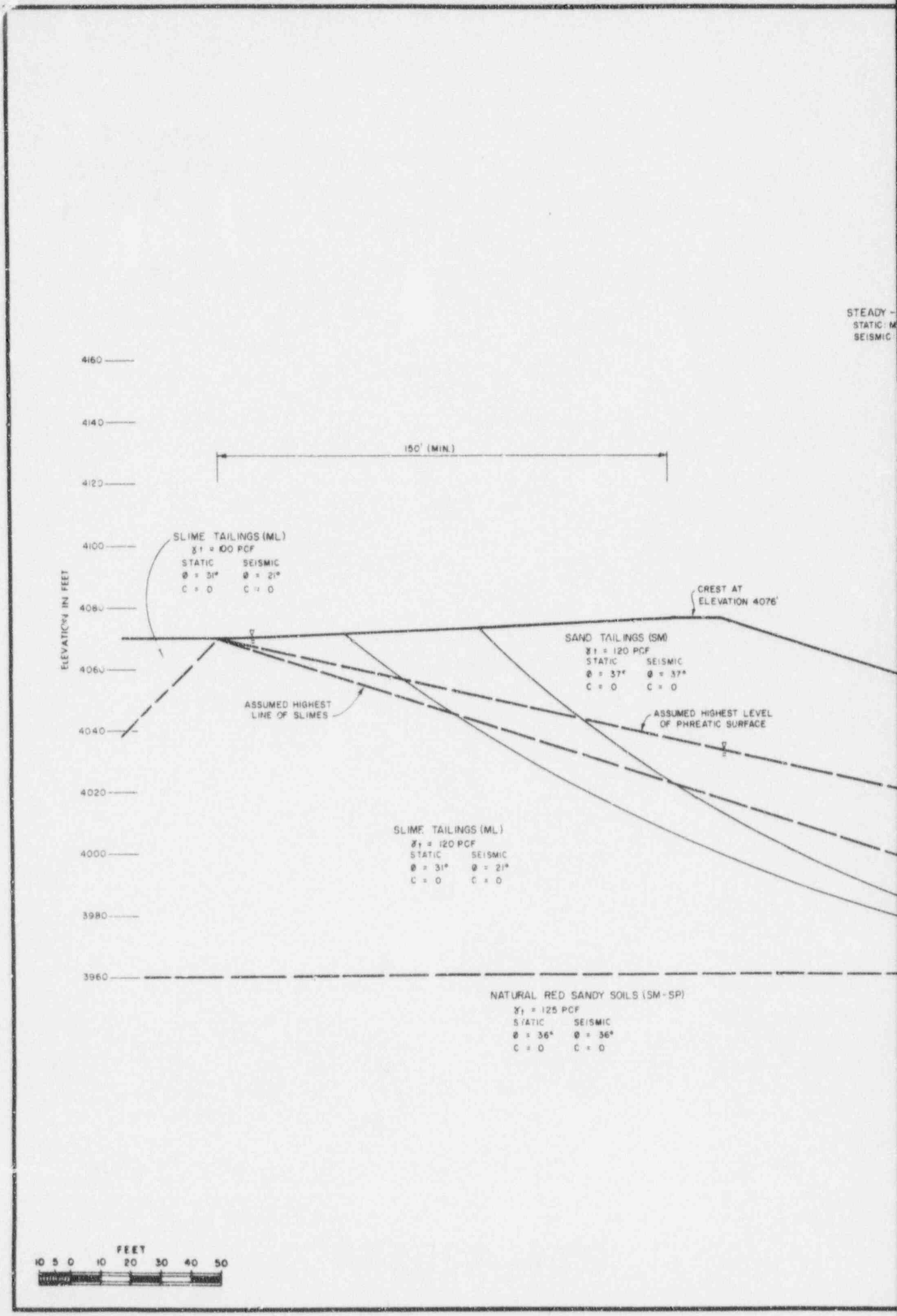
DAMES & MOORE

PLATE 5

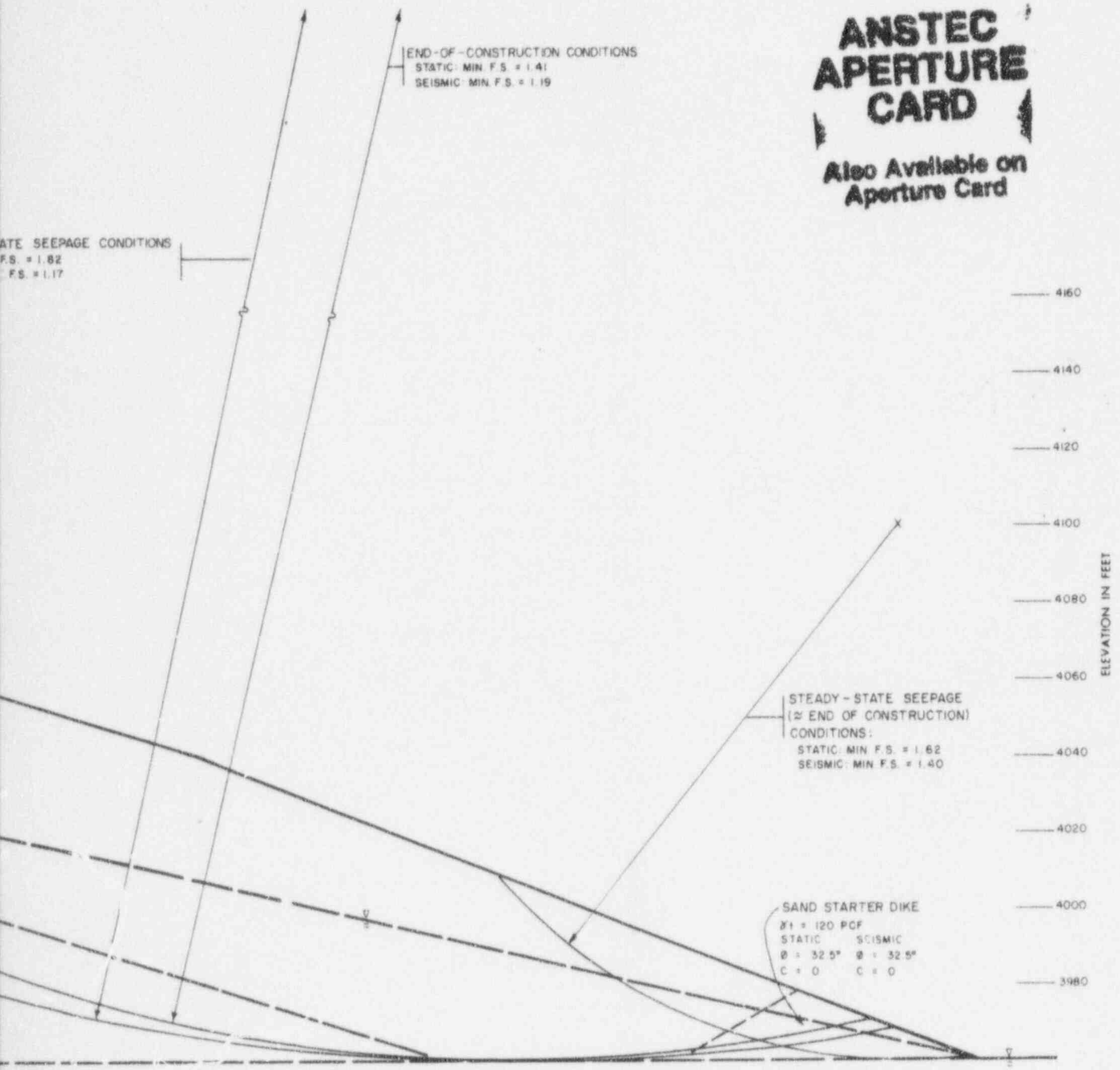
REMAINING CAPACITY TO ELEVATION 4034'	170	ACRE FEET
REMAINING BEACH WEDGE	30	ACRE FEET
TOTAL	200	ACRE FEET



ELEVATION VERSES REMAINING CUMULATIVE CAPACITY FOR EXISTING TAILINGS IMPOUNDMENT, ATLAS MINERALS, MOAB, UTAH MILL



STEADY -
STATIC: M
SEISMIC
4160
4140
4120
4100
4080
4060
4040
4020
4000
3980
3960
ELEVATION IN FEET
150' (MIN.)
SLIME TAILINGS (ML)
γt = 100 PCF
STATIC SEISMIC
Ø = 31° Ø = 21°
C = 0 C = 0
CREST AT ELEVATION 4076'
SAND TAILINGS (SM)
γt = 120 PCF
STATIC SEISMIC
Ø = 37° Ø = 37°
C = 0 C = 0
ASSUMED HIGHEST LINE OF SLIMES
ASSUMED HIGHEST LEVEL OF PHREATIC SURFACE
SLIME TAILINGS (ML)
γT = 120 PCF
STATIC SEISMIC
Ø = 31° Ø = 21°
C = 0 C = 0
NATURAL RED SANDY SOILS (SM-SP)
γT = 125 PCF
STATIC SEISMIC
Ø = 36° Ø = 36°
C = 0 C = 0
FEET
10 5 0 10 20 30 40 50

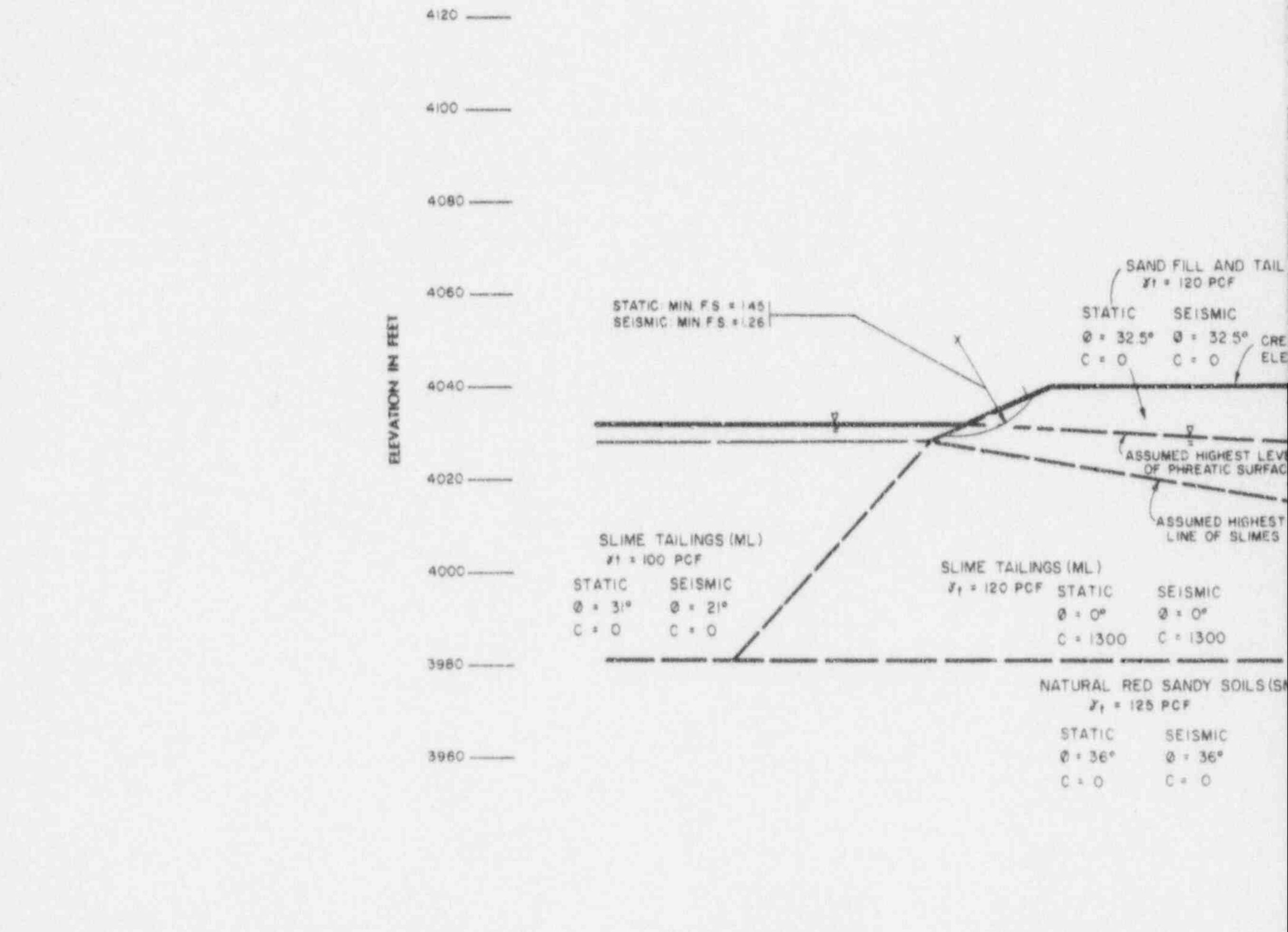


RESULTS OF SLOPE STABILITY ANALYSIS
SECTION A-A
THIRD ADDITIONAL RAISE

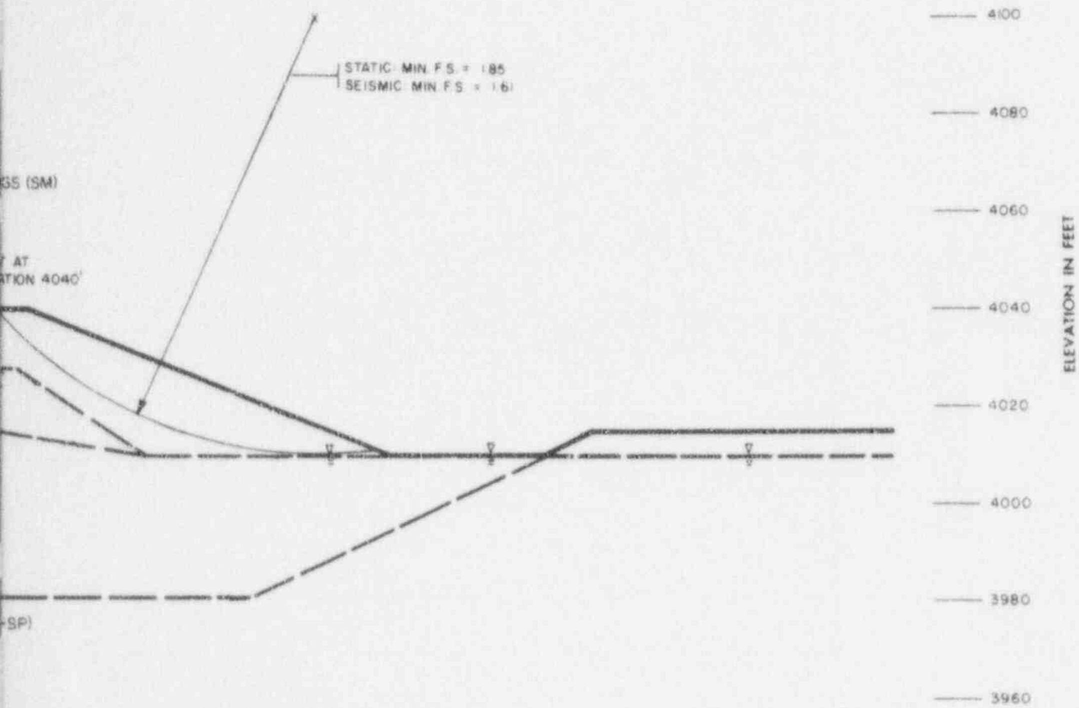
9705230158 - 06

DAMES & MOORE

PLATE 7



4120
4100
4080
4060
4040
4020
4000
3980
3960
ELEVATION IN FEET
STATIC: MIN. F.S. = 1.45
SEISMIC: MIN. F.S. = 1.26
SAND FILL AND TAIL
γt = 120 PCF
STATIC
SEISMIC
Ø = 32.5°
Ø = 32.5°
C = 0
C = 0
CRE
ELE
ASSUMED HIGHEST LEV
OF PHREATIC SURFAC
ASSUMED HIGHEST
LINE OF SLIMES
SLIME TAILINGS (ML)
γt = 100 PCF
STATIC
SEISMIC
Ø = 31°
Ø = 21°
C = 0
C = 0
SLIME TAILINGS (ML)
γt = 120 PCF
STATIC
SEISMIC
Ø = 0°
Ø = 0°
C = 1300
C = 1300
NATURAL RED SANDY SOILS (SM
γt = 125 PCF
STATIC
SEISMIC
Ø = 36°
Ø = 36°
C = 0
C = 0
FEET
10 5 0 10 20 30 40 50

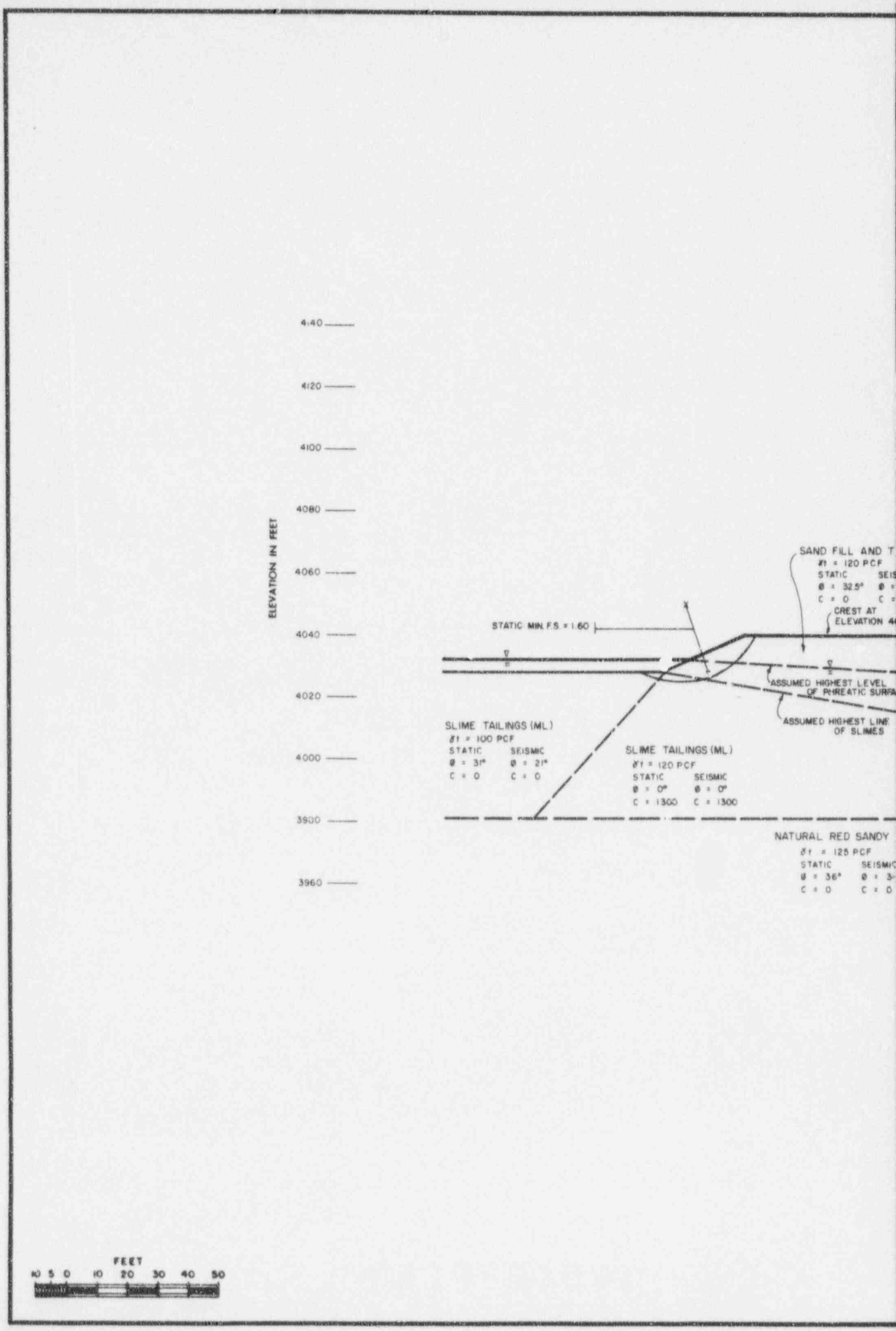


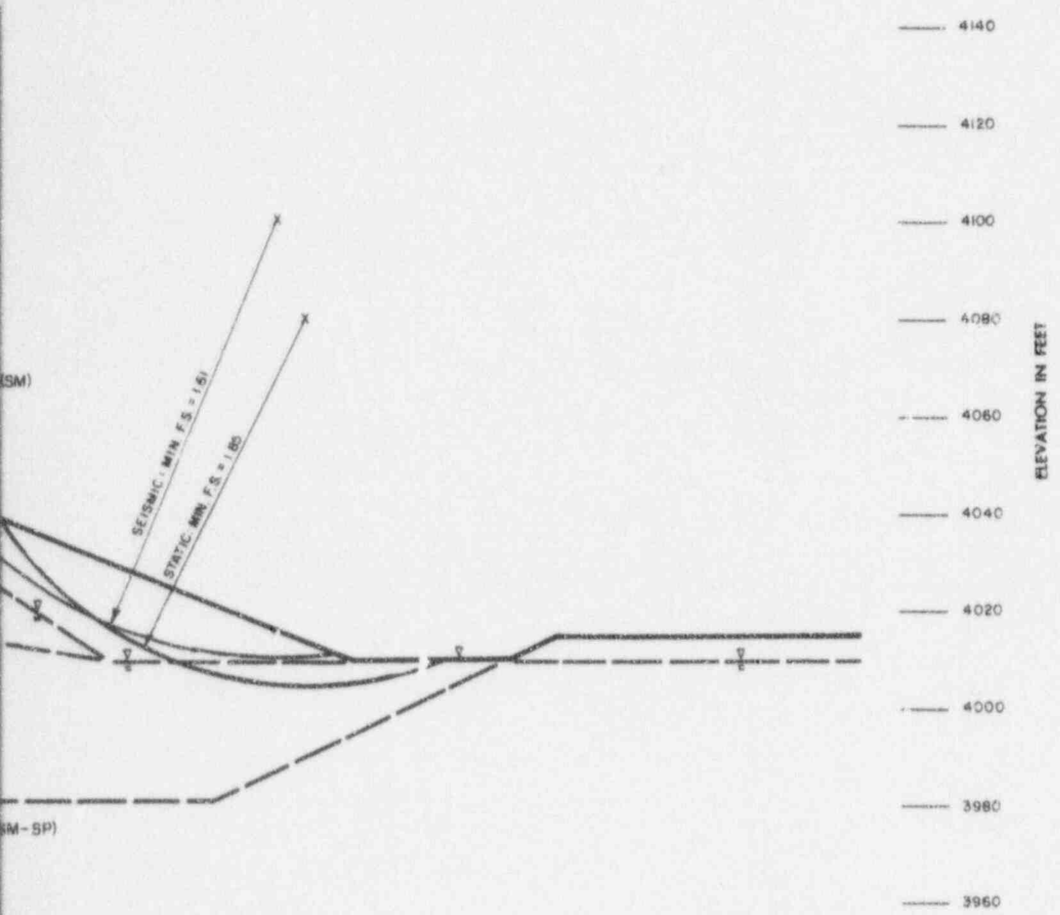
RESULTS OF SLOPE STABILITY ANALYSIS
SECTION C-C
EXISTING EMBANKMENT
END-OF-CONSTRUCTION CONDITIONS

9705230158 - 07

DAMES & MOORE

PLATE 8A


ELEVATION IN FEET
4140
4120
4100
4080
4060
4040
4020
4000
3980
3960
STATIC MIN. F.S. = 1.60
SAND FILL AND T
γt = 120 PCF
STATIC SEIS
Ø = 32.5° Ø =
C = 0 C =
CREST AT
ELEVATION 40
ASSUMED HIGHEST LEVEL
OF PHREATIC SURFA
ASSUMED HIGHEST LINE
OF SLIMES
SLIME TAILINGS (ML)
γt = 100 PCF
STATIC SEISMIC
Ø = 31° Ø = 21°
C = 0 C = 0
SLIME TAILINGS (ML)
γt = 120 PCF
STATIC SEISMIC
Ø = 0° Ø = 0°
C = 1300 C = 1300
NATURAL RED SANDY
γt = 125 PCF
STATIC SEISMIC
Ø = 36° Ø = 3
C = 0 C = 0
FEET
10 5 0 10 20 30 40 50



RESULTS OF SLOPE STABILITY ANALYSIS

SECTION C-C

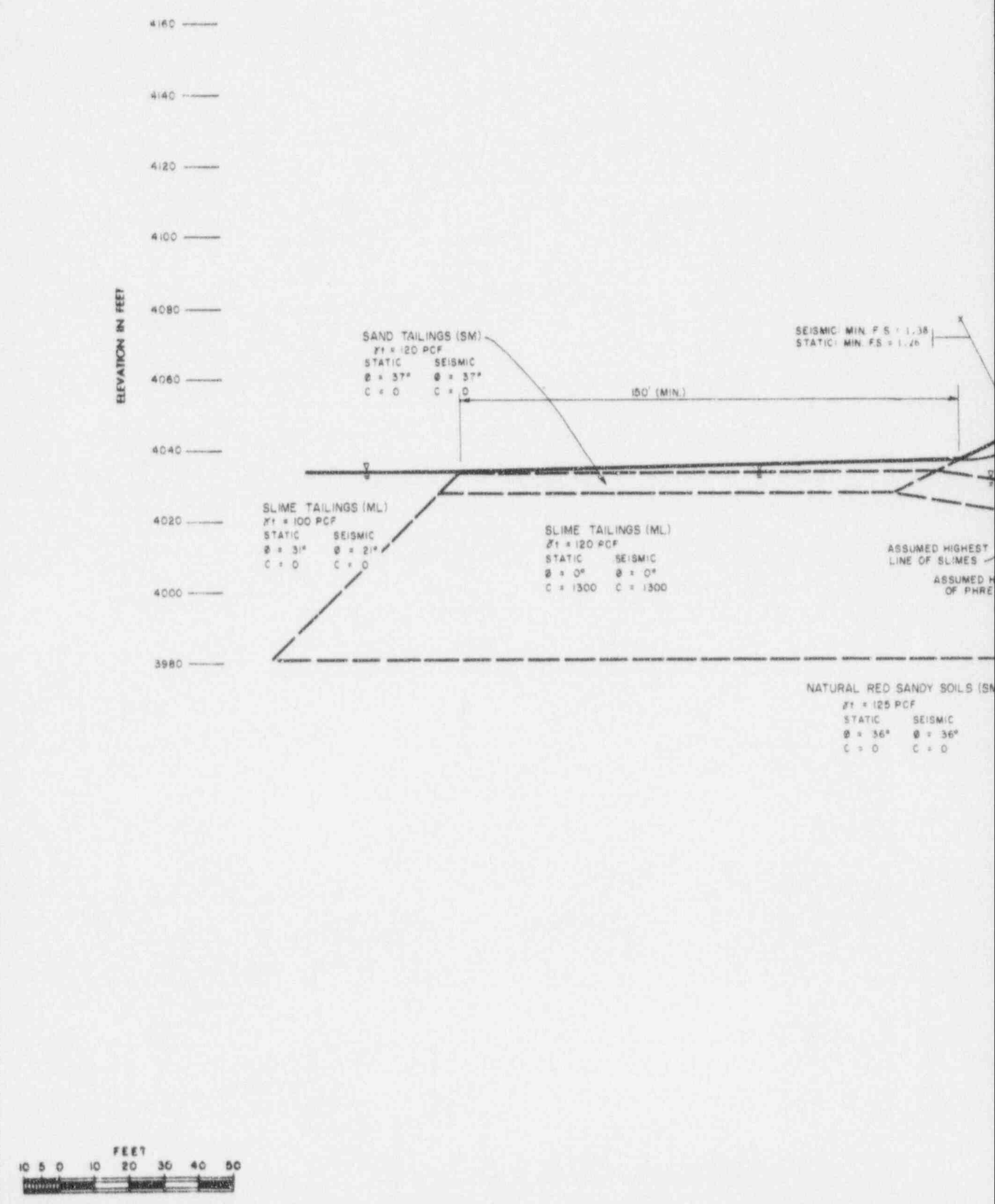
EXISTING EMBANKMENT

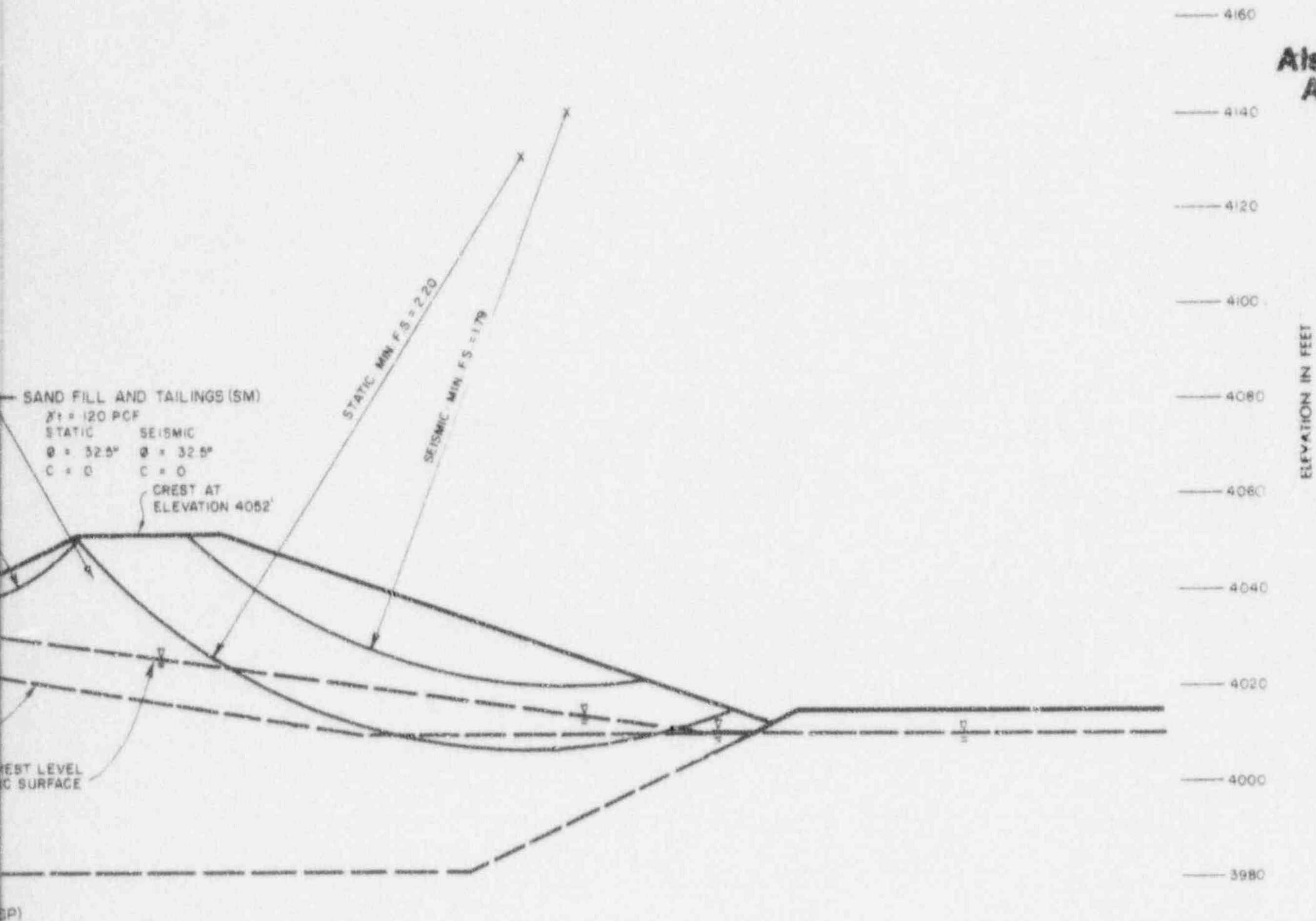
STEADY-STATE SEEPAGE CONDITIONS

9705230158 -08

DAMES & MOORE

PLATE 88


ELEVATION IN FEET
4160
4140
4120
4100
4080
4060
4040
4020
4000
3980
SAND TAILINGS (SM)
γt = 120 PCF
STATIC SEISMIC
Ø = 37° Ø = 37°
C = 0 C = 0
150' (MIN.)
SEISMIC: MIN. F.S. = 1.38
STATIC: MIN. F.S. = 1.26
SLIME TAILINGS (ML)
γt = 100 PCF
STATIC SEISMIC
Ø = 31° Ø = 21°
C = 0 C = 0
SLIME TAILINGS (ML)
γt = 120 PCF
STATIC SEISMIC
Ø = 0° Ø = 0°
C = 1300 C = 1300
ASSUMED HIGHEST
LINE OF SLIMES
ASSUMED H
OF PHRE
NATURAL RED SANDY SOILS (SM
γt = 125 PCF
STATIC SEISMIC
Ø = 36° Ø = 36°
C = 0 C = 0
FEET
10 5 0 10 20 30 40 50

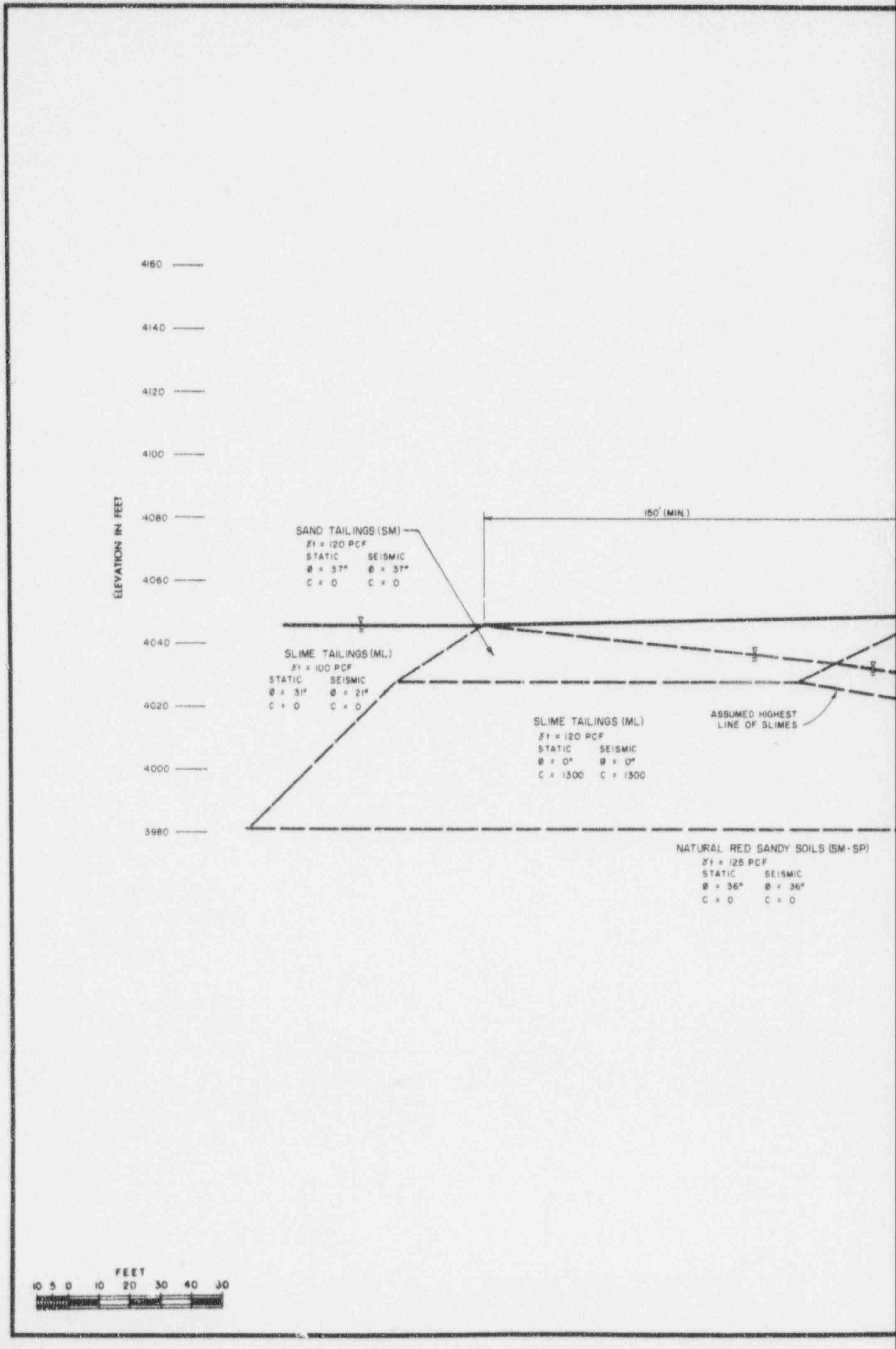


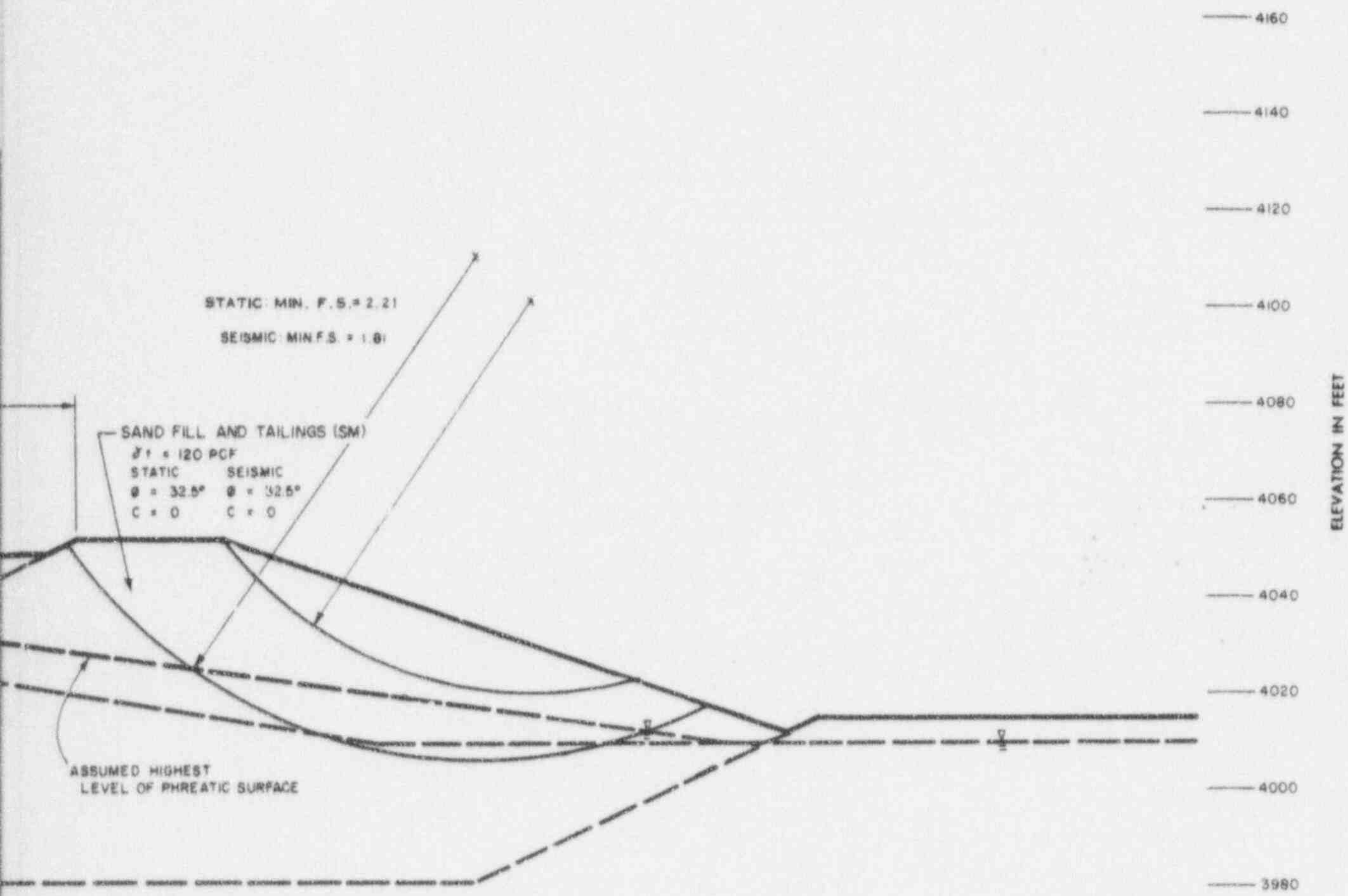
RESULTS OF SLOPE STABILITY ANALYSIS
SECTION C - C
FIRST ADDITIONAL RAISE
END-OF-CONSTRUCTION CONDITIONS

9705230158 - 09

DAMES & MOORE

PLATE ~~8C~~


4160
4140
4120
4100
4080
4060
4040
4020
4000
3980
ELEVATION IN FEET
150' (MIN.)
SAND TAILINGS (SM)
γt = 120 PCF
STATIC SEISMIC
Ø = 37° Ø = 37°
C = 0 C = 0
SLIME TAILINGS (ML)
γt = 100 PCF
STATIC SEISMIC
Ø = 31° Ø = 21°
C = 0 C = 0
SLIME TAILINGS (ML)
γt = 120 PCF
STATIC SEISMIC
Ø = 0° Ø = 0°
C = 1300 C = 1300
ASSUMED HIGHEST LINE OF SLIMES
NATURAL RED SANDY SOILS (SM-SP)
γt = 125 PCF
STATIC SEISMIC
Ø = 36° Ø = 36°
C = 0 C = 0
FEET
10 5 0 10 20 30 40 50



RESULTS OF SLOPE STABILITY ANALYSIS

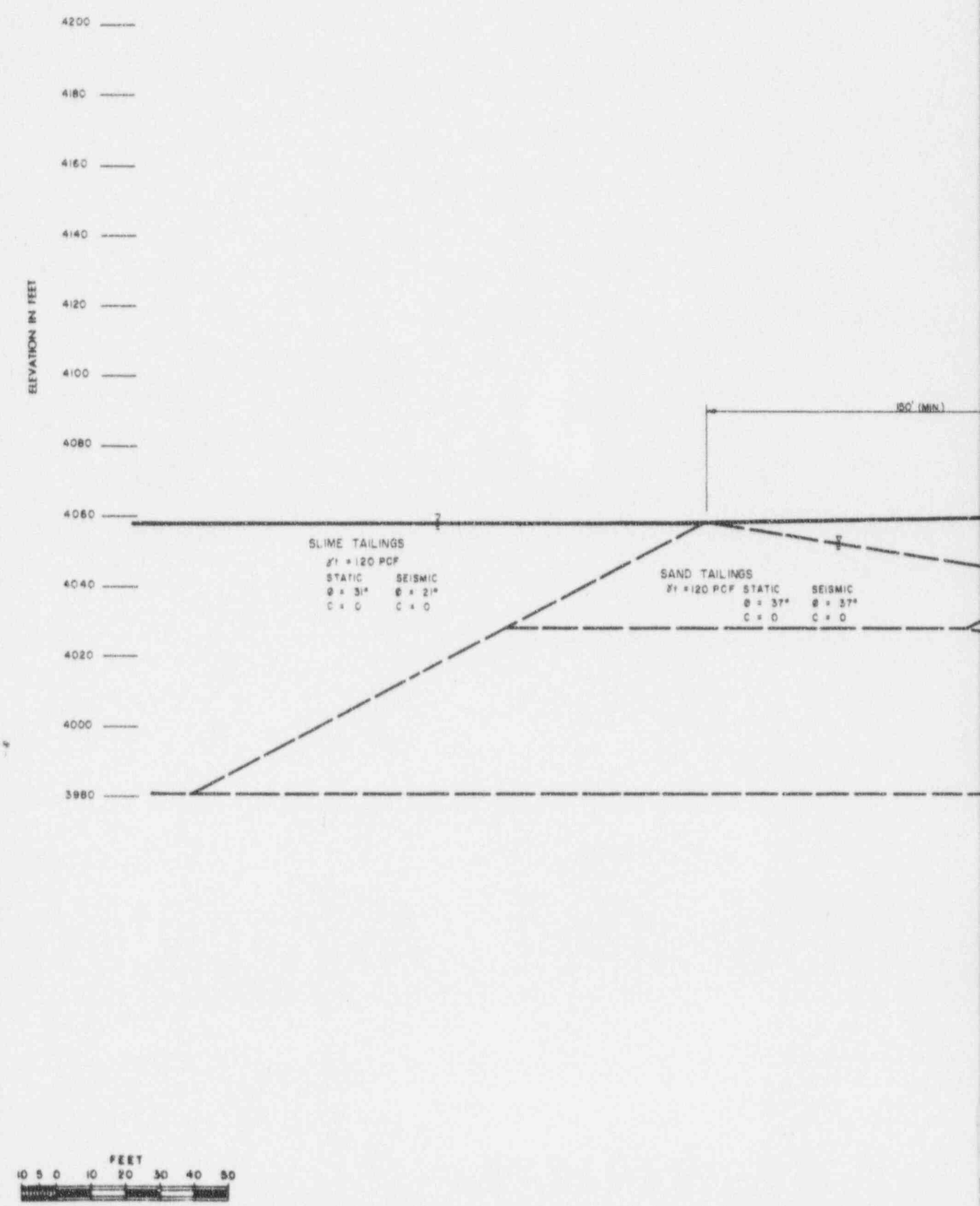
SECTION C-C

FIRST ADDITIONAL RAISE

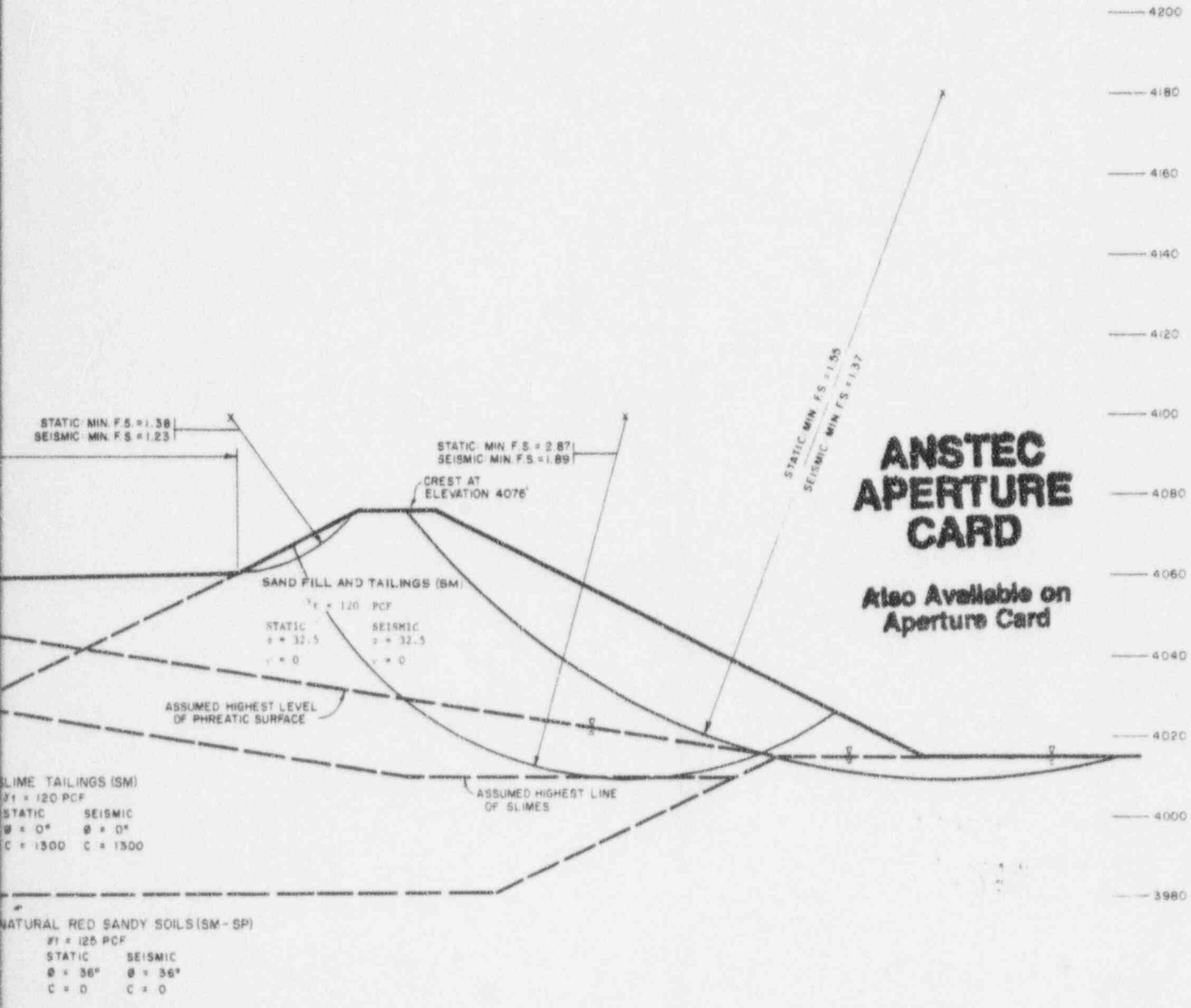
STEADY-STATE SEEPAGE CONDITIONS

9705230158 -10

DAMES & MOORE



ELEVATION IN FEET
4200
4180
4160
4140
4120
4100
4080
4060
4040
4020
4000
3980
150' (MIN.)
SLIME TAILINGS
γt = 120 PCF
STATIC
SEISMIC
Ø = 31°
Ø = 21°
C = 0
C = 0
SAND TAILINGS
γt = 120 PCF
STATIC
SEISMIC
Ø = 37°
Ø = 37°
C = 0
C = 0
FEET
10 5 0 10 20 30 40 50



ANSTEC APERTURE CARD

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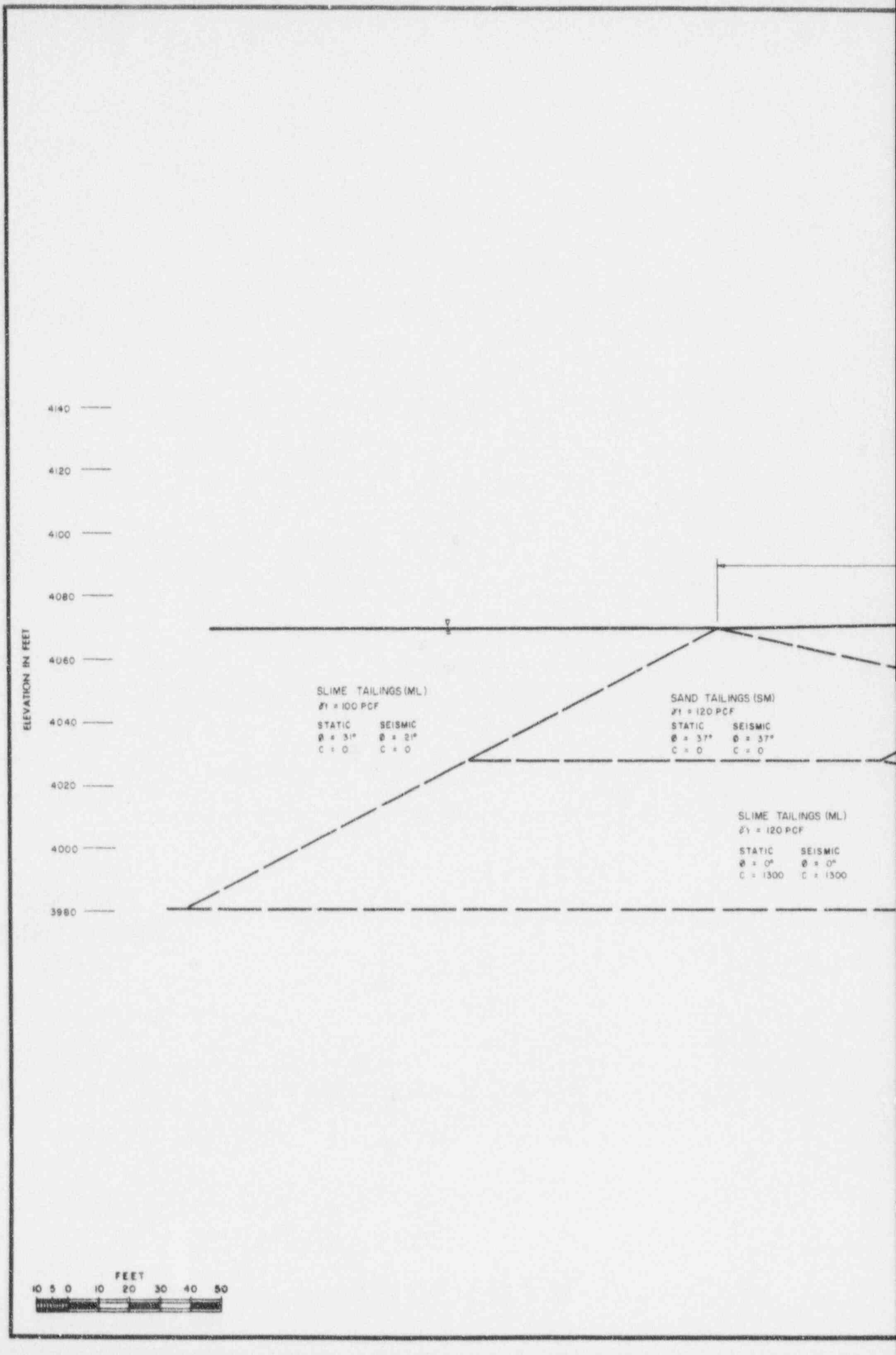
RESULTS OF SLOPE STABILITY ANALYSIS
SECTION C-C
THIRD ADDITIONAL RAISE
END-OF-CONSTRUCTION CONDITIONS

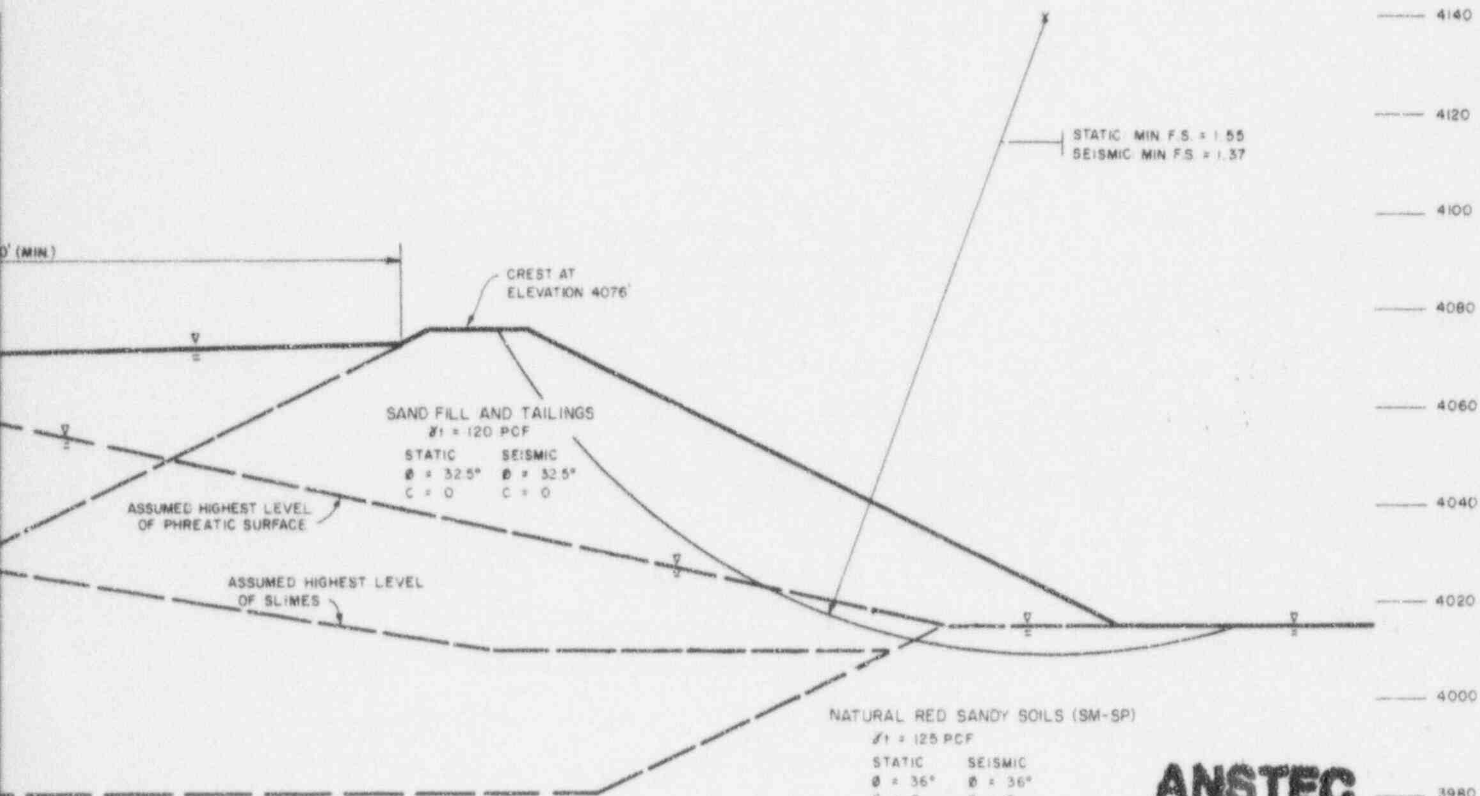
9705230158 - 11

DAMES & MOORE

PLATE 8E



ELEVATION IN FEET
4140
4120
4100
4080
4060
4040
4020
4000
3980
SLIME TAILINGS (ML)
γ_t = 100 PCF
STATIC
ϕ = 31°
C = 0
SEISMIC
ϕ = 21°
C = 0
SAND TAILINGS (SM)
γ_t = 120 PCF
STATIC
ϕ = 37°
C = 0
SEISMIC
ϕ = 37°
C = 0
SLIME TAILINGS (ML)
γ_t = 120 PCF
STATIC
ϕ = 0°
C = 1300
SEISMIC
ϕ = 0°
C = 1300
FEET
10 5 0 10 20 30 40 50



ANSTEC APERTURE CARD

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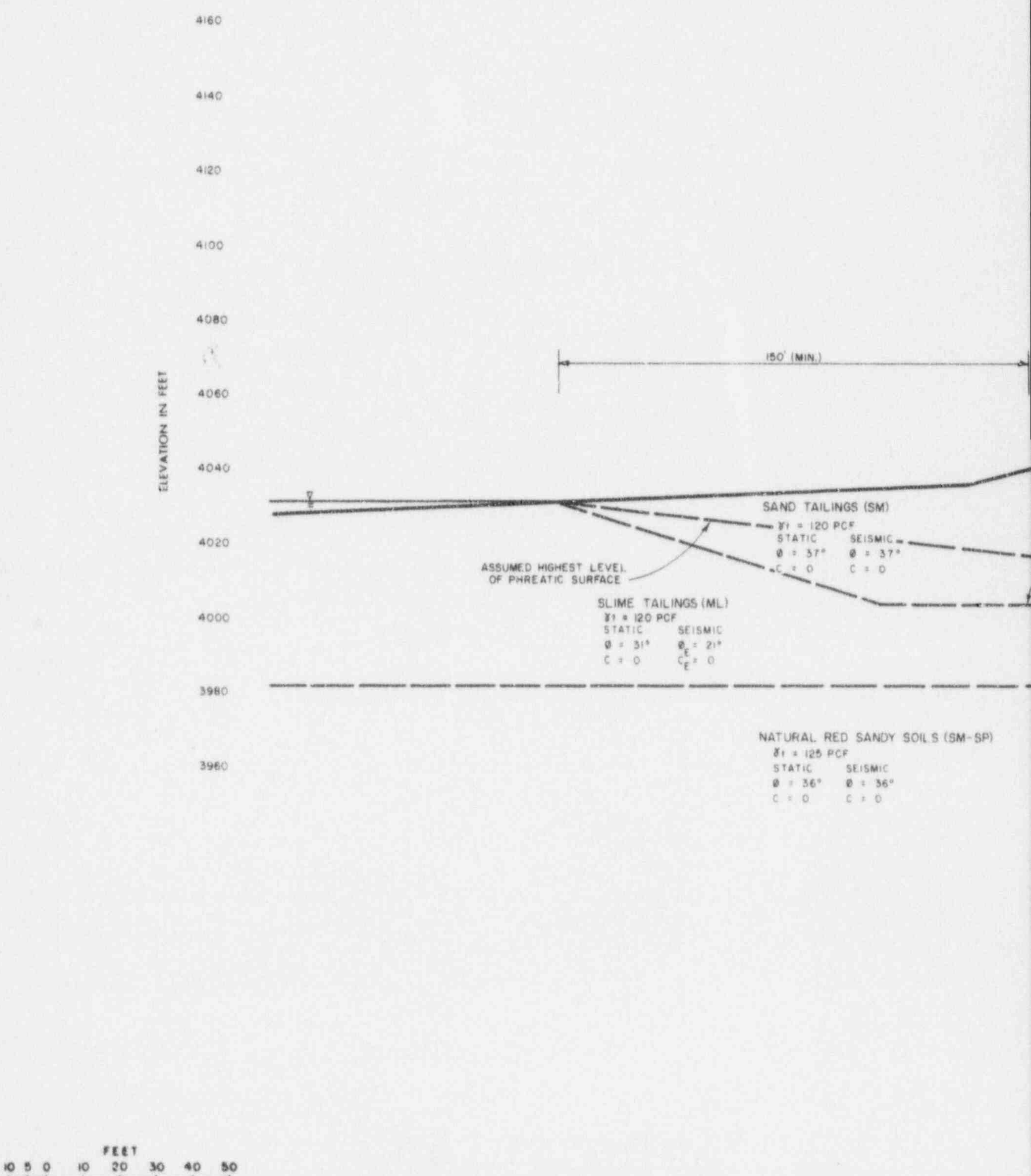
RESULTS OF SLOPE STABILITY ANALYSIS
SECTION C-C
THIRD ADDITIONAL RAISE
STEADY-STATE SEEPAGE CONDITIONS

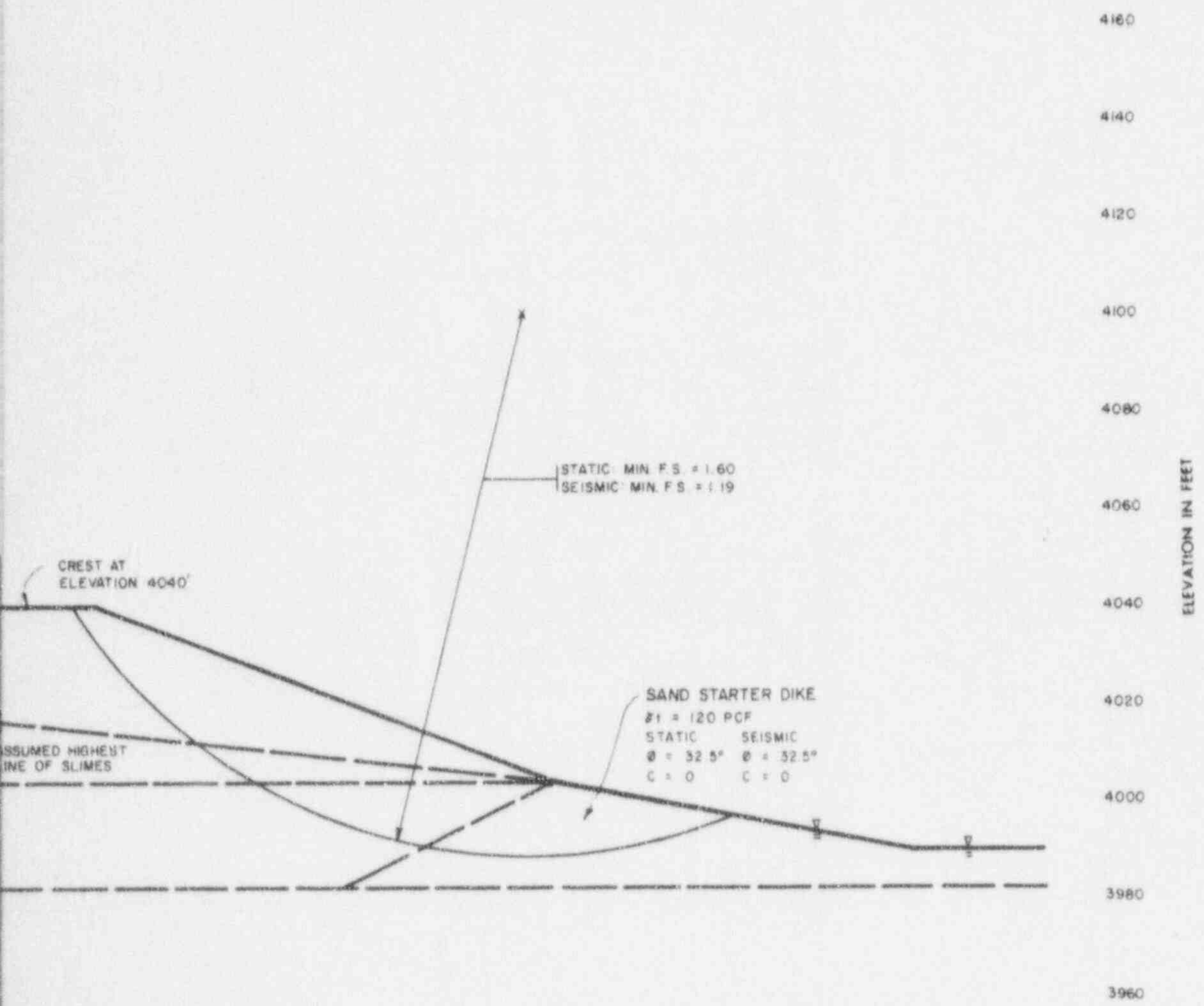
9705230158 - 12

DAMES & MOORE

PLATE 8F



ELEVATION IN FEET
4160
4140
4120
4100
4080
4060
4040
4020
4000
3980
3960
150' (MIN.)
SAND TAILINGS (SM)
γ_t = 120 PCF
STATIC SEISMIC
Ø = 37° Ø = 37°
C = 0 C = 0
ASSUMED HIGHEST LEVEL OF PHREATIC SURFACE
SLIME TAILINGS (ML)
γ_t = 120 PCF
STATIC SEISMIC
Ø = 31° $Ø_E$ = 21°
C = 0 C_E = 0
NATURAL RED SANDY SOILS (SM-SP)
γ_t = 125 PCF
STATIC SEISMIC
Ø = 36° Ø = 36°
C = 0 C = 0
FEET
10 5 0 10 20 30 40 50

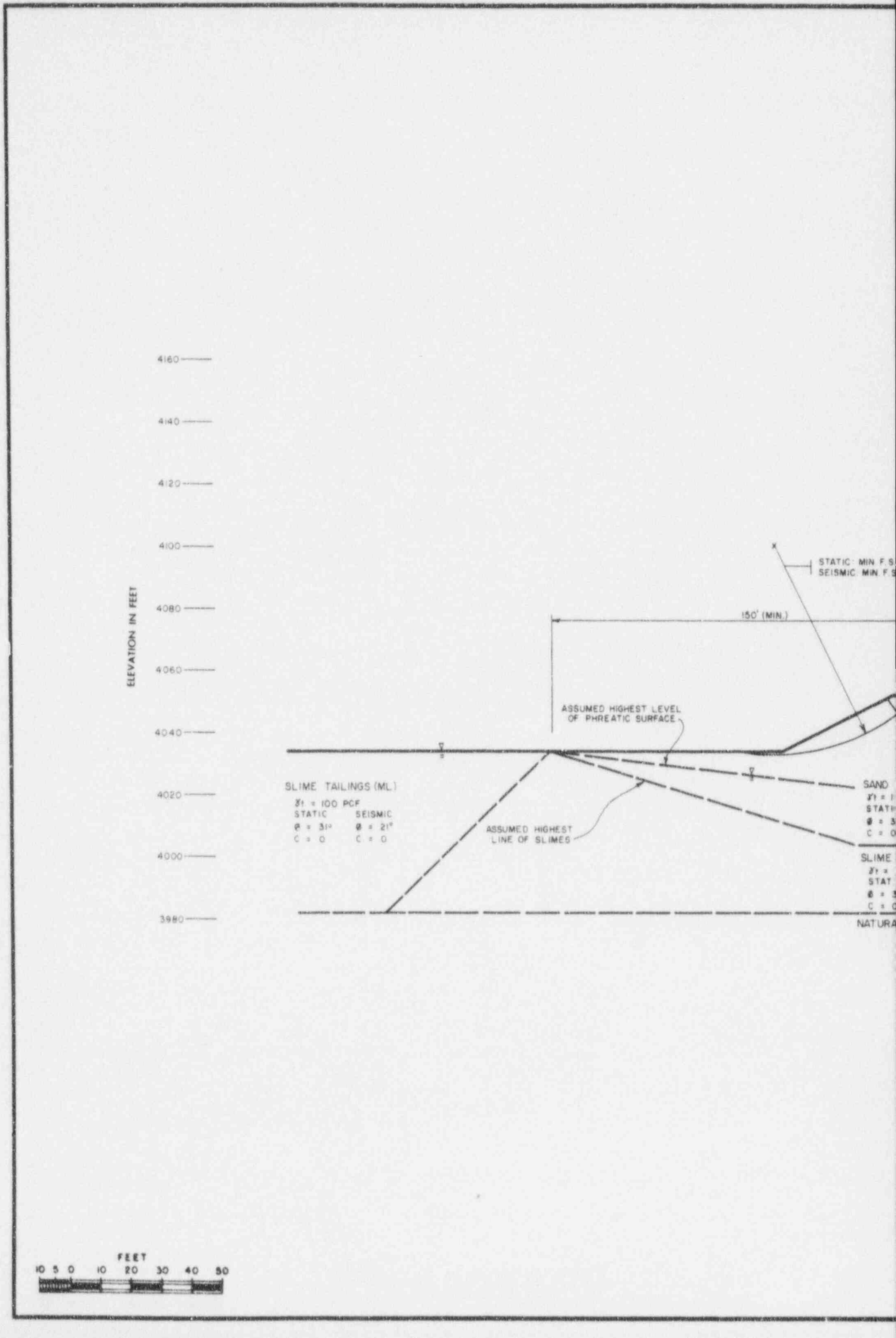


RESULTS OF SLOPE STABILITY ANALYSIS
SECTION E-E
EXISTING EMBANKMENT
STEADY-STATE SEEPAGE CONDITIONS

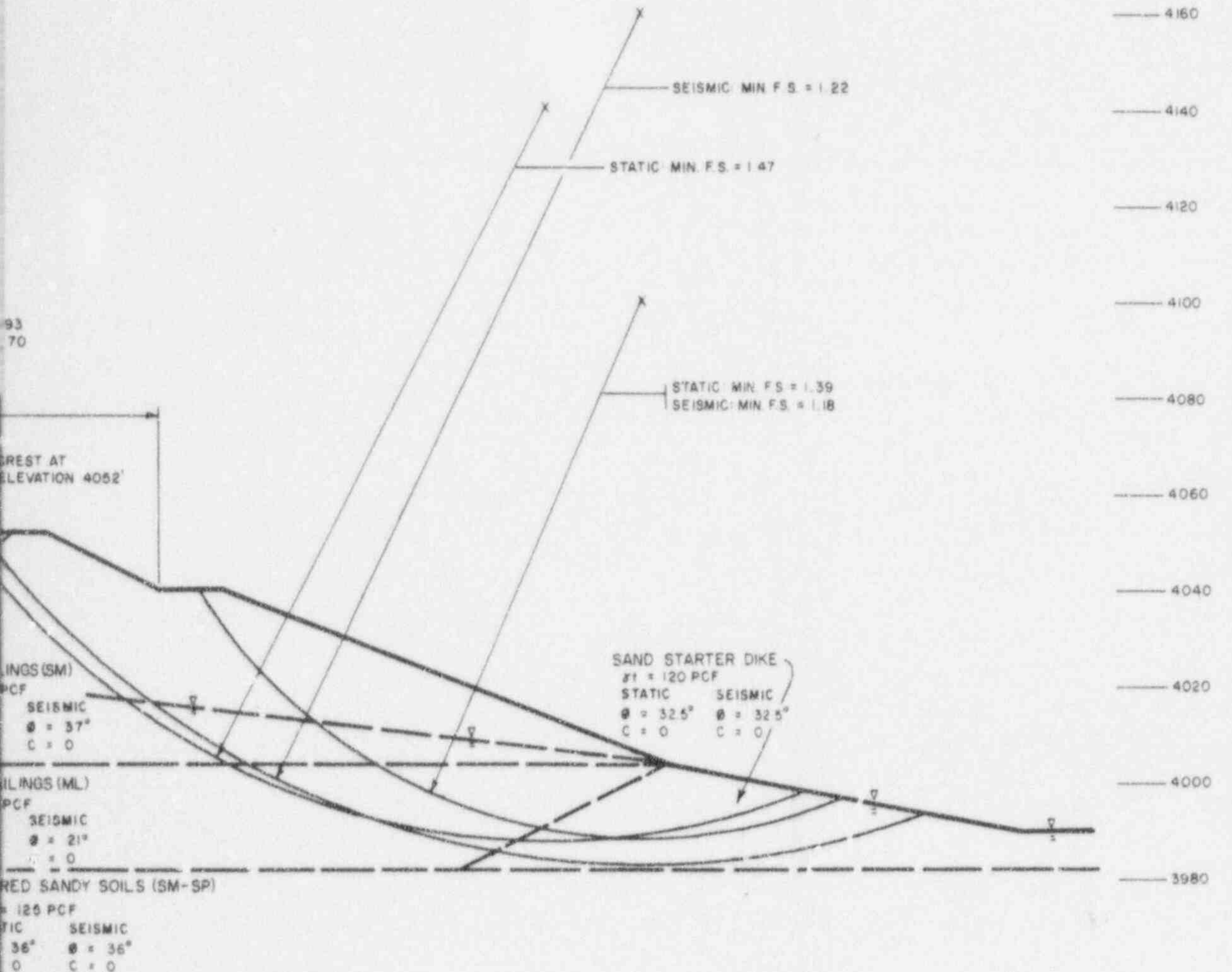
9705230158 - 13

DAMES & MOORE

PLATE 9A



ELEVATION IN FEET
4160
4140
4120
4100
4080
4060
4040
4020
4000
3980
STATIC: MIN F.S
SEISMIC: MIN F.S
150' (MIN.)
ASSUMED HIGHEST LEVEL
OF PHREATIC SURFACE
SLIME TAILINGS (ML)
γ_t = 100 PCF
STATIC
ϕ = 31°
C = 0
SEISMIC
ϕ = 21°
C = 0
ASSUMED HIGHEST
LINE OF SLIMES
SAND
SLIME
NATURA
FEET
10 5 0 10 20 30 40 50



ANSTEC APERTURE CARD

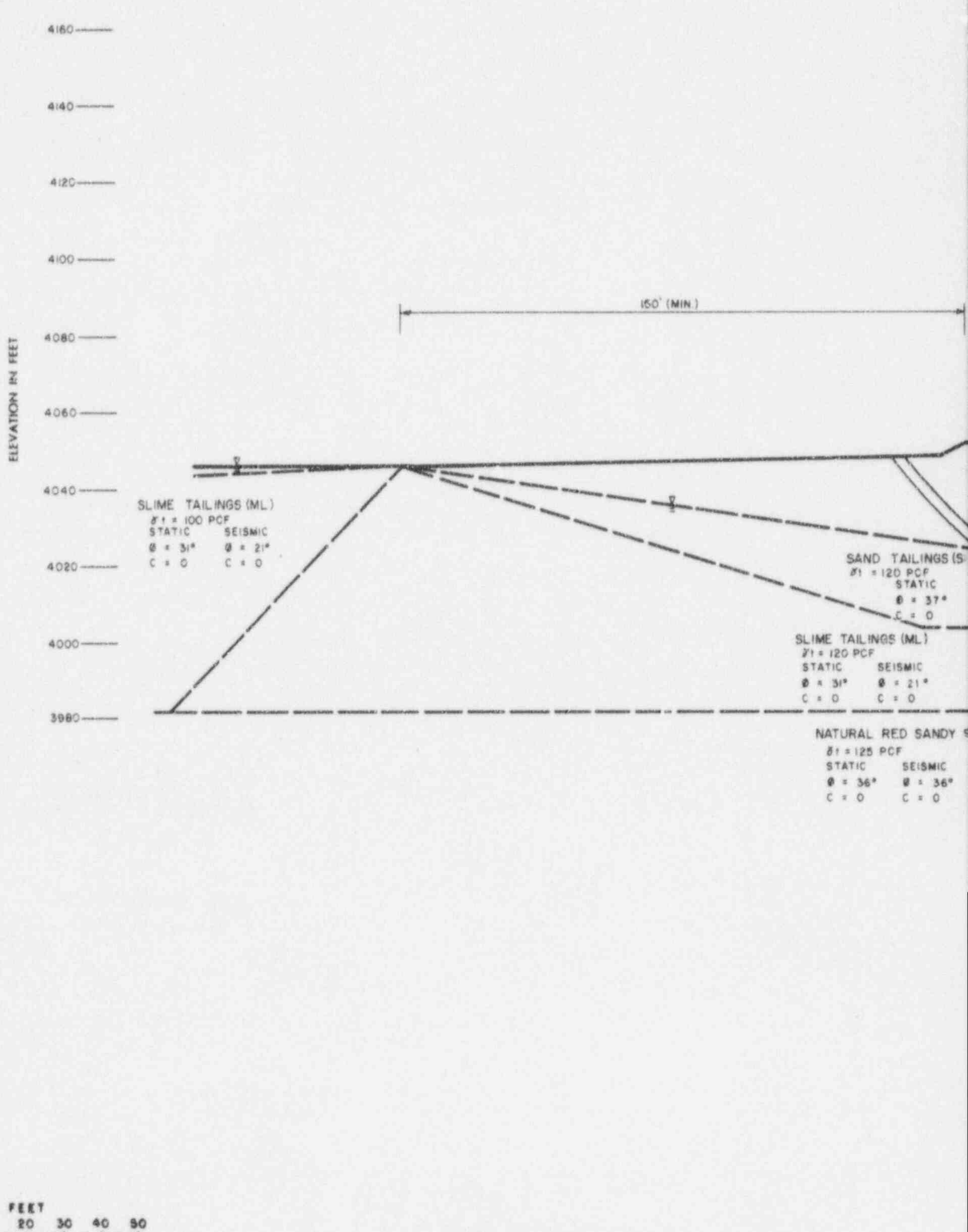
Also Available on Aperture Card

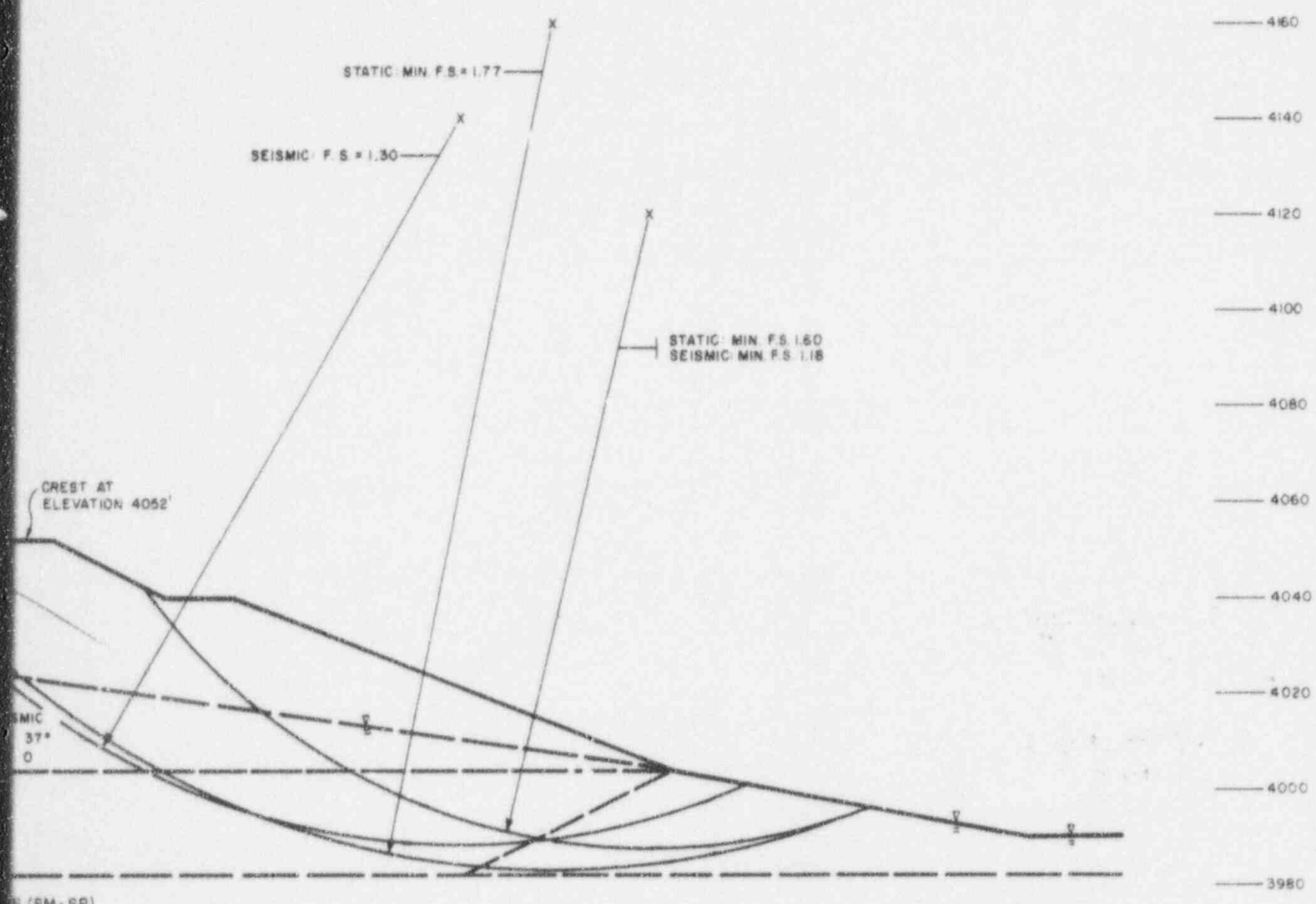
RESULTS OF SLOPE STABILITY ANALYSIS
SECTION E-E
FIRST ADDITIONAL RAISE
END-OF-CONSTRUCTION CONDITIONS

9705230158 - 14

DAMES & MOORE

PLATE 9B


ELEVATION IN FEET
4160
4140
4120
4100
4080
4060
4040
4020
4000
3980
150' (MIN.)
SLIME TAILINGS (ML)
γt = 100 PCF
STATIC SEISMIC
Ø = 31° Ø = 21°
C = 0 C = 0
SAND TAILINGS (S
γt = 120 PCF
STATIC
Ø = 37°
C = 0
SLIME TAILINGS (ML)
γt = 120 PCF
STATIC SEISMIC
Ø = 31° Ø = 21°
C = 0 C = 0
NATURAL RED SANDY S
γt = 125 PCF
STATIC SEISMIC
Ø = 36° Ø = 36°
C = 0 C = 0
FEET
10 5 0 10 20 30 40 50



ANSTEC APERTURE CARD

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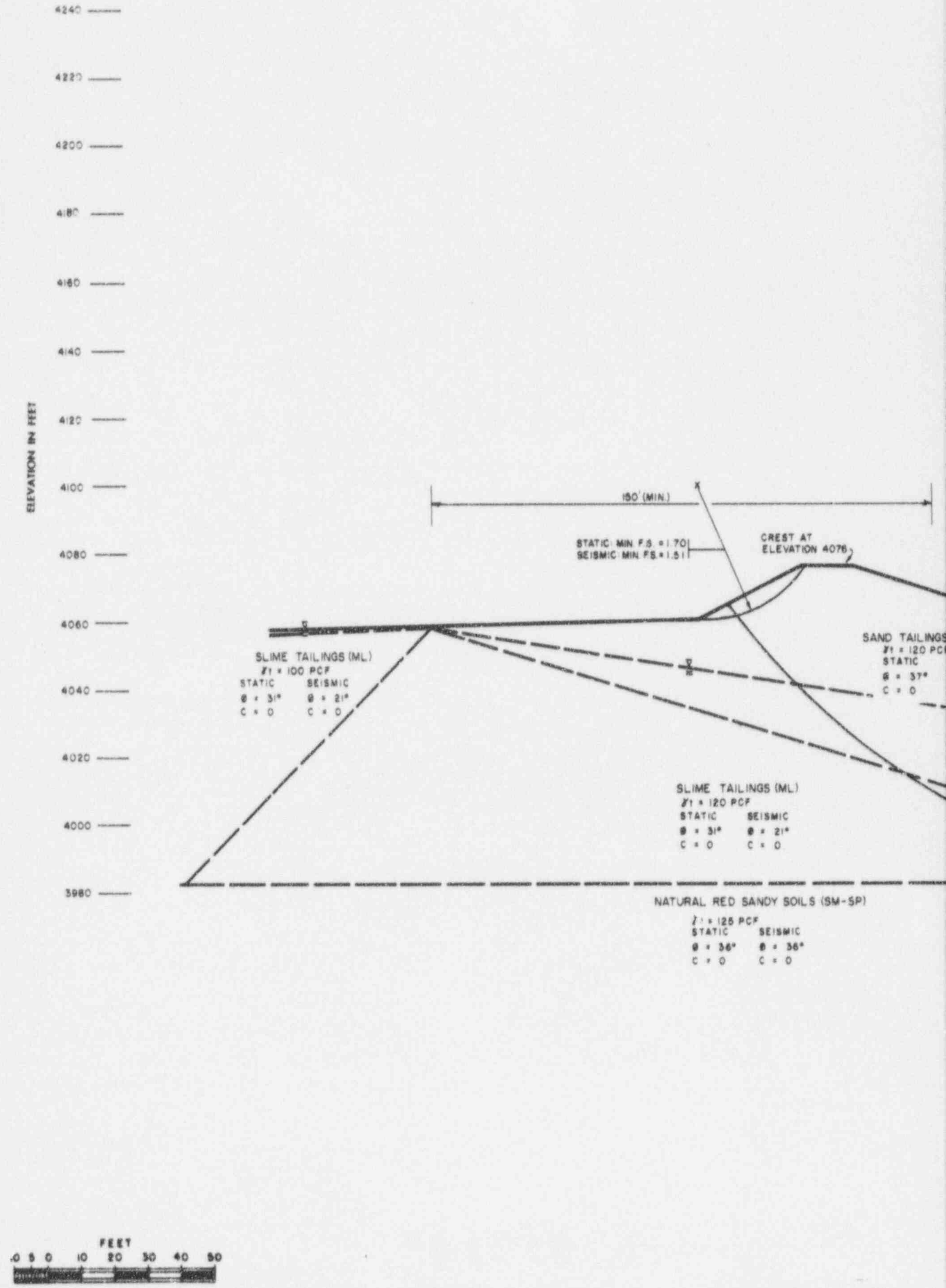
RESULTS OF SLOPE STABILITY ANALYSIS
SECTION E-E
FIRST ADDITIONAL RAISE
STEADY-STATE SEEPAGE CONDITIONS

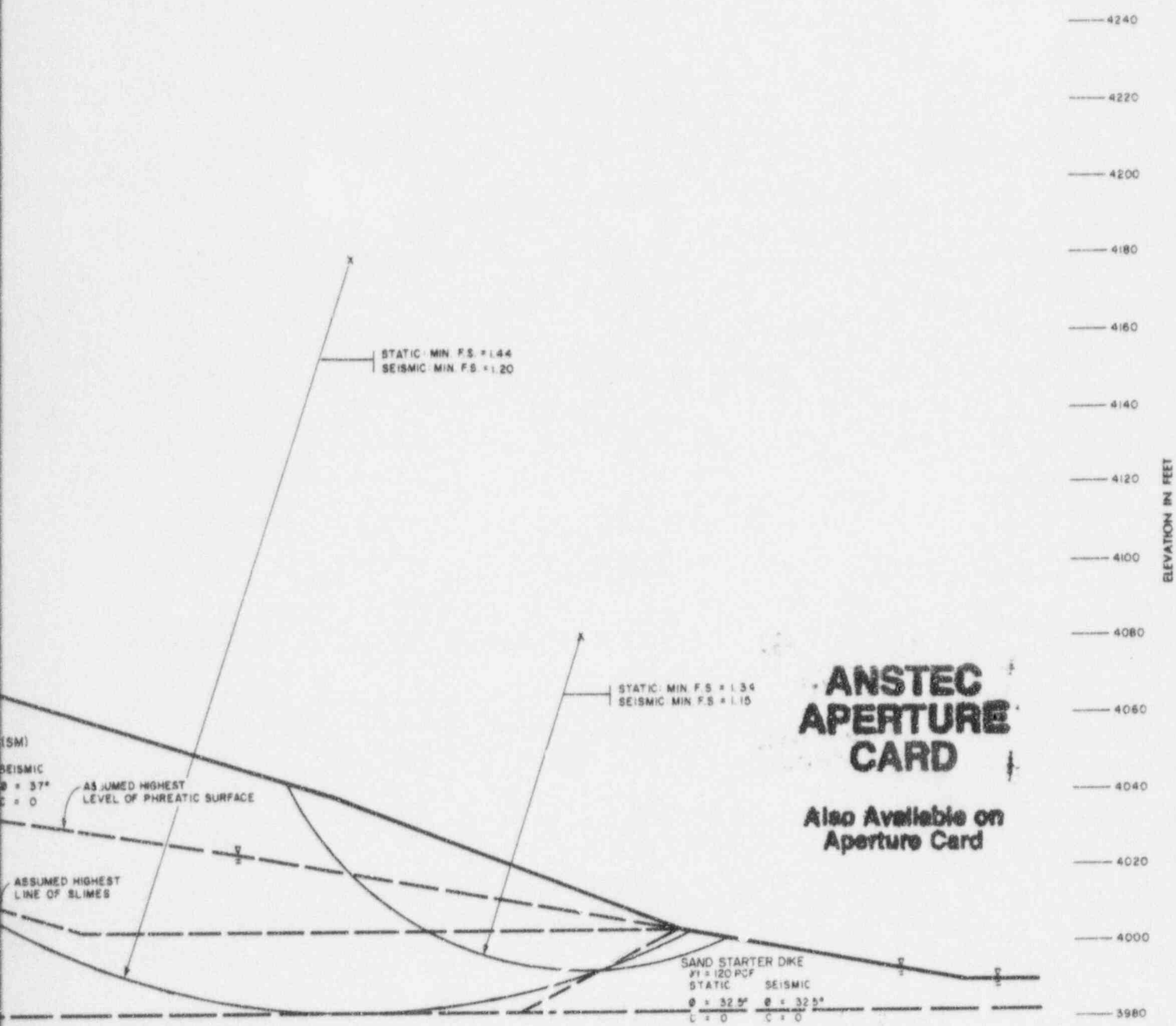
9705280158 - 15

DAMES & MOORE

PLATE 9C



ATLAS MINERALS
BY F. S. YOUNG DATE 9-29-77
CHECKED BY DATE

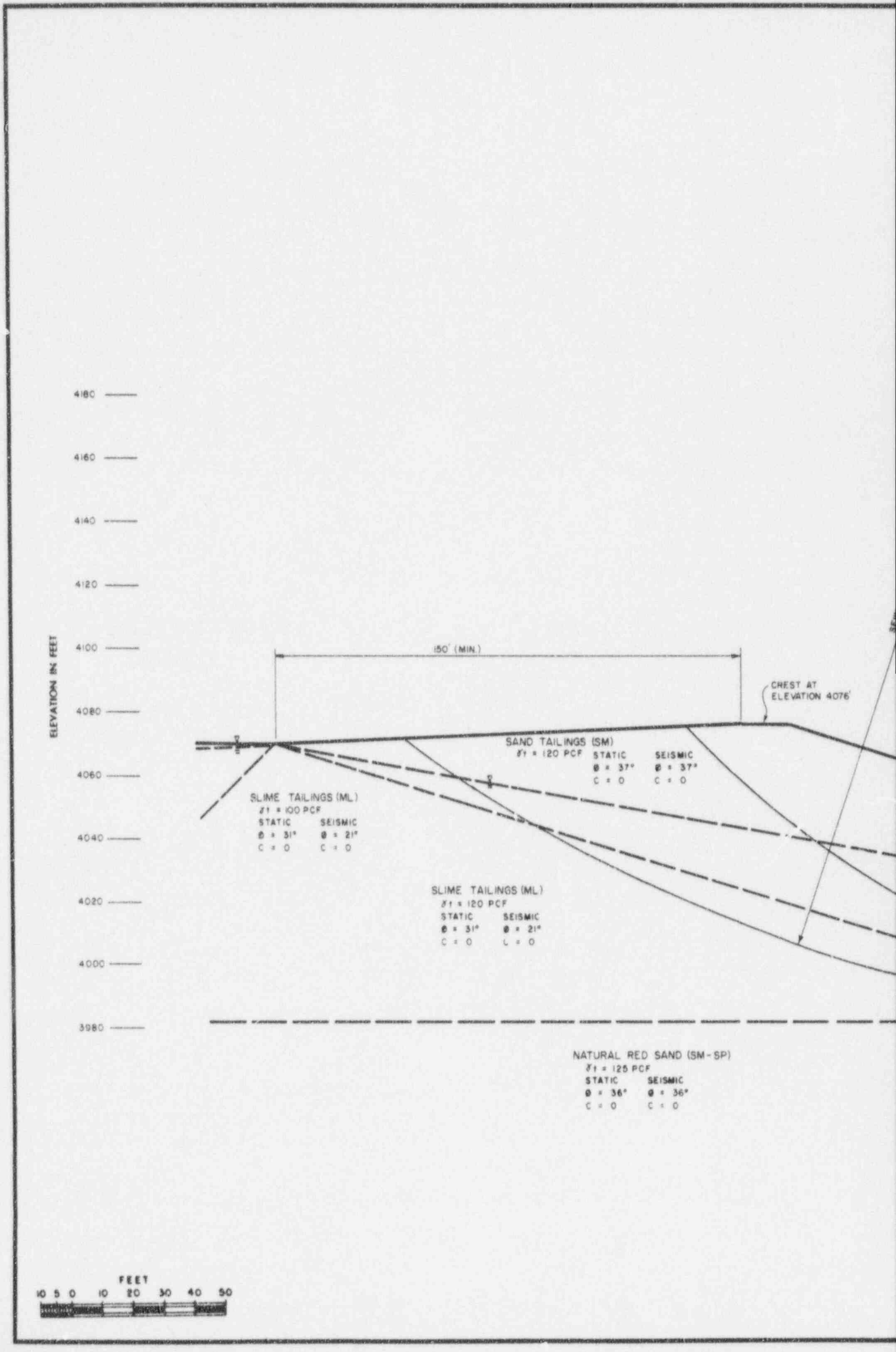


RESULTS OF SLOPE STABILITY ANALYSIS
SECTION E-E
THIRD ADDITIONAL RAISE
END-OF-CONSTRUCTION CONDITIONS

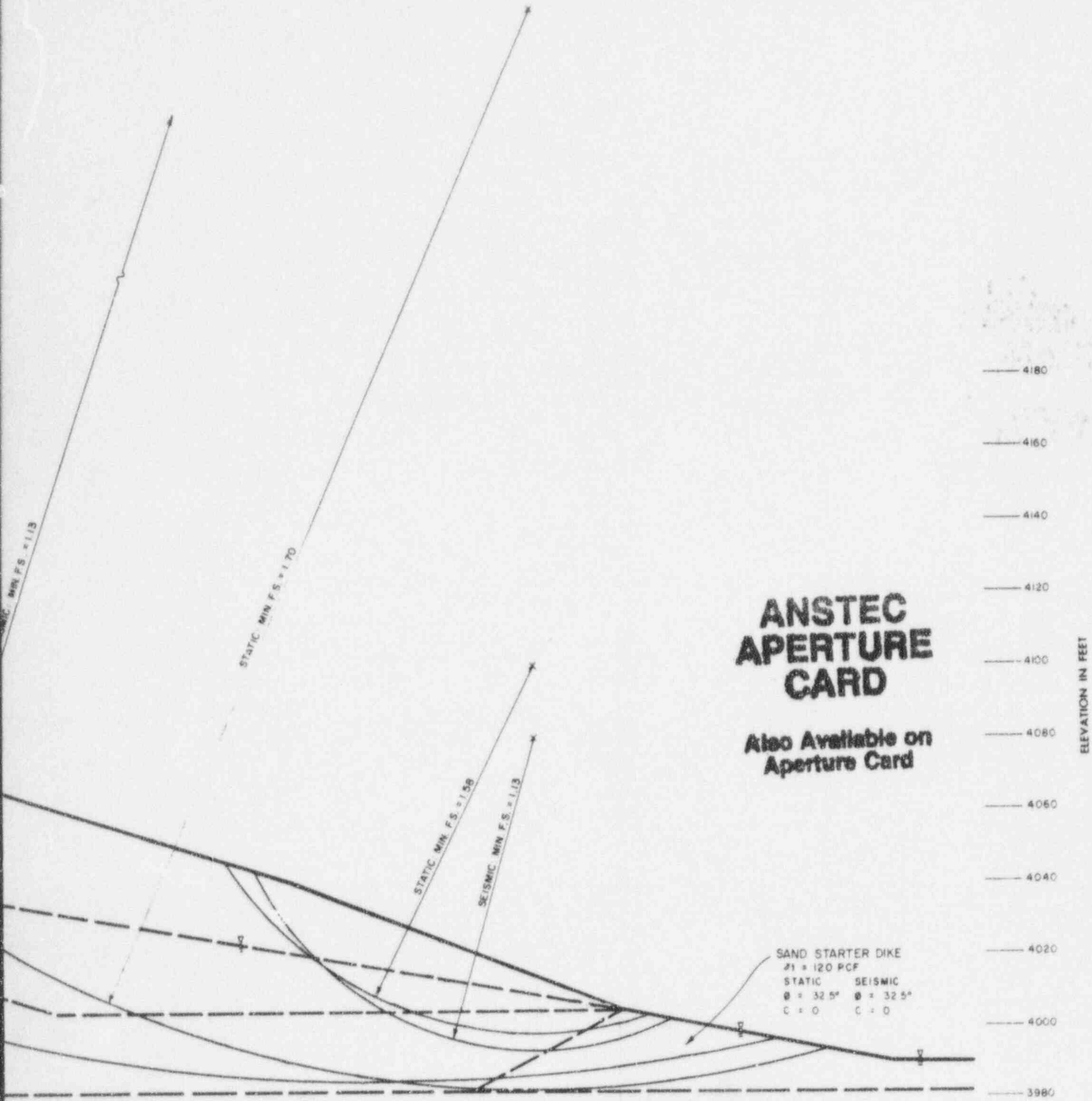
9705230158 - 16

DAMES & MOORE

PLATE 90



4180
4160
4140
4120
4100
4080
4060
4040
4020
4000
3980
ELEVATION IN FEET
150' (MIN.)
CREST AT ELEVATION 4076'
SAND TAILINGS (SM)
γt = 120 PCF
STATIC
Ø = 37°
C = 0
SEISMIC
Ø = 37°
C = 0
SLIME TAILINGS (ML)
γt = 100 PCF
STATIC
Ø = 31°
C = 0
SEISMIC
Ø = 21°
C = 0
SLIME TAILINGS (ML)
γt = 120 PCF
STATIC
Ø = 31°
C = 0
SEISMIC
Ø = 21°
C = 0
NATURAL RED SAND (SM-SP)
γt = 125 PCF
STATIC
Ø = 36°
C = 0
SEISMIC
Ø = 36°
C = 0
FEET
10 5 0 10 20 30 40 50

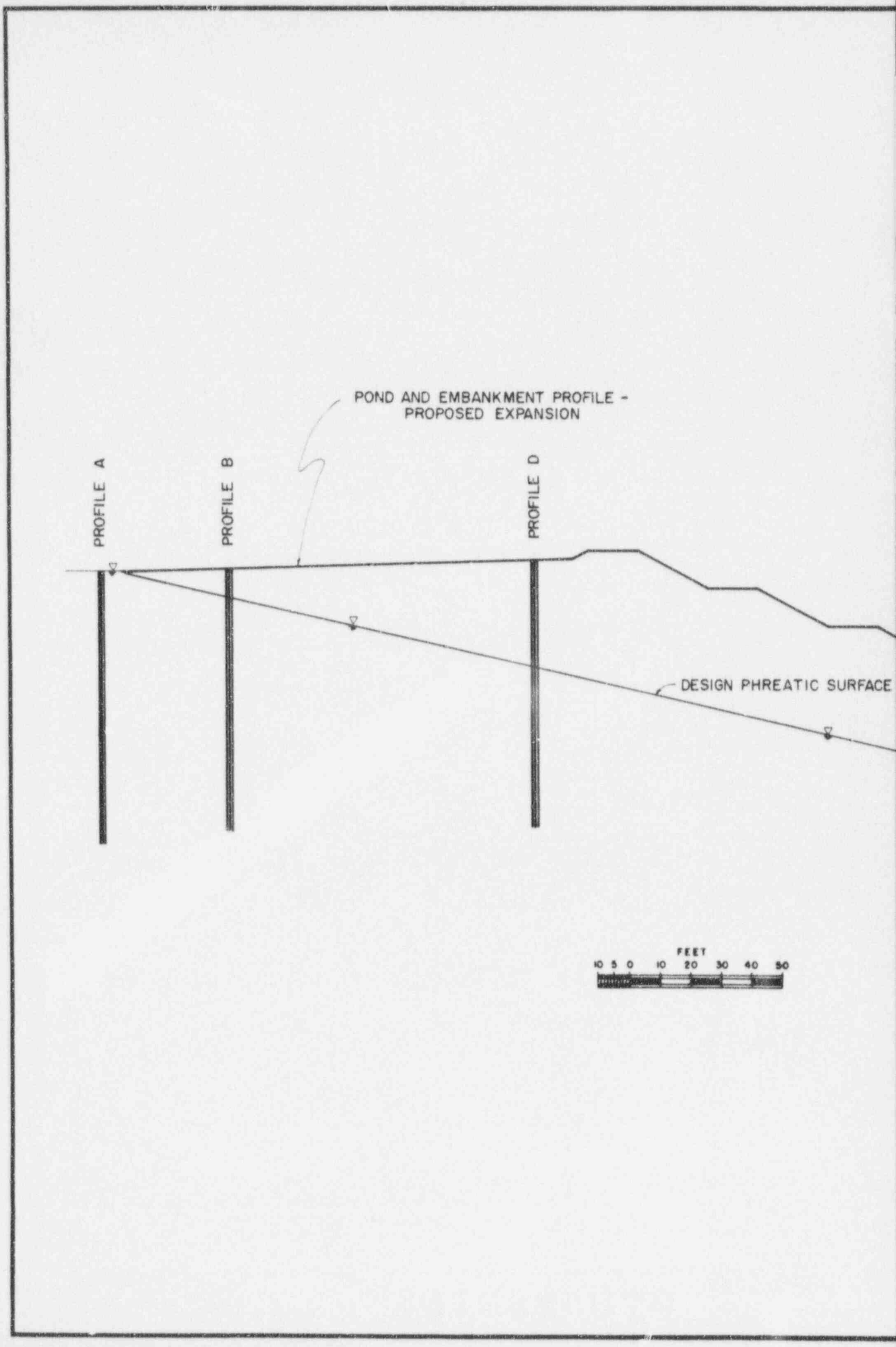


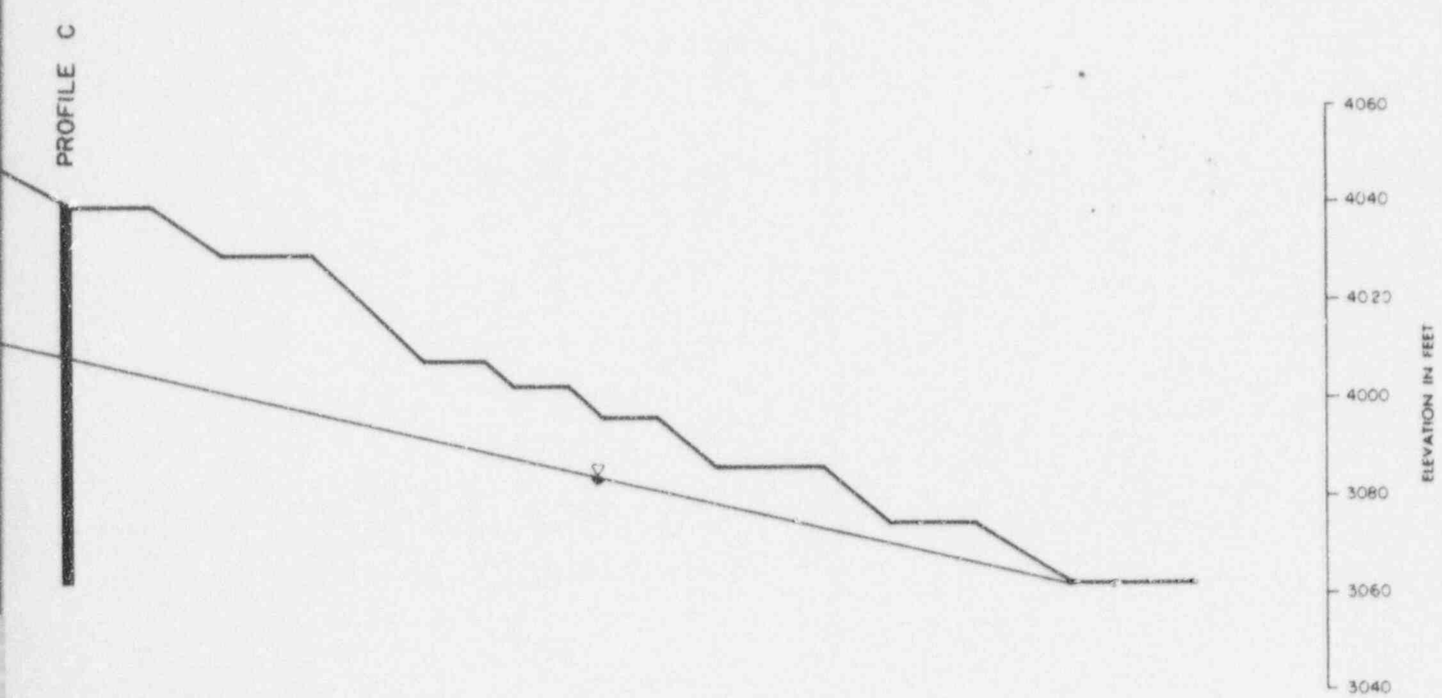
RESULTS OF SLOPE STABILITY ANALYSIS
SECTION E-E
THIRD ADDITIONAL RAISE
STEADY-STATE SEEPAGE CONDITIONS

9705280158 - 17

DAMES & MOORE

PLATE 9E


POND AND EMBANKMENT PROFILE -
PROPOSED EXPANSION
PROFILE A
PROFILE B
PROFILE D
DESIGN PHREATIC SURFACE
FEET
10 5 0 10 20 30 40 50



TYPICAL EMBANKMENT SECTION SHOWING
TAILINGS PROFILES CONSIDERED IN
LIQUEFACTION ANALYSES

9705230158 - 18

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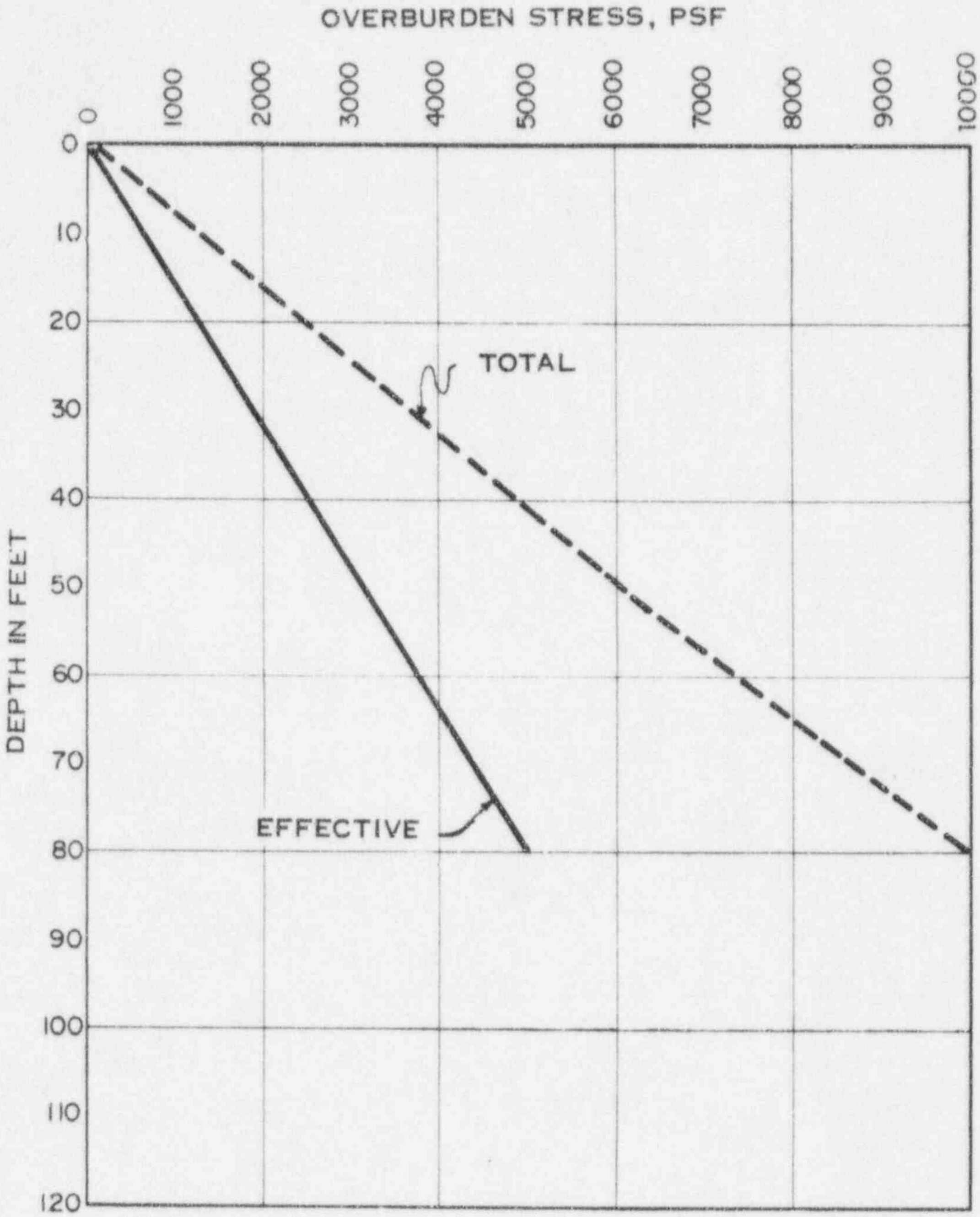
PLATE 10



OVERBURDEN STRESSES
VERSUS DEPTH OF TAILINGS
(PROFILE A)

DAMES & MOORE

PLATE 11A

BY DATE
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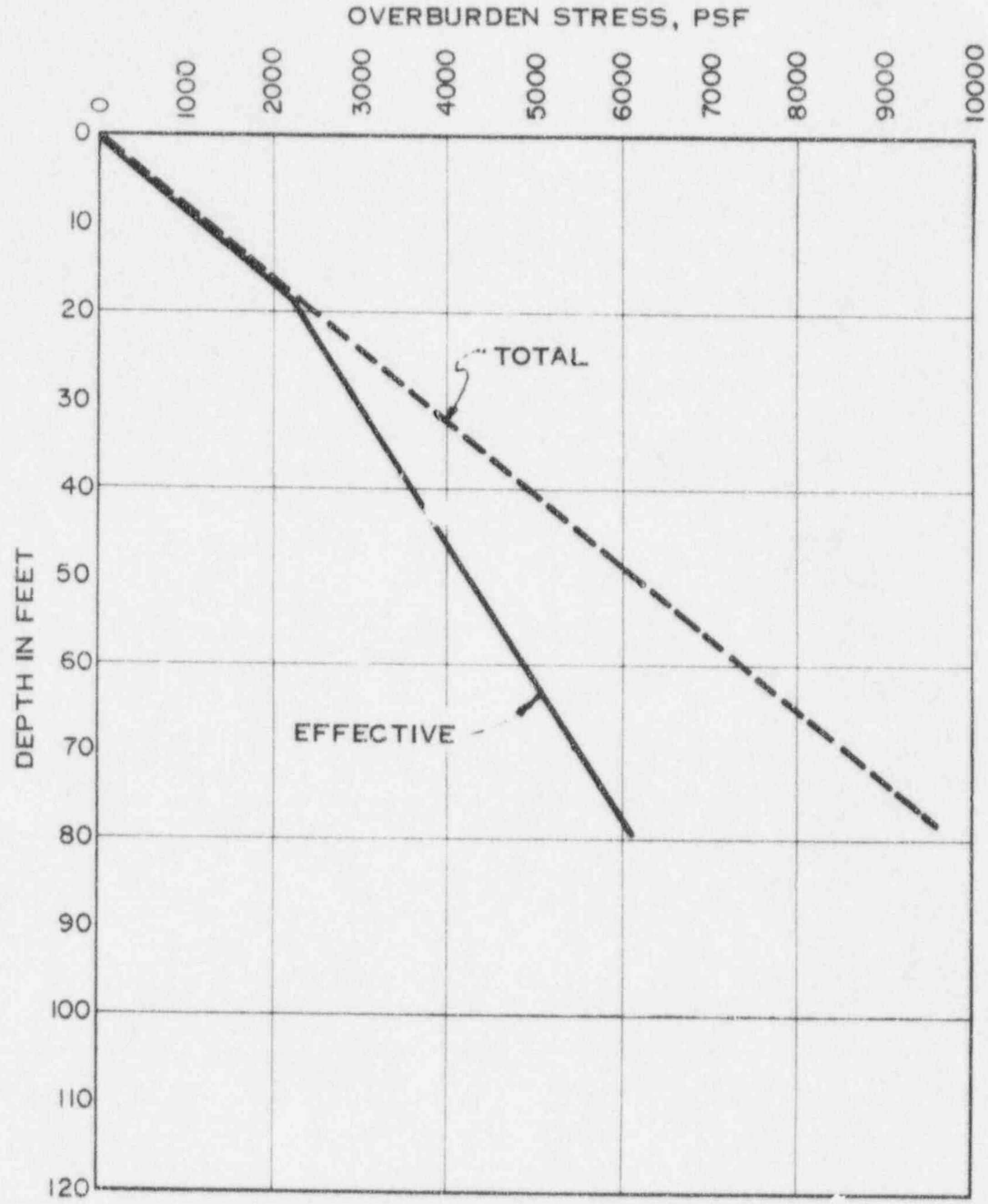
FILE 05467-018
REVISIONS
BY DATE
946.7 (REV. 6



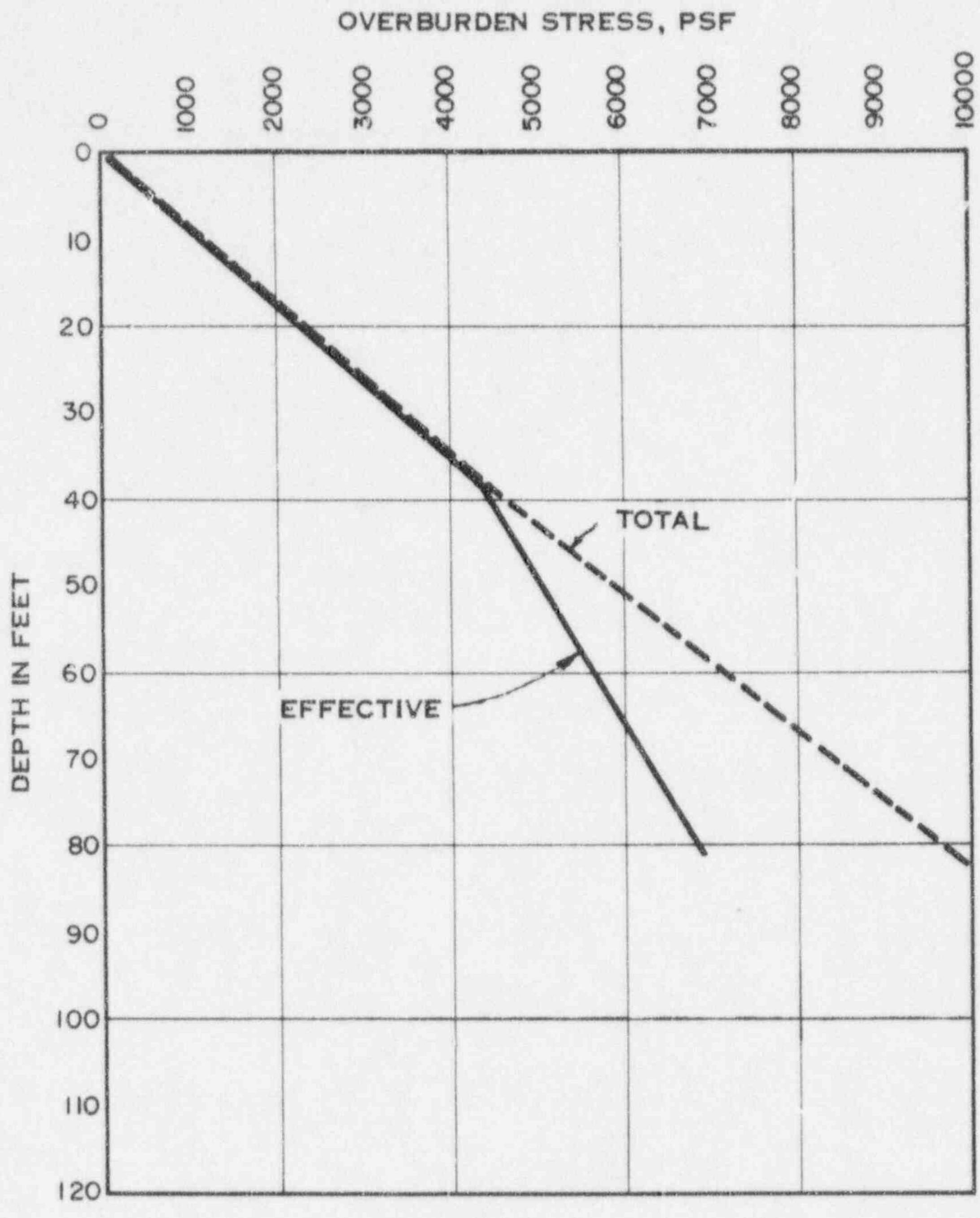
OVERBURDEN STRESSES
VERSUS DEPTH OF TAILINGS
(PROFILE B)

DAMES & MOORE

PLATE 11B

BY ______ DATE ______ CHECKED BY ______ FILE ______ REVISIONS BY ______ DATE ______

946.7 (REV. 6-68)



OVERBURDEN STRESSES
VERSUS DEPTH OF TAILINGS
(PROFILE C)

DAMES & MOORE

PLATE 11C

946.7 REV. 6-

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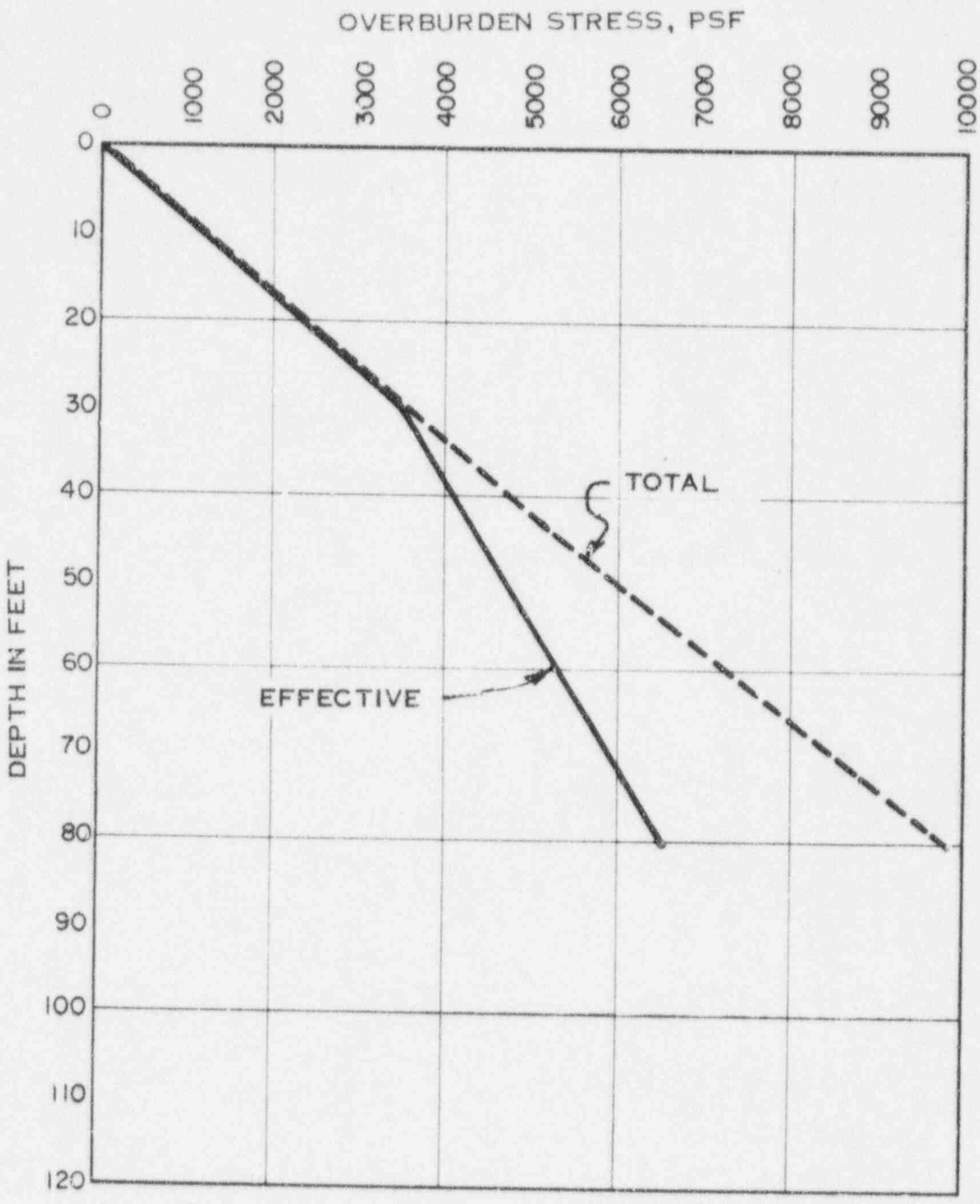

OVERBURDEN STRESSES
VERSUS DEPTH OF TAILINGS
(PROFILE D)

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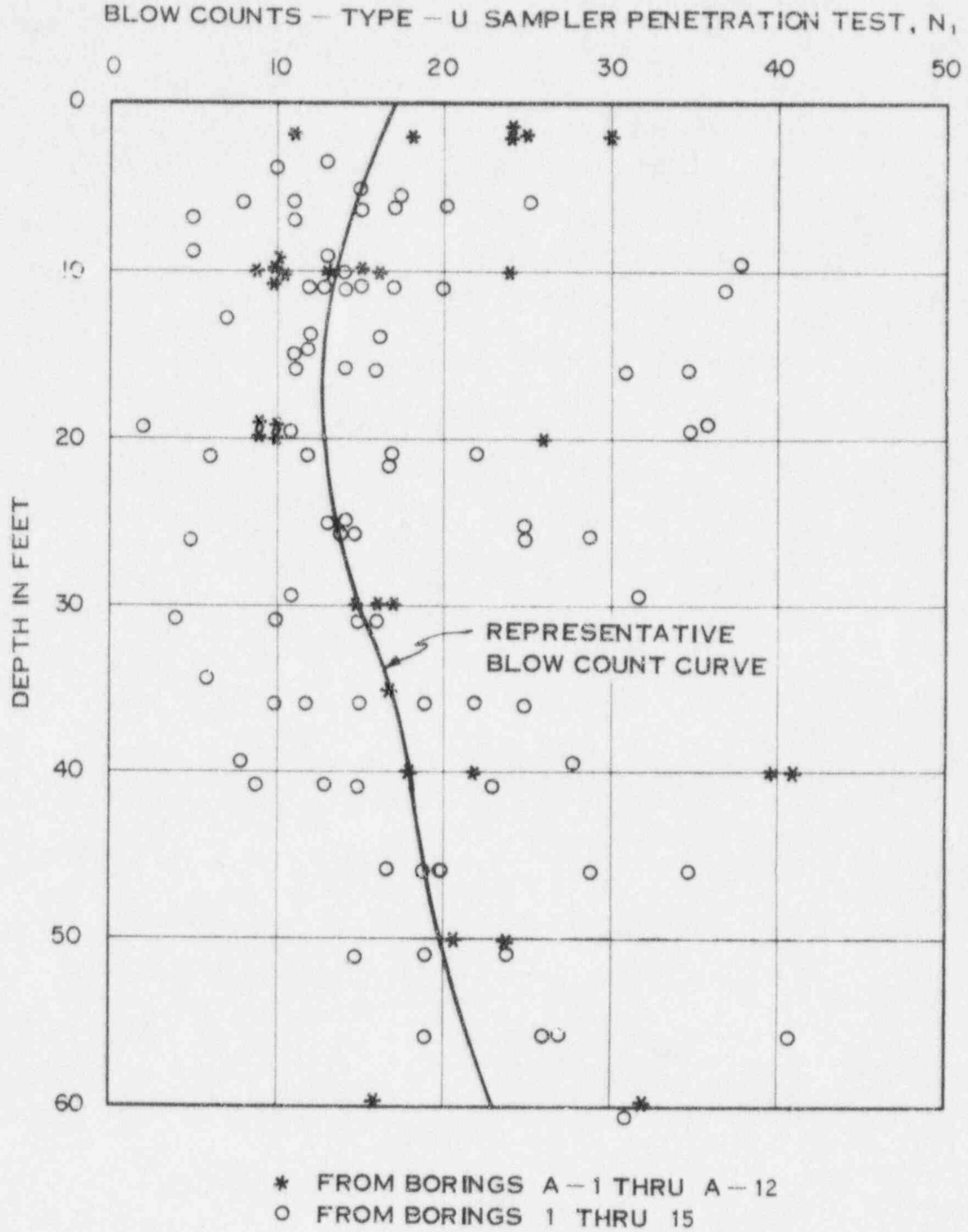
DAMES & MOORE



BLOW COUNTS - TYPE - U SAMPLER
PENETRATION TEST VERSUS DEPTH

DAMES & MOORE

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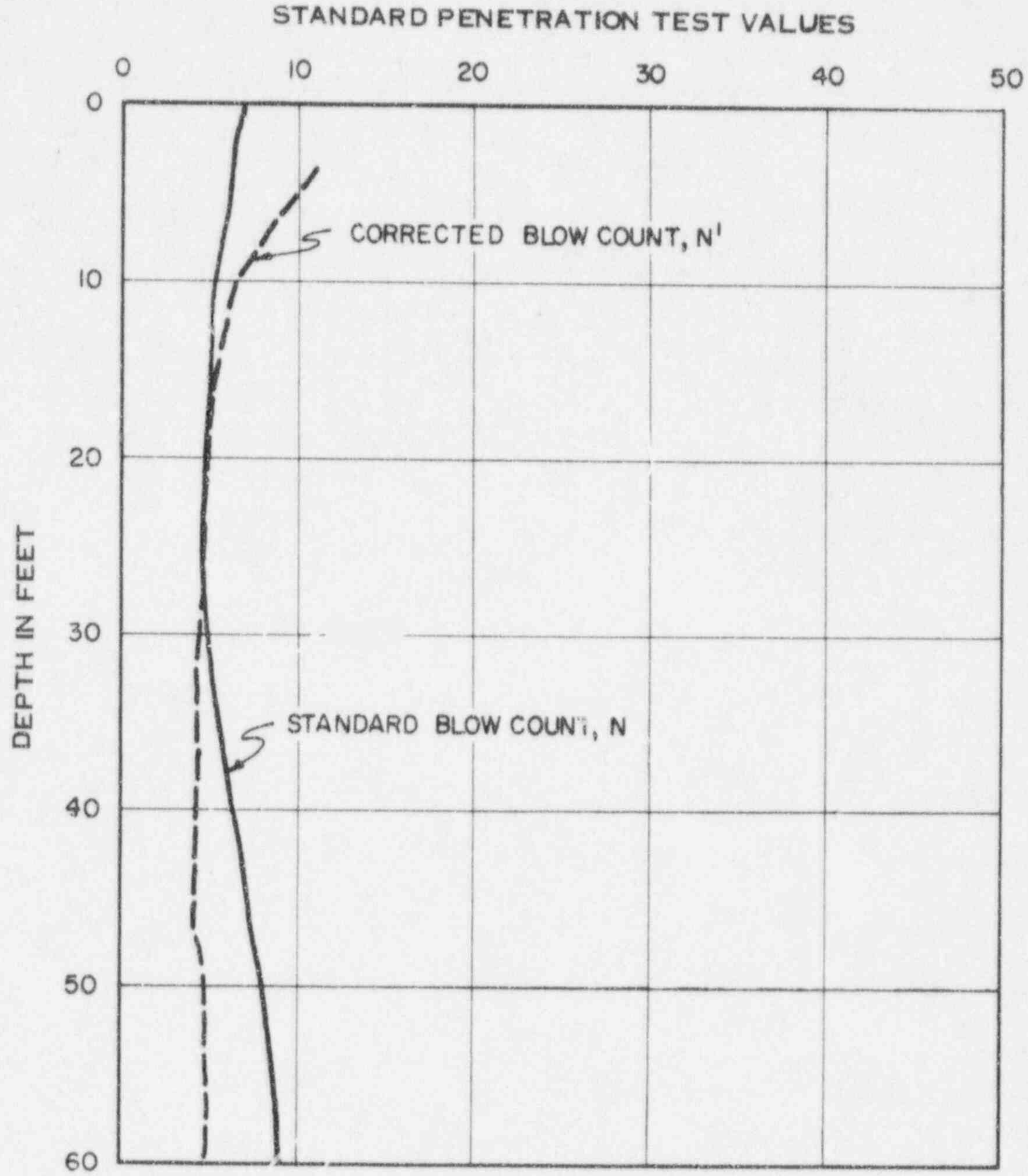

DEPTH	N	C_N	N'
5	6	1.68	10
10	5	1.30	6.5
15	5	1.08	5.5
20	5	0.92	5
25	5.5	0.84	4.5
30	6	0.75	4.5
35	6.5	0.69	4.5
40	7	0.64	4.5
45	7	0.60	4
50	8	0.56	4.5
60	9	0.50	4.5

STANDARD PENETRATION TEST VALUES VERSUS DEPTH

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BY ________ DATE ________
CHECKED BY ________ DATE ________

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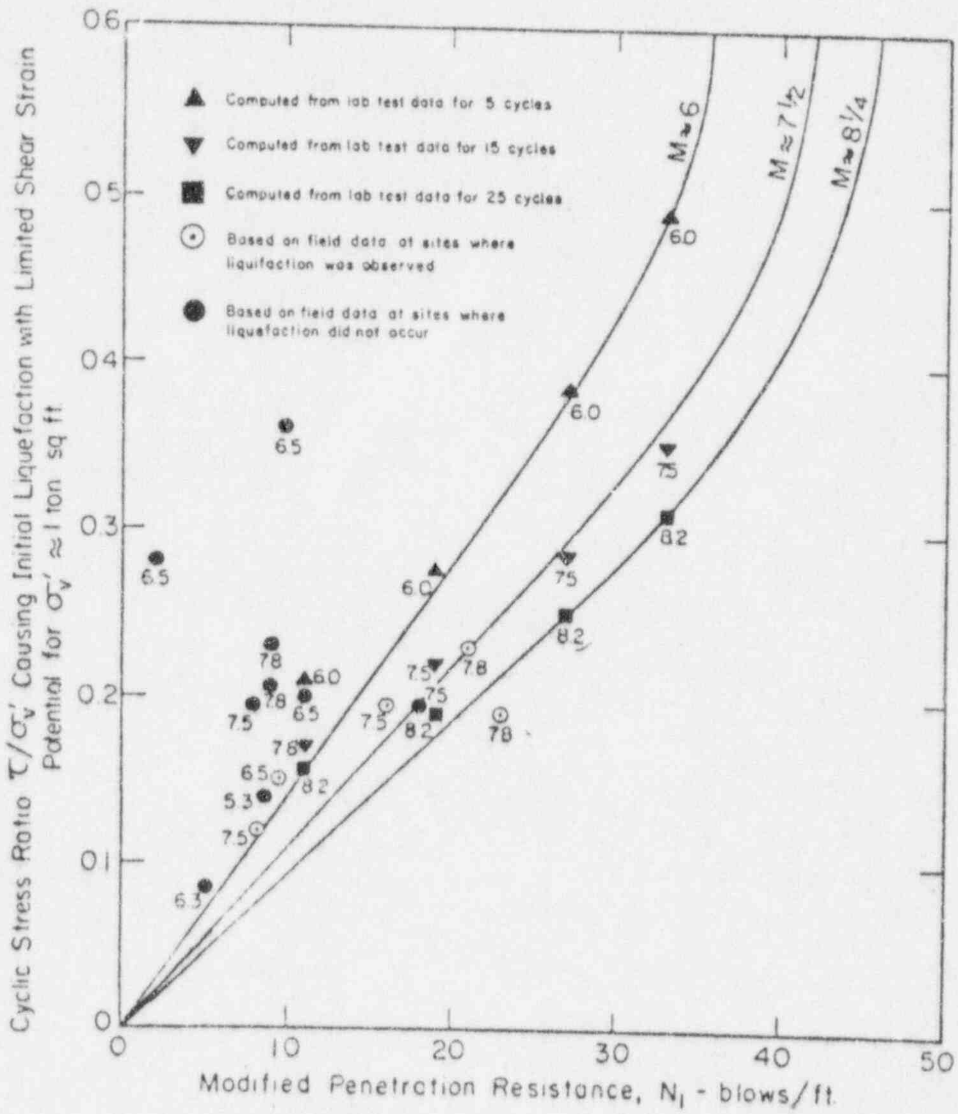


CORRELATION BETWEEN FIELD LIQUEFACTION BEHAVIOR OF SANDS FOR LEVEL GROUND CONDITIONS AND PENETRATION RESISTANCE

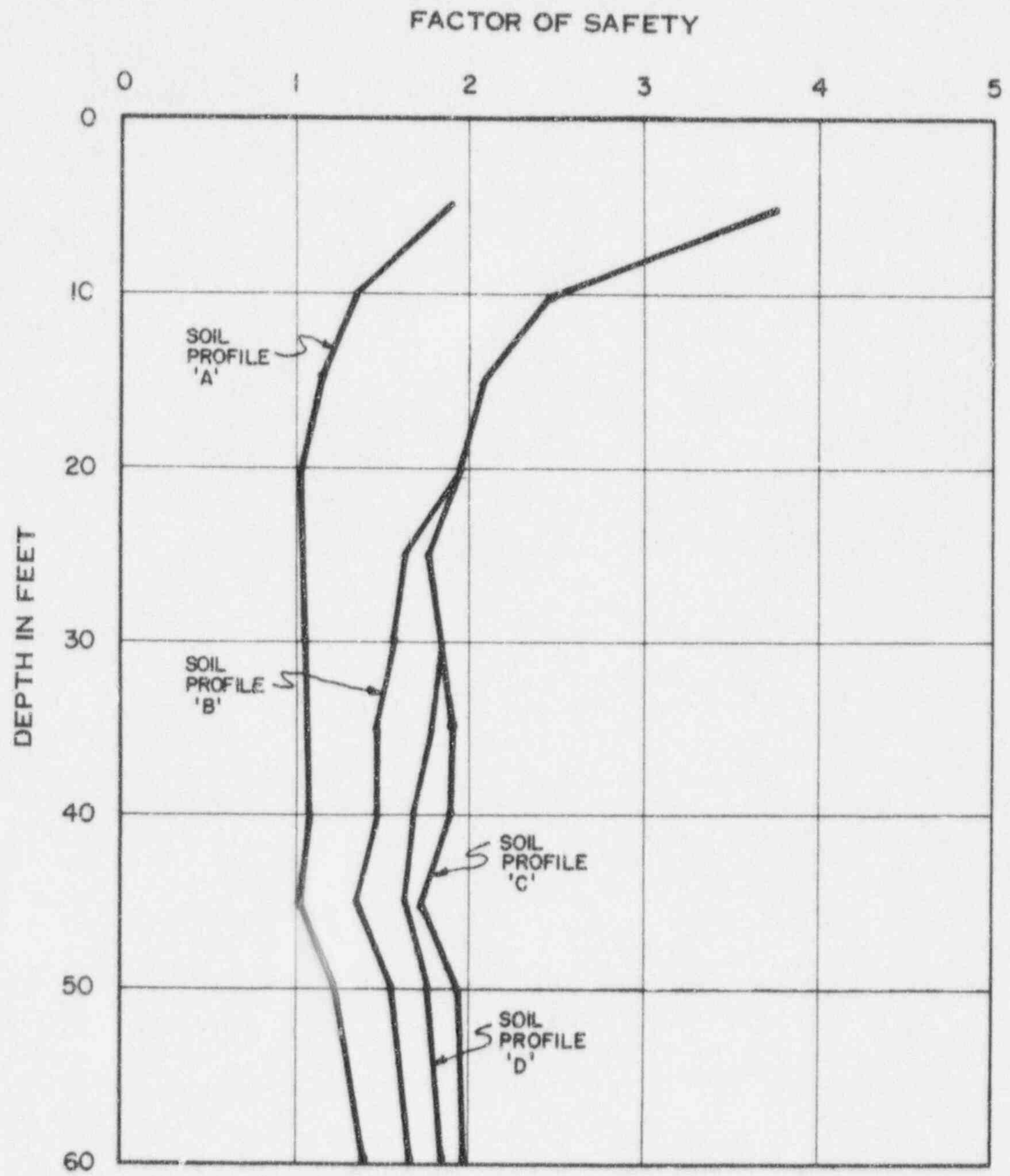
(Seed, 1976)

DAMES & MOORE

DEPTH IN FEET	σ_o	$\bar{\sigma}_o$	$\sigma_o/\bar{\sigma}_o$	r_d	$\tau_{ave}/\bar{\sigma}_o$	$(\tau/\bar{\sigma}_o)_{field}$	F.S.
PROFILE A							
5	600	300	2.0	.99	.064	.120	1.88
10	1100	600	1.83	.93	.058	.078	1.34
15	1700	900	1.88	.97	.059	.066	1.12
20	2300	1200	1.91	.95	.059	.060	1.02
25	2800	1600	1.75	.94	.053	.055	1.04
30	3400	1900	1.78	.92	.053	.055	1.04
35	4000	2200	1.82	.89	.053	.055	1.04
40	4600	2500	1.84	.85	.051	.055	1.08
45	5200	2800	1.86	.80	.048	.050	1.04
50	5700	3100	1.8[illegible]	.75	.045	.055	1.22
60	6900	3800	1.[illegible]	.68	.040	.055	1.38
PROFILE B							
5	600	600	1	.99	.032	.120	3.75
10	1100	1100	1	.98	.032	.078	2.43
15	1700	1700	1	.97	.032	.066	2.06
20	2300	2300	1	.95	.031	.060	1.94
25	2900	2600	1.12	.94	.034	.055	1.62
30	3500	2900	1.21	.92	.036	.055	1.53
35	4200	3200	1.31	.89	.038	.055	1.45
40	4800	3500	1.37	.85	.038	.055	1.45
45	5400	3800	1.42	.80	.037	.050	1.35
50	6000	4200	1.43	.75	.035	.055	1.57
60	7200	4800	1.50	.68	.033	.055	1.67
PROFILE C							
5	600	600	1	.99	.032	.120	3.75
10	1100	1100	1	.98	.032	.078	2.43
15	1700	1700	1	.97	.032	.066	2.06
20	2300	2300	1	.95	.031	.060	1.94
25	2800	2800	1	.94	.031	.055	1.77
30	3400	3400	1	.92	.030	.055	1.83
35	4000	4000	1	.89	.029	.055	1.90
40	4600	4400	1.05	.85	.029	.055	1.90
45	5200	4700	1.11	.80	.029	.050	1.72
50	5800	5000	1.16	.75	.028	.055	1.96
60	7000	5600	1.25	.68	.028	.055	1.96
PROFILE D							
5	600	600	1	.99	.032	.120	3.75
10	1100	1100	1	.98	.032	.079	2.43
15	1700	1700	1	.97	.032	.066	2.06
20	2300	2300	1	.95	.031	.060	1.94
25	2800	2800	1	.94	.031	.055	1.77
30	3400	3400	1	.92	.030	.055	1.83
35	4100	3800	1.07	.89	.031	.055	1.77
40	4700	4000	1.18	.85	.033	.055	1.67
45	5300	4400	1.20	.80	.031	.050	1.61
50	5900	4700	1.26	.75	.031	.055	1.77
60	7100	5200	1.37	.68	.030	.055	1.83

SUMMARY OF LIQUEFACTION ANALYSIS
EMPIRICAL METHOD

DAMES & MOORE



FACTOR OF SAFETY AGAINST LIQUEFACTION BASED ON EMPIRICAL CORRELATION FOR M= 7 EVENT

DAMES & MOORE

PLATE 16

946.7 (REV. 6-...)

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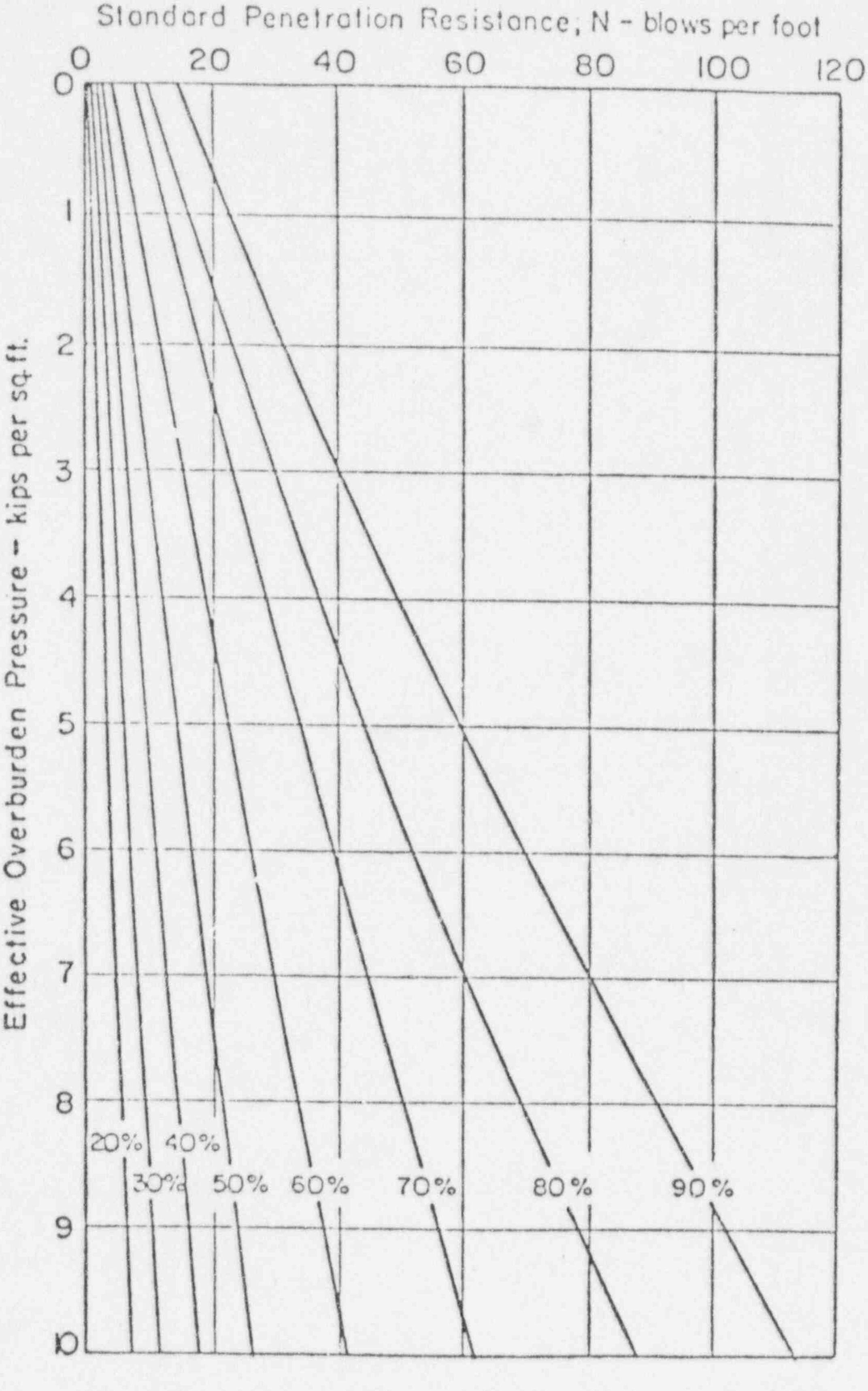
REVISIONS BY ______ DATE ______



RELATIONSHIP BETWEEN STANDARD PENETRATION TEST RESISTANCE, RELATIVE DENSITY, AND EFFECTIVE OVERBURDEN PRESSURE

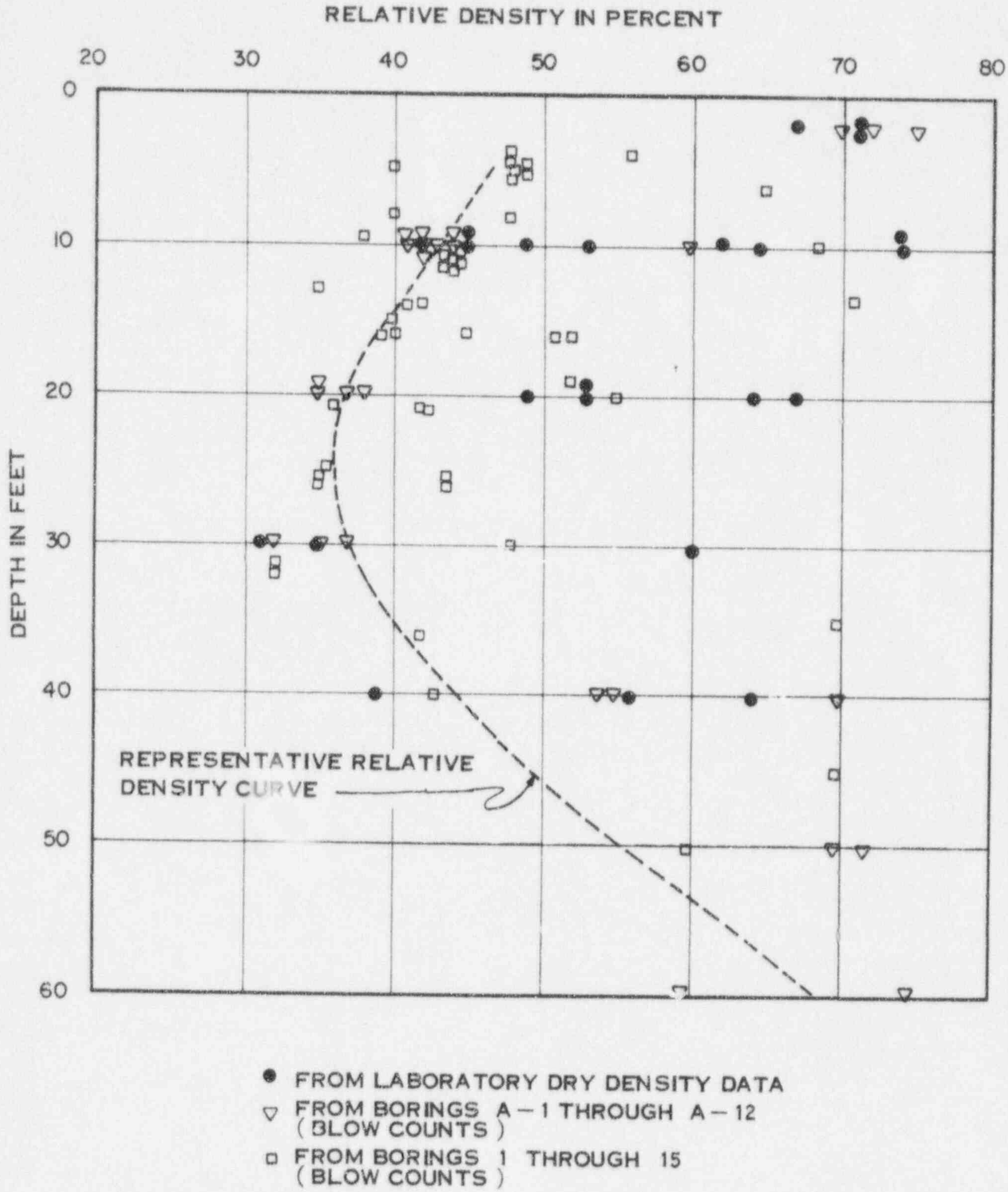
(GIBBS AND HOLTZ, 1957)

DAMES & MOORE

PLATE 17

946.7 (REV. 6-

BY DATE CHECKED BY FILE 05867-013 REVISIONS BY DATE



RELATIVE DENSITY VERSUS DEPTH
OF IN-SITU TAILINGS

DAMES & MOORE

PLATE 18

REVISIONS BY ______ DATE ______

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BY DATE
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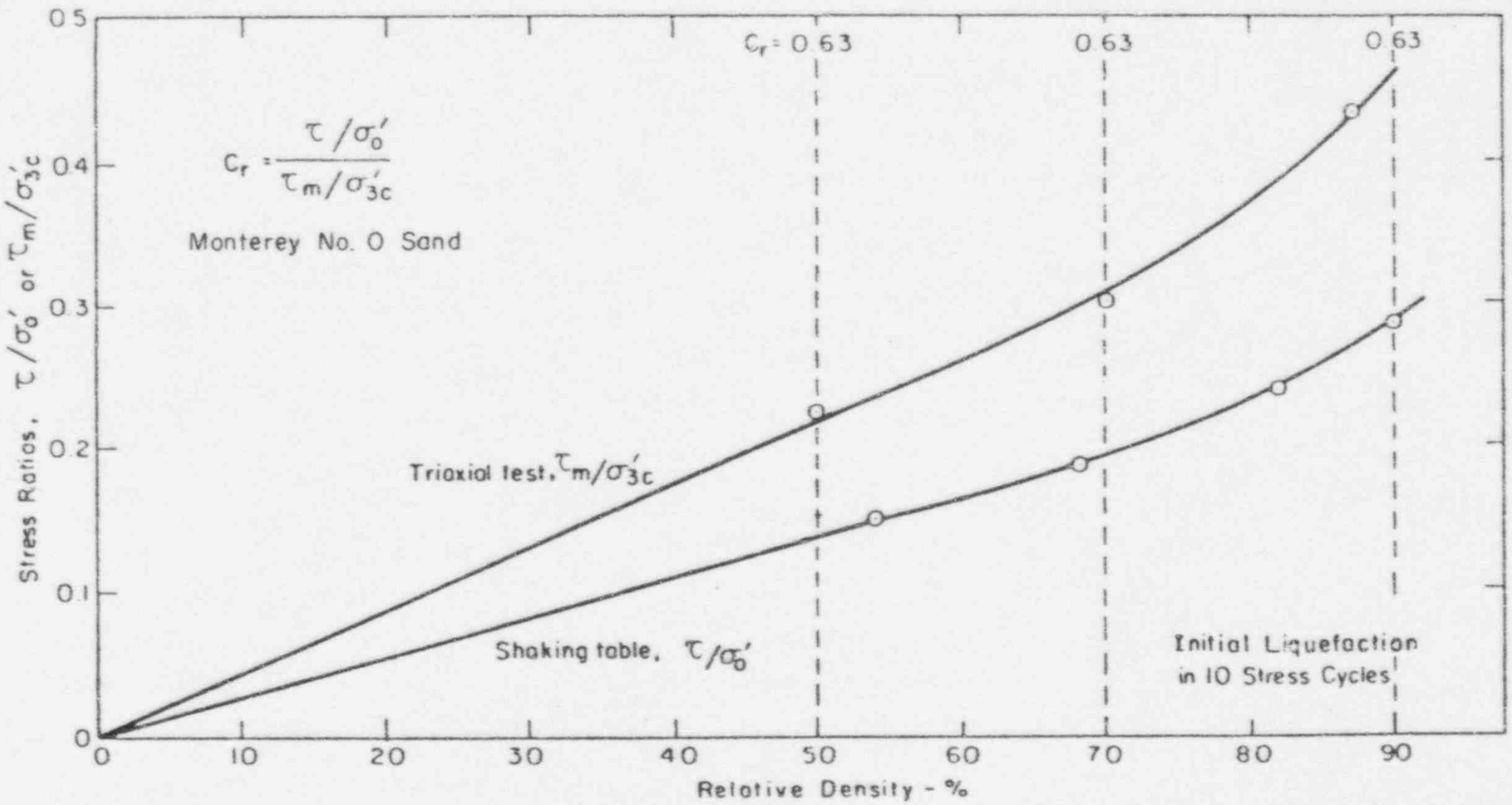
REVISIONS
BY DATE



COMPARISON OF SHAKING TABLE AND TRIAXIAL TEST RESULTS

(De Alba, Chan and Seed, 1975)

DAMES & MOORE

PLATE 19

946.7 (REV. 6

BY ____________ DATE ____________

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REVISIONS

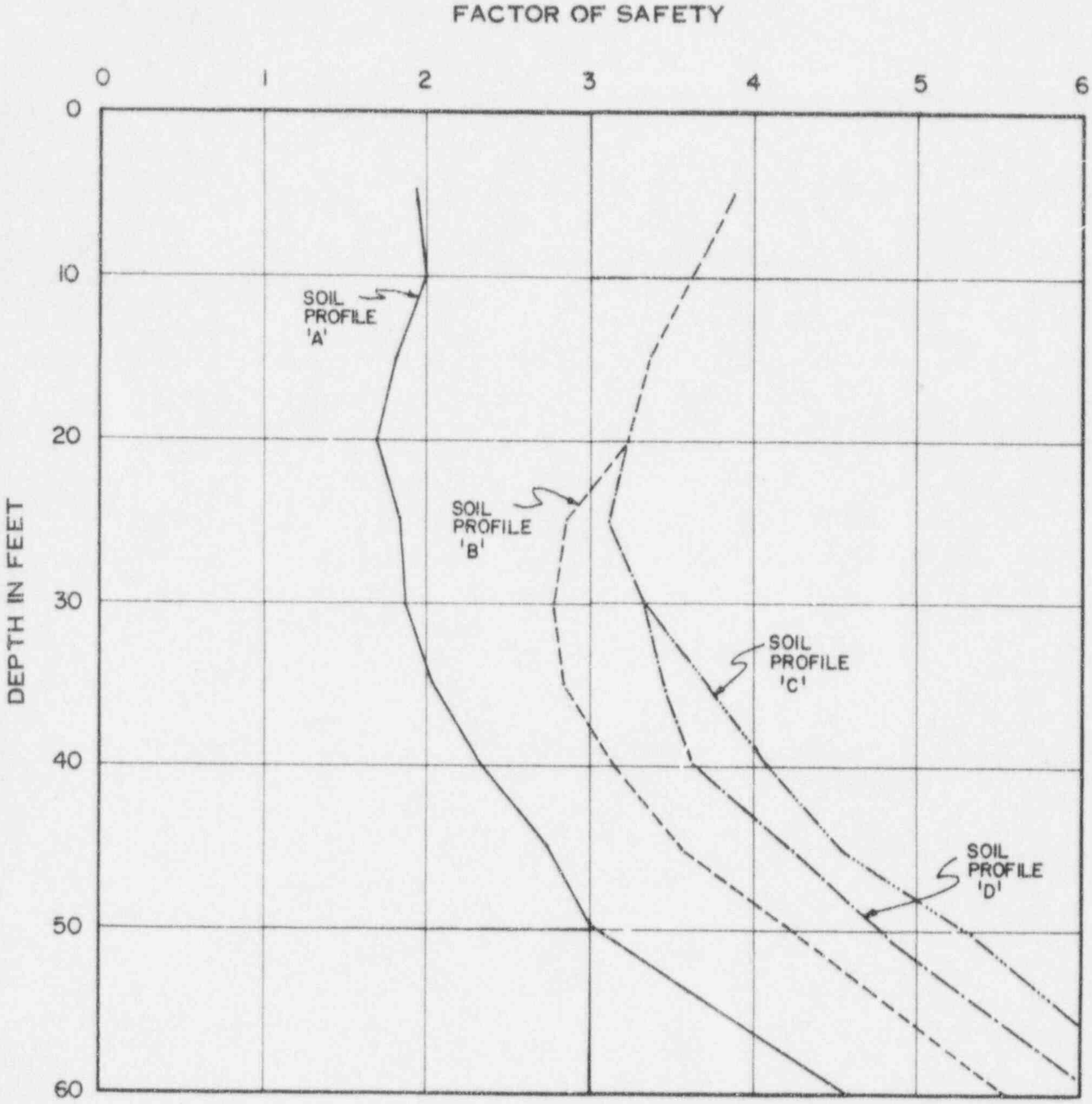
BY ________ DATE ______

DEPTH IN FEET	D_R	SHAKING TABLE TEST DATA	AVERAGE CYCLIC STRESS RATIO INDUCED IN GROUND (Eq. 1)				FACTORS OF SAFETY FOR VARIOUS PROFILES			
		$(\tau/\bar{\sigma}_o)$	A	B	C	D	A	B	C	D
5	46	0.124	.064	.032	.032	.032	1.94	3.88	3.88	3.88
10	43	.116	.058	.032	.032	.032	2.00	3.63	3.63	3.63
15	40	.108	.059	.032	.032	.032	1.83	3.38	3.38	3.38
20	37	.100	.059	.031	.031	.031	1.70	3.23	3.23	3.23
25	36	.097	.053	.034	.031	.031	1.83	2.85	3.13	3.13
30	37	.100	.053	.036	.030	.030	1.89	2.78	3.33	3.33
35	40	.108	.053	.038	.029	.031	2.04	2.84	3.72	3.48
40	44	.119	.051	.038	.029	.033	2.33	3.13	4.10	3.61
45	49	.132	.048	.037	.029	.031	2.75	3.57	4.55	4.26
50	55	.149	.045	.035	.028	.031	3.04	4.26	5.32	4.80
60	68	.184	.040	.033	.028	.030	4.60	5.58	6.57	6.13

COMPARISON OF INDUCED CYCLIC STRESS RATIO
AND THAT REQUIRED TO PRODUCE INITIAL LIQUEFACTION
IN SHAKING TABLE TESTS

DAMES & MOORE

PLATE 20



FACTOR OF SAFETY AGAINST LIQUEFACTION
BASED ON LABORATORY DATA AND RELATIVE DENSITY

DAMES & MOORE

PLATE 21

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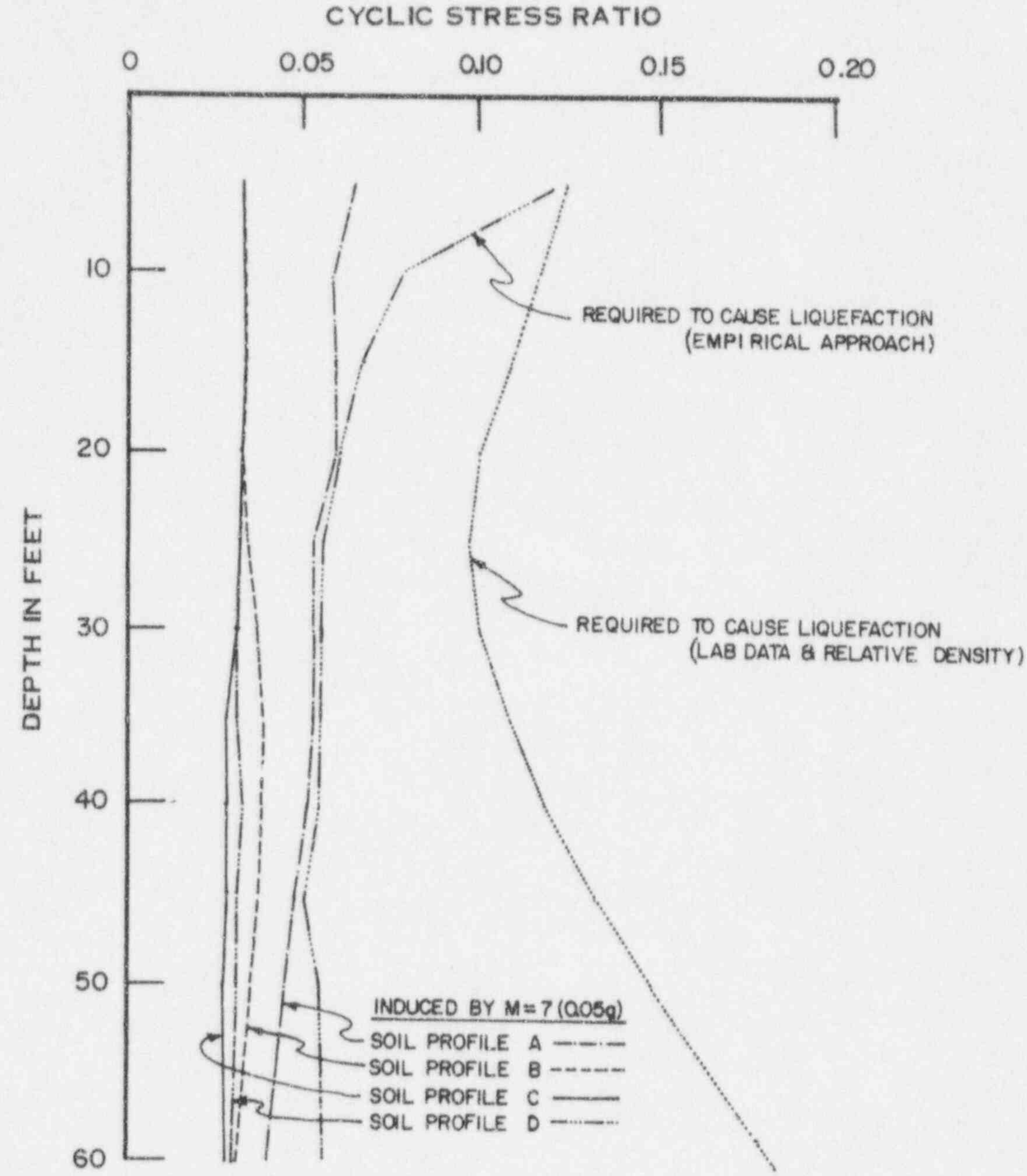
REVISIONS BY DATE
946.7 (REV. 6-



CYCLIC STRESS RATIO VERSUS DEPTH

BY ________ DATE ________ CHECKED BY ________ FILE 05467-018 REVISIONS BY ________ DATE ________

946.7 (REV. 6-

DAMES & MOORE

APPENDIX A

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FIELD EXPLORATION AND LABORATORY TESTING

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APPENDIX A

FIELD EXPLORATION AND LABORATORY TESTING

LIST OF PLATES

APPENDIX A

FIELD EXPLORATION AND LABORATORY TESTING

FIELD EXPLORATION

PRESENT INVESTIGATION

The field exploration program conducted for this study consisted of the drilling, sampling, and logging of 13 exploratory borings extending to depths ranging from 9.5 to 82.5 feet (Borings A-1 through A-14), the installation of three piezometers to depths of from 60.0 to 79.0 feet and the excavation of three test pits (A-1 through A-3) along the north embankment starter dike to depths ranging from 5.0 to 8.0 feet. In addition, standpipes were installed in 13 of the borings in order to measure water levels. The borings were drilled using a truck-mounted rotary-wash drill rig. In general, casing was used to advance the first 7.5 feet of the boring, after which the hole was advanced using the rotary-wash techniques. The test pits were excavated utilizing a rubber tired tractor mounted backhoe.

All field operations were conducted and supervised by an experienced Dames & Moore geotechnical engineer. The borings were sampled using a Dames & Moore Type U Split-Barrel Sampler, as shown on Plate A-1. A complete log was maintained in the field for each boring and test pit, and the materials were classified by visual and textural examination. The method utilized in classifying the soils is defined on Plate A-2, Unified Soil Classification System. A verbal description of the relative density of the material, as noted on the logs, was determined using blows per foot of penetration with the Dames & Moore Sampler. The correlation relating, in an approximate manner, the verbal density descriptions shown on the logs, relative density, Dames & Moore blow counts, and standard penetration blow counts are presented as follows:

TABLE A1

CORRELATION OF PENETRATION RESISTANCE AND SOIL PROPERTIES

Verbal Description Of Relative Density	Approximate Relative Density, D_r (In Percent)	Dames & Moore Sampler*, Blows Per Foot	Standard Penetration Blows Per Foot
Very Loose	<20	<11	<4
Loose	20-40	11-26	4-10
Medium Dense	40-70	26-72	10-30
Dense	70-90	72-104	30-50
Very Dense	>90	>104	>50

*Using 140 pound hammer falling 30 inches.

Graphical representations of the soils penetrated by the borings drilled in conjunction with this study are presented on Plates A-3A through A-3F, Log of Borings. The logs of the test pits conducted along the northern embankment seepage area are shown on Plate A-4A.

The locations and ground surface elevations of each boring were determined by Mr. John Keough, Registered Utah Land Surveyor, No. 1963. The ground surface elevations are presented above the respective log of each boring and refer to USGS datum. The locations of the borings are shown on Plate 3, Plot Plan, in the text of this report.

OTHER PREVIOUS INVESTIGATIONS

Other Dames & Moore studies carried out at the site provide valuable exploratory information, and the logs of the borings and test pits from these studies have been incorporated into this report (Dames & Moore, 1977(a); Dames & Moore, 1975; Dames & Moore, 1973; Dames & Moore, 1974). Logs of the exploratory borings (P-1 through P-3 and 1 through 15) drilled at the site in conjunction with other field programs by Dames & Moore are presented on Plates A-3G through A-3Q, Log of Borings. Logs of the test pits (Test Pits 1 through 6 and P-1 through P-8) from these other studies are presented on Plates A-4B through A-4D, Logs of Test Pits. The locations of all borings and test pits drilled and excavated at the site in conjunction with the other Dames & Moore studies are shown on Plate 2, Plot Plan, in the text of this report.

WATER LEVEL MEASUREMENTS

In order to monitor the water levels at the site, 13 of the 14 exploratory borings drilled in conjunction with this study were equipped with two-inch diameter plastic pipe, the lower 20 feet of which were slotted. Sandy gravel backfill was placed around the plastic pipe adjacent to the slotted interval and the remainder of the hole was backfilled with the cuttings from the drilling operation.

In addition to the stand pipes placed within the exploratory borings, piezometers were installed at three locations along the crest of the tailings embankment. The locations of the piezometers are presented on Plate 2, Plot Plan, in the text of this report. Each piezometer was installed by drilling a hole four and three-quarters inches in diameter and placing an open-tube "Casagrande" type piezometer tip (two feet in length) at the bottom of the hole. A three-quarters inch PVC tube was connected to the piezometer tip and extended to the top of the drilled hole. Well-graded sand backfill was placed in the bottom 12 feet of the hole, around the piezometer tip. Approximately five feet of pelletized bentonite was placed on top the well-graded sand backfill and well-tamped around the PVC pipe.

Sandy drill cuttings were placed in the next five foot interval upon which a second five-foot layer of pelletized bentonite was placed. The remainder of the hole was then backfilled with more available sandy cuttings. The surface of the piezometer was protected by an eight-inch steel pipe. A diagram of the piezometer installation is shown on Plate A-5, Diagram of Typical Piezometer. The piezometers numbered P1, P2, and P3 were installed to depth of 60, 65, and 79 feet, respectively.

Ground water levels were observed in the borings and piezometers during the course of the field exploration program. The water levels recorded within each of the borings during the time of drilling are presented on the Log of Borings. Additional water levels have been periodically obtained from the borings and piezometers by representatives of Atlas Minerals. These water levels are presented on Table A2.

TABLE A2

RECORDED PERIODIC WATER LEVELS

Location	Piezometer or Stand Pipe Tip Installation in feet	WATER LEVEL DEPTH BELOW SURFACE, FEET — DATE OF MEASUREMENT 3/08/77 3/10/77	4/13/77	4/28/77	5/12/77	5/26/77	6/09/77	6/23/77	7/07/77	7/21/77	8/29/77	9/07/77	10/17/77	2/14/78	Condition of Stand Pipe or Piezometer 8/29/77
Boring A-1	65.0	44.0	43.6	43.6	43.6	43.4	43.3	43.8	43.6	43.6	43.4	43.7	42.4	42.9	Open to 59.8 ft
Boring A-2	59.5	24.0	24.5	24.3	24.1	24.0	24.1	24.2	24.2	24.3	24.5	24.0	24.5	24.5	Open to 38.5 ft
Boring A-3	42.0	17.0	21.1	21.2	21.0	21.1	20.8	20.9	20.8	20.9	21.1	20.7	20.3	21.1	Open to 28.5 ft
Boring A-4	69.5	39.8	44.0	44.6	45.0	45.5	45.8	46.2	46.5	46.7	46.6	46.8	47.5	49.6	Open to 49.5 ft
Boring A-5	81.0	42.0	61.4	63.8	61.4	61.4	61.2	61.4	61.3	61.4	61.4	61.0	60.8	61.6	Open to 71.9 ft
Boring A-6	61.0	35.5	37.1	37.1	37.0	36.9	36.9	36.6	36.6	36.6	*	34.7	34.1	34.4	----
Boring A-7	29.0	25.0	22.9	23.7	25.3	20.0	24.1	24.7	25.4	25.6	24.5	24.6	26.6	25.8	Open to 29.2 ft
Boring A-8	20.0	17.2	19.5	19.5	18.6	16.9	15.9	17.3	17.4	18.3	16.5	15.8	16.7	19.7**	Open to 19.5 ft
Boring A-9	49.0	36.0	29.7	30.0	29.9	29.8	29.7	29.7	29.7	29.8	29.8	29.5	29.3	31.1	Open to 38.1 ft
Boring A-10	19.5	19.5	19.5	19.5	19.5	19.5	17.4	19.4	19.5	19.5**	19.5**	19.5**	19.5**	19.5**	Open to 19.5 ft**
Boring A-11	49.5	28.5	26.1	26.0	26.1	26.1	26.2	26.1	26.1	26.2	*	-			Destroyed
Boring A-12	20.0	17.0	20.4	19.4	19.4	19.4	18.8	-	-	-	-	-			Destroyed
Boring A-13	20.0	11.5	19.3	20.2	19.7	20.2	20.2**	19.4	19.5**	19.4**	*	-	19.5**	19.5**	Destroyed
Piezometer P-1	60.0	-	13.1	13.2	13.3	13.2	13.2	13.2	13.2	13.2	13.1	16.1	17.0	Plugged	Plugged 15.5 ft
Piezometer P-2	65.0	-	20.3	22.3	24.0	25.8	27.2	29.7	29.9	31.3	34.1	34.8	37.6	46.4	Open to 58.5 ft
Piezometer P-3	79.0	-	60.2	66.2	65.8	67.6	63.9	70.2	71.0	71.7	40.0**	40.0**	40.0**	39.5	Plugged 40.0 ft

* not read
** dry

LABORATORY TESTS

GENERAL

This section of the Appendix presents a summary of all laboratory test data conducted by Dames & Moore in this and previous studies. All of the laboratory test data presented was reviewed and where applicable, utilized as a basis for engineering analyses in this study. Data tabulated in this section can be cross-referenced to the previous or present reports by boring numbers.

Laboratory tests were performed on selected samples of the tailings materials as well as the natural in-situ soils directly beneath and surrounding the tailings pond. The purpose of the laboratory testing program was to obtain data in order to determine the strength, compressibility, permeability, and engineering index properties of the tailings pond embankment foundation and fill materials. In general, the engineering properties of the foundation and tailings construction material were classified in the following four groups: (1) sand tailings material (SM), (2) slime tailings material (ML), (3) natural red sand soils, and (4) recompacted red sandy soils and sand tailings. The laboratory tests performed include moisture and density determinations, relative densisty, Atterberg limits, compaction, gradation, direct shear, triaxial compression, one-dimensional compression (consolidation), and permeability testing.

MOISTURE AND DENSITY

Moisture and density determinations were performed in order to aid in classifying materials and to correlate other engineering properties. The results of these tests are shown to the left of each boring log adjacent to the respective sample locations (Plates A-3A through A-3Q).

RELATIVE DENSITY

To provide data utilized in our study of the liquefaction potential of the embankment tailings a relative density test was performed in accordance with the ASTM* Designation D-2049 using a representative sample of the "clean" beach tailings from which a majority of the embankments have been constructed.

The gradation of the tailings is such that 7 to 20 percent by weight of the particles pass the No.200 sieve. The ASTM Designation D-2049 states that the test should be performed on granular soils containing 12 percent by weight or less of soil particles passing the No.200 sieve.

The minimum dry density and the maximum dry density of the tested tailings as determined by the relative density test were 81.6 pounds per cubic foot and 109.0 pounds per cubic foot, respectively.

ATTERBERG LIMITS

Two Atterberg limits tests were performed on slime samples obtained from the submerged ponded water area. Sample 1 had a liquid limit (LL) of 63.8, a plastic limit (PL) of 22.8, a plasticity index (PI) of 41 and is classified as a CH type material. Sample 2 had a LL of 35.8, a PL of 22.8, a PI of 8.3 and is identified as an ML soil.

---

*American Society for Testing and Materials.

COMPACTION

A compaction test was performed on bulk samples taken from Test Pit P-1. The test was performed in accordance with the ASTM 698, Method of Compaction, as described on Plate A-6, Method of Performing Compaction Tests. The results of the compaction tests are presented on Plate A-7, Compaction Test Data.

GRADATION

To aid in classifying the soils, gradation tests were performed on various samples taken within the natural soils, recompacted natural soils, sand tailings material and slime tailings material. Both complete and partial sieve analyses were conducted. The results of the complete sieve analyses are presented on Plates A-8A through A-8L. The results of the partial sieves are tabulated in Table A2, on the following page.

TABLE A3

RESULTS OF PARTIAL GRADATION ANALYSES

Boring	Depth in ft	Percent by Weight Passing No. 60 Sieve	No. 100 Sieve	No. 200 Sieve	Soil Classification
A-1	40.5	100	100	98.9	ML
A-1	50	100	100	99.2	ML
A-2	40	99.9	99.3	89.2	ML
A-3	30	100	100	97.6	ML
A-5	70	100	100	99.9	ML
A-6	40	100	98.6	90.9	ML
A-6	50	100	99.9	98.6	ML
A-11	30	100	99.7	93.3	ML
A-11	40	100	100	100	ML
1	10.5	59.9	--	17.3	SM
1	20.5	97.1	--	38.2	SM
1	25.5	100	--	26.0	SM
1	30.5	98.9	--	41.5	SM/ML
1	35.5	96.6	--	47.0	SM/ML
1	45.5	63.2	--	15.7	SM
1	65.5	94.6	--	18.5	SM
1	70.5	100	--	98.8	ML
2	10.5	73.2	--	18.3	SM
2	25.5	72.6	--	24.5	SM
3	5.5	100	--	44.2	SM/ML
3	15.5	100	--	83.3	ML
3	50.5	100	--	100	ML
4	10.5	78.6	--	24.9	SM
4	20.5	98.6	--	25.6	SM
4	25.5	100	--	27.0	SM
4	30.5	100	--	42.0	SM/ML
4	60.5	92.8	--	48.1	SM/ML
5	10.5	82.1	--	16.5	SM
5	30.5	100	--	100	ML
Test Pit A-3	5-6'	--	--	99.3	ML

DIRECT SHEAR TESTS

Direct shear tests were performed on the sand tailings material, the slime tailings material, the natural red sandy soils and the recompacted red sandy soils. The tests were performed according to the general procedures described on Plate A-9, Method of Performing Direct Shear Tests. On all of the sandy soils (excluding slimes tailings) the tests were run at a strain rate of .005 inches per minute so as to produce consolidated-drained (CD) strained behavior. The tests performed on the fine-grained slimes tailings were run at higher strain rates and therefore must be assumed as reflecting shear strengths under partially-drained or perhaps undrained conditions. The results of the tests are shown on Table A4, on the following page.

TABLE A4

DIRECT SHEAR TEST DATA

Boring	Depth in ft	Soil Classification	Normal Stress PSF	Peak Shearing Strength in PSF
NATURAL RED SANDY SOILS				
2	25.5	SM	2,500	2,800
2	25.5	SM	4,000	5,020
4	60.5	SM/ML	1,500	1,030
4	60.5	SM/ML	3,500	1,450
5	30.5	ML	2,500	1,500
5	30.5	ML	4,500	2,650
10	34.5	SM	2,000	1,650
10	34.5	SM	3,500	3,100
10	44.0	SM	4,500	4,000
12	44.0	SM	3,500	3,400*
12	49.5	SM	2,500	2,530*
12	49.5	SM	5,000	3,350
P-1	5.5	SM	500	350
P-1	5.5	SM	1,000	780
P-1	10.5	SM	2,000	1,500
P-1	10.5	SM	3,000	2,320
P-3	4.5	SM	4,000	2,800
P-3	9.5	SM	5,000	3,690
SAND TAILINGS MATERIAL				
1	10.5	SM	1,000	1,050
1	10.5	SM	2,500	2,100
1	25.5	SM	2,500	2,450
1	25.5	SM	4,000	3,050
1	45.5	SM	3,000	2,530
1	45.5	SM	5,500	4,350
1	65.5	SM	1,500	1,450
1	65.5	SM	3,500	2,900
2	10.5	SM	3,500	2,900
2	10.5	SM	1,500	1,350
4	10.5	SM	3,500	2,750
4	10.5	SM	5,000	3,850
4	25.5	SM	2,500	2,050
4	25.5	SM	4,000	3,200
5	10.5	SM	1,750	1,650
5	10.5	SM	3,250	2,830
11	13.5	SM	1,500	1,000
11	13.5	SM	3,000	1,850
11	35.5	SM	3,500	1,030
11	35.5	SM	5,500	3,850
12	13.5	SM	2,000	1,250
12	13.5	SM	4,000	2,400
13	29.5	SM	3,000	2,550
13	29.5	SM	5,000	3,750
13	39.5	SM	1,500	1,050

*Piece of gravel in sample

TABLE A4 (continued)

Boring	Depth in ft	Soil Classification	Normal Stress PSF	Peak Shearing Strength in PSF
SLIMES TAILINGS				
1	70.5	ML	2,500	1,550
3	15.5	ML	2,000	2,570
5	15.5	ML	4,500	3,850
-	-	ML	600	220*
-	-	ML	900	410*
RECOMPACTED NATURAL RED SAND**				
Test Pit P1	3-5	SM	500	350
P1	3-5	SM	1,000	650
P1	3-5	SM	2,000	1,200
P1	3-5	SM	3,000	1,900
P1	3-5	SM	4,000	2,500
P1	3-5	SM	5,000	3,200

* Hand pushed Shelby tube sample taken from approximately two feet depth in the partially consolidated slimes.

** Materials compacted to an average of 100.5 pounds per cubic foot (90.5% of the maximum dry density as determined by ASTM D-698, Method of Compaction).

TRIAXIAL COMPRESSION TESTS

General

Triaxial compression tests were performed on samples of the sand tailings (SM), slime tailings (ML), and the natural red sandy soils. Triaxial testing included consolidated-undrained (CU) tests both with and without measured pore pressures, consolidated-drained tests (CD) and unconsolidated-undrained (UU) tests, both singular and multi-phase.

All of the triaxial tests, with the exception of the unconsolidated-undrained test samples, were back pressure saturated to a minimum B-value of 0.95 prior to shearing. A portion of the UU tests were also back pressure saturated to a minimum B-value of 0.90, while other UU test samples were not saturated prior to sheaing.

The single-phase unconsolidated-undrained triaxial tests were sheared until the peak total deviator stress was reached. The multi-phase unconsolidated-undrained tests were confined at a low pressure and sheared to a strain of five percent. The total deviator stress was then recorded. At this point, an additional confining pressure was applied to the sample and the shearing process was repeated. This procedure was continued until a total of three confining pressures and total deviator stresses were recorded.

Summarization of Triaxial Compression Test Data

The following subsections summarize the triaxial test data for each material type tested.

Sand Tailings - Triaxial tests were conducted on sand tailings samples including - 1) consolidated-undrained tests both with and without pore pressure measurements, 2) consolidated-drained tests, and 3) unconsolidated-undrained tests. The results of triaxial test data for the sand tailings material are presented on Table A-5, beginning on the following page.

Slime Tailings - Triaxial testing were conducted on the slime tailings including - 1) Consolidated-undrained tests with pore pressure measurements, and 2) both single and multi-phase unconsolidated-undrained tests. The results of these tests are tabulated on Table A-6 as presented on the following pages.

Natural Red Sandy Soils - A series of unconsolidated-undrained multi-phase triaxial compression tests were conducted on selected samples of the natural red sandy soils. The results of these tests are tabulated in Table A-7 on page A-30.

TABLE A5

TRIAXIAL TEST DATA ON SAND TAILINGS

CONSOLIDATED UNDRAINED TRIAXIAL TEST
WITH PORE PRESSURE MEASUREMENT

ATLAS MINERALS 5467-018-06, APRIL 7,1977
BORING A4, SAMPLE 7, DEPTH 60.0 FEET, BROWN SANDY SILT

SAMPLE HEIGHT	=	5.77 IN
SAMPLE AREA	=	4.40 SQ IN
CONSOLIDATION PRESSURE	=	8.00 KSF
INIT. MAX. PRIN. STRESS	=	8.00 KSF
INITIAL PORE PRESSURE	=	86.0 PSI
BEFORE CONSOLIDATION		
DRY DENSITY	=	94.3 PCF
WATER CONTENT	=	27.40 PERCENT
AFTER CONSOLIDATION		
DRY DENSITY	=	99.2 PCF
WATER CONTENT	=	26.70 PERCENT
BEAM LOAD FACTOR	=	47.92 LBS/PERCENT

AXIAL STRAIN PCT	PORE PRESSURE KSF	DEVIATOR STRESS KSF	SIGMA3 EFFECTIVE KSF	SIGMA1 EFFECTIVE KSF	STRESS RATIO	ABAR	PBAR KSF	QBAR KSF
.35	2.88	5.78	5.12	10.90	2.13	.50	8.01	2.89
.52	3.38	6.86	4.62	11.48	2.49	.49	8.05	3.43
.69	3.64	7.63	4.36	11.99	2.75	.48	8.17	3.82
.87	3.79	8.40	4.21	12.61	2.99	.45	8.41	4.20
1.07	3.83	9.00	4.17	13.17	3.16	.43	8.67	4.50
1.21	3.83	9.45	4.17	13.62	3.27	.41	8.89	4.73
1.39	3.77	10.05	4.23	14.28	3.38	.38	9.25	5.03
1.56	3.72	10.65	4.28	14.94	3.49	.35	9.61	5.33
1.91	3.56	11.69	4.44	16.14	3.63	.30	10.29	5.85
2.43	3.18	13.31	4.82	18.13	3.76	.24	11.47	6.66
2.77	2.92	14.49	5.08	19.56	3.85	.20	12.32	7.24
3.76	2.09	17.66	5.91	23.57	3.99	.12	14.74	8.83
4.61	1.35	20.20	6.65	26.84	4.04	.07	16.74	10.10
5.37	.68	22.56	7.32	29.88	4.08	.03	18.60	11.28
6.51	-.32	25.51	8.32	33.83	4.07	-.01	21.07	12.76
7.57	-1.22	28.27	9.22	37.49	4.06	-.04	23.36	14.13
8.18	-1.73	29.66	9.73	39.39	4.05	-.06	24.56	14.83
8.78	-2.23	31.04	10.23	41.27	4.03	-.07	25.75	15.52
9.79	-2.98	33.25	10.98	44.23	4.03	-.09	27.60	16.62
10.78	-3.67	34.84	11.67	46.51	3.99	-.11	29.09	17.42
11.21	-3.96	35.65	11.96	47.61	3.98	-.11	29.78	17.82
11.85	-4.33	36.50	12.33	48.83	3.96	-.12	30.58	18.25
12.61	-4.74	37.41	12.74	50.15	3.94	-.13	31.44	18.71
13.50	-5.18	38.26	13.18	51.44	3.90	-.14	32.31	19.13
14.09	-5.47	38.54	13.47	52.01	3.86	-.14	32.74	19.27
14.64	-5.69	38.82	13.69	52.51	3.84	-.15	33.10	19.41
15.56	-6.03	38.67	14.03	52.70	3.76	-.16	33.37	19.33
16.22	-6.18	37.84	14.18	52.02	3.67	-.16	33.10	18.92
16.72	-6.19	36.05	14.19	50.24	3.54	-.17	32.22	18.02

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TABLE A5
TRIAXIAL TEST DATA ON SAND TAILINGS
(CONTINUED)

CONSOLIDATED UNDRAINED TRIAXIAL TEST
WITH PORE PRESSURE MEASUREMENT

ATLAS MINERALS ,MOAB,UTAH, 5467-018-06.APRIL 14,1977
BORINGA5,SAMPLE 3,DEPTH 20 FEET, BROWN SANDY SILT

SAMPLE HEIGHT	=	5.97 IN
SAMPLE AREA	=	4.42 SQ IN
CONSOLIDATION PRESSURE	=	2.00 KSF
INIT. MAX. PRIN. STRESS	=	2.00 KSF
INITIAL PORE PRESSURE	=	87.4 PSI
BEFORE CONSOLIDATION		
DRY DENSITY	=	99.1 PCF
WATER CONTENT	=	18.70 PERCENT
AFTER CONSOLIDATION		
DRY DENSITY	=	103.6 PCF
WATER CONTENT	=	25.70 PERCENT
BEAM LOAD FACTOR	=	9.58 LBS/PERCENT

AXIAL STRAIN PCT	PORE PRESSURE KSF	DEVIATOR STRESS KSF	SIGMA3 EFFECTIVE KSF	SIGMA1 EFFECTIVE KSF	STRESS RATIO	ABAR	PBAR KSF	QBAR KSF
.28	.33	2.05	1.67	3.72	2.23	.16	2.70	1.03
.59	.27	3.26	1.73	4.99	2.89	.08	3.36	1.63
.84	.03	4.02	1.97	6.00	3.04	.01	3.98	2.01
1.34	-.42	6.22	2.42	8.64	3.57	-.07	5.53	3.11
1.68	-.78	7.37	2.78	10.15	3.65	-.11	6.46	3.68
2.01	-1.15	8.84	3.15	11.99	3.81	-.13	7.57	4.42
2.51	-1.64	10.74	3.64	14.39	3.95	-.15	9.01	5.37
3.02	-2.23	12.54	4.23	16.77	3.96	-.18	10.50	6.27
3.35	-2.58	13.73	4.58	18.31	4.00	-.19	11.44	6.86
4.19	-3.57	16.45	5.57	22.02	3.95	-.22	13.80	8.23
5.03	-4.44	18.83	6.44	25.26	3.93	-.24	15.85	9.41
5.86	-5.10	20.66	7.10	27.76	3.91	-.25	17.43	10.33
6.70	-5.66	22.25	7.66	29.91	3.91	-.25	18.79	11.13
7.54	-6.19	23.67	8.19	31.86	3.89	-.26	20.03	11.84
8.38	-6.74	24.94	8.74	33.68	3.85	-.27	21.21	12.47
9.21	-7.11	26.02	9.11	35.13	3.86	-.27	22.12	13.01
10.05	-7.42	26.71	9.42	36.12	3.84	-.28	22.77	13.35
10.89	-7.69	27.32	9.69	37.01	3.82	-.28	23.35	13.66
11.73	-7.86	27.56	9.86	37.42	3.79	-.29	23.64	13.78
12.56	-8.01	27.57	10.01	37.58	3.76	-.29	23.79	13.79
13.40	-8.08	27.58	10.08	37.66	3.74	-.29	23.87	13.79
14.24	-8.12	27.52	10.12	37.65	3.72	-.30	23.88	13.76
15.08	-8.16	27.49	10.16	37.66	3.70	-.30	23.91	13.75
15.92	-8.19	27.49	10.19	37.68	3.70	-.30	23.94	13.74
16.42	-8.24	27.32	10.24	37.56	3.67	-.30	23.90	13.66
16.75	-8.27	26.98	10.27	37.24	3.63	-.31	23.75	13.49

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TABLE A 5

TRIAXIAL TEST DATA ON SAND TAILINGS

(CONTINUED)

CONSOLIDATED UNDRAINED TRIAXIAL TEST
WITH PORE PRESSURE MEASUREMENT

ATLAS MINERALS,5467-018-06,MOAB,UTAH,APRIL 13,1977
BORING A5,SAMPLE 5,DEPTH 40 FT.

SAMPLE HEIGHT	=	5.58 IN
SAMPLE AREA	=	4.59 SQ IN
CONSOLIDATION PRESSURE	=	4.00 KSF
INIT. MAX. PRIN. STRESS	=	4.00 KSF
INITIAL PORE PRESSURE	=	72.2 PSI
BEFORE CONSOLIDATION		
DRY DENSITY	=	98.2 PCF
WATER CONTENT	=	24.30 PERCENT
AFTER CONSOLIDATION		
DRY DENSITY	=	99.3 PCF
WATER CONTENT	=	25.20 PERCENT
BEAM LOAD FACTOR	=	15.97 LBS/PERCENT

AXIAL STRAIN PCT	PORE PRESSURE KSF	DEVIATOR STRESS KSF	SIGMA3 EFFECTIVE KSF	SIGMA1 EFFECTIVE KSF	STRESS RATIO	ABAR	PBAR KSF	QBAR KSF
.18	1.17	2.25	2.83	5.08	1.79	.52	3.96	1.13
.54	1.68	3.34	2.32	5.65	2.44	.50	3.98	1.67
.95	1.71	4.42	2.29	6.70	2.93	.39	4.49	2.21
1.27	1.61	5.34	2.39	7.73	3.24	.30	5.06	2.67
1.79	1.09	6.74	2.91	9.65	3.32	.16	6.28	3.37
2.69	.43	9.65	3.57	13.22	3.71	.04	8.39	4.83
3.58	-.39	12.66	4.39	17.05	3.88	-.03	10.72	6.33
4.48	-1.21	15.75	5.21	20.95	4.02	-.08	13.08	7.87
5.37	-2.20	19.01	6.20	25.21	4.06	-.12	15.71	9.51
6.27	-3.20	22.07	7.20	29.27	4.07	-.14	18.23	11.04
7.17	-4.06	24.93	8.06	32.99	4.09	-.16	20.53	12.47
8.06	-5.00	27.50	9.00	36.50	4.06	-.18	22.75	13.75
8.96	-5.83	29.38	9.83	39.21	3.99	-.20	24.52	14.69
9.49	-6.28	30.65	10.28	40.93	3.98	-.20	25.60	15.33
10.03	-6.78	31.46	10.78	42.25	3.92	-.22	26.51	15.73
10.39	-7.19	30.89	11.19	42.07	3.76	-.23	26.63	15.44
10.75	-7.23	29.07	11.23	40.29	3.59	-.25	25.76	14.53

TABLE A 5

TRIAXIAL TEST DATA ON SAND TAILINGS

(CONTINUED)

CONSOLIDATED UNDRAINED TRIAXIAL TEST
WITH PORE PRESSURE MEASUREMENT

ATLAS MINERALS, MOAB, UTAH, 5467-018-06, APRIL 15, 1977
BORING A5, SAMPLE 7, DEPTH 60 FEET, BROWN SILTY FINE SAND

SAMPLE HEIGHT	=	5.46 IN
SAMPLE AREA	=	4.43 SQ IN
CONSOLIDATION PRESSURE	=	6.00 KSF
INIT. MAX. PRIN. STRESS	=	6.00 KSF
INITIAL PORE PRESSURE	=	78.3 PSI
BEFORE CONSOLIDATION		
DRY DENSITY	=	105.7 PCF
WATER CONTENT	=	21.10 PERCENT
AFTER CONSOLIDATION		
DRY DENSITY	=	110.2 PCF
WATER CONTENT	=	20.00 PERCENT
BEAM LOAD FACTOR	=	15.33 LBS/PERCENT

AXIAL STRAIN PCT	PORE PRESSURE KSF	DEVIATOR STRESS KSF	SIGMA3 EFFECTIVE KSF	SIGMA1 EFFECTIVE KSF	STRESS RATIO	ABAR	PBAR KSF	QBAR KSF
.09	.73	2.24	5.27	7.51	1.43	.33	6.39	1.12
.18	1.27	3.78	4.73	8.51	1.80	.34	6.62	1.89
.38	1.92	5.91	4.08	9.99	2.45	.32	7.04	2.95
.73	2.19	8.21	3.81	12.02	3.15	.27	7.92	4.11
.92	2.16	9.28	3.84	13.12	3.42	.23	8.48	4.64
1.17	2.02	10.64	3.98	14.62	3.67	.19	9.30	5.32
1.67	1.57	13.33	4.43	17.76	4.01	.12	11.09	6.66
2.29	.86	16.55	5.14	21.69	4.22	.05	13.41	8.28
3.63	-.17	20.70	6.17	26.87	4.35	-.01	16.52	10.35
3.99	-1.50	25.83	7.50	33.33	4.45	-.06	20.41	12.92
5.15	-3.21	31.90	9.21	41.12	4.46	-.10	25.16	15.95
6.26	-4.91	37.41	10.91	48.32	4.43	-.13	29.62	18.71
6.98	-5.95	40.75	11.95	52.69	4.41	-.15	32.32	20.37
7.71	-6.96	43.78	12.96	56.74	4.38	-.16	34.85	21.89
8.13	-7.52	45.46	13.52	58.98	4.36	-.17	36.25	22.73
8.63	-8.12	47.35	14.12	61.48	4.35	-.17	37.80	23.68
9.40	-8.93	49.66	14.93	64.59	4.33	-.18	39.76	24.83
10.35	-9.76	51.82	15.76	67.59	4.29	-.19	41.67	25.91
11.03	-10.20	52.76	16.20	68.96	4.26	-.19	42.58	26.38
11.47	-10.43	52.94	16.43	69.37	4.22	-.20	42.90	26.47
11.89	-10.60	53.13	16.60	69.73	4.20	-.20	43.16	26.56
12.18	-10.71	52.95	16.71	69.67	4.17	-.20	43.19	26.48
12.58	-10.84	52.71	16.84	69.55	4.13	-.21	43.20	26.35
13.41	-11.02	51.78	17.02	68.80	4.04	-.21	42.91	25.89

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A-19

TABLE A 5
TRIAXIAL TEST DATA ON SAND TAILINGS
(CONTINUED)

05/16/74 15:05:17 ATM 000111003 000111 1 50

CONSOLIDATED DRAINED TRIAXIAL TEST

A[illegible]AS MINERALS 5467-003-06 UTAH
BORING 10 SAMPLE 5 DEPTH 16.5 FT.
SAMPLE HEIGHT .... = 4.592 INCHES
SAMPLE VOLUME .... = 339.670 C.C.
SAMPLE AREA .... = 4.514 SQUARE INCHES
CONSOLIDATION PRESSURE = 1500. PSF
BEAM LOAD FACTOR ..... = 6.389 LBS/PERCENT

DELH	ASTRAIN	VSTRAIN	LSTRAIN	POISR	SIGDEVE	SIGMA1E	SIRATIO
29.	.63	.622	-.005	.35	1304.4	2804.4	1.870
52.	1.13	.984	-.074	.34	1872.4	3372.4	2.248
69.	1.50	1.002	-.250	.37	2230.8	3730.8	2.487
121.	2.64	1.409	-.613	.37	3261.0	4761.0	3.174
135.	2.94	1.479	-.730	.38	3474.0	4974.0	3.316
[illegible]3.	3.33	1.508	-.912	.39	3681.0	5181.0	3.454
161.	3.51	1.539	-.984	.39	3835.3	5335.3	3.557
211.	4.59	1.639	-1.478	.40	4606.5	6106.5	4.071
236.	5.14	1.705	-1.717	.41	4721.0	6221.0	4.147
297.	6.47	1.622	-2.423	.43	5406.7	6906.7	4.604
330.	7.19	1.490	-2.848	.44	5626.8	7126.8	4.751
398.	8.67	1.268	-3.700	.46	6128.0	7628.0	5.085
423.	9.21	1.098	-4.057	.46	6155.9	7655.9	5.104
[illegible]7.	10.61	.762	-4.922	.48	6242.8	7742.8	5.162
496.	10.80	.710	-5.046	.48	6225.9	7725.9	5.151
505.	11.00	.643	-5.177	.48	6208.0	7708.0	5.139
513.	11.17	.607	-5.282	.48	6193.6	7693.6	5.129

TABLE A 5

TRIAXIAL TEST DATA ON SAND TAILINGS

(CONTINUED)

ATM,111003,1,50

CONSOLIDATED DRAINED TRIAXIAL TEST

ATLAS MINERALS 5467-003-06 UTAH
BORING 10 SAMPLE 6 DEPTH 19.5 FT.
SAMPLE HEIGHT .... = 5.683 INCHES
AMPLE VOLUME .... = 395.950 C.C.
SAMPLE AREA .... = 4.251 SQUARE INCHES
CONSOLIDATION PRESSURE = 3000. PSF
BEAM LOAD FACTOR ..... = 9.580 LBS/PERCENT

DELH	ASTRAIN	VSTRAIN	LSTRAIN	POISR	SIGDEVE	SIGMA1E	SIRATIO
20.	.35	.263	-.045	.45	1069.8	4069.8	1.357
31.	.55	.431	-.057	.42	1588.1	4588.1	1.529
107.	1.88	1.359	-.262	.38	3389.0	6389.0	2.130
125.	2.20	1.547	-.326	.37	4029.2	7029.2	2.343
145.	2.55	1.746	-.403	.37	4473.4	7473.4	2.491
211.	3.71	2.315	-.699	.36	6077.0	9077.0	3.026
231.	4.06	2.457	-.804	.36	6414.6	9414.6	3.138
260.	4.58	2.619	-.978	.36	6836.3	9836.3	3.279
321.	5.65	2.964	-1.342	.36	8140.1	11140.1	3.713
352.	6.19	3.206	-1.494	.36	8553.5	11553.5	3.851
372.	6.55	3.191	-1.678	.37	8896.0	11896.0	3.965
445.	7.83	3.350	-2.240	.38	9902.2	12902.2	4.301
484.	8.52	3.396	-2.560	.39	10294.1	13294.1	4.431
560.	9.85	3.490	-3.182	.40	11214.3	14214.3	4.738
595.	10.47	3.464	-3.503	.40	11285.1	14285.1	4.762
672.	11.82	3.380	-4.222	.42	11578.5	14578.5	4.859
684.	12.04	3.371	-4.332	.42	11520.1	14520.1	4.840

TABLE A 5

TRIAXIAL TEST DATA ON SAND TAILINGS

(CONTINUED)

ATM,111003,1,50

CONSOLIDATED DRAINED TRIAXIAL TEST

ATLAS MINERALS 5467-003-06 UTAH
BORING 10 SAMPLE 6 DEPTH 19.5 FT.
SAMPLE HEIGHT .... = 5.683 INCHES
SAMPLE VOLUME .... = 395.950 C.C.
SAMPLE AREA .... = 4.251 SQUARE INCHES
CONSOLIDATION PRESSURE = 3000. PSF
BEAM LOAD FACTOR ..... = 9.580 LBS/PERCENT

DELH	ASTRAIN	VSTRAIN	LSTRAIN	POISR	SIGDEVE	SIGMA1E	SIRATIO
20.	.35	.263	-.045	.45	1069.8	4069.8	1.357
31.	.55	.431	-.057	.42	1588.1	4588.1	1.529
107.	1.88	1.359	-.262	.38	3389.0	6389.0	2.130
125.	2.20	1.547	-.326	.37	4029.2	7029.2	2.343
145.	2.55	1.746	-.403	.37	4473.4	7473.4	2.491
211.	3.71	2.315	-.699	.36	6077.0	9077.0	3.026
231.	4.06	2.457	-.804	.36	6414.6	9414.6	3.138
260.	4.58	2.619	-.978	.36	6836.3	9836.3	3.279
321.	5.65	2.964	-1.342	.36	8140.1	11140.1	3.713
352.	6.19	3.206	-1.494	.36	8553.5	11553.5	3.851
372.	6.55	3.191	-1.678	.37	8896.0	11896.0	3.965
445.	7.83	3.350	-2.240	.38	9902.2	12902.2	4.301
484.	8.52	3.396	-2.560	.39	10294.1	13294.1	4.431
560.	9.85	3.490	-3.182	.40	11214.3	14214.3	4.738
595.	10.47	3.464	-3.503	.40	11285.1	14285.1	4.762
672.	11.82	3.380	-4.222	.42	11578.5	14578.5	4.859
684.	12.04	3.371	-4.332	.42	11520.1	14520.1	4.840

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REVISIONS BY ______ DATE ______

TABLE A 5

TRIAXIAL TEST DATA ON SAND TAILINGS

(CONTINUED)

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CONSOLIDATED DRAINED TRIAXIAL TEST

A. AS MINERALS 5467-003-06 UTAH 29
BORING 10, SAMPLE 7, DEPTH 26 FEET, MINE TAILUNG
SAMPLE HEIGHT .... = 5.649 INCHES
SAMPLE VOLUME .... = 379.750 C.C.
SAMPLE AREA .... = 4.102 SQUARE INCHES
CONSOLIDATION PRESSURE = 4500. PSF
BEAM LOAD FACTOR ..... = 9.563 LBS/PERCENT

DELH	ASTRAIN	VSTRAIN	LSTRAIN	POISR	SIGDEVE	SIGMA1E	SIRATIO
81.	1.43	1.753	.160	.31	3712.5	8212.5	1.825
304.	5.38	1.840	-1.771	.43	8755.3	13255.3	2.946
325.	5.75	2.111	-1.821	.42	8939.3	13439.3	2.987
425.	7.52	3.535	-1.994	.39	10836.0	15336.0	3.408
447.	7.91	3.808	-2.053	.38	11239.5	15739.5	3.498
20.	9.21	5.014	-2.096	.36	12380.2	16880.2	3.751
532.	9.42	5.057	-2.180	.36	12517.3	17017.3	3.782
556.	9.84	5.170	-2.336	.36	12729.3	17229.3	3.829
631.	11.17	5.374	-2.898	.37	13421.5	17921.5	3.983
652.	11.54	5.436	-3.053	.37	13688.9	18188.9	4.042
671.	11.88	5.474	-3.202	.37	13924.5	18424.5	4.094
747.	13.22	5.572	-3.826	.38	14529.9	19029.9	4.229
982.	17.38	5.597	-5.893	.41	15014.7	19514.7	4.337
21.	18.07	5.606	-6.234	.41	15240.9	19740.9	4.387
1100.	19.47	5.662	-6.905	.42	15506.6	20006.6	4.446
1207.	21.37	5.764	-7.801	.42	15439.0	19939.0	4.431
1227.	21.72	5.772	-7.974	.43	15231.0	19731.0	4.385
1248.	22.09	5.808	-8.142	.43	15164.5	19664.5	4.370

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TABLE A 5

TRIAXIAL COMPRESSION TEST DATA ON SAND TAILINGS

(CONTINUED)

CONSOLIDATED UNDRAINED TRIAXIAL TESTS

Boring No.	Depth in Feet	Soil Classification	Moisture Content In Percent: Before Consolidation	Moisture Content In Percent: After Consolidation	Dry Density in lbs/cu ft: Before Consolidation	Dry Density in lbs/cu ft: After Consolidation	Confining Pressure in lbs/sq ft	Deviator Stress in lbs/sq ft
1	20.5	SM	11.2	26.5	96	96	1,500	14,352
1	30.5	SM/ML	22.8	25.9	96	96	3,500	14,987
1	35.5	SM/ML	23.5	24.3	97	97	5,500	37,026
4	20.5	SM	10.9	28.3	93	93	2,500	21,161
4	30.5	SM/ML	28.2*	25.8*	89*	89*	4,500	36,897

*Moisture and Densities are approximate.

UNCONSOLIDATED UNDRAINED TRIAXIAL TESTS

Boring No.	Depth in Feet	Soil Classification	Moisture Content In Percent	Dry Density in lbs/cu ft	Confining Pressure in lbs/sq ft	Deviator Stress in lbs/sq ft
A-2	10.0	SM	14.3	95	1,500	6,014
A-6	20.0	SM	24.5	96	3,000	9,294
A-9	20.0	SM	26.0	95	4,500	10,034

TABLE A 6
TRIAXIAL COMPRESSION TEST DATA ON SLIME TAILINGS

CONSOLIDATED UNDRAINED TRIAXIAL TEST
WITH PORE PRESSURE MEASUREMENT

ATLAS MINERALS 5467-018-06 UTAH SEPT 23 77
BORING A1 SAMPLE 5B DEPTH 40.5 FT

SAMPLE HEIGHT = 5.21 IN
SAMPLE AREA = 4.50 SQ IN
CONSOLIDATION PRESSURE = 1.00 KSF
INIT. MAX. PRIN. STRESS = 1.00 KSF
INITIAL PORE PRESSURE = 0.0 PSI

BEFORE CONSOLIDATION
DRY DENSITY = 86.0 PCF
WATER CONTENT = 35.30 PERCENT
AFTER CONSOLIDATION
DRY DENSITY = 88.0 PCF
WATER CONTENT = 35.20 PERCENT
BEAM LOAD FACTOR = 1.92 LBS/PERCENT

AXIAL STRAIN PCT	PORE PRESSURE KSF	DEVIATOR STRESS KSF	SIGMA3 EFFECTIVE KSF	SIGMA1 EFFECTIVE KSF	STRESS RATIO	ABAR	PBAR KSF	QBAR KSF
.29	.23	.42	.77	1.19	1.54	.55	.98	.21
.58	.32	.54	.68	1.23	1.79	.58	.95	.27
.96	.40	.65	.60	1.25	2.09	.62	.92	.32
1.34	.43	.74	.57	1.31	2.30	.59	.94	.37
1.92	.46	.85	.54	1.39	2.57	.54	.96	.42
2.50	.46	.96	.54	1.50	2.79	.48	1.02	.48
3.26	.43	1.07	.57	1.64	2.88	.40	1.10	.53
4.03	.42	1.18	.58	1.76	3.02	.35	1.17	.59
4.80	.40	1.24	.60	1.84	3.08	.32	1.22	.62
5.57	.37	1.30	.63	1.92	3.07	.29	1.27	.65
6.33	.36	1.33	.64	1.97	3.08	.27	1.31	.67
7.10	.35	1.36	.65	2.02	3.08	.25	1.34	.68
7.87	.33	1.37	.67	2.04	3.05	.24	1.36	.69
8.64	.33	1.39	.67	2.06	3.08	.24	1.36	.69
9.60	.32	1.40	.68	2.08	3.04	.23	1.38	.70
10.56	.30	1.41	.70	2.11	3.02	.21	1.40	.70
11.52	.29	1.42	.71	2.13	3.00	.20	1.42	.71
12.48	.29	1.42	.71	2.13	3.00	.20	1.42	.71
13.44	.29	1.43	.71	2.14	3.01	.20	1.43	.71
14.40	.29	1.44	.71	2.15	3.02	.20	1.43	.72
15.36	.29	1.45	.71	2.16	3.03	.20	1.44	.72
16.31	.29	1.46	.71	2.17	3.05	.20	1.44	.73

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FILE 05467-018 Atlas Minerals
BY ____ DATE ____
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TABLE A6

TRIAXIAL COMPRESSION TEST DATA ON SLIME TAILINGS

(CONTINUED)

CONSOLIDATED UNDRAINED TRIAXIAL TEST
WITH PORE PRESSURE MEASUREMENT

ATLAS MINERALS, 5467-018-06 APRIL 7,1977
BORING A2, SAMPLE 4, DEPTH 30.0 FEET BROWN SANDY SILT

SAMPLE HEIGHT	=	5.03 IN
SAMPLE AREA	=	4.36 SQ IN
CONSOLIDATION PRESSURE	=	4.00 KSF
INIT. MAX. PRIN. STRESS	=	4.00 KSF
INITIAL PORE PRESSURE	=	72.5 PSI
BEFORE CONSOLIDATION		
DRY DENSITY	=	88.4 PCF
WATER CONTENT	=	31.80 PERCENT
AFTER CONSOLIDATION		
DRY DENSITY	=	95.6 PCF
WATER CONTENT	=	28.20 PERCENT
BEAM LOAD FACTOR	=	9.58 LBS/PERCENT

AXIAL STRAIN PCT	PORE PRESSURE KSF	DEVIATOR STRESS KSF	SIGMA3 EFFECTIVE KSF	SIGMA1 EFFECTIVE KSF	STRESS RATIO	ABAR	PBAR KSF	QBAR KSF
.22	1.15	1.71	2.85	4.55	1.60	.68	3.70	.85
.99	2.28	2.57	1.72	4.29	2.49	.89	3.01	1.28
1.99	2.45	3.04	1.55	4.59	2.96	.81	3.07	1.52
2.38	2.51	3.09	1.49	4.58	3.07	.81	3.04	1.54
2.78	2.51	3.29	1.49	4.79	3.20	.76	3.14	1.65
3.18	2.51	3.37	1.49	4.87	3.26	.74	3.18	1.69
3.58	2.51	3.51	1.49	5.00	3.35	.71	3.25	1.75
3.97	2.49	3.59	1.51	5.10	3.38	.69	3.30	1.79
4.37	2.49	3.63	1.51	5.14	3.41	.69	3.32	1.82
4.97	2.45	3.76	1.55	5.31	3.42	.65	3.43	1.88
5.96	2.33	3.87	1.67	5.54	3.32	.60	3.60	1.93
6.95	2.26	4.06	1.74	5.80	3.34	.56	3.77	2.03
7.95	2.22	4.17	1.78	5.95	3.34	.53	3.87	2.08
8.34	2.16	4.24	1.84	6.08	3.30	.51	3.96	2.12
8.74	2.13	4.25	1.87	6.11	3.27	.50	3.99	2.12
9.14	2.12	4.26	1.88	6.14	3.26	.50	4.01	2.13
9.54	2.10	4.27	1.90	6.16	3.25	.49	4.03	2.13
9.93	2.09	4.25	1.91	6.16	3.22	.49	4.04	2.12
0.33	2.07	4.23	1.93	6.16	3.20	.49	4.04	2.11
0.93	2.06	4.17	1.94	6.11	3.15	.49	4.03	2.09
1.92	2.04	4.13	1.96	6.06	3.11	.50	4.02	2.06
2.91	2.03	3.94	1.97	5.91	3.00	.52	3.94	1.97

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TABLE A6
TRIAXIAL COMPRESSION TEST DATA ON SLIME TAILINGS
(CONTINUED)

CONSOLIDATED UNDRAINED TRIAXIAL TEST
WITH PORE PRESSURE MEASUREMENT

ATLAS MINERALS 5467-018-06 UTAH SEPT 23 77
BORING A3 SAMPLE 4 DEPTH 30 FT

SAMPLE HEIGHT	=	4.94	IN
SAMPLE AREA	=	4.39	SQ IN
CONSOLIDATION PRESSURE	=	2.00	KSF
INIT. MAX. PRIN. STRESS	=	2.00	KSF
INITIAL PORE PRESSURE	=	0.0	PSI
BEFORE CONSOLIDATION			
DRY DENSITY	=	91.0	PCF
WATER CONTENT	=	32.40	PERCENT
AFTER CONSOLIDATION			
DRY DENSITY	=	94.9	PCF
WATER CONTENT	=	29.10	PERCENT
BEAM LOAD FACTOR	=	1.92	LBS/PERCENT

AXIAL STRAIN PCT	PORE PRESSURE KSF	DEVIATOR STRESS KSF	SIGMA3 EFFECTIVE KSF	SIGMA1 EFFECTIVE KSF	STRESS RATIO	ABAR	PBAR KSF	QBAR KSF
.20	.46	.76	1.54	2.30	1.49	.61	1.92	.38
.40	.69	1.01	1.31	2.32	1.77	.69	1.81	.50
.71	.86	1.24	1.14	2.37	2.09	.70	1.75	.62
1.01	.94	1.37	1.06	2.43	2.29	.68	1.75	.68
1.42	.99	1.54	1.01	2.55	2.53	.64	1.78	.77
2.02	1.01	1.78	.99	2.77	2.79	.57	1.88	.89
2.83	.99	2.06	1.01	3.07	3.05	.48	2.04	1.03
3.64	.95	2.36	1.05	3.41	3.24	.40	2.23	1.18
4.25	.91	2.54	1.09	3.63	3.32	.36	2.36	1.27
4.86	.86	2.73	1.14	3.87	3.41	.32	2.50	1.37
5.47	.82	2.91	1.18	4.09	3.47	.28	2.64	1.46
6.07	.76	3.07	1.24	4.31	3.48	.25	2.77	1.54
7.09	.69	3.33	1.31	4.64	3.54	.21	2.97	1.66
8.10	.60	3.58	1.40	4.98	3.57	.17	3.19	1.79
9.11	.52	3.83	1.48	5.31	3.58	.14	3.40	1.91
10.12	.43	4.04	1.57	5.61	3.58	.11	3.59	2.02
11.13	.35	4.25	1.65	5.90	3.57	.08	3.78	2.12
12.15	.29	4.39	1.71	6.10	3.56	.07	3.91	2.20
13.16	.20	4.53	1.80	6.33	3.52	.04	4.06	2.27

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TABLE A6
TRIAXIAL COMPRESSION TEST DATA ON SLIME TAILINGS
(CONTINUED)

CONSOLIDATED UNDRAINED TRIAXIAL TEST
WITH PORE PRESSURE MEASUREMENT

ATLAS MINERALS, 5467-018-06 APRIL 7,1977
BORING A6 SAMPLE 6 DEPTH 50 FT

SAMPLE HEIGHT	=	4.81 IN
SAMPLE AREA	=	4.11 SQ IN
CONSOLIDATION PRESSURE	=	8.00 KSF
INIT. MAX. PRIN. STRESS	=	8.00 KSF
INITIAL PORE PRESSURE	=	57.0 PSI
BEFORE CONSOLIDATION		
DRY DENSITY	=	82.4 PCF
WATER CONTENT	=	36.30 PERCENT
AFTER CONSOLIDATION		
DRY DENSITY	=	95.9 PCF
WATER CONTENT	=	28.30 PERCENT
BEAM LOAD FACTOR	=	7.67 LBS/PERCENT

AXIAL STRAIN PCT	PORE PRESSURE KSF	DEVIATOR STRESS KSF	SIGMA3 EFFECTIVE KSF	SIGMA1 EFFECTIVE KSF	STRESS RATIO	ABAR	PBAR KSF	QBAR KSF
.48	1.86	2.35	6.14	8.49	1.38	.79	7.32	1.18
.85	2.46	2.40	5.54	7.93	1.43	1.03	6.74	1.20
1.39	3.17	3.15	4.83	7.98	1.65	1.01	6.41	1.58
2.79	4.13	3.66	3.87	7.52	1.95	1.13	5.70	1.83
3.18	4.31	3.77	3.69	7.47	2.02	1.14	5.58	1.89
3.87	4.54	3.95	3.46	7.41	2.14	1.15	5.44	1.98
4.33	4.65	4.03	3.35	7.38	2.20	1.15	5.37	2.02
5.09	4.78	4.13	3.22	7.35	2.28	1.16	5.28	2.06
5.74	4.87	4.20	3.13	7.34	2.34	1.16	5.23	2.10
6.63	4.94	4.26	3.06	7.32	2.39	1.16	5.19	2.13
7.05	4.95	4.27	3.05	7.32	2.40	1.16	5.18	2.13
7.53	4.98	4.30	3.02	7.31	2.42	1.16	5.17	2.15
8.13	5.00	4.32	3.00	7.32	2.44	1.16	5.16	2.16
8.84	5.01	4.31	2.99	7.30	2.44	1.16	5.14	2.15
9.44	5.01	4.33	2.99	7.32	2.45	1.16	5.15	2.16
9.88	5.01	4.33	2.99	7.32	2.45	1.16	5.16	2.17
10.96	5.01	4.26	2.99	7.25	2.42	1.18	5.12	2.13
11.92	4.98	4.19	3.02	7.21	2.39	1.19	5.11	2.09
13.23	4.98	4.08	3.02	7.10	2.35	1.22	5.06	2.04
13.60	4.98	4.02	3.02	7.03	2.33	1.24	5.03	2.01

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BY ______ DATE ______
CHECKED BY ______

TABLE A 6

TRIAXIAL COMPRESSION TEST DATA ON SLIME TAILINGS

(CONTINUED)

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CONSOLIDATED PORE PRESSURE TRIAXIAL TEST

ATLAS MINERALS, UTAH, 5467-003-06 16 MAY 1974

BORING 12, SAMPLE 6, DEPTH 34.5 FEET, LT. BRN. CL) SILT

SAMPLE HEIGHT .... = 5.677 INCHES
SAMPLE AREA .... = 4.319 SQUARE INCHES
CONSOLIDATION PRESSURE = 2500. PSF
INIT. MAX. PRIN. STRESS= 2500. PSF
INITIAL PORE PRESSURE = 70.0 PSI

DEFL MILS	STRAIN	UPRESS PSF	SIGDEV PSF	SIGMA3E PSF	SIGMA1E PSF	RATIO	ABAR
20.	.35	432.0	1135.4	2068.0	3203.4	1.55	.38
40.	.70	1152.0	1787.0	1348.0	3135.0	2.33	.64
60.	1.06	1353.6	2086.2	1146.4	3232.6	2.82	.65
80.	1.41	1454.4	2204.8	1045.6	3250.4	3.11	.66
100.	1.76	1526.4	2343.4	973.6	3317.0	3.41	.65
120.	2.11	1584.0	2460.1	916.0	3376.1	3.69	.64
140.	2.47	1584.0	2565.5	916.0	3481.5	3.80	.62
160.	2.82	1584.0	2618.3	916.0	3534.3	3.86	.60
180.	3.17	1569.6	2732.5	930.4	3662.9	3.94	.57
200.	3.52	1569.6	2804.8	930.4	3735.2	4.01	.56
220.	3.88	1555.2	2886.7	944.8	3831.5	4.06	.54
240.	4.23	1540.8	2978.1	959.2	3937.3	4.10	.52
260.	4.58	1497.6	3048.4	1002.4	4050.8	4.04	.49
280.	4.93	1483.2	3148.5	1016.8	4165.3	4.10	.47
300.	5.28	1440.0	3227.6	1060.0	4287.6	4.04	.45
350.	6.17	1382.4	3457.4	1117.6	4575.0	4.09	.40
400.	7.05	1353.6	3652.7	1146.4	4799.1	4.19	.37
450.	7.93	1296.0	3833.8	1204.0	5037.9	4.18	.34
500.	8.81	1252.8	3981.6	1247.2	5228.8	4.19	.31
550.	9.69	1195.2	4135.5	1304.8	5440.3	4.17	.29
600.	10.57	1152.0	4285.6	1348.0	5633.6	4.18	.27
650.	11.45	1123.2	4413.2	1376.8	5790.0	4.21	.25
700.	12.33	1051.2	4509.3	1448.8	5958.1	4.11	.23
750.	13.21	979.2	4574.9	1520.8	6095.7	4.01	.21
850.	14.97	936.0	4708.4	1564.0	6272.4	4.01	.20
900.	15.85	921.6	4767.2	1578.4	6345.6	4.02	.19
950.	16.73	864.0	4806.0	1636.0	6442.0	3.94	.18
1000.	17.61	849.6	4816.5	1650.4	6466.9	3.92	.18
1020.	17.97	849.6	4813.4	1650.4	6463.8	3.92	.18
1040.	18.32	849.6	4827.5	1650.4	6477.9	3.93	.18
1060.	18.67	849.6	4832.7	1650.4	6483.1	3.93	.18
1080.	19.02	849.6	4837.6	1650.4	6488.0	3.93	.18
1100.	19.38	849.6	4833.8	1650.4	6484.2	3.93	.18
1120.	19.73	849.6	4829.7	1650.4	6480.1	3.93	.18
1140.	20.08	849.6	4834.1	1650.4	6484.5	3.93	.18
1160.	20.43	849.6	4821.2	1650.4	6471.6	3.92	.18
1180.	20.79	849.6	4799.9	1650.4	6450.3	3.91	.18

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REVISIONS BY ______ DATE ______

FILE 20467-008 Atlas Min

BY R. Witt DATE ______ CHECKED BY ______ DATE ______

946.7 (REV.

TABLE A6

TRIAXIAL COMPRESSION TEST DATA ON SLIME TAILINGS

(CONTINUED)

05/10/74 14:25:28 ATM 000111003 000111 1 50

CONSOLIDATED PORE PRESSURE TRIAXIAL TEST

ATLAS MINERALS, UTAH, 5467-003-06

BORING 12, SAMPLE 7, DEPTH 39.5 FEET, LT. BRN. CL) SILT

SAMPLE HEIGHT .... = 5.642 INCHES
SAMPLE AREA .... = 4.430 SQUARE INCHES
CONSOLIDATION PRESSURE = 4500. PSF
INIT. MAX. PRIN. STRESS = 4500. PSF
INITIAL PORE PRESSURE = 72.0 PSI

DEFL MILS	STRAIN	UPRESS PSF	SIGDEV PSF	SIGMA3E PSF	SIGMA1E PSF	RATIO	ABAR
52.	.92	2001.6	1629.5	2498.4	4127.9	1.65	1.23
70.	1.24	2289.6	2104.1	2210.4	4314.5	1.95	1.09
93.	1.65	2548.8	2279.3	1951.2	4230.5	2.17	1.12
119.	2.11	2721.6	2439.3	1778.4	4217.7	2.37	1.12
140.	2.48	2851.2	2539.4	1648.8	4188.2	2.54	1.12
157.	2.78	2894.4	2628.5	1605.6	4234.1	2.64	1.10
182.	3.23	2980.8	2700.9	1519.2	4220.1	2.78	1.10
205.	3.63	3052.8	2773.5	1447.2	4220.7	2.92	1.10
231.	4.09	3096.0	2855.9	1404.0	4259.9	3.03	1.08
254.	4.50	3139.2	2903.2	1360.8	4264.0	3.13	1.08
276.	4.89	3160.0	2950.6	1332.0	4282.6	3.22	1.07
308.	5.46	3211.2	3015.5	1288.8	4304.3	3.34	1.06
343.	6.08	3225.6	3077.6	1274.4	4352.0	3.41	1.05
352.	6.24	3254.4	3107.4	1245.6	4353.0	3.49	1.05
375.	6.65	3254.4	3140.4	1245.6	4386.0	3.52	1.04
400.	7.09	3254.4	3171.8	1245.6	4417.4	3.55	1.03
450.	7.98	3254.4	3244.8	1245.6	4490.4	3.60	1.00
500.	8.86	3268.8	3304.4	1231.2	4535.6	3.68	.99
550.	9.75	3268.8	3351.0	1231.2	4582.2	3.72	.98
600.	10.63	3268.8	3384.9	1231.2	4616.1	3.75	.97
650.	11.52	3268.8	3461.5	1231.2	4692.7	3.81	.94
700.	12.41	3268.8	3437.8	1231.2	4669.0	3.79	.95
750.	13.29	3268.8	3435.4	1231.2	4666.6	3.79	.95
770.	13.65	3268.8	3432.1	1231.2	4663.3	3.79	.95
790.	14.00	3268.8	3418.0	1231.2	4649.2	3.78	.96
810.	14.36	3268.8	3393.3	1231.2	4624.5	3.76	.96
830.	14.71	3268.8	3358.0	1231.2	4589.2	3.73	.97
850.	15.07	3268.8	3312.3	1231.2	4543.5	3.69	.99

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FILE 05462-... Atlas Minerals

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TABLE A6
TRIAXIAL COMPRESSION TEST DATA ON SLIME TAILINGS
(CONTINUED)

UNCONSOLIDATED UNDRAINED SINGLE PHASE TRIAXIAL TESTS

Boring No.	Depth in Feet	Soil Classification	Moisture Content in Percent		Dry Density in lbs/cu ft		Confining Pressure in lbs/sq ft	Deviator Stress in lbs/sq ft
			Before Test	After Test	Before Test	After Test		
3	50.5	ML	38.4	43.7	82	82	2,500	4,672
3	55.5	ML	34.5	36.4	84	84	3,500	5,890
3	60.5	ML	39.9	41.0	79	79	4,500	4,626
14A	2.0	ML	21.8		99.9		200	1,985
14A	3.5	ML	26.7		94.5		350	1,376
14A	5.0	ML	26.9		98.5		500	1,401
14A	8.5	ML	23.2		103.9		850	2,638

UNCONSOLIDATED UNDRAINED MULTI-PHASE TRIAXIAL TESTS

Boring No.	Depth in Feet	Soil Classification	Moisture Content in Percent	Dry Density in lbs/cu ft	Confining Pressure in lbs/sq ft	Deviator Stress in lbs/sq ft
A-2	40.0	ML	23.8	101	1,500	11,400
					3,000	17,858
					4,500	20,839

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TABLE A7

TRIAXIAL COMPRESSION TEST DATA
ON NATURAL RED SANDY SOIL

Unconsolidated-Undrained, Multi-Phase

Boring Number	Depth in ft	Soil Classification	Moisture Content in %	Dry Density in lbs/ cu ft	Confining Pressure in lbs/ cu ft	Deviator Stress in lbs/ sq ft
A-8	20.0	SP	18.1	102	1,000	4,463
					2,000	7,222
					3,000	9,459
A-11	50.0	SP	7.5	102	1,000	4,608
					2,000	7,492
					3,000	9,885
A-12	8.5	SM/GM	6.3	110	1,000	4,030
					2,000	6,537
					3,000	9,081
A-13	11.0	GM	11.9	117	1,000	4,333
					2,000	6,669
					3,000	8,909

ONE-DIMENSIONAL COMPRESSION

One-dimensional compression (consolidation) tests were performed on several samples in order to estimate the compressibility of the natural foundation material. The tests were performed according to the general procedure shown on Plate A-10, Method of Performing Consolidation Tests. Resulting compression curves are shown on Plates A-11A and A-11B, Consolidation Test Data.

PERCOLATION

Several series of percolation tests were performed in order to determine the permeability of undisturbed materials. The tests were categorized into three main groups: (1) sand tailings material (SM), (2) slimes tailings material (ML), and (3) natural red sandy foundation material. The test results are presented in Table A8, on the following page. The method used on performing percolation tests is presented on Plate A-12.

TABLE A8

PERCOLATION TEST DATA

Boring Number	Depth in feet	Soil Type	Surcharge Pressure in lbs/sq ft	Permeability K in feet per year
		Sand Tailings Material		
10	6.5	SM	200	4.0
10	12.5	SM	400	2.0
10	39.5	SP/SM	800	1200.0*
11	18.5	SM	400	20.0
12	8.5	SM	200	35.0
13	17.5	SM	200	14.0
A4	10.0	SM	4,000	550.0
A4	20.0	SM	4,000	54.0
A4	40.0	SM	4,000	73.0
A5	10.0	SM	4,000	16.0
A5	30.0	SM	4,000	210.0
A6	30.0	SM	4,000	58.0
		Slime Tailings Material		
A1	50.0	ML	4,000	1.3
A5	70.0	ML	4,000	0.2
A6	40.0	ML	4,000	2.4
A11	30.0	ML	4,000	0.1
A11	40.0	ML	4,000	0.1
		Natural Red Sandy Soils		
8	5.5	SM	200	13.0
8	18.5	SM/GP	400	99.0
13	49.5	SM	800	490.0
15	8.5	SP	200	370.0
15	13.5	SP	400	110.0
15	18.5	SP	800	230.0
15	39.5	SP/SM	800	180.0
A1	60.5	SP	4,000	1.3
A3	43.0	SP	4,000	88.0
A6	62.0	SP	4,000	2000.0

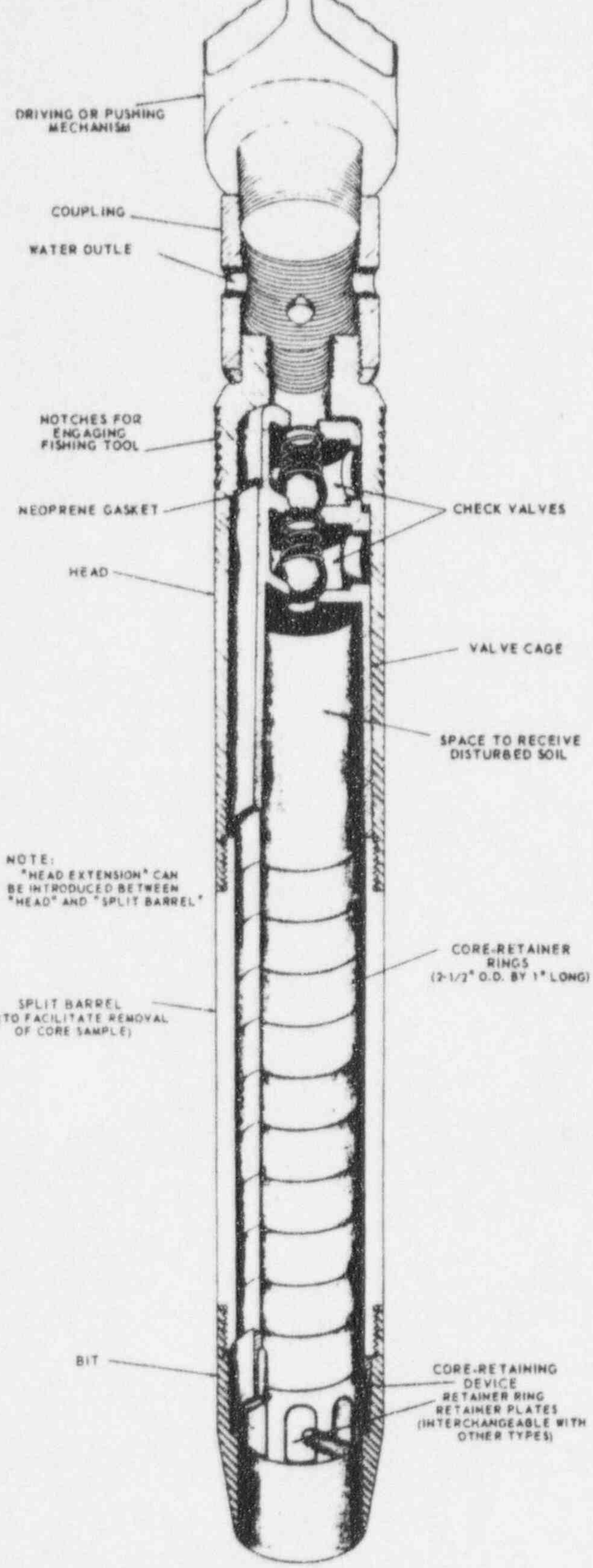
*Loss of soil during test - results invalid.

In addition, field permeability tests were performed where the natural red sandy soils were encountered in the borings. The results of these tests are shown in Table A9.

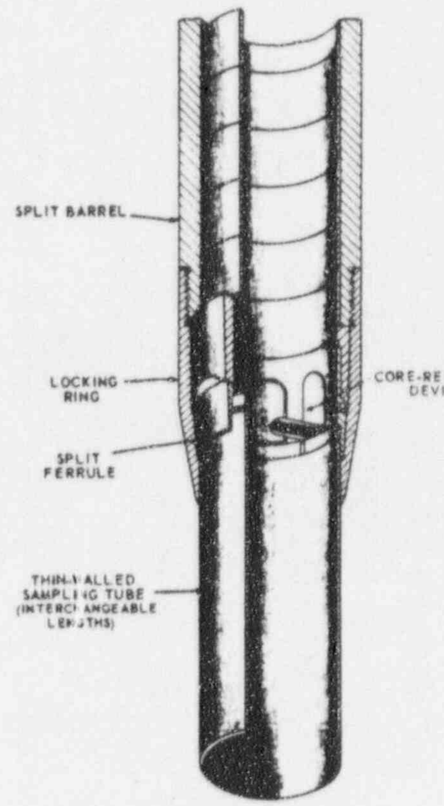
TABLE A9

FIELD PERMEABILITY TESTS ON NATURAL SOILS

Boring Number	Depth in feet	Soil Type	Permeability K in feet/year
8	13.0	SP-GP	850.0
8	18.0	SP-GP	1400.0
8	28.0	SP-GP	8000.0
15	8.0	SP	256.0
15	13.0	SP	430.0
15	18.0	SP	220.0



ALTERNATE ATTACHMENTS



SOIL SAMPLER TYPE U

U. S. PATENT NO. 2,318,062

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PLATE A-1

REVISED 1-73 SLC BY [signature] DATE
CHECKED BY DATE
FILE 05721-013 [handwritten]
REVISIONS BY DATE

MAJOR DIVISIONS			GRAPH SYMBOL	LETTER SYMBOL	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS	GRAVEL AND GRAVELLY SOILS	CLEAN GRAVELS (LITTLE OR NO FINES)		GW	WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES
				GP	POORLY-GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES
	MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)		GM	SILTY GRAVELS, GRAVEL-SAND-SILT MIXTURES
				GC	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES
	SAND AND SANDY SOILS	CLEAN SAND (LITTLE OR NO FINES)		SW	WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE				SP	POORLY-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
	MORE THAN 50% OF COARSE FRACTION PASSING NO. 4 SIEVE	SANDS WITH FINES (APPRECIABLE AMOUNT OF FINES)		SM	SILTY SANDS, SAND-SILT MIXTURES
				SC	CLAYEY SANDS, SAND-CLAY MIXTURES
FINE GRAINED SOILS	SILTS AND CLAYS	LIQUID LIMIT LESS THAN 50		ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS OR CLAYEY SILTS WITH SLIGHT PLASTICITY
				CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
				OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS	LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILTY SOILS
MORE THAN 50% OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE				CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS
				OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC SOILS				PT	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS

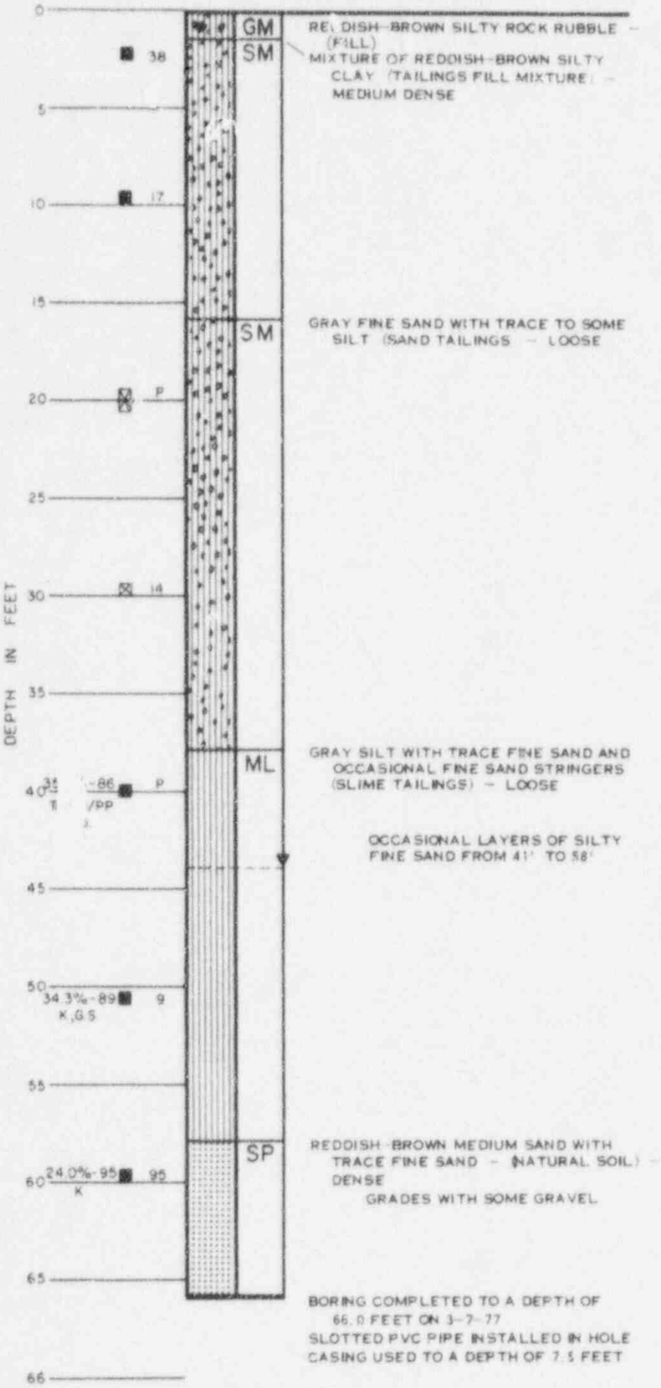
NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS.

SOIL CLASSIFICATION CHART

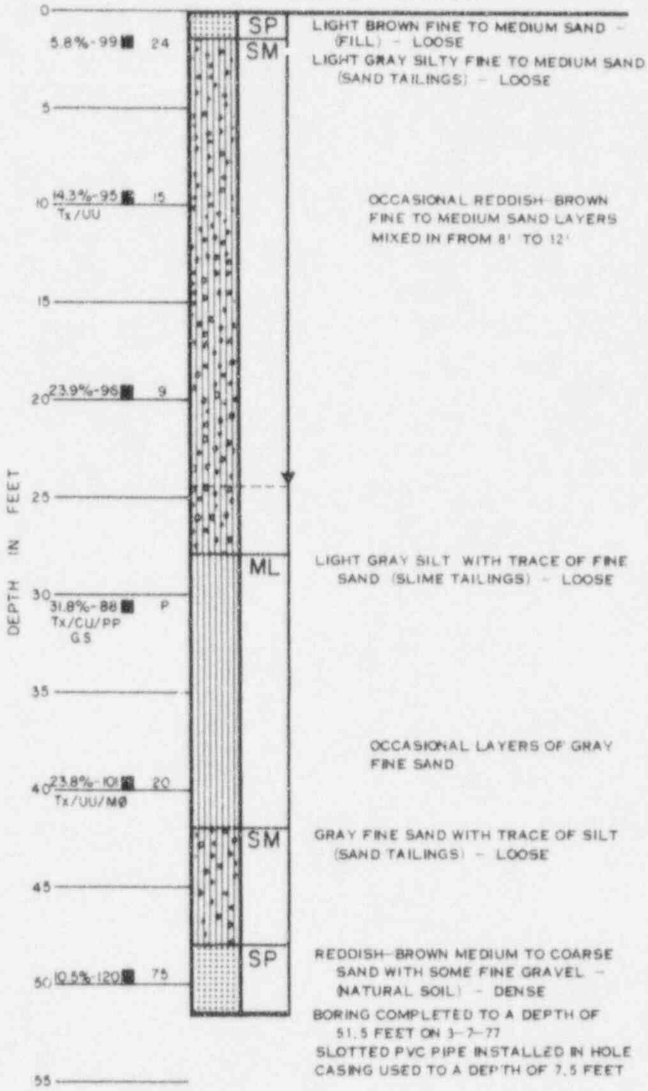
UNIFIED SOIL CLASSIFICATION SYSTEM

DAMES & MOORE

BORING A1



BORING A2



KEY

A - B ■ C
D

A FIELD MOISTURE EXPRESSED AS A PERCENTAGE OF THE DRY WEIGHT OF SOIL
B DRY DENSITY EXPRESSED IN LBS. PER CUBIC FOOT
C BLOWS PER FOOT OF PENETRATION USING A 140 LB. HAMMER DROPPING 30 INCHES
D TYPE OF TESTS PERFORMED ON SAMPLE:

G.S. - GRAIN SIZE ANALYSIS
D.S. - DIRECT SHEAR TEST
K - PERCOLATION (PERMEABILITY) TEST
CONSOL - CONSOLIDATION TEST
Tx/_/_ - TRIAXIAL COMPRESSION TEST

■ DEPTH AT WHICH UNDISTURBED SAMPLE WAS EXTRACTED
⊠ DEPTH AT WHICH DISTURBED SAMPLE WAS EXTRACTED
□ SAMPLING ATTEMPT WITH NO RECOVERY

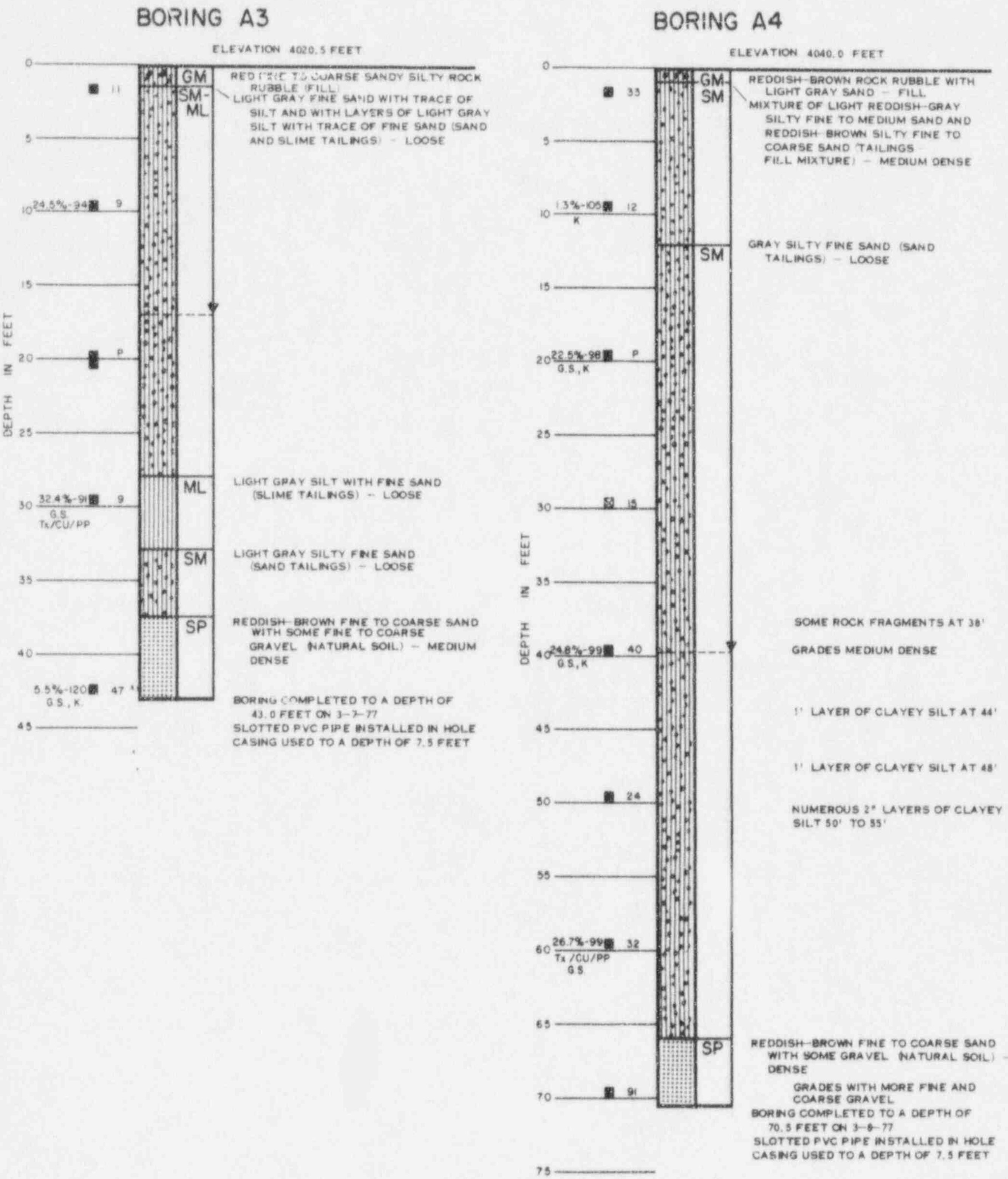
NOTES

ELEVATIONS OF BORINGS ARE BASED ON U.S.G.S. DATUM. PLANT ELEVATION 500.0 CORRESPONDS TO 4028.2 MSL.

LOG OF BORINGS

DAMES & MOORE

PLATE A-3A



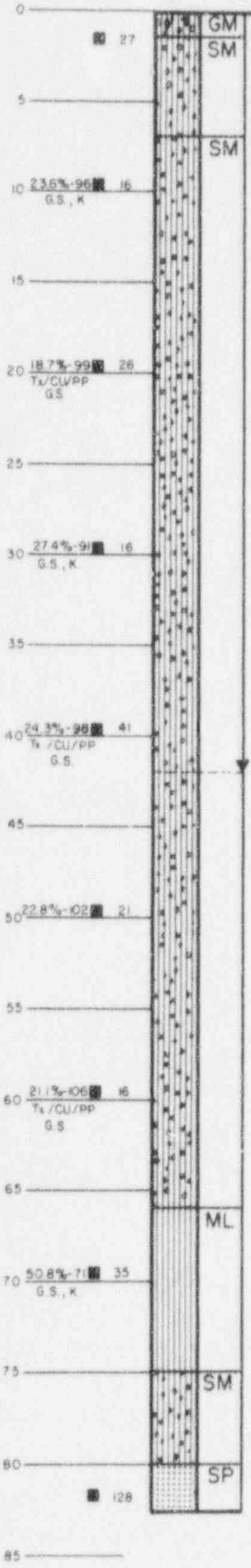
LOG OF BORINGS

DAMES & MOORE

PLATE A-3B

BORING A5

ELEVATION 4039.8 FEET



GM — REDDISH-BROWN SILTY ROCK FRAGMENTS - (FILL)

SM — MIXTURE OF REDDISH-GRAY SILTY FINE TO MEDIUM SAND AND REDDISH-BROWN SILTY FINE TO MEDIUM SAND (TAILINGS - FILL MIXTURE) - MEDIUM DENSE

SM — LIGHT GRAY SILTY FINE TO MEDIUM SAND (SAND TAILINGS) - LOOSE

GRADES WITH MORE SILT

GRADES MEDIUM DENSE

GRADES WITH LESS SILT

GRADES WITH MEDIUM SAND, LESS SILT

ML — LIGHT GRAY CLAYEY SILT WITH FINE SAND LAYERS (SLIME TAILINGS) - MEDIUM DENSE

SM — LIGHT GRAY SILTY FINE TO MEDIUM SAND (SAND TAILINGS)

SP — REDDISH BROWN FINE TO MEDIUM SAND WITH FINE GRAVEL (NATURAL SOIL) VERY DENSE

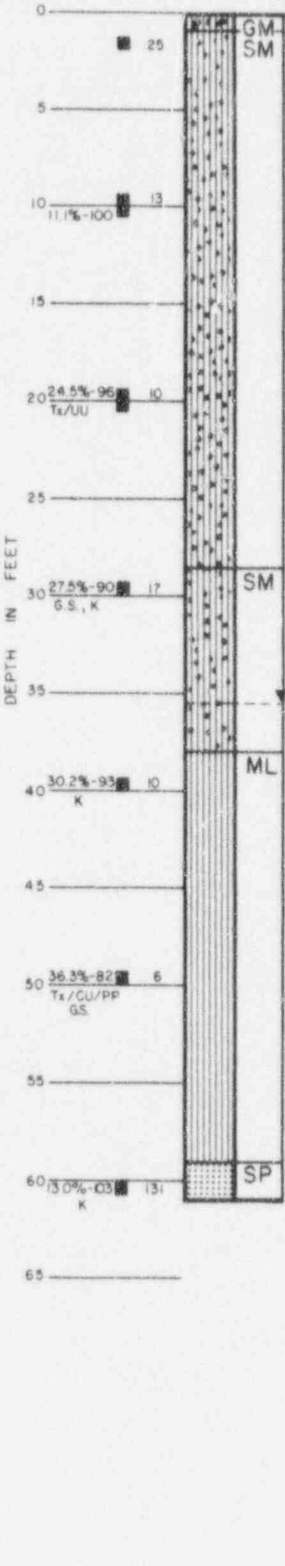
BORING COMPLETED TO A DEPTH OF 82.5 FEET ON 3-8-77

SLOTTED PVC PIPE INSTALLED IN HOLE

CASING USED TO A DEPTH OF 7.5 FEET

BORING A6

ELEVATION 4040.7 FEET



GM/SM — REDDISH-BROWN ROCK RUBBLE WITH SILT AND SAND - (FILL)
MIXTURE OF REDDISH LIGHT GRAY SILTY FINE SAND AND REDDISH-BROWN SILTY FINE TO MEDIUM SAND (TAILINGS - FILL MIXTURE) - MEDIUM DENSE

GRADES LOOSE

SM — TANNISH-GRAY SILTY FINE SAND (SAND TAILINGS) - LOOSE

ML — LIGHT GRAY CLAYEY SILT WITH FINE SAND (SLIME TAILINGS) - LOOSE

GRADES WITH LESS CLAY AND MORE FINE SAND

GRADES WITH MORE CLAY AND LESS FINE SAND - VERY LOOSE

SP — REDDISH-BROWN FINE TO COARSE SAND WITH SOME SANDSTONE GRAVEL (NATURAL SOIL) - VERY DENSE

BORING COMPLETED TO A DEPTH OF 61.0 FEET ON 3-8-77

SLOTTED PVC PIPE INSTALLED IN HOLE

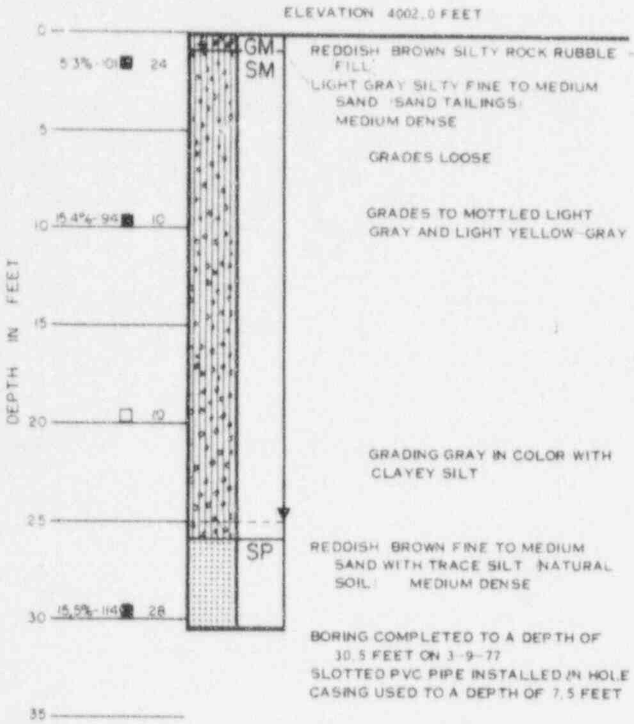
CASING USED TO A DEPTH OF 7.5 FEET

LOG OF BORINGS

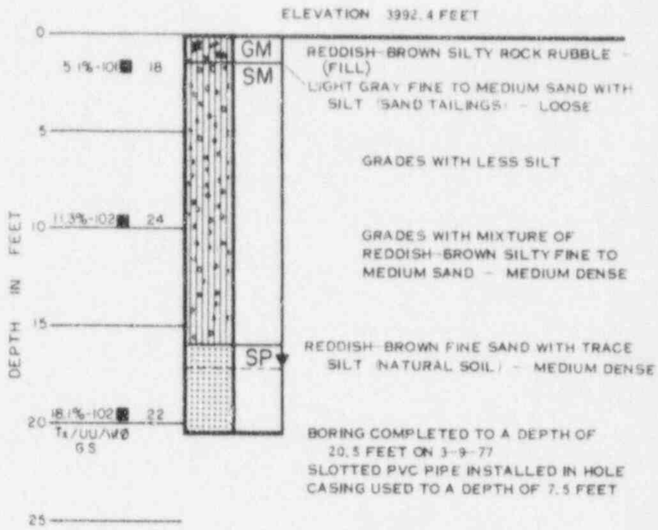
DAMES & MOORE

FILE 05467-018
ATLAS MINERALS
BY [illegible] DATE 4-5-77
CHECKED BY [illegible] DATE [illegible]
REVISIONS
BY DATE
BY DATE
PLATE OF

BORING A7



BORING A8

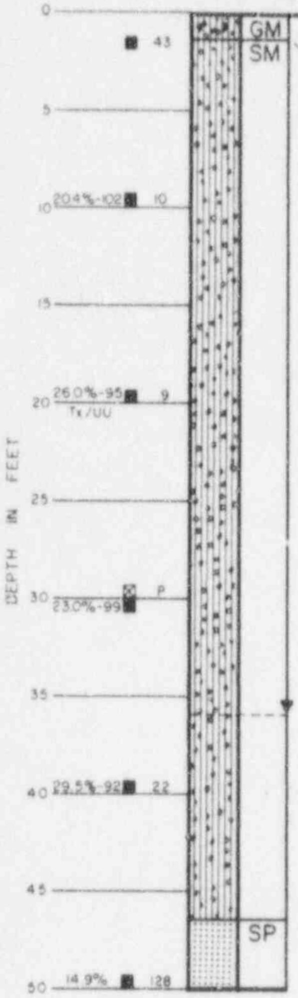


LOG OF BORINGS

DAMES & MOORE

BORING A9

ELEVATION 4007.9 FEET



GM — REDDISH-BROWN SILTY ROCK RUBBLE - (FILL)

SM — MIXTURE OF REDDISH-BROWN SILTY SANDY FINE GRAVEL AND GRAY SILTY FINE TO MEDIUM SAND (TAILINGS - FILL MIXTURE) - MEDIUM DENSE

GRADES LOOSE

GRADES WITH SOME SMALL ZONES OF LIGHT GRAY CLAYEY SILT - LOOSE

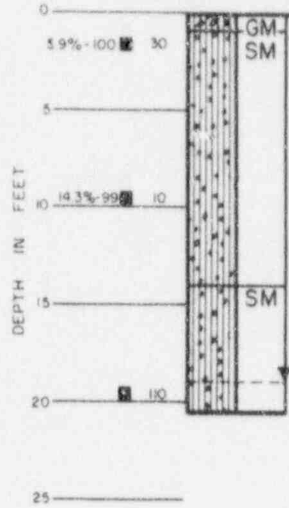
GRADES TO BROWNISH-GRAY SILTY FINE TO MEDIUM SAND - LOOSE

SP — REDDISH-BROWN FINE SAND WITH SOME MEDIUM SAND (NATURAL SOIL) - DENSE

BORING COMPLETED TO A DEPTH OF 50.0 FEET ON 3-9-77
SLOTTED PVC PIPE INSTALLED IN HOLE
CASING USED TO A DEPTH OF 7.5 FEET

BORING A10

ELEVATION 3986.6 FEET



GM — REDDISH-GRAY SANDSTONE ROCK RUBBLE - (FILL)

SM — LIGHT GRAY SILTY FINE TO MEDIUM SAND (SAND TAILINGS) MEDIUM DENSE

SM — REDDISH-BROWN SILTY FINE TO MEDIUM SAND WITH SOME COARSE SAND AND FINE GRAVEL - (FILL) - DENSE

BORING COMPLETED TO A DEPTH OF 20.5 FEET ON 3-9-77
SLOTTED PVC PIPE INSTALLED IN HOLE
CASING USED TO A DEPTH OF 7.5 FEET

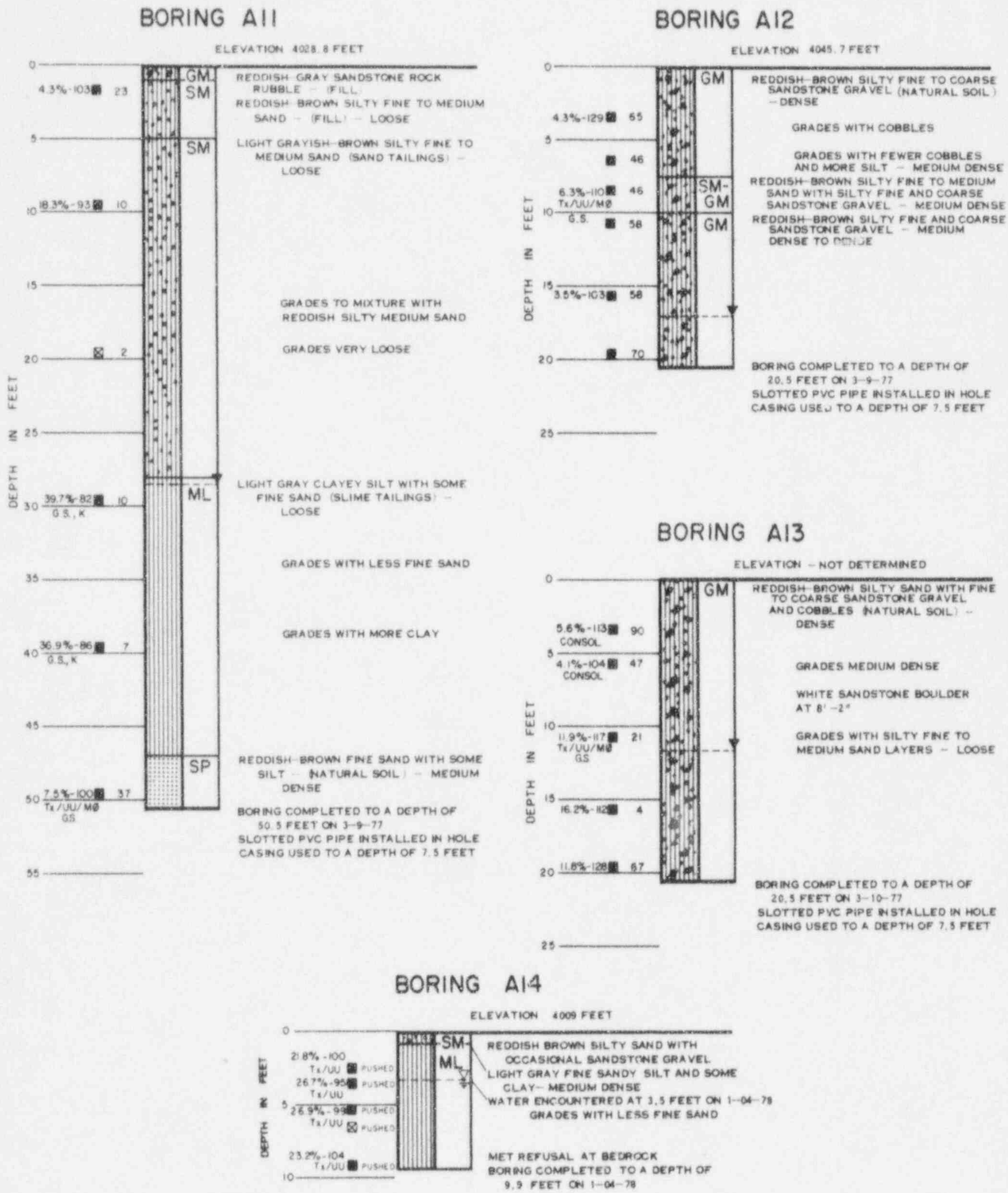
LOG OF BORINGS

DAMES & MOORE

PLATE A-3E

REVISIONS BY ___ DATE ___ BY ___ DATE ___ PLATE ___ OF ___

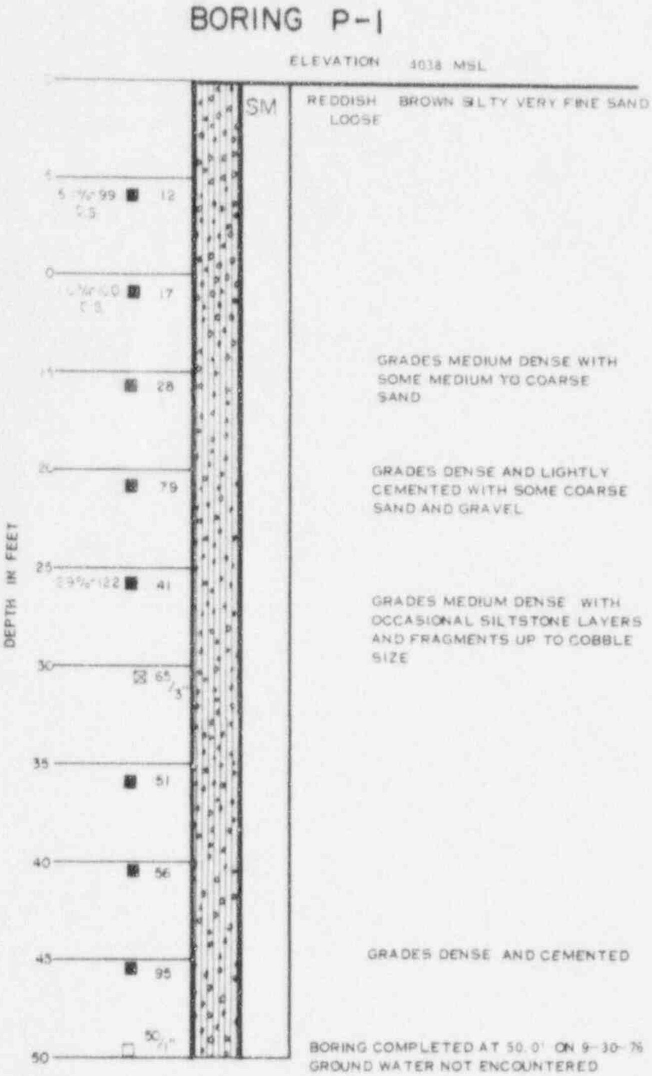
FILE 05467-018
ATLAS MINERALS
BY NBM DATE 4-5-77
CHECKED BY ___ DATE ___

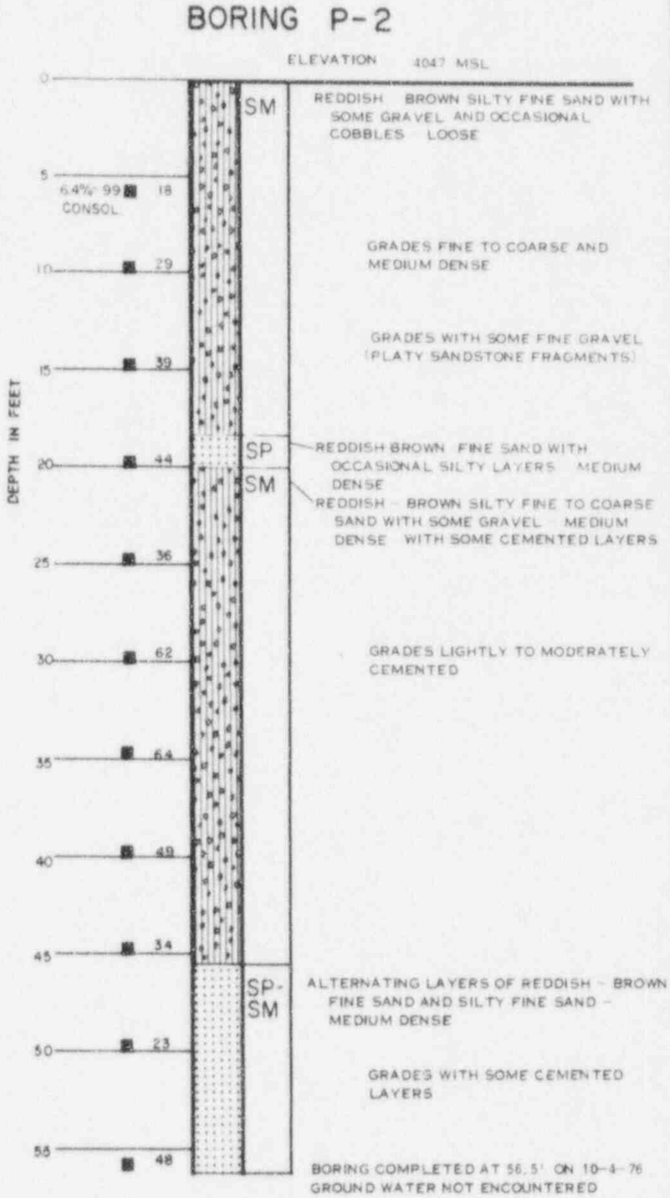


LOG OF BORINGS

DAMES & MOORE

PLATE A-3F





LOG OF BORINGS

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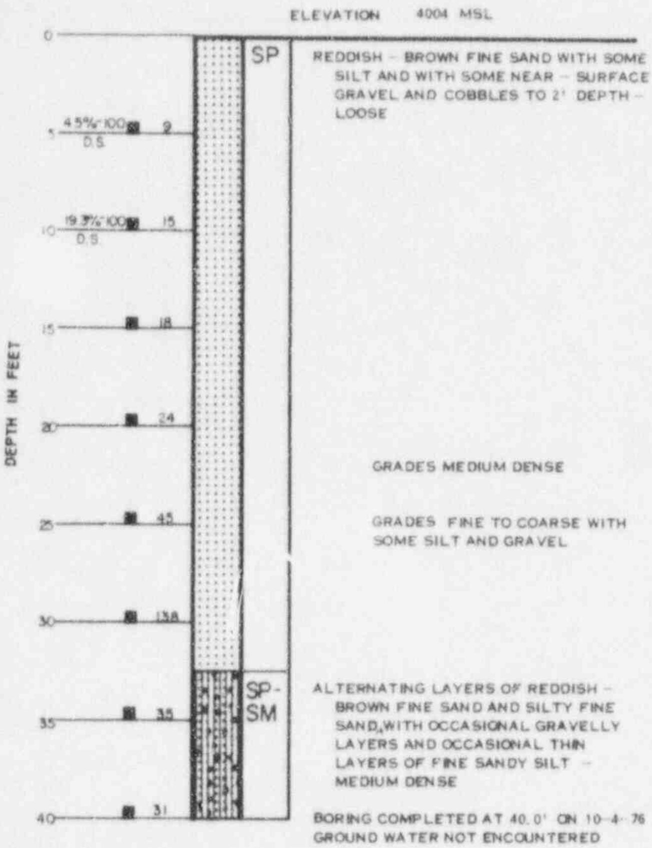
PLATE A-3G

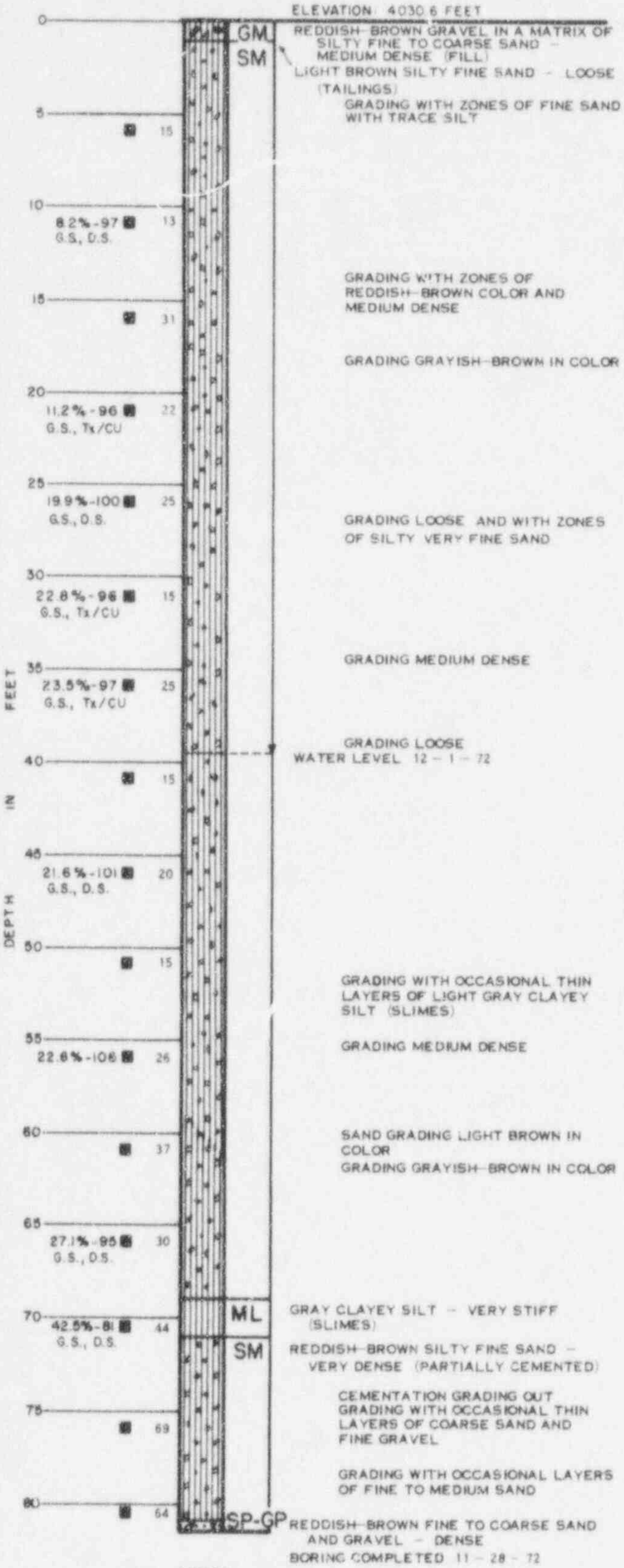
BORING P-3

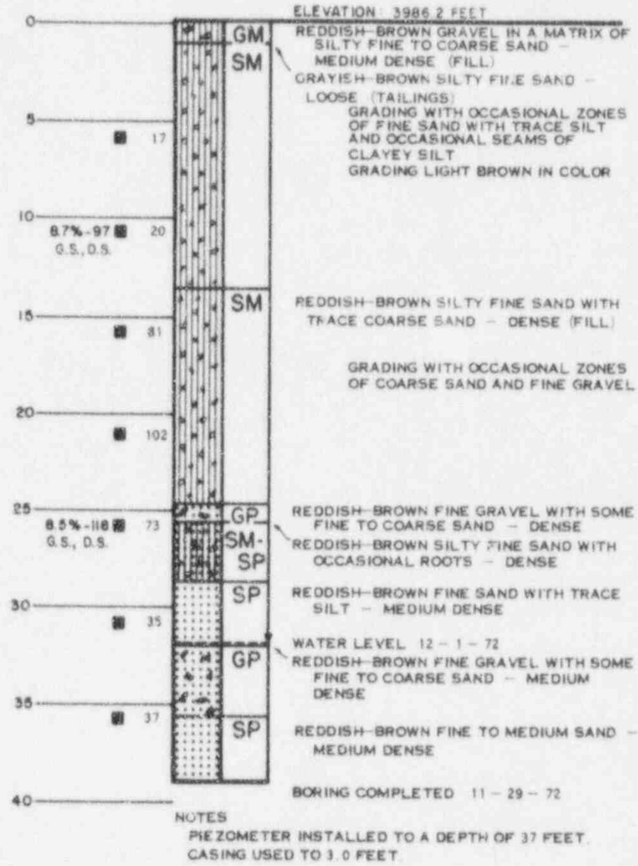


LOG OF BORINGS

DAMES & MOORE

PLATE A-3H





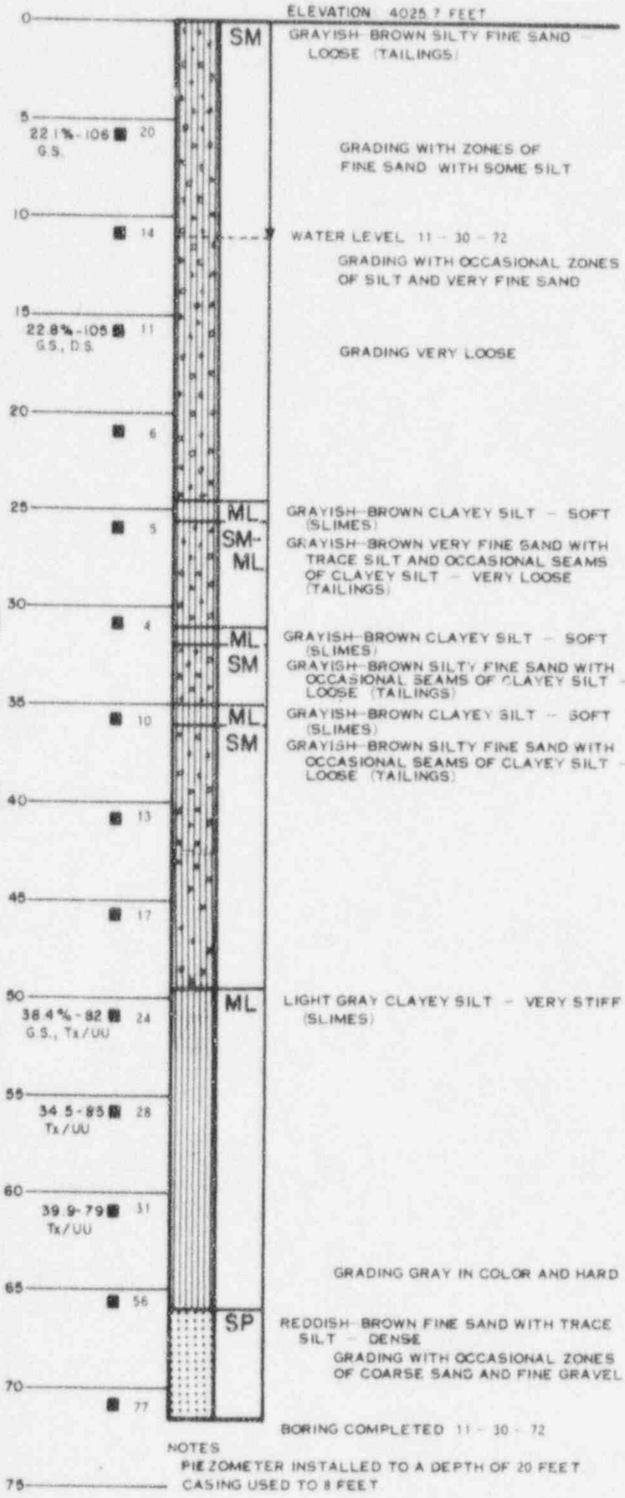
LOG OF BORINGS

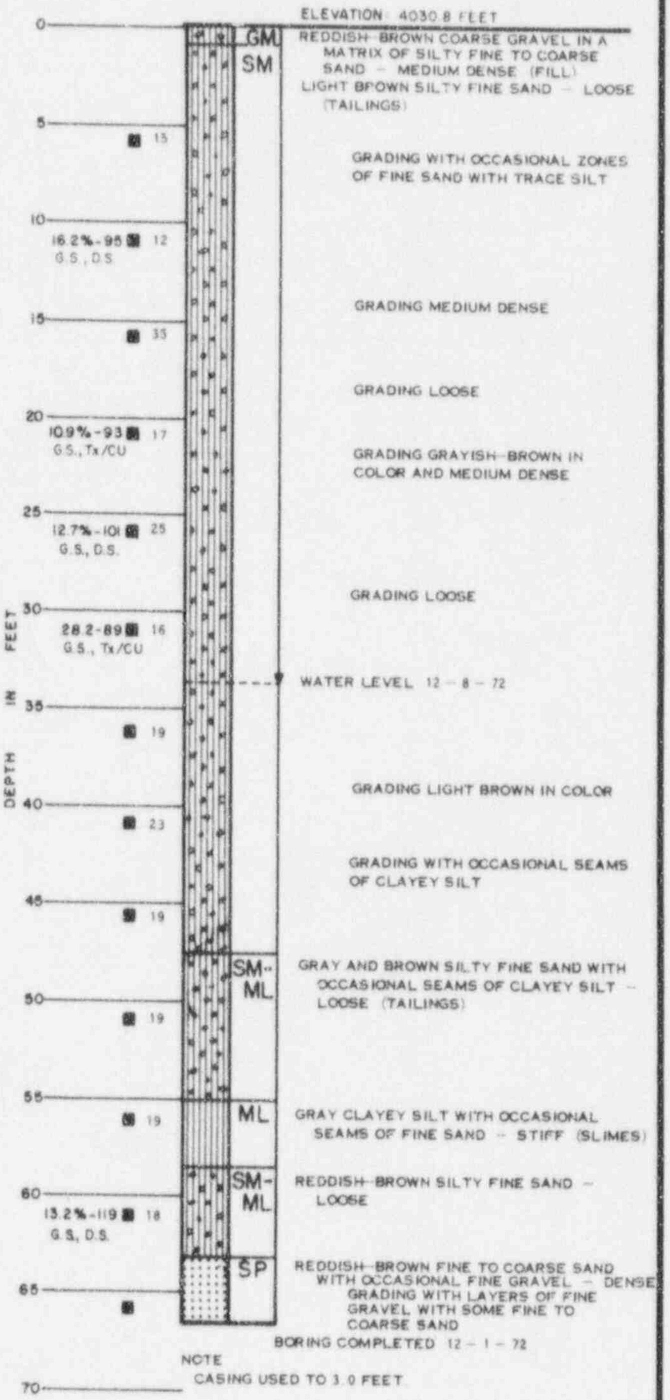
REVISIONS
BY _X_ DATE ____
BY ____ DATE ____
PLATE ____ OF ____

FILE 5467-002
ATLAS MINERALS - MOAB, UTAH
BY ABD DATE 12-19-72
CHECKED BY ____ DATE ____

DAMES & MOORE

PLATE A-31

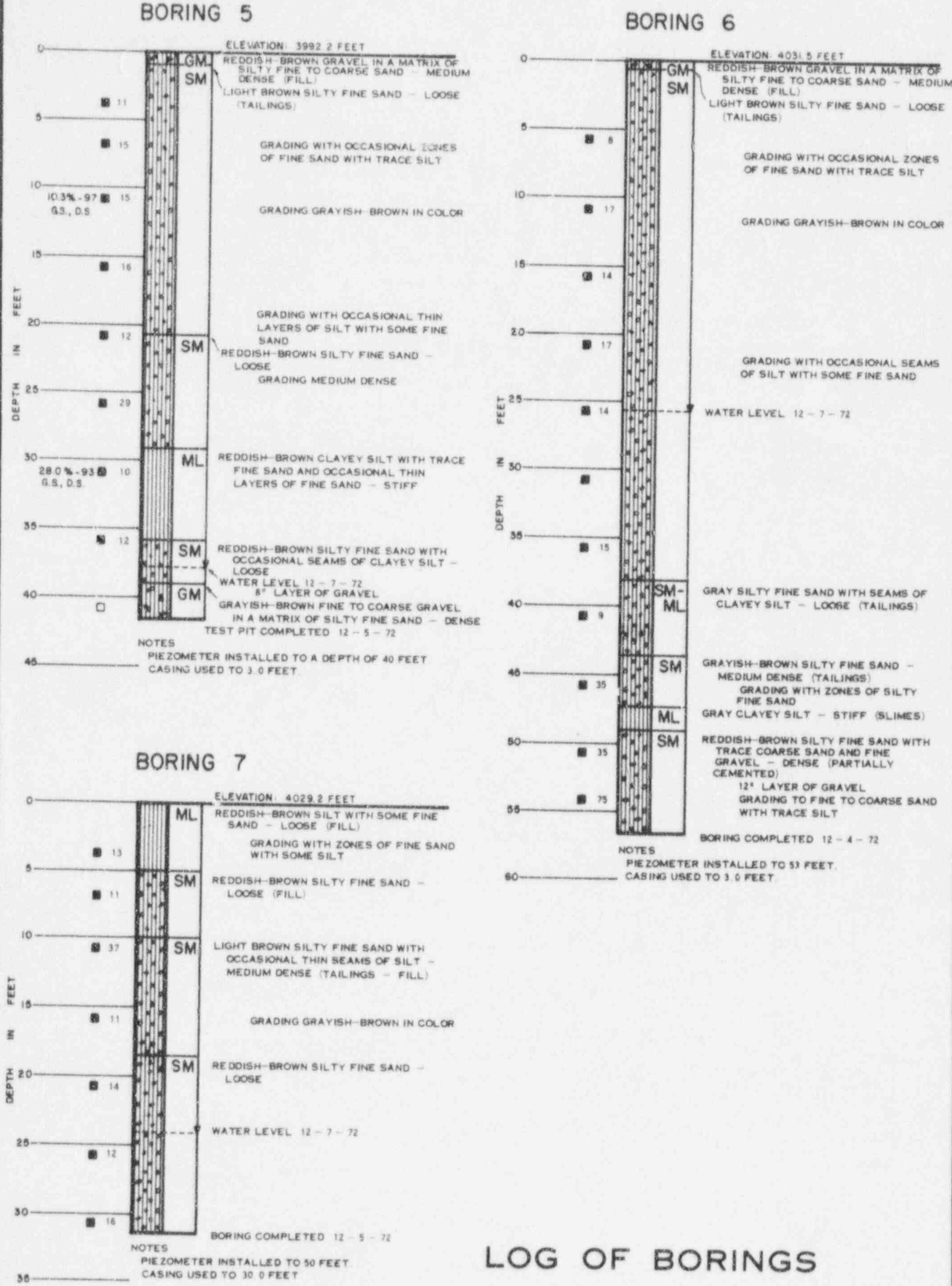




LOG OF BORINGS

DAMES & MOORE

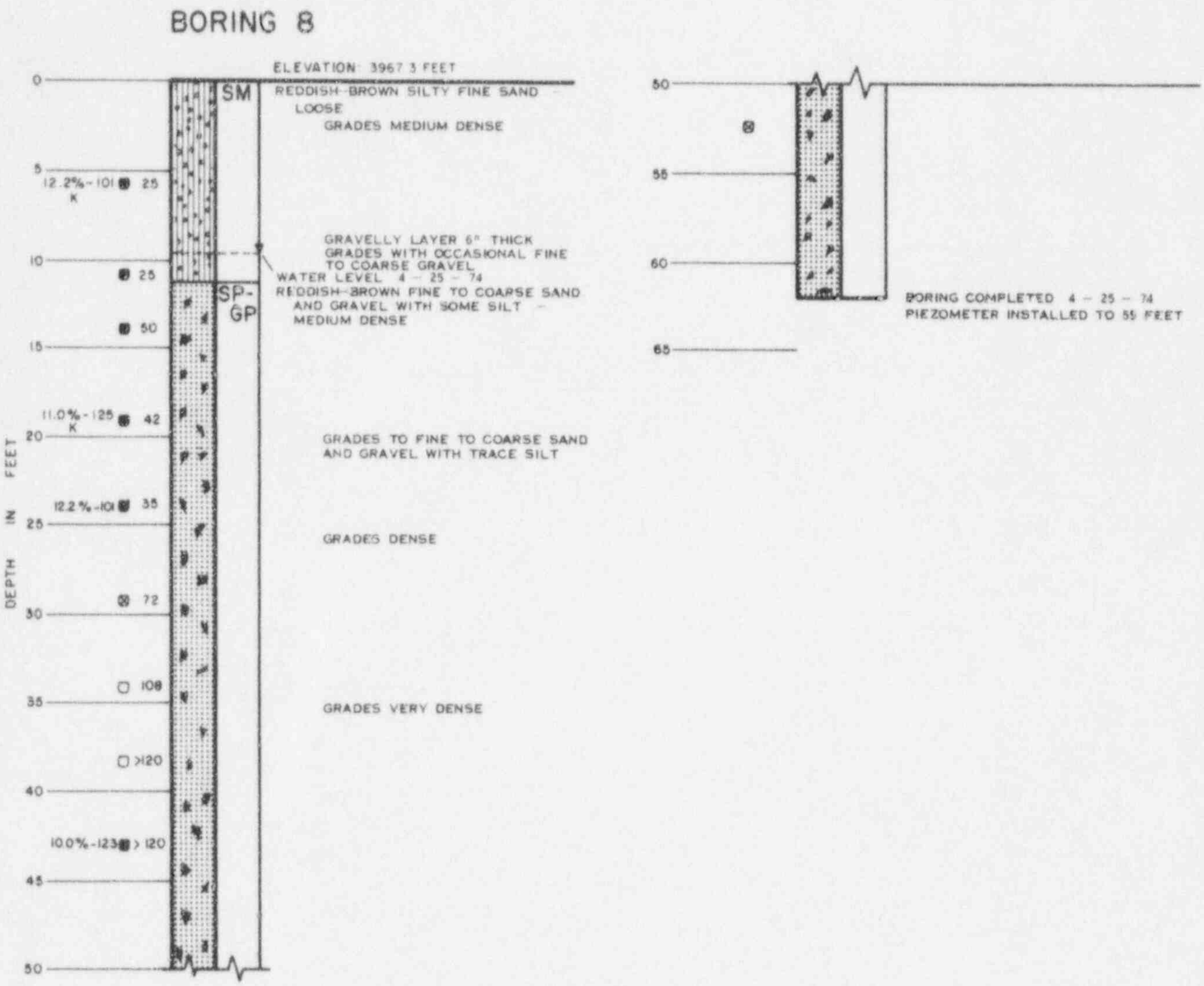
PLATE A-3J



LOG OF BORINGS

DAMES & MOORE

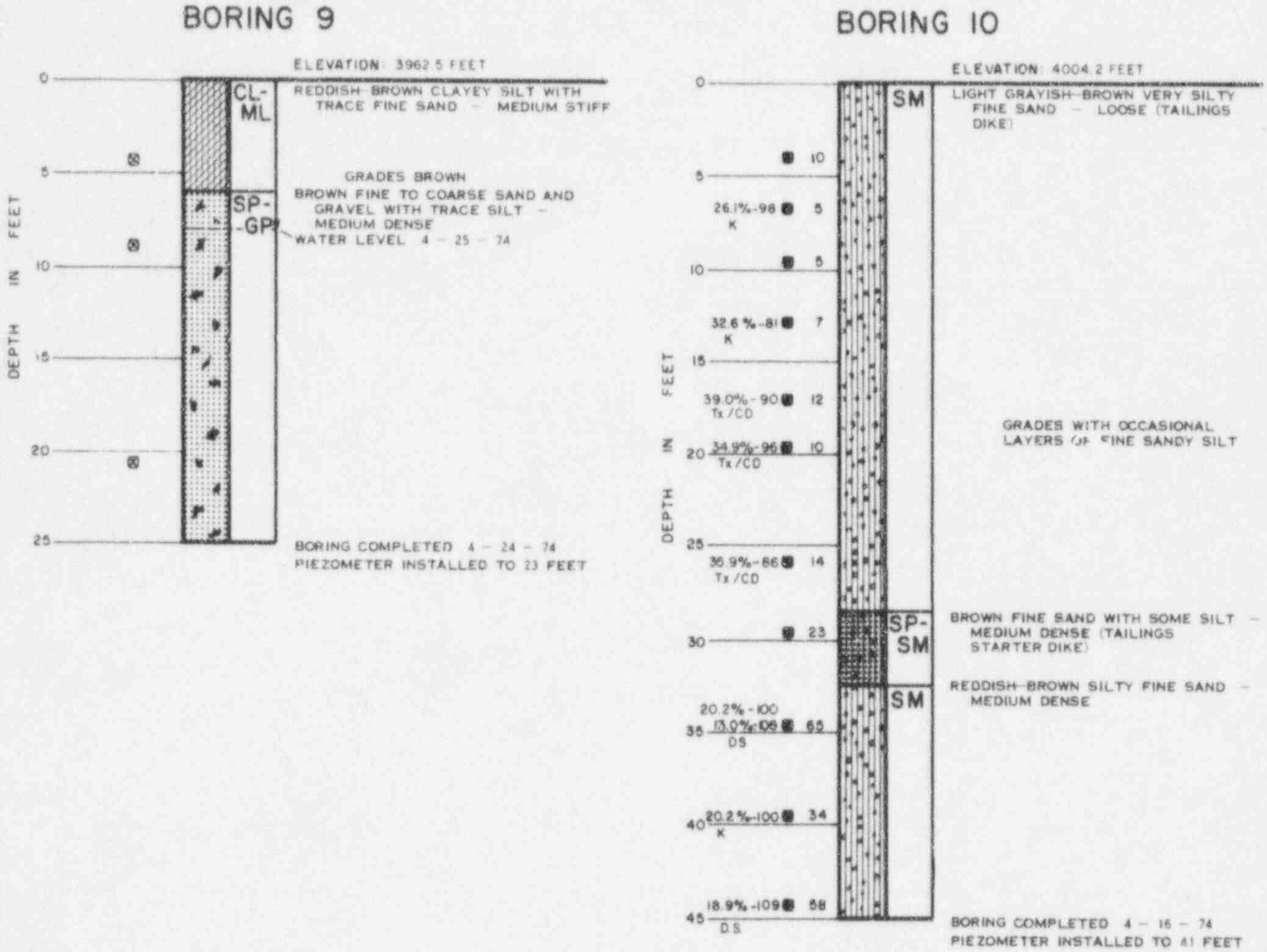
PLATE A-3K



LOG OF BORING

DAMES & MOORE

PLATE A-3L

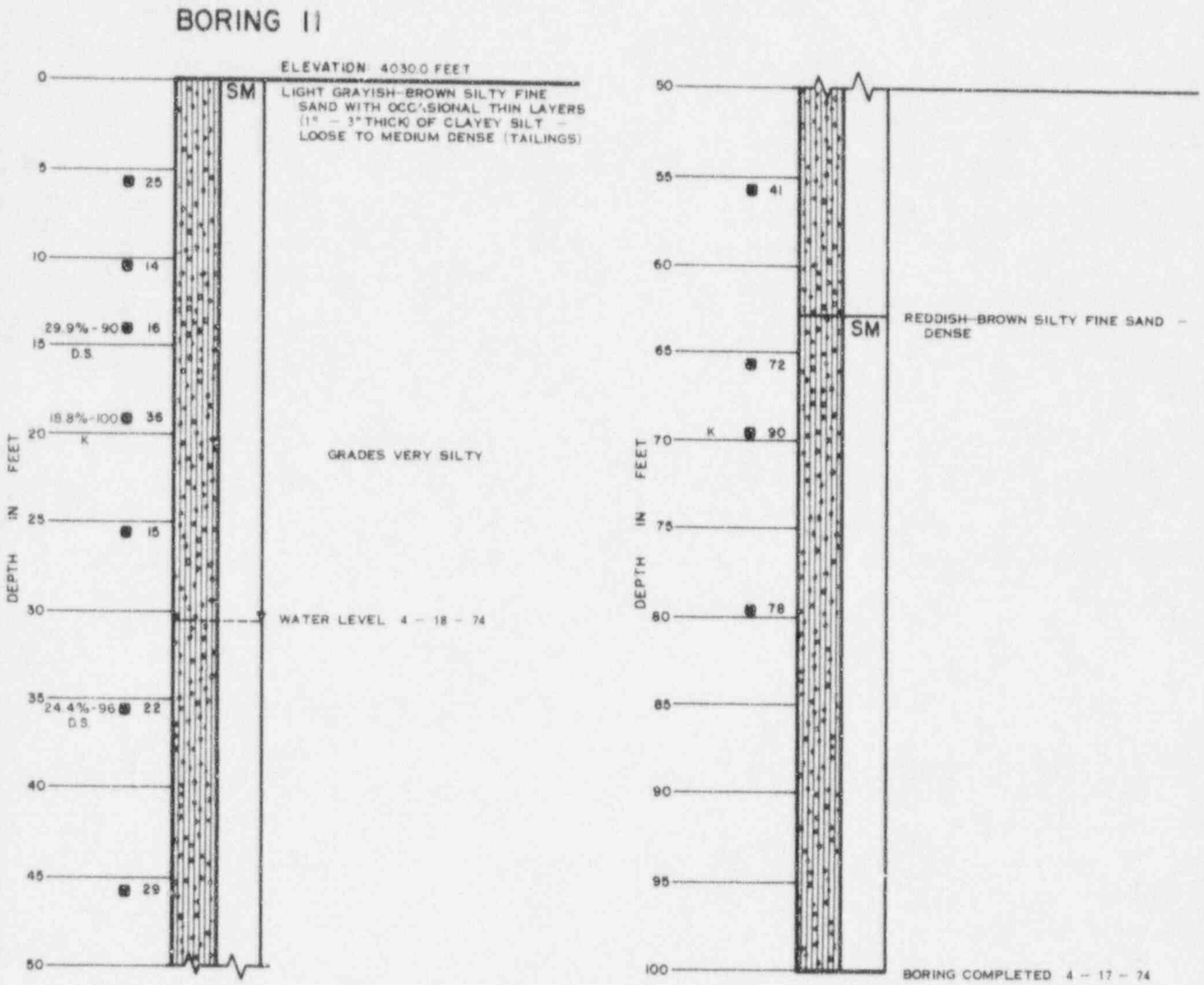


LOG OF BORINGS

DAMES & MOORE

PLATE A-3M

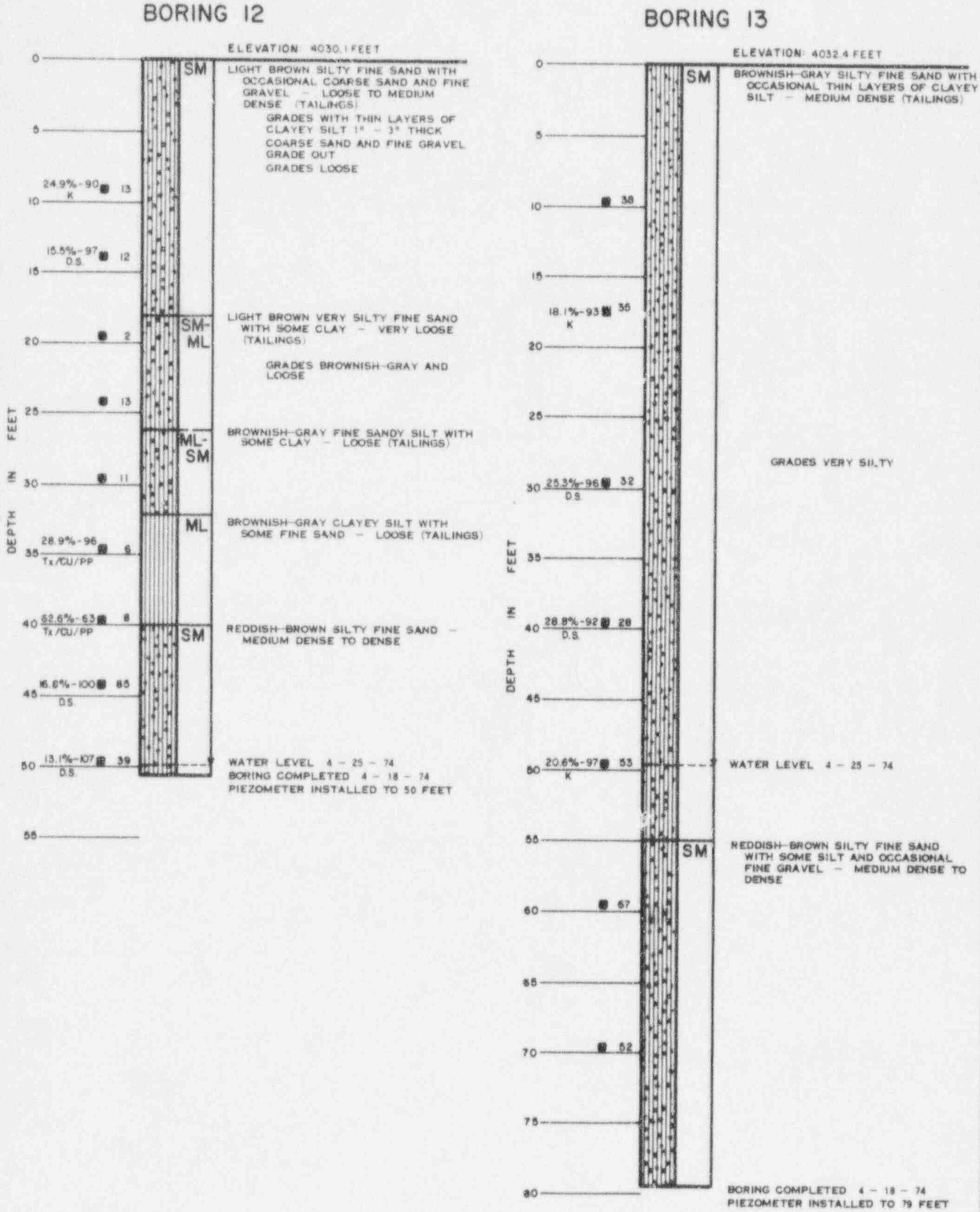
FILE 3469-003
ATLAS MINERALS - MOAB
CHECKED BY CGL DATE 6-5-74



LOG OF BORING

DAMES & MOORE

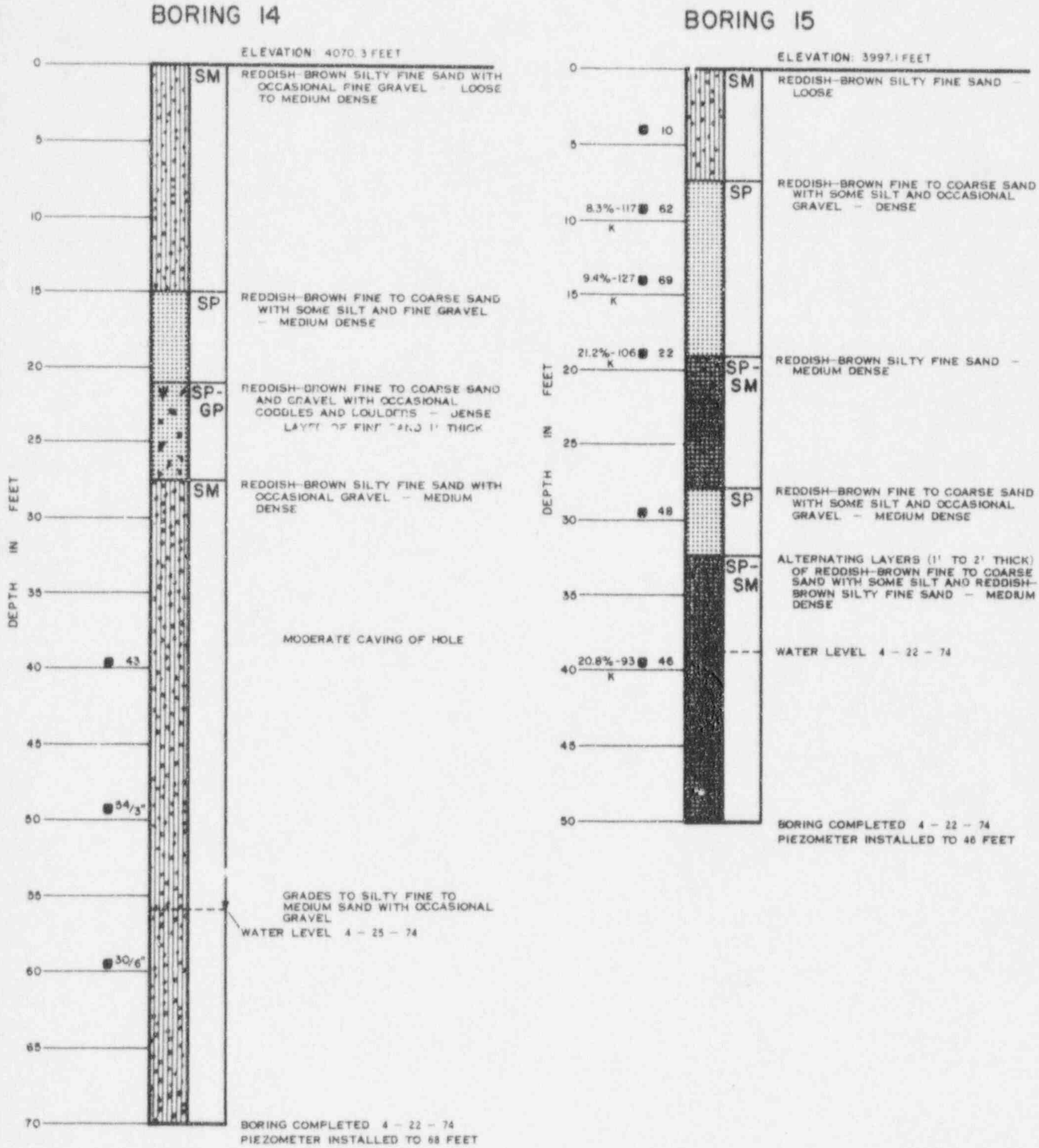
PLATE A-3N



LOG OF BORINGS

DAMES & MOORE

PLATE A-3P



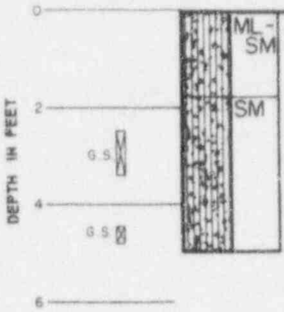
LOG OF BORINGS

DAMES & MOORE

PLATE A-30

TEST PIT - A-1

ELEVATION 4000



REDDISH BROWN SANDY SILT WITH COBBLES AND OCCASIONAL BOULDERS (FILL)
SEEPING WATER BELOW BOULDER - GRAY COLORATION

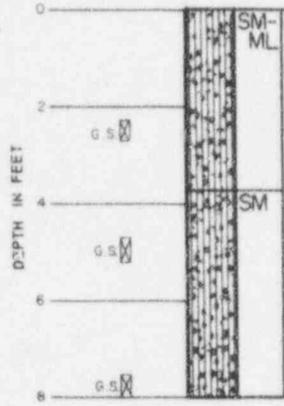
MOTTLED REDDISH BROWN AND GRAY SILTY FINE SAND TAILINGS MIXED WITH WITH RED SAND

CRADES WITH MORE SILT

TEST PIT COMPLETED AT 5.0' ON 8-5-77
GROUND WATER WAS NOT ENCOUNTERED

TEST PIT - A-2

ELEVATION 4000'



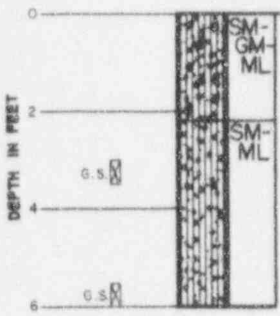
REDDISH-BROWN SILTY FINE TO COARSE SAND (FILL)

GRAY FINE SAND WITH SOME SILT AND SILT WITH SOME FINE SAND (SAND AND SLIME TAILINGS MIXTURE) - SATURATED

TEST PIT COMPLETED AT 8.0' ON 8-5-77
GROUND WATE WAS NOT ENCOUNTERED

TEST PIT - A-3

ELEVATION 4000'



REDDISH-BROWN SILT, GRAVEL AND SAND (FILL)
SEEPING WATER

GRAY SILTY FINE SAND - (SAND TAILINGS AND SLIMES)

TEST PIT COMPLETED AT 6.0' ON 8-5-77
GROUND WATER WAS NOT ENCOUNTERED

NOTES

THE DISCUSSION IN THE TEXT UNDER THE SECTION TITLED, "SITE CONDITIONS, SUBSURFACE", IS NECESSARY TO A PROPER UNDERSTANDING OF THE NATURE OF THE SUBSURFACE MATERIALS.

ELEVATIONS OF BORINGS ARE BASED ON U.S.G.S. DATUM. PLANT ELEVATION 500.0 CORRESPONDS TO 4028.2 MSL.

KEY:

A ☒ DEPTH AT WHICH BULK SAMPLE WAS EXTRACTED

TYPE OF TEST PERFORMED ON SAMPLE:

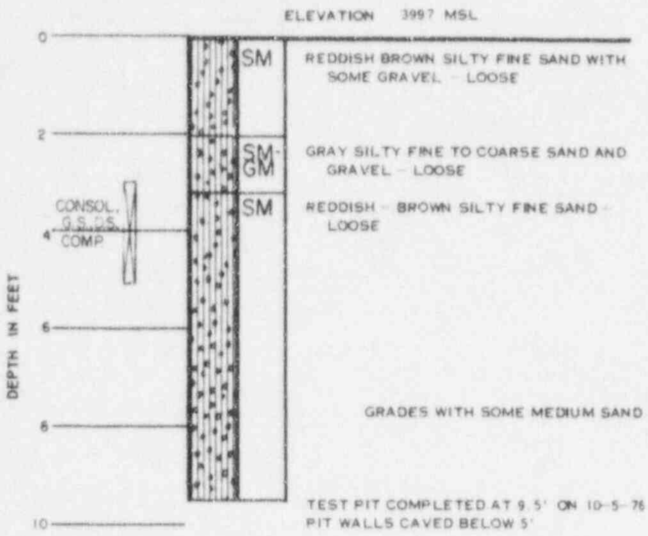
G.S. - GRAIN SIZE ANALYSIS
D.S. - DIRECT SHEAR TEST
COMP. - COMPACTION TEST
CONSOL. - CONSOLIDATION TEST

LOG OF TEST PITS

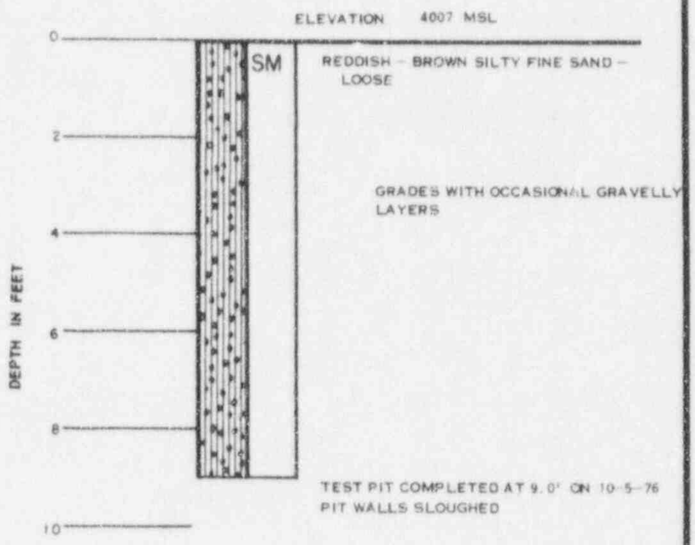
DAMES & MOORE

PLATE A-4A

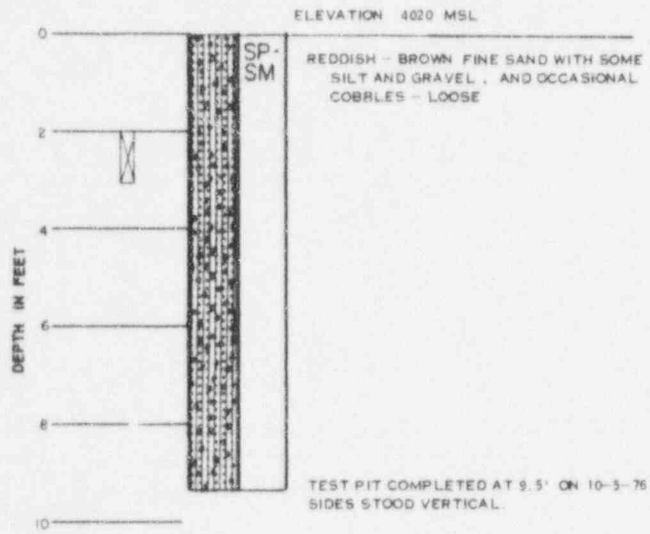
TEST PIT P-1



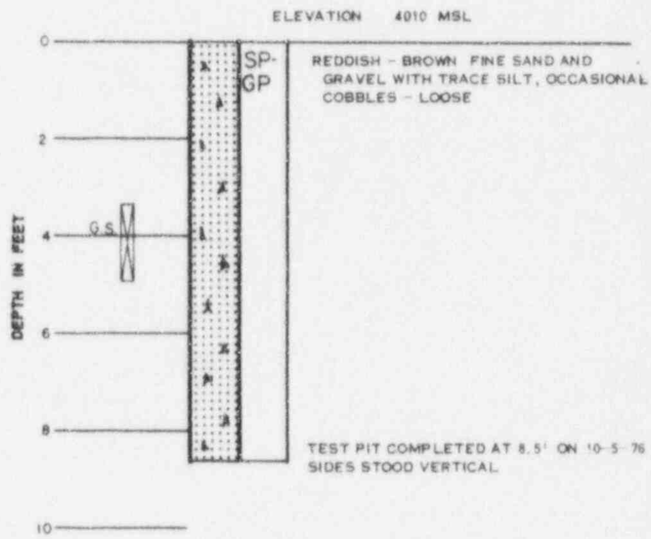
TEST PIT P-2



TEST PIT P-3



TEST PIT P-4

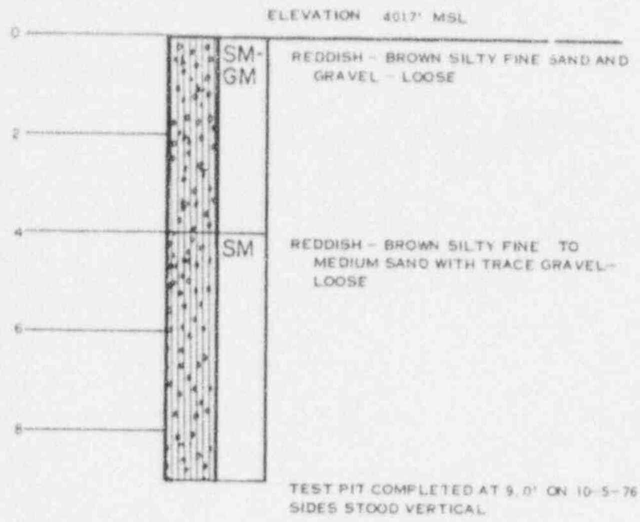


LOG OF TEST PITS

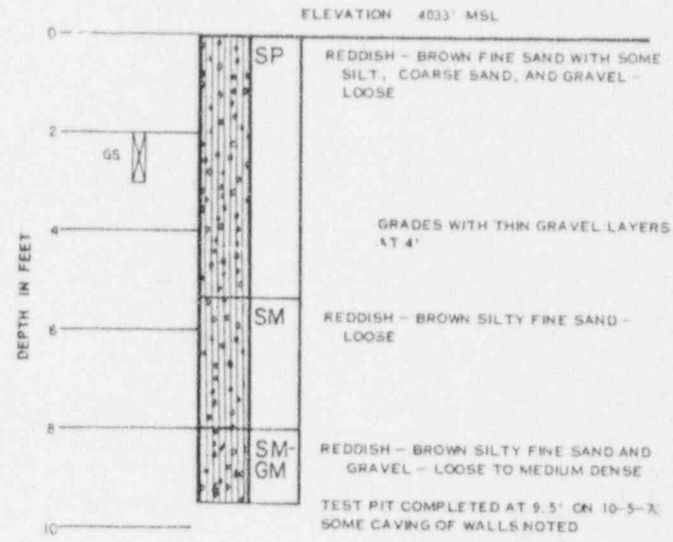
DAMES & MOORE

PLATE A-4B

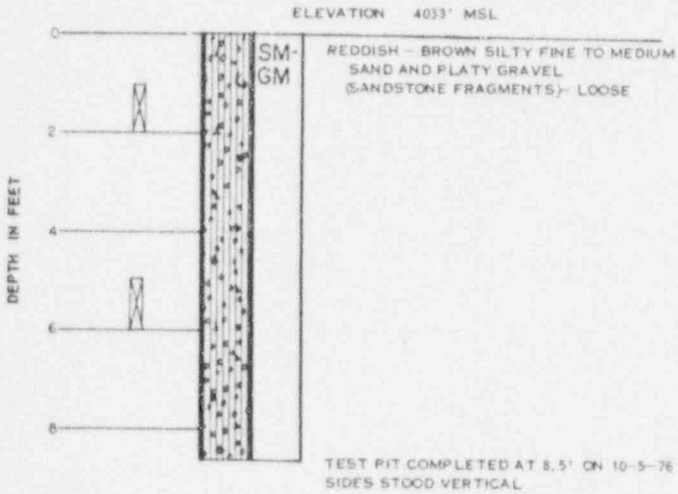
TEST PIT P-5



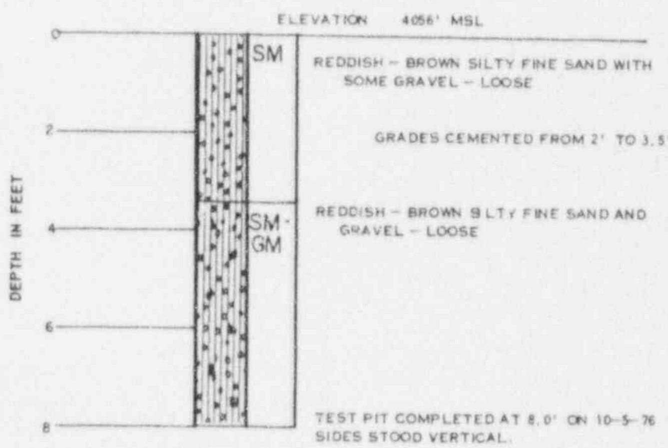
TEST PIT P-6



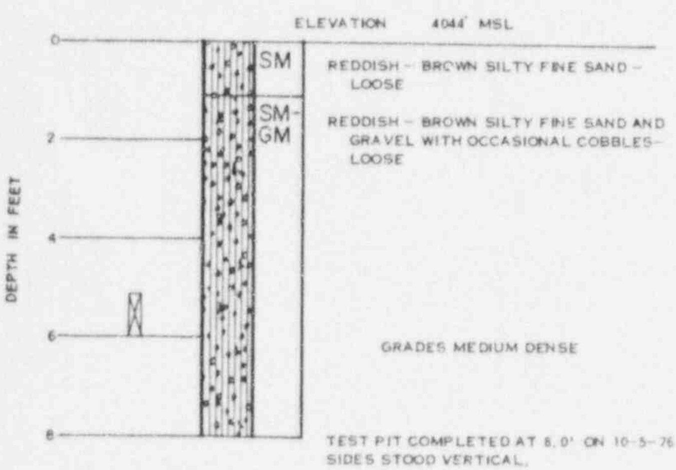
TEST PIT P-7



TEST PIT P-8



TEST PIT P-9



LOG OF TEST PITS

DAMES & MOORE

LOG OF TEST PITS

TEST PIT 1

ELEVATION: 3962.0 FEET

Depth (feet)	Symbol	Description
0	SP	LIGHT BROWN FINE SAND WITH TRACE COARSE SAND, FINE GRAVEL AND SILT - LOOSE
	SM	REDDISH-BROWN SILTY FINE SAND - LOOSE
5		GRADING WITH TRACE MEDIUM SAND
		WATER LEVEL 12 - 7 - 72
10		TEST PIT COMPLETED 12 - 6 - 72

NOTE
PIEZOMETER INSTALLED TO 10.0 FEET

TEST PIT 2

ELEVATION: 3959.6 FEET

Depth (feet)	Symbol	Description
0	ML	MOTTLED BROWN AND REDDISH-BROWN CLAYEY SILT WITH OCCASIONAL THIN SEAMS OF SILTY CLAY - MEDIUM STIFF
		GRADING WITH TRACE FINE SAND AND OCCASIONAL SEAMS OF FINE SAND
5	ML-SM	REDDISH-BROWN SILT WITH FINE SAND - LOOSE
		WATER LEVEL 12 - 7 - 72
	SM	REDDISH-BROWN SILTY FINE SAND - LOOSE
10		TEST PIT COMPLETED 12 - 6 - 72

NOTE
PIEZOMETER INSTALLED TO 10.5 FEET

TEST PIT 3

ELEVATION: 3962.5 FEET

Depth (feet)	Symbol	Description
0	SM	REDDISH-BROWN SILTY FINE SAND - LOOSE
		GRADING WITH SOME SILT - MEDIUM DENSE AND PARTIALLY CEMENTED
5	ML	GRAYISH-BROWN SILT WITH SOME FINE SAND AND TRACE CLAY - LOOSE
		WATER LEVEL 12 - 7 - 72
10		GRADING TO A CLAYEY SILT WITH SOME FINE SAND
		TEST PIT COMPLETED 12 - 6 - 72

NOTE
PIEZOMETER INSTALLED TO 12.5 FEET

TEST PIT 4

ELEVATION: 3967.8 FEET

Depth (feet)	Symbol	Description
0	ML	GRAYISH-BROWN CLAYEY SILT WITH SOME FINE SAND - LOOSE
	SM	LIGHT REDDISH-BROWN SILTY FINE SAND - LOOSE
5		GRADING WITH OCCASIONAL 2" - 3" LAYERS OF CLAYEY SILT AND SILTY GRAVEL
		WATER LEVEL 12 - 7 - 72
10		TEST PIT COMPLETED 12 - 7 - 72

NOTE
PIEZOMETER INSTALLED TO 10.0 FEET

TEST PIT 5

ELEVATION: 3953.6 FEET

Depth (feet)	Symbol	Description
0	SP	REDDISH-BROWN FINE SAND WITH SOME SILT - LOOSE
		GRADING TO FINE TO MEDIUM SAND
	ML	REDDISH-BROWN CLAYEY SILT WITH SOME FINE SAND - LOOSE
5	SP-SM	REDDISH-BROWN FINE SAND WITH SOME SILT - LOOSE
		WATER LEVEL 12 - 7 - 72
10		TEST PIT COMPLETED 12 - 7 - 72

NOTE
PIEZOMETER INSTALLED TO 10.0 FEET

TEST PIT 6

ELEVATION: 3979.2 FEET

Depth (feet)	Symbol	Description
0	SM	REDDISH-BROWN SILTY FINE SAND - LOOSE
5		GRADING WITH OCCASIONAL COARSE GRAVEL
	SP-GP	BROWN FINE TO COARSE SAND AND GRAVEL WITH OCCASIONAL COBBLES - MEDIUM DENSE
10	SP	BROWN FINE SAND WITH TRACE FINE TO COARSE GRAVEL, SILT AND COBBLES - MEDIUM DENSE
15		TEST PIT COMPLETED 12 - 7 - 72

NOTES
PIEZOMETER INSTALLED TO 14.0 FEET
GROUND WATER WAS NOT ENCOUNTERED IN THIS TEST PIT.

DAMES & MOORE

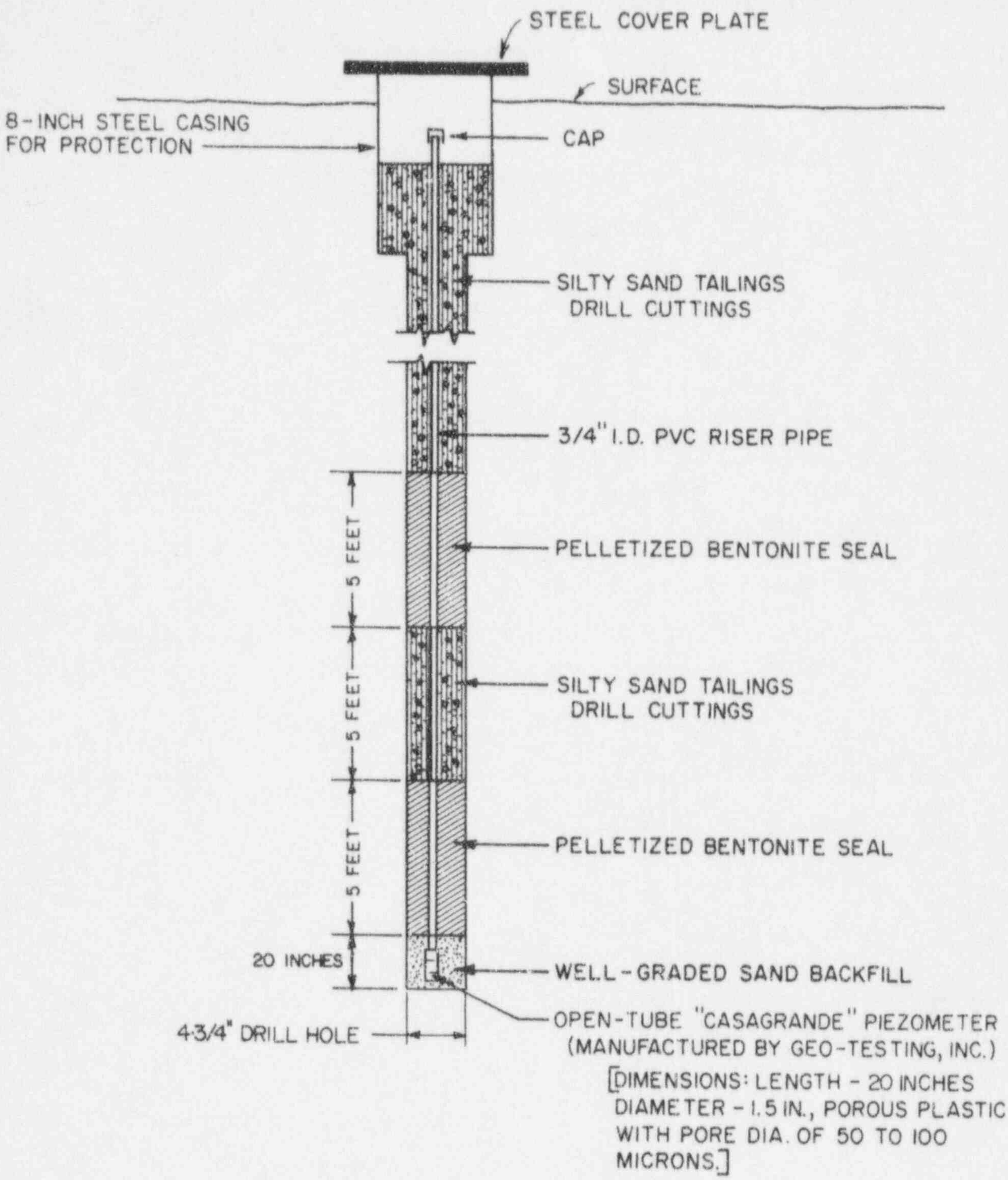


DIAGRAM OF TYPICAL PIEZOMETER

BY H.B.T. DATE 5-27-77 FILE 05467-018 ATLAS REVISIONS
CHECKED BY JKR 5/31/77 BY ____ DATE ____

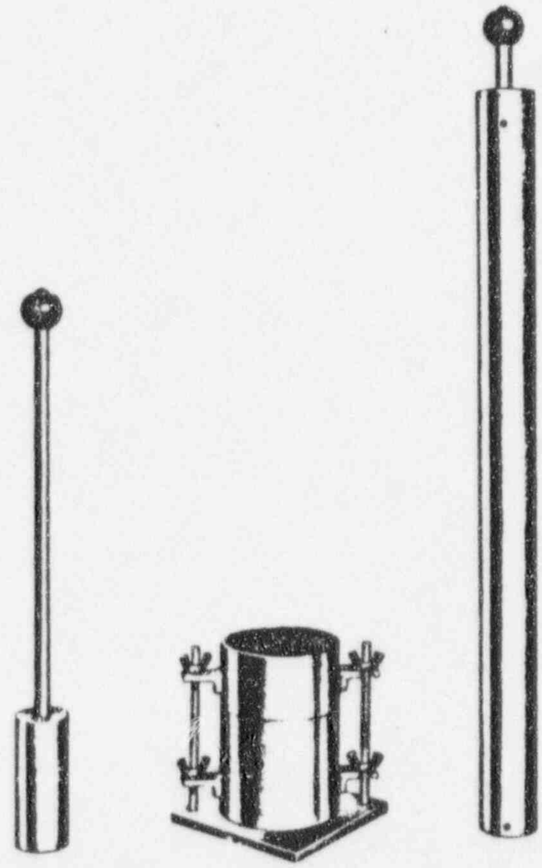
946.7 (REV. 6-

DAMES & MOORE

PLATE A-5

IT HAS BEEN ESTABLISHED THAT WHEN COMPACTING EFFORT IS HELD CONSTANT, THE DENSITY OF A ROLLED EARTH FILL INCREASES WITH ADDED MOISTURE UNTIL A MAXIMUM DRY DENSITY IS OBTAINED AT A MOISTURE CONTENT TERMED THE "OPTIMUM MOISTURE CONTENT," AFTER WHICH THE DRY DENSITY DECREASES. THE COMPACTION CURVE SHOWING THE RELATIONSHIP BETWEEN DENSITY AND MOISTURE CONTENT FOR A SPECIFIC COMPACTING EFFORT IS DETERMINED BY EXPERIMENTAL METHODS. TWO COMMONLY USED METHODS ARE DESCRIBED IN THE FOLLOWING PARAGRAPHS.

FOR THE "STANDARD A.A.S.H.O." (A.S.T.M. D698-58T & A.A.S.H.O. T99-57) METHOD OF COMPACTION A PORTION OF THE SOIL SAMPLE PASSING THE NO. 4 SIEVE IS COMPACTED AT A SPECIFIC MOISTURE CONTENT IN THREE EQUAL LAYERS IN A STANDARD COMPACTION CYLINDER HAVING A VOLUME OF 1/30 CUBIC FOOT, USING TWENTY-FIVE 12-INCH BLOWS OF A STANDARD 5-1/2 POUND RAMMER TO COMPACT EACH LAYER.



SOME APPARATUS FOR PERFORMING COMPACTION TESTS
Shows, from left to right, 5-1/2 pound rammer (sleeve controlling 12" height of drop removed), 1/30 cubic-foot cylinder with removable collar and base plate, and 10 pound rammer within sleeve.

IN THE "MODIFIED A.A.S.H.O." (A.S.T.M. D-1557-58T & A.A.S.H.O. T 180-57) METHOD OF COMPACTION A PORTION OF THE SOIL SAMPLE PASSING THE NO. 4 SIEVE IS COMPACTED AT A SPECIFIC MOISTURE CONTENT IN FIVE EQUAL LAYERS IN A STANDARD COMPACTION CYLINDER HAVING A VOLUME OF 1/30 CUBIC FOOT, USING TWENTY-FIVE 18-INCH BLOWS OF A 10-POUND RAMMER TO COMPACT EACH LAYER. SEVERAL VARIATIONS OF THESE COMPACTION TESTING METHODS ARE OFTEN USED AND THESE ARE DESCRIBED IN A.A.S.H.O. & A.S.T.M. SPECIFICATIONS.

FOR BOTH METHODS, THE WET DENSITY OF THE COMPACTED SAMPLE IS DETERMINED BY WEIGHING THE KNOWN VOLUME OF SOIL; THE MOISTURE CONTENT, BY MEASURING THE LOSS OF WEIGHT OF A PORTION OF THE SAMPLE WHEN OVEN DRIED; AND THE DRY DENSITY, BY COMPUTING IT FROM THE WET DENSITY AND MOISTURE CONTENT. A SERIES OF SUCH COMPACTIONS IS PERFORMED AT INCREASING MOISTURE CONTENTS UNTIL A SUFFICIENT NUMBER OF POINTS DEFINING THE MOISTURE-DENSITY RELATIONSHIP HAVE BEEN OBTAINED TO PERMIT THE PLOTTING OF THE COMPACTION CURVE. THE MAXIMUM DRY DENSITY AND OPTIMUM MOISTURE CONTENT FOR THE PARTICULAR COMPACTING EFFORT ARE DETERMINED FROM THE COMPACTION CURVE.

METHOD OF PERFORMING COMPACTION TESTS (STANDARD AND MODIFIED A.A.S.H.O. METHODS)

DAMES & MOORE

PLATE A-6

TEST PIT

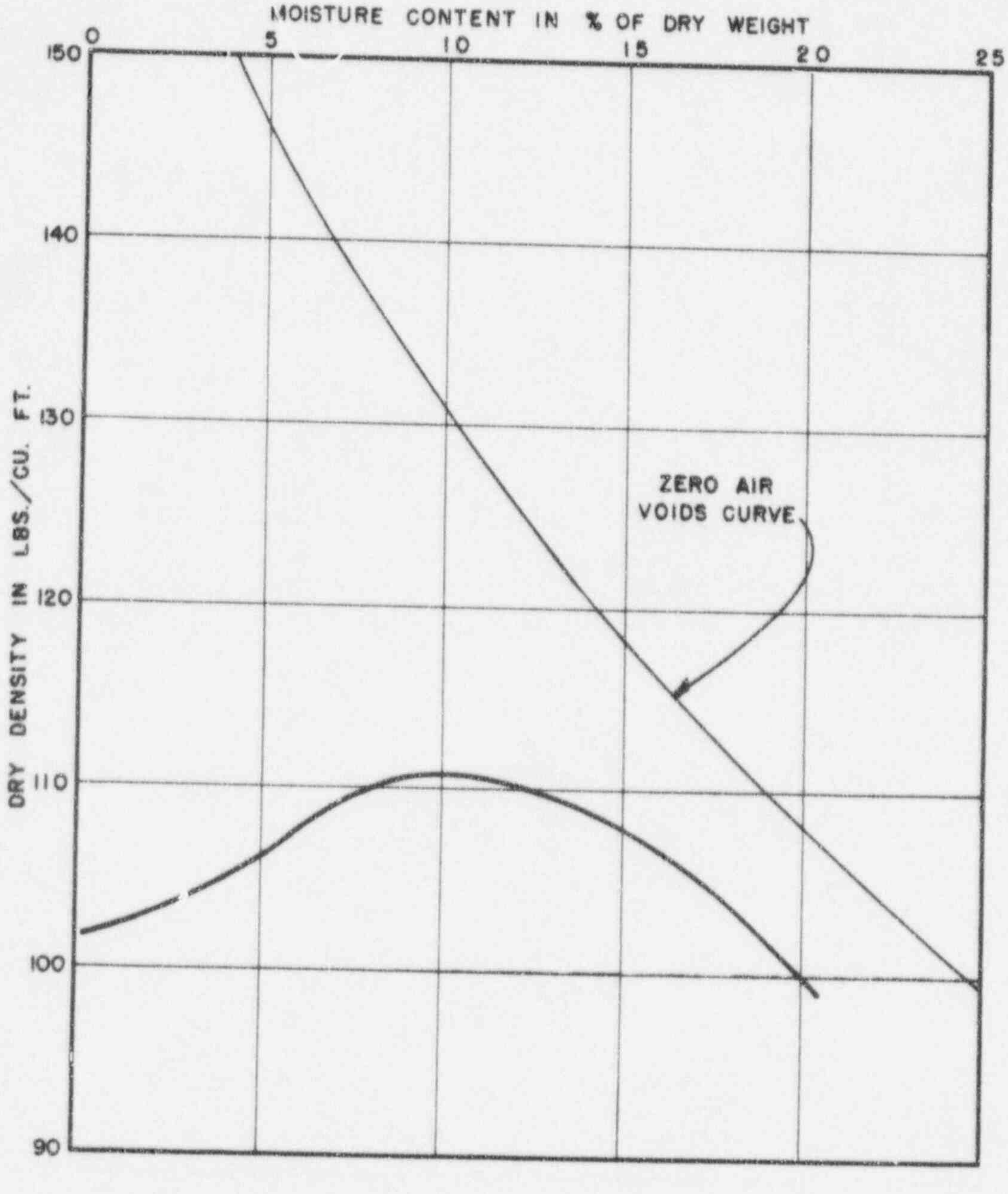
SAMPLE NO. P - 1 DEPTH 3'-5'

SOIL SM (NATURAL SOIL)

OPTIMUM MOISTURE CONTENT 10.2 PERCENT

MAXIMUM DRY DENSITY 111 LBS. PER CUBIC FOOT

METHOD OF COMPACTION AASHTO T-99



COMPACTION TEST DATA

DRAWN BY DATE 12-22

CHECKED BY DATE 23-76

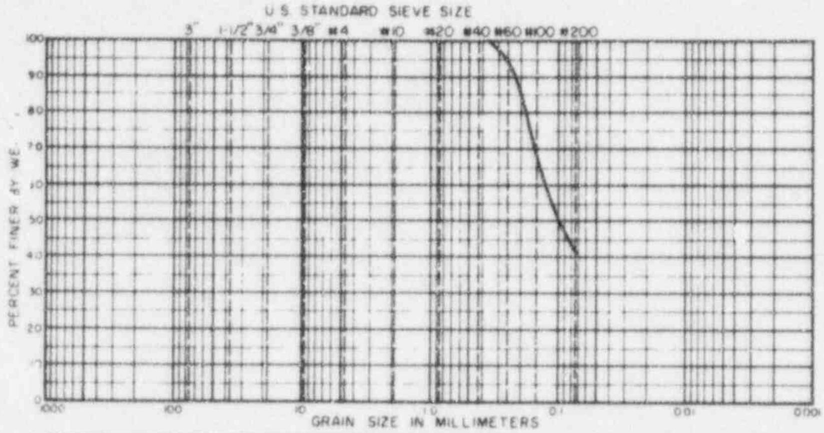
REVISION 2 BY DATE

REVISION BY DATE

FILE

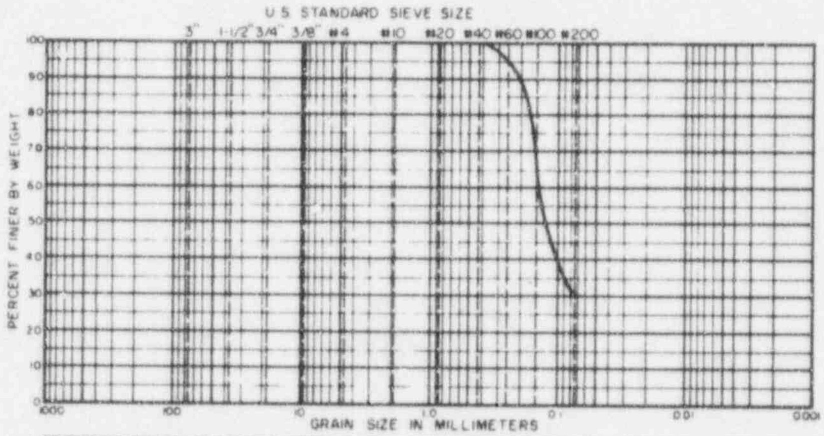
956.1 (REV. 4-57)

DAMES & MOORE



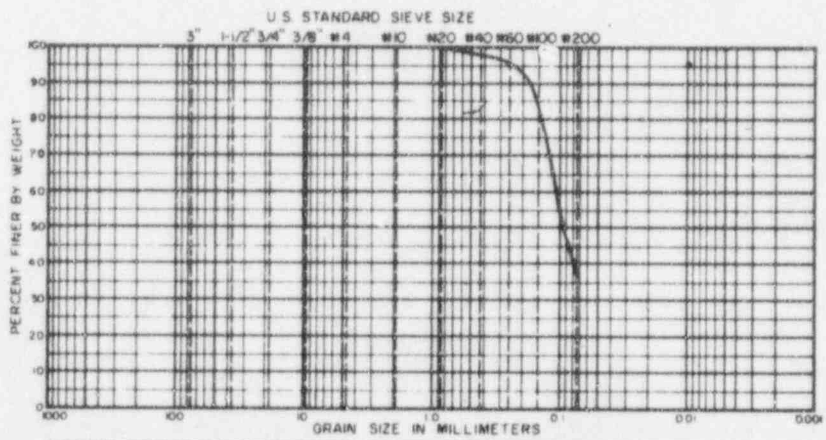
COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

LOCATION	DEPTH	CLASSIFICATION
BORING A4	20'	GRAY SILTY FINE SAND (SM)



COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

LOCATION	DEPTH	CLASSIFICATION
BORING A4	40'	GRAY SILTY FINE SAND (SM)



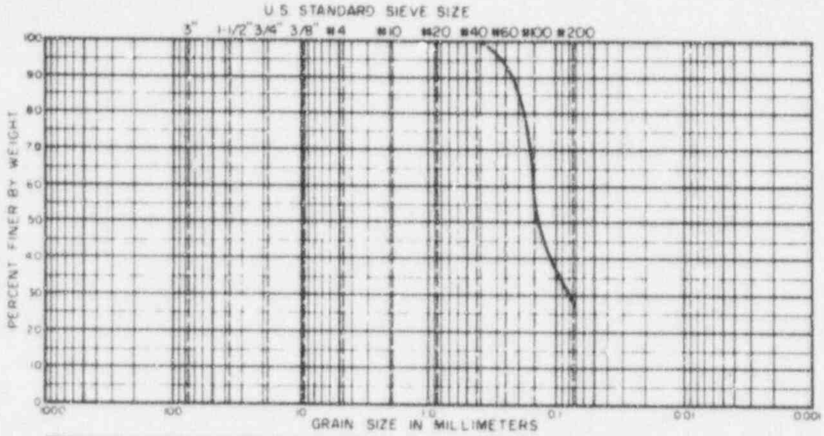
COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

LOCATION	DEPTH	CLASSIFICATION
BORING A4	60'	GRAY SILTY FINE TO MEDIUM SAND (SM)

GRADATION CURVES
SAND TAILINGS

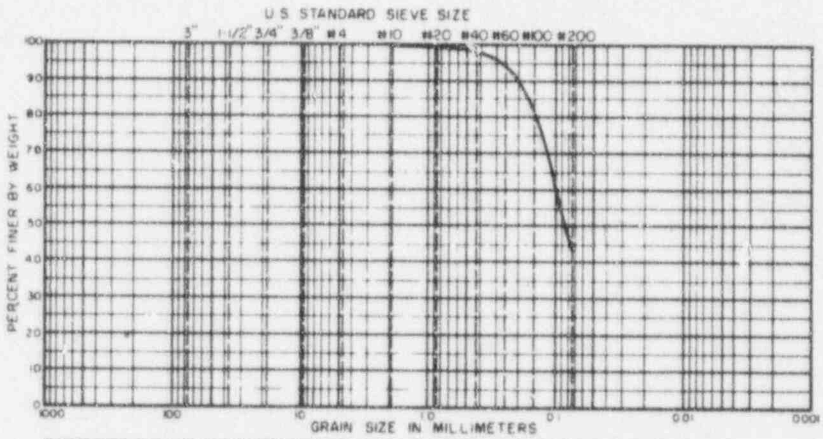
DAMES & MOORE

PLATE A-8A



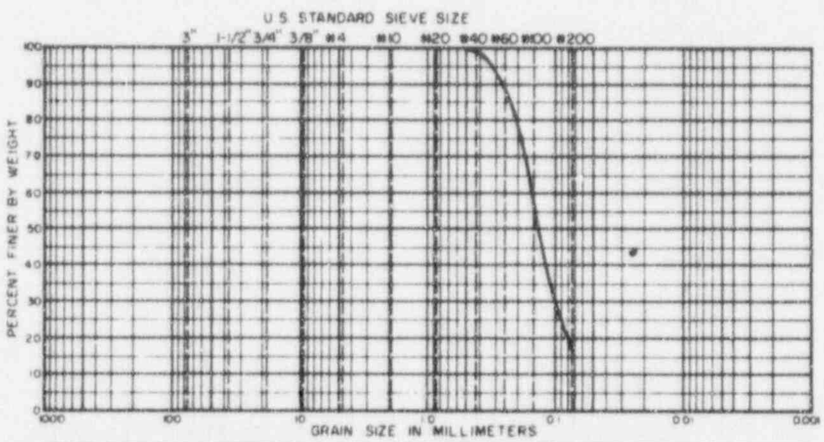
COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

LOCATION	DEPTH	CLASSIFICATION
BORING A5	10'	LIGHT GRAY SILTY FINE SAND (SM)



COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

LOCATION	DEPTH	CLASSIFICATION
BORING A5	20'	LIGHT GRAY SILTY FINE TO MEDIUM SAND (SM)



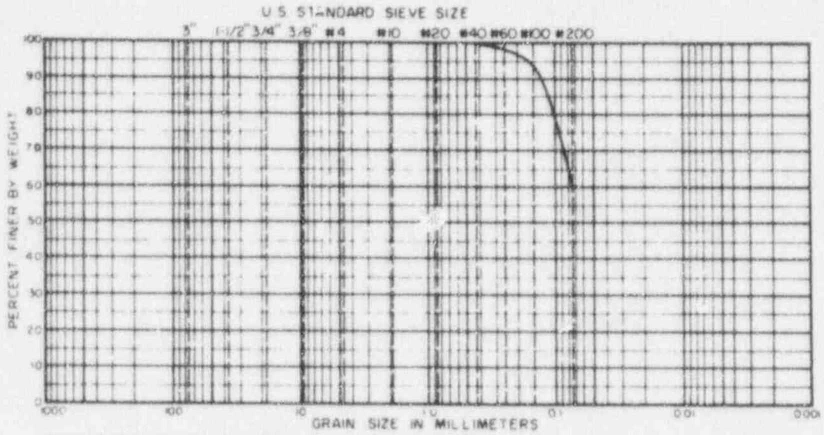
COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

LOCATION	DEPTH	CLASSIFICATION
BORING A5	30'	TANNISH-GRAY SILTY FINE SAND (SM)

GRADATION CURVES
SAND TAILINGS

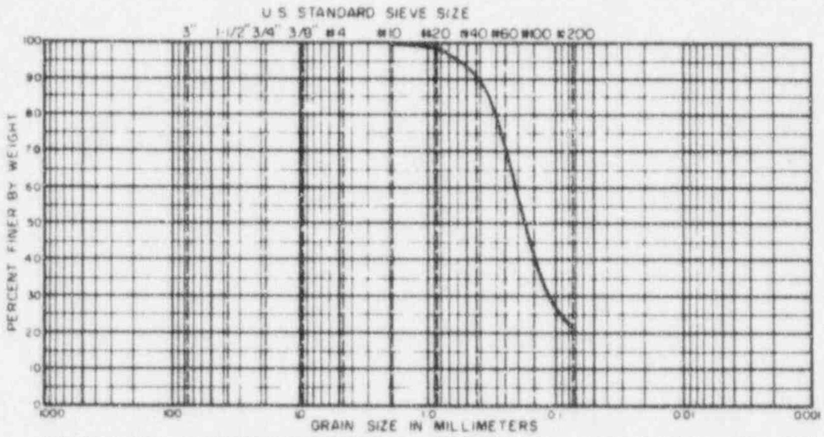
DAMES & MOORE

PLATE A-8B



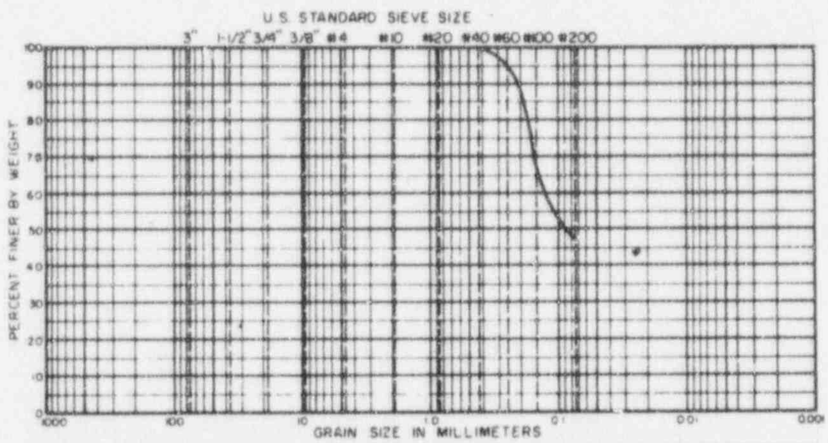
COBBLES	GRAVEL COARSE	GRAVEL FINE	SAND COARSE	SAND MEDIUM	SAND FINE	SILT OR CLAY

LOCATION	DEPTH	CLASSIFICATION
BORING A5	40'	LIGHT GRAY SILTY FINE SAND (SM)



COBBLES	GRAVEL COARSE	GRAVEL FINE	SAND COARSE	SAND MEDIUM	SAND FINE	SILT OR CLAY

LOCATION	DEPTH	CLASSIFICATION
BORING A5	60'	LIGHT GRAY SILTY FINE TO MEDIUM SAND (SM)



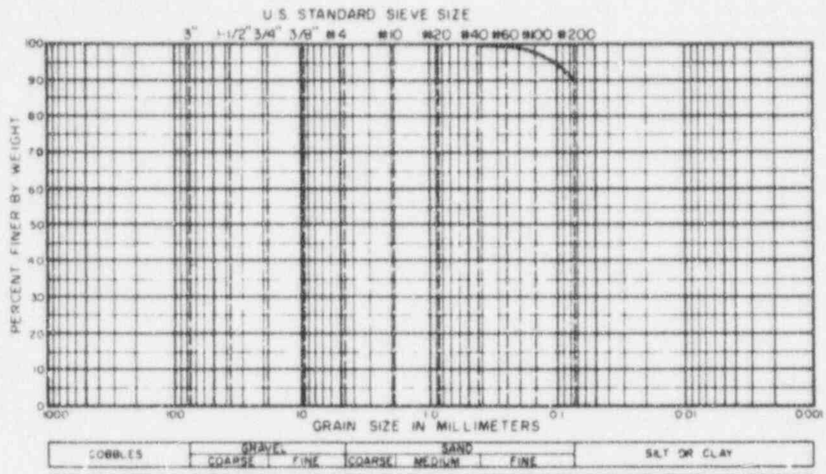
COBBLES	GRAVEL COARSE	GRAVEL FINE	SAND COARSE	SAND MEDIUM	SAND FINE	SILT OR CLAY

LOCATION	DEPTH	CLASSIFICATION
BORING A6	30'	TANNISH-GRAY SILTY FINE SAND (SM)

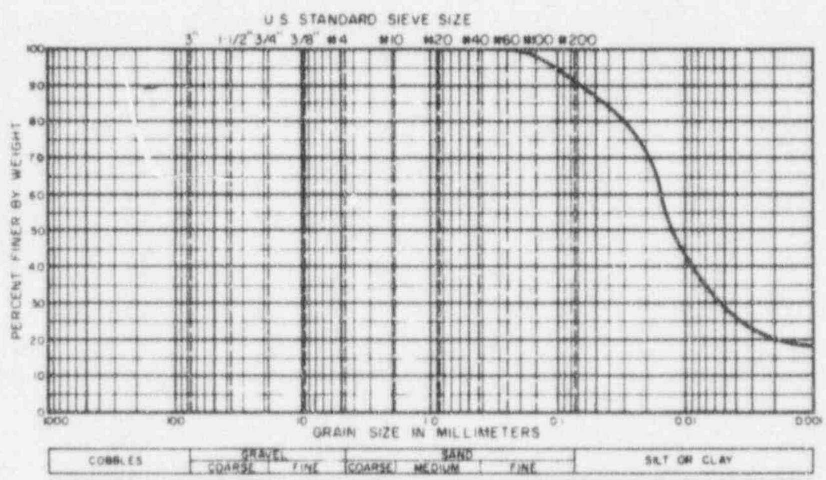
GRADATION CURVES
SAND TAILINGS

DAMES & MOORE

PLATE A-8C



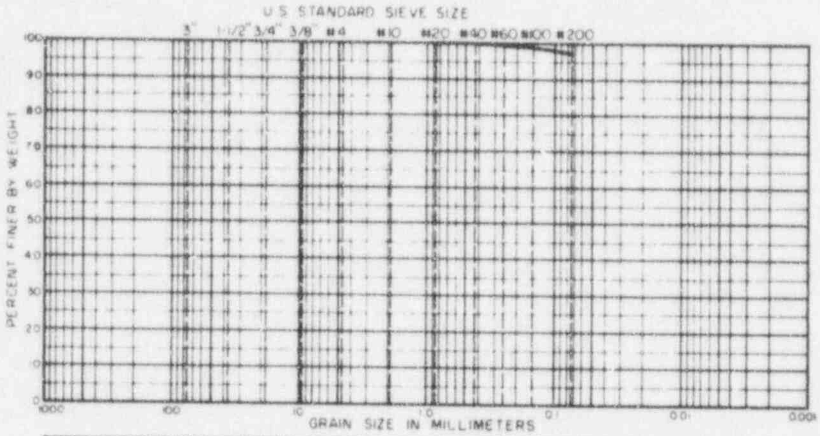
LOCATION	DEPTH	CLASSIFICATION
BORING A2	30'	LIGHT GRAY SILT WITH TRACE OF FINE SAND (ML)



LOCATION	DEPTH	CLASSIFICATION
BORING A6	40'	LIGHT GRAY CLAYEY SILT WITH TRACE OF FINE SAND (ML)

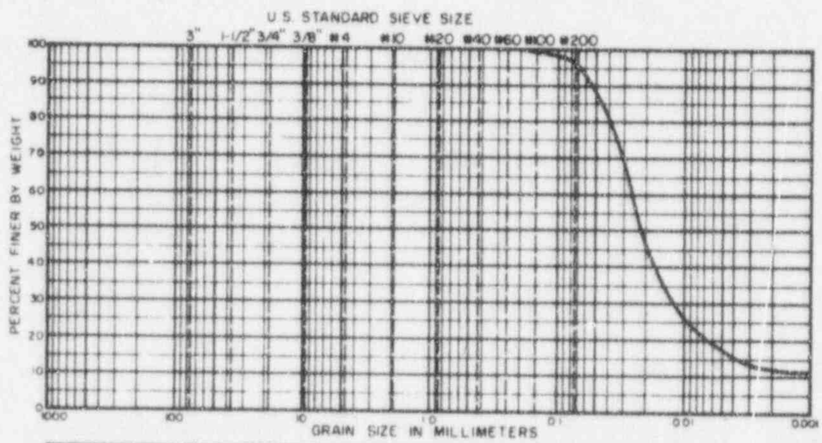
GRADATION CURVES - SLIME TAILINGS

DAMES & MOORE



COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

LOCATION	DEPTH	CLASSIFICATION
BORING A-6	50'	LIGHT GRAY SILT WITH A TRACE OF FINE SAND (ML)

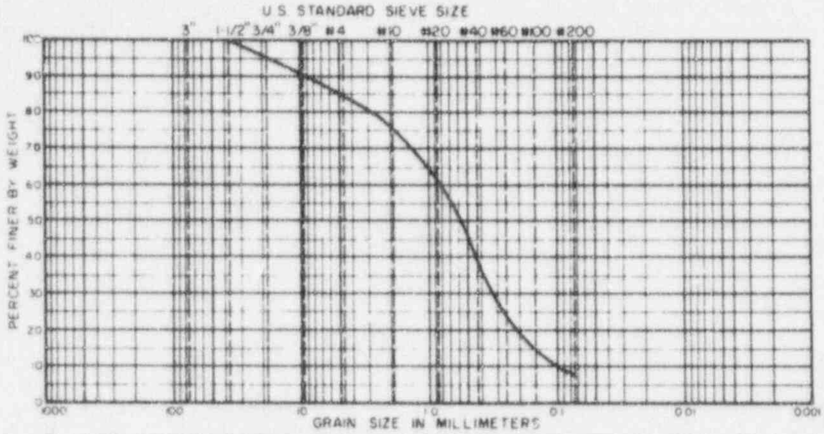


COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

LOCATION	DEPTH	CLASSIFICATION
BORING A11	30'	LIGHT GRAY CLAYEY SILT WITH TRACE FINE SAND (ML)

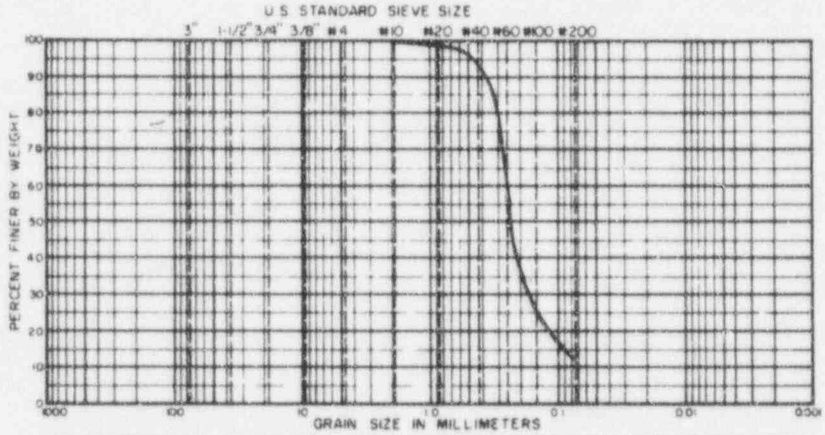
GRADATION CURVES - SLIME TAILINGS

DAMES & MOORE



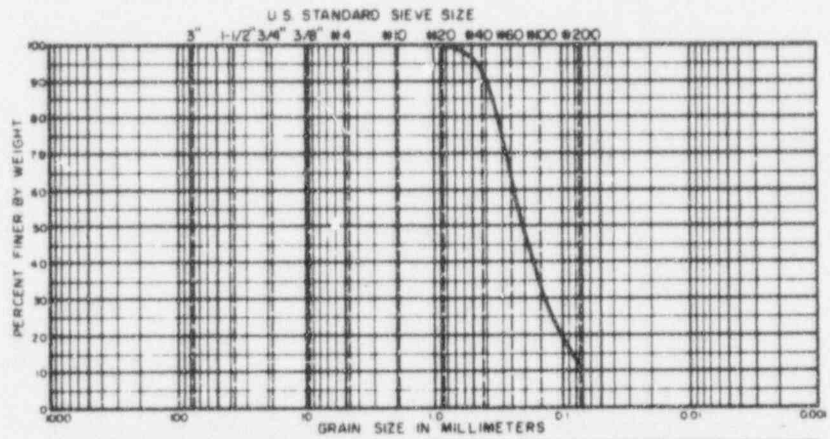
COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

LOCATION	DEPTH	CLASSIFICATION
BORING A3	40'	REDDISH-BROWN FINE TO COARSE SAND WITH SOME FINE AND COARSE SANDSTONE GRAVEL (SP)



COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

LOCATION	DEPTH	CLASSIFICATION
BORING A8	20'	REDDISH-BROWN FINE SAND WITH TRACE SILT AND FINE TO COARSE SAND (SP)



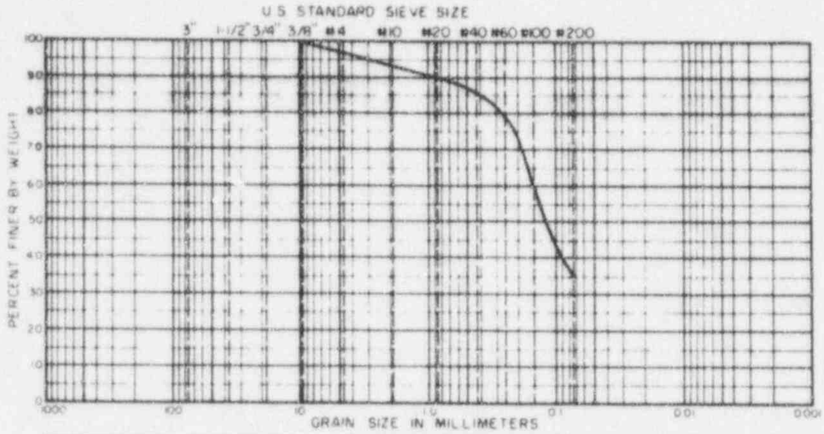
COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

LOCATION	DEPTH	CLASSIFICATION
BORING A11	50'	REDDISH-BROWN FINE SAND WITH SOME SILT AND TRACE OF MEDIUM SAND (SP)

GRADATION CURVES - IN SITU FOUNDATION SOILS

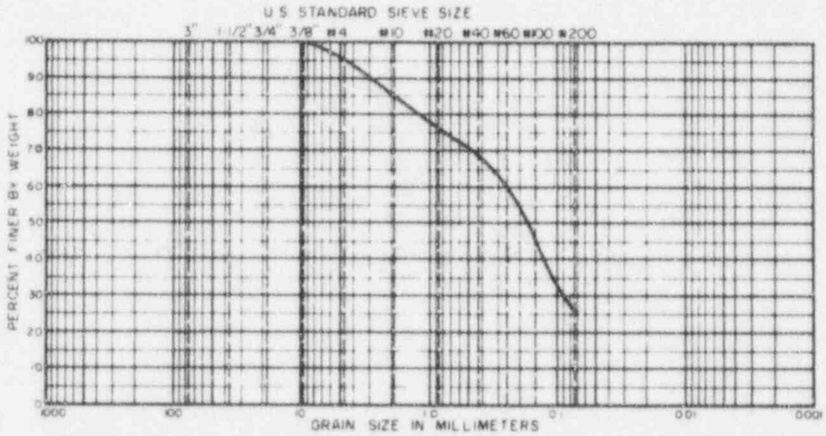
DAMES & MOORE

PLATE A-8F



COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

LOCATION	DEPTH	CLASSIFICATION
BORING A12	8.5'	SILTY FINE TO MEDIUM SAND WITH SOME COARSE SAND AND TRACE FINE SANDSTONE GRAVEL (SM)



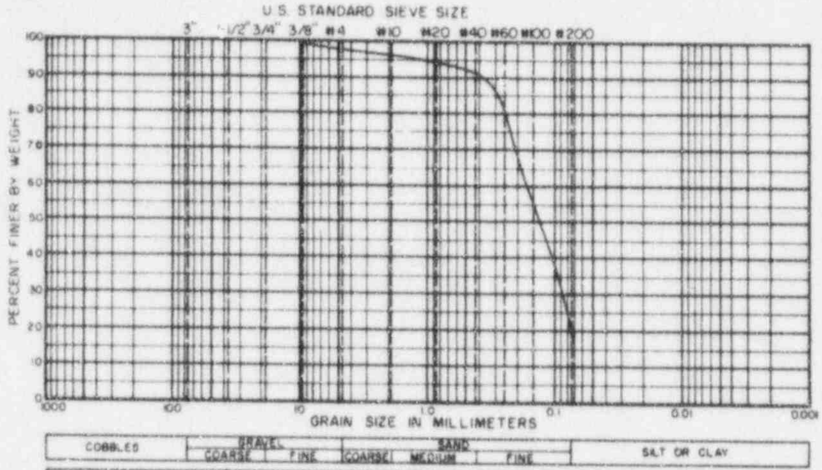
COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

LOCATION	DEPTH	CLASSIFICATION
BORING A13	11'	REDDISH-BROWN SILTY FINE TO COARSE SAND WITH SOME FINE GRAVEL (SM)

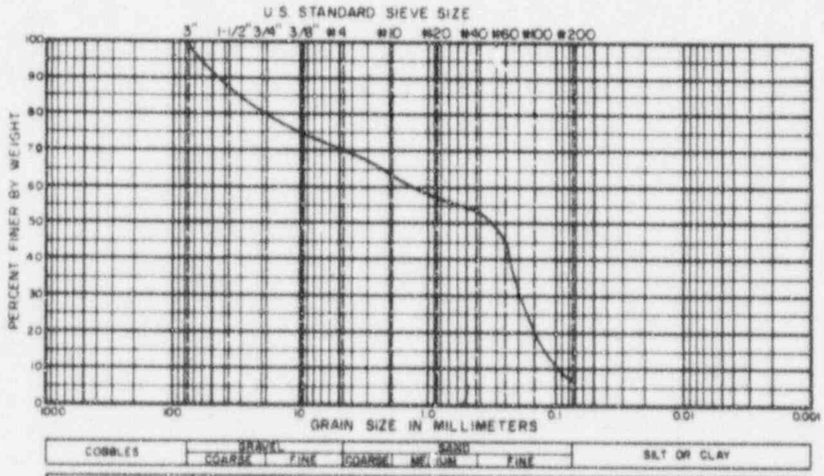
GRADATION CURVES - IN SITU FOUNDATION SOILS

DAMES & MOORE

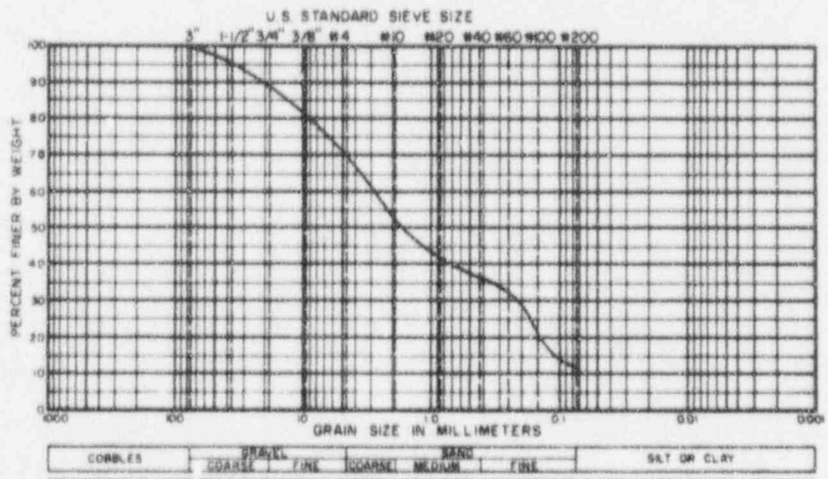
PLATE A-8G



LOCATION	DEPTH	CLASSIFICATION
TEST PIT P-1	3' - 5'	SM



LOCATION	DEPTH	CLASSIFICATION
TEST PIT P - 4	3.5'	SP

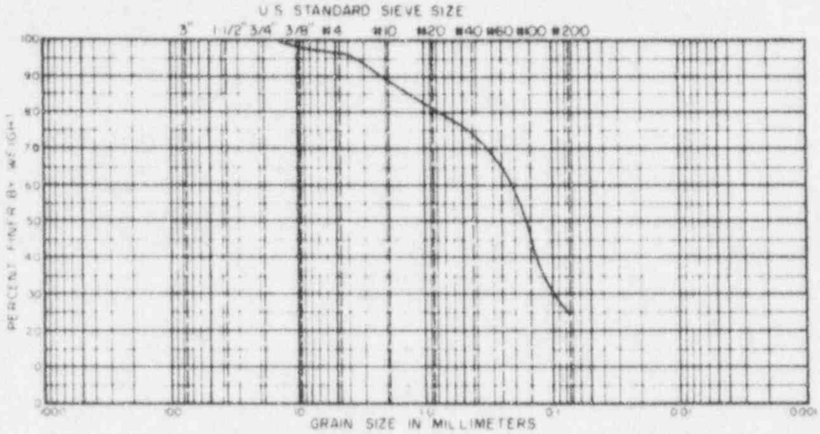


LOCATION	DEPTH	CLASSIFICATION
TEST PIT P-6	2'	SP

GRADATION CURVES
IN SITU FOUNDATION SOILS

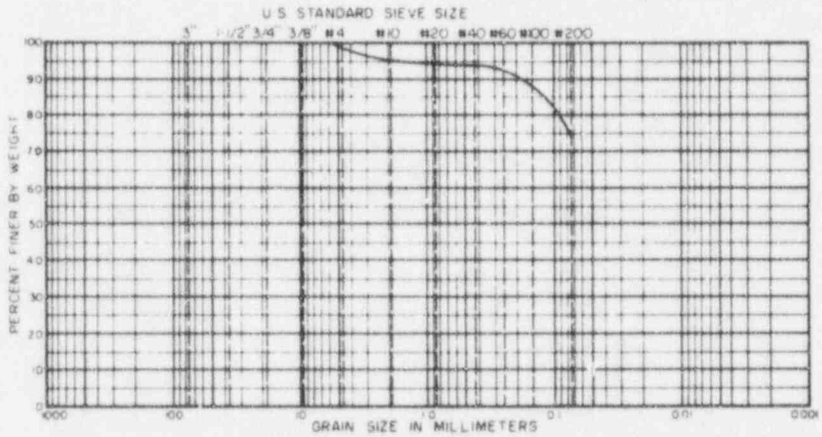
DAMES & MOORE

PLATE A-8H



COBBLES	GRAVEL COARSE	GRAVEL FINE	SAND COARSE	SAND MEDIUM	SAND FINE	SILT OR CLAY

LOCATION	DEPTH	CLASSIFICATION
TEST PIT A1	3'	REDDISH BROWN AND GRAY SILTY FINE TO COARSE SAND WITH TRACE OF FINE GRAVEL (SM)



COBBLES	GRAVEL COARSE	GRAVEL FINE	SAND COARSE	SAND MEDIUM	SAND FINE	SILT OR CLAY

LOCATION	DEPTH	CLASSIFICATION
TEST PIT A1	5'	SILT WITH SOME FINE SAND AND TRACE OF MEDIUM TO COARSE SAND (ML)

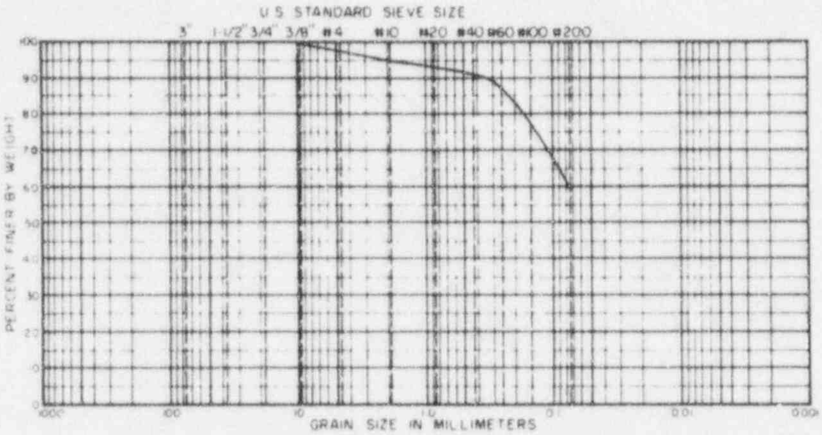
GRADATION CURVES
TAILINGS AND SAND FILL

DAMES & MOORE

PLATE A-81

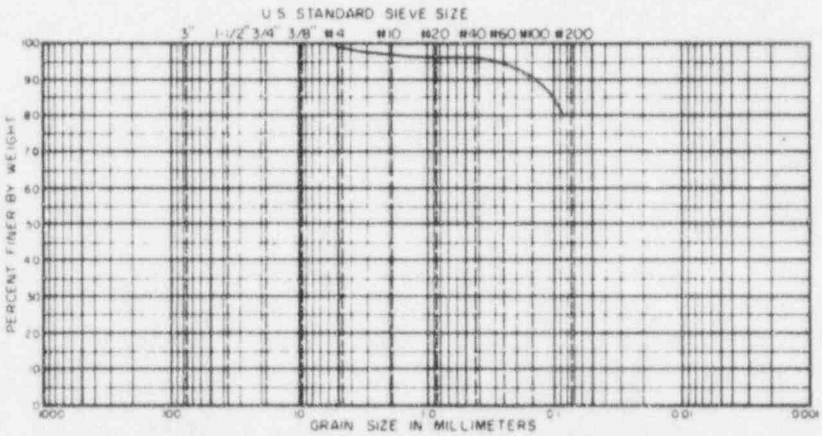
FILE 05467-008 ATLAS MINERALS BY H.B. TOLAND DATE 10-5-78 CHECKED BY DATE

REVISIONS BY DATE BY DATE PLATE OF



COBBLES	GRAVEL COARSE	GRAVEL FINE	SAND COARSE	SAND MEDIUM	SAND FINE	SILT OR CLAY

LOCATION	DEPTH	CLASSIFICATION
TEST PIT A 3	3.7'	GRAY SILT AND FINE SAND WITH TRACE OF MEDIUM TO COARSE SAND AND FINE GRAVEL (SM-ML)



COBBLES	GRAVEL COARSE	GRAVEL FINE	SAND COARSE	SAND MEDIUM	SAND FINE	SILT OR CLAY

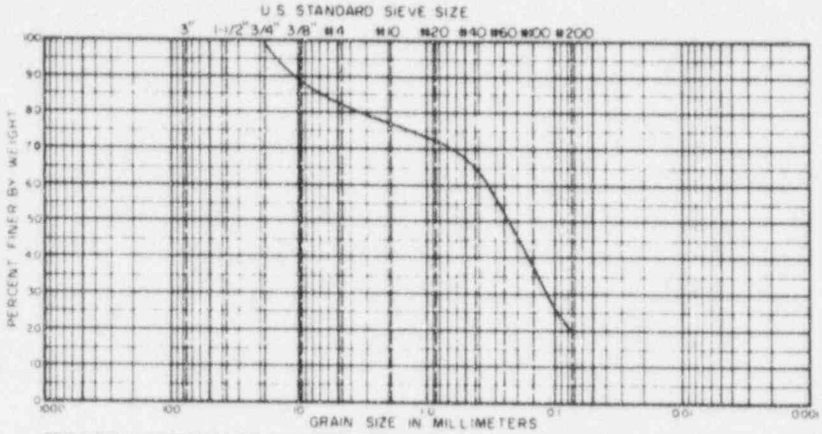
LOCATION	DEPTH	CLASSIFICATION
TEST PIT A 3	6'	SILT WITH SOME FINE SAND AND TRACE OF MEDIUM TO COARSE SAND AND FINE GRAVEL (ML)

GRADATION CURVES

TAILINGS AND SAND FILL

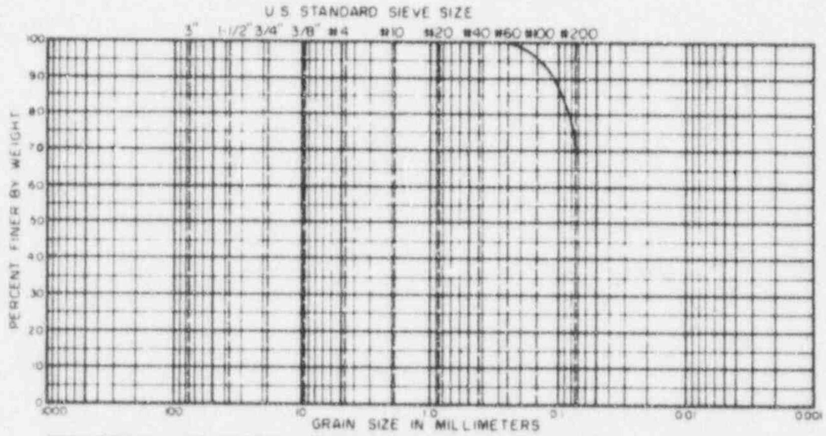
DAMES & MOORE

PLATE A-8J



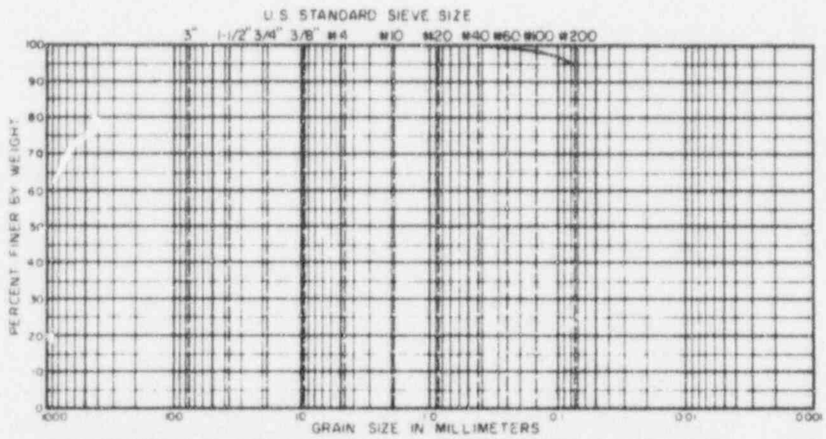
COBBLES	GRAVEL COARSE	GRAVEL FINE	SAND COARSE	SAND MEDIUM	SAND FINE	SILT OR CLAY

LOCATION	DEPTH	CLASSIFICATION
TEST PIT A 2	3'	REDDISH BROWN SILTY FINE TO COARSE SAND WITH SOME FINE GRAVEL (SM)



COBBLES	GRAVEL COARSE	GRAVEL FINE	SAND COARSE	SAND MEDIUM	SAND FINE	SILT OR CLAY

LOCATION	DEPTH	CLASSIFICATION
TEST PIT A 2	5'	SILT WITH SOME FINE SAND (ML)



COBBLES	GRAVEL COARSE	GRAVEL FINE	SAND COARSE	SAND MEDIUM	SAND FINE	SILT OR CLAY

LOCATION	DEPTH	CLASSIFICATION
TEST PIT A 2	8'	SILT WITH TRACE OF FINE SAND (ML)

GRADATION CURVES
TAILINGS AND SAND FILL

DAMES & MOORE

PLATE A-8K

REVISIONS
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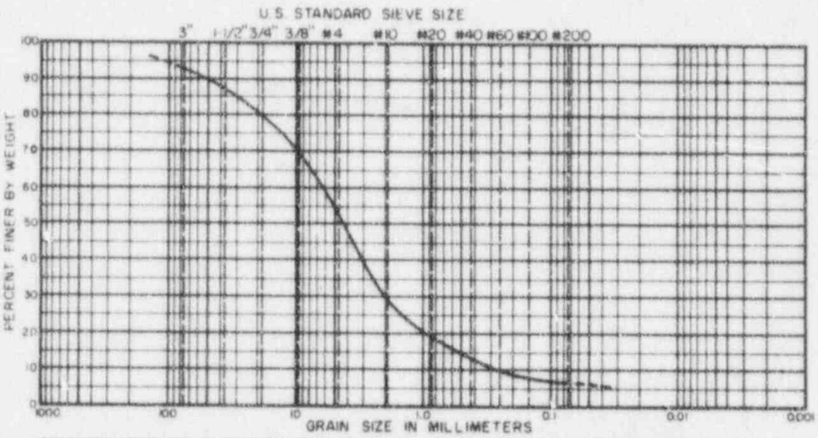
PLATE ____ OF ____

FILE 3861-003
ATLAS MINERALS - MOAB
BY ____ DATE ____
CHECKED BY ____ DATE ____



COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

LOCATION	DEPTH	CLASSIFICATION
EMBANKMENT COVER	SURFACE	REDDISH-BROWN FINE TO COARSE SAND AND GRAVEL (SP-GP)

GRADATION CURVE

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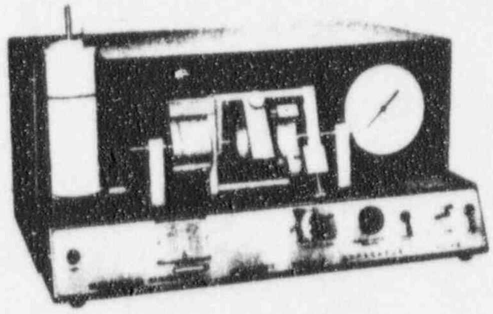
DIRECT SHEAR TESTS ARE PERFORMED TO DETERMINE THE SHEARING STRENGTHS OF SOILS. FRICTION TESTS ARE PERFORMED TO DETERMINE THE FRICTIONAL RESISTANCES BETWEEN SOILS AND VARIOUS OTHER MATERIALS SUCH AS WOOD, STEEL, OR CONCRETE. THE TESTS ARE PERFORMED IN THE LABORATORY TO SIMULATE ANTICIPATED FIELD CONDITIONS.



DIRECT SHEAR TESTING
& RECORDING APPARATUS

EACH SAMPLE IS TESTED WITHIN THREE BRASS RINGS, TWO AND ONE-HALF INCHES IN DIAMETER AND ONE INCH IN LENGTH. UNDISTURBED SAMPLES OF IN-PLACE SOILS ARE TESTED IN RINGS TAKEN FROM THE SAMPLING DEVICE IN WHICH THE SAMPLES WERE OBTAINED. LOOSE SAMPLES OF SOILS TO BE USED IN CONSTRUCTING EARTH FILLS ARE COMPACTED IN RINGS TO PREDETERMINED CONDITIONS AND TESTED.

DIRECT SHEAR TESTS

A THREE-INCH LENGTH OF THE SAMPLE IS TESTED IN DIRECT DOUBLE SHEAR. A CONSTANT PRESSURE, APPROPRIATE TO THE CONDITIONS OF THE PROBLEM FOR WHICH THE TEST IS BEING PERFORMED, IS APPLIED NORMAL TO THE ENDS OF THE SAMPLE THROUGH POROUS STONES. A SHEARING FAILURE OF THE SAMPLE IS CAUSED BY MOVING THE CENTER RING IN A DIRECTION PERPENDICULAR TO THE AXIS OF THE SAMPLE. TRANSVERSE MOVEMENT OF THE OUTER RINGS IS PREVENTED.

THE SHEARING FAILURE MAY BE ACCOMPLISHED BY APPLYING TO THE CENTER RING EITHER A CONSTANT RATE OF LOAD, A CONSTANT RATE OF DEFLECTION, OR INCREMENTS OF LOAD OR DEFLECTION. IN EACH CASE, THE SHEARING LOAD AND THE DEFLECTIONS IN BOTH THE AXIAL AND TRANSVERSE DIRECTIONS ARE RECORDED AND PLOTTED. THE SHEARING STRENGTH OF THE SOIL IS DETERMINED FROM THE RESULTING LOAD-DEFLECTION CURVES.

FRICTION TESTS

IN ORDER TO DETERMINE THE FRICTIONAL RESISTANCE BETWEEN SOIL AND THE SURFACES OF VARIOUS MATERIALS, THE CENTER RING OF SOIL IN THE DIRECT SHEAR TEST IS REPLACED BY A DISK OF THE MATERIAL TO BE TESTED. THE TEST IS THEN PERFORMED IN THE SAME MANNER AS THE DIRECT SHEAR TEST BY FORCING THE DISK OF MATERIAL FROM THE SOIL SURFACES.

METHOD OF PERFORMING DIRECT SHEAR AND FRICTION TESTS

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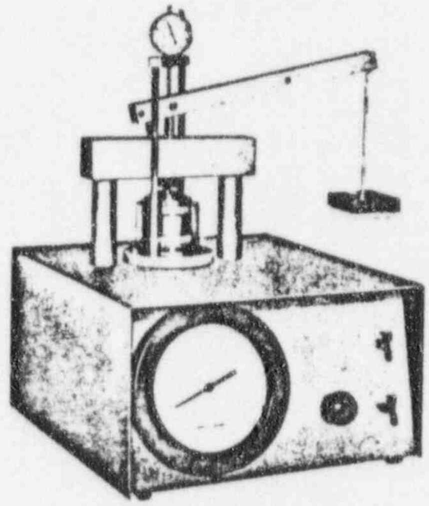
REVISIONS BY ______ DATE ______

Consolidation tests are performed to evaluate the volume changes of soils subjected to increased loads. Time-consolidation and pressure-consolidation curves may be plotted from the data obtained in the tests. Engineering analyses based on these curves permit estimates to be made of the probable magnitude and rate of settlement of the tested soils under applied loads.

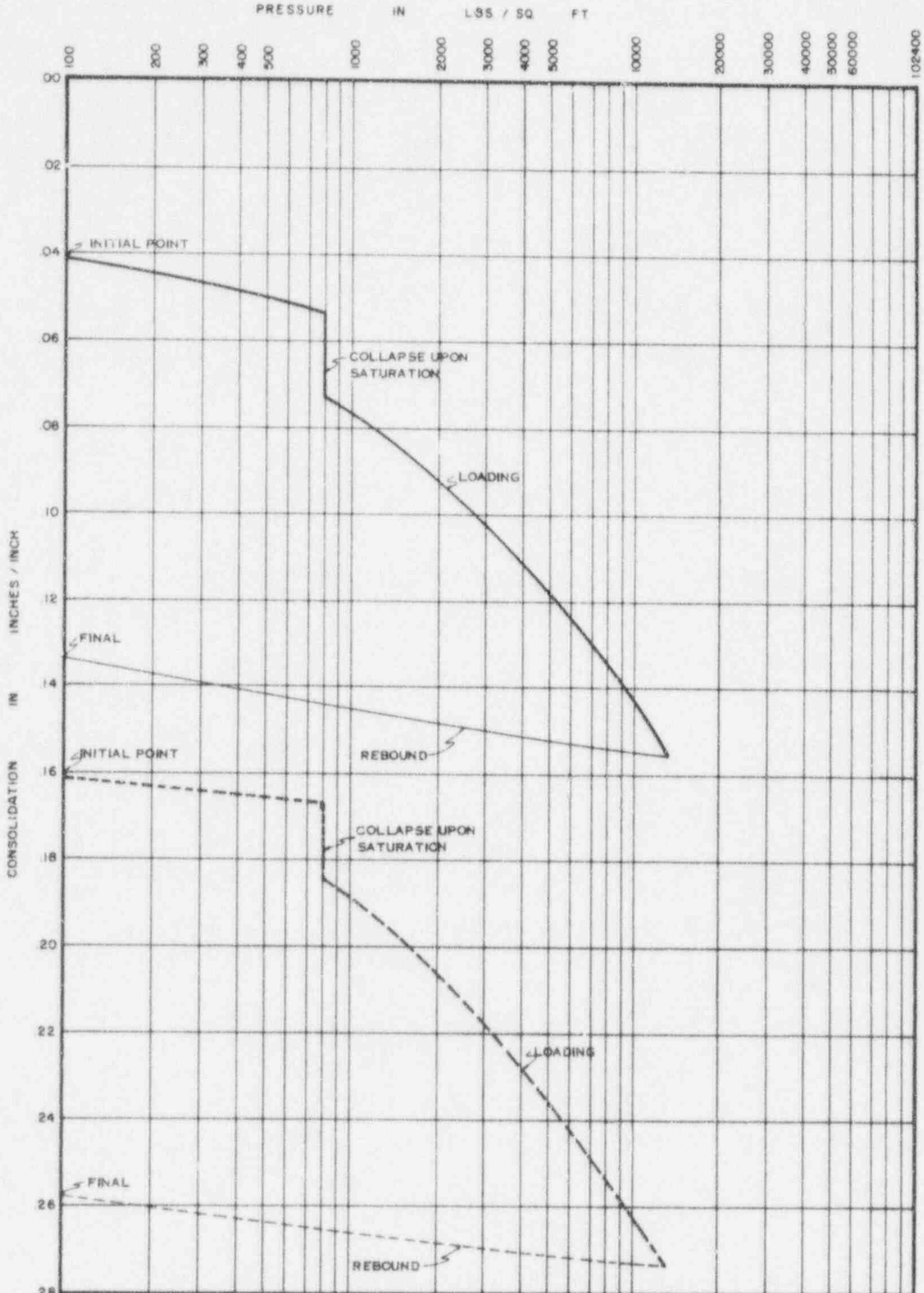
Each sample is tested within brass rings two and one-half inches in diameter and one inch in length. Undisturbed samples of in-place soils are tested in rings taken from the sampling device in which the samples were obtained. Loose samples of soils to be used in constructing earth fills are compacted in rings to predetermined conditions and tested.



DEAD LOAD-PNEUMATIC CONSOLIDOMETER

In testing, the sample is rigidly confined laterally by the brass ring. Axial loads are transmitted to the ends of the sample by porous disks. The disks allow drainage of the loaded sample. The axial compression or expansion of the sample is measured by a micrometer dial indicator at appropriate time intervals after each load increment is applied. Each load is ordinarily twice the preceding load. The increments are selected to obtain consolidation data representing the field loading conditions for which the test is being performed. Each load increment is allowed to act over an interval of time dependent on the type and extent of the soil in the field.

METHOD OF PERFORMING CONSOLIDATION TESTS

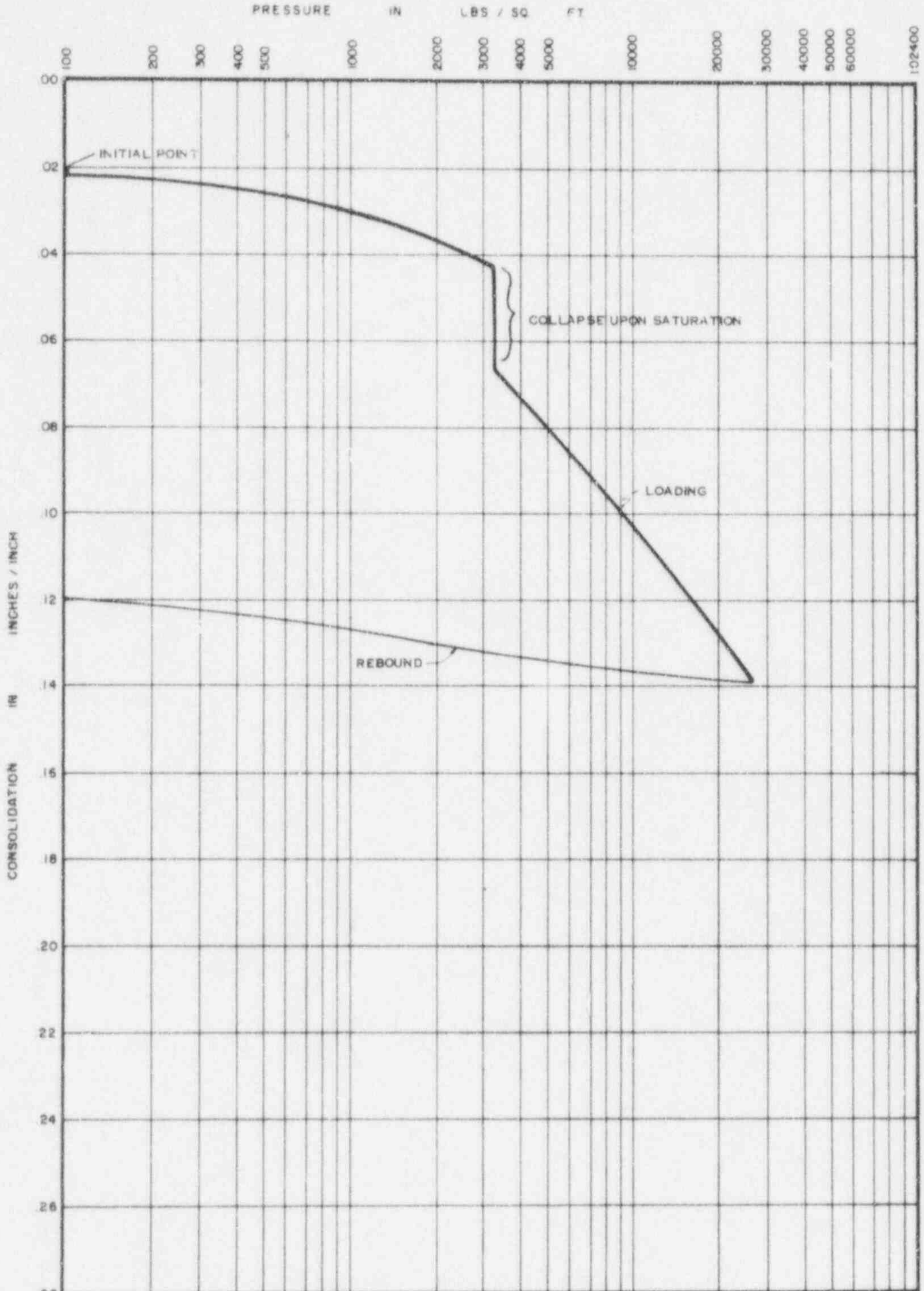


BORING		SOIL TYPE	MOISTURE CONTENT IN PERCENT		DRY DENSITY IN LBS. / CU. FT.	SYMBOL
NO.	DEPTH		BEFORE	AFTER		
A-13	3.5'	REDDISH BROWN SILTY SAND (SM) — NATURAL SOIL	5.6	13.0	112.7	
A-13	6.0'	REDDISH BROWN SILTY SAND (SM) — NATURAL SOIL	4.1	16.3	103.9	

CONSOLIDATION TEST DATA

DAMES & MOORE

PLATE A-11A



BORING		SOIL TYPE	MOISTURE CONTENT IN PERCENT		DRY DENSITY IN LBS. / CU. FT.	SYMBOL
NO.	DEPTH		BEFORE	AFTER		
P-2	5.5'	REDDISH BROWN SILTY SAND (SM) - NATURAL SOIL	6.4	17.7	98.5	———

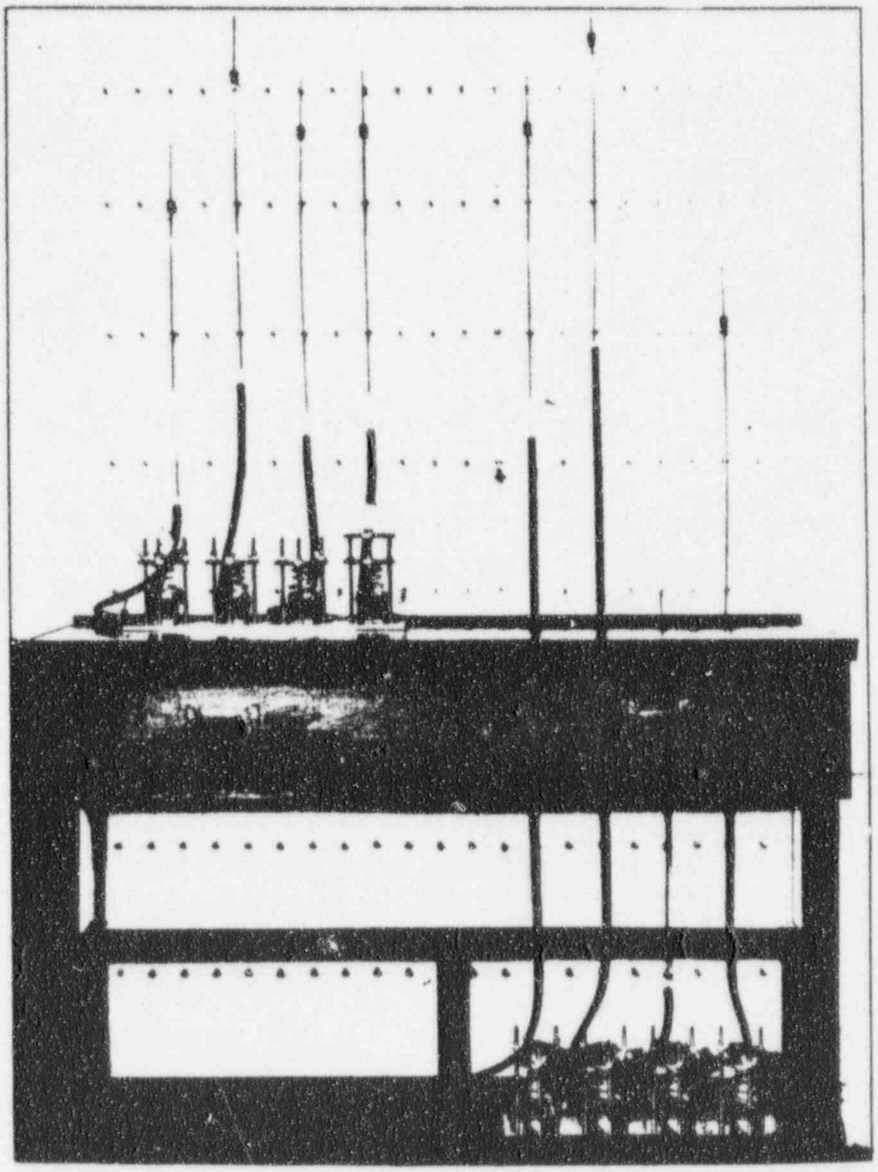
CONSOLIDATION TEST DATA

DAMES & MOORE

PLATE A-11B

The quantity and the velocity of flow of water which will escape through an earth structure or percolate through soil are dependent upon the permeability of the earth structure or soil. The permeability of soil has often been calculated by empirical formulas but is best determined by laboratory tests, especially in the case of compacted soils.

A one-inch length of the core sample is sealed in the perco ation apparatus, placed under a confining load, or surcharge pressure, and subjected to the pressure of a known head of water. The percolation rate is computed from the measurements of the volume of water which flows through the sample in a series of time intervals. These rates are usually expressed as the velocity of flow in feet per year under a hydraulic gradient of one and at a temperature of 20 degrees Centigrade. The rate sc expressed may be adjusted for any set of conditions involving the same soil by employing established physical laws. Generally, the percolation rate varies over a wide range at the beginning of the test and gradually approaches equilibrium as the test progresses.



APPARATUS FOR PERFORMING PERCOLATIONS TESTS
Shows tests in progress on eight samples simultaneously.

During the performance of the test, continuous readings of the deflection of the sample are taken by means of micrometer dial gauges. The amount of compression or expansion, expressed as a percentage of the original length of the sample, is a valuable indication of the compression of the soil which will occur under the action of load or the expansion of the soil as saturation takes place.

METHOD OF PERFORMING PERCOLATION TESTS

DAMES & MOORE

APPENDIX B

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APPENDIX B

EVALUATION OF ENGINEERING PROPERTIES

GENERAL

The materials that comprise the tailings pond-embankment system may generally be classified into four types according to their origin and textural differences; they are: (1) sand tailings, (2) slime tailings, (3) natural red sandy soils, and (4) recompacted fill soils. The engineering properties of each of the four types are pertinent to the evaluation of the stability of the tailings pond-embankment system. The properties were determined by the performance of various laboratory tests that included moisture-density determinations, Atterberg limits tests, grain-size analyses, a relative density test, permeability tests, compaction tests, direct shear tests, triaxial compression tests, and consolidation tests. The procedures used for performing these tests and their results are described in Appendix A. The following subsections present the design parameters for each soil type that were derived from the results of these laboratory tests. Emphasis of the presentation will be placed on the determination of design shear strength parameters as they apply to performing stability analysis under end-of-construction, steady state seepage, and pseudo static (seismic) loading conditions. The selected design shear strength parameters are summarized in Table B-1.

TABLE B-1

SUMMARY OF DESIGN PARAMETERS FOR EMBANKMENT MATERIALS [a]

Design Parameter	Sand Tailings (SM)	Slime Tailings (ML)	Natural Red Sandy Soils (SM-SP)	Compacted Fill (SM-SP)
Total Unit Weight, pcf	120	120 (Consolidated) 100 (Lightly Consolidated)	125	120
Drained Shear Strength Parameters:	$\phi = 37^{\circ}$ $c = 0$	$\phi = 31^{\circ}$ $c = 0$	$\phi = 36^{\circ}$ $c = 0$	$\phi = 32.5^{\circ}$ $c = 0$
Undrained Shear Strength Parameters: (Main Embankment: Sections A-A & E-E)	$\phi = 37^{\circ}$ $c = 0$	(S_u/σ_c)[b] $= \tan \alpha = .38$	$\phi = 36^{\circ}$ $c = 0$	$\phi = 32.5^{\circ}$ $c = 0$
Undrained Shear Strength Parameters: (Western Embankment: Section C-C)	$\phi = 37^{\circ}$ $c = 0$	Directly Under Pond Area (S_u/σ_c)[b] $= \tan \alpha = .38$ Directly Under Embankment Area $\phi = 0$ $c = 1{,}300$ psf	$\phi = 36^{\circ}$ $c = 0$	$\phi = 32.5^{\circ}$ $c = 0$

---

[a] Values of the shear strength parameter are conservatively chosen or assumed based on results of laboratory tests as shown on Plates 5 through 8.

[b] The undrained shear strength varies directly with the effective overburden (see explanation on page B-5).

SAND TAILINGS

GENERAL SOIL PROPERTIES

Sand tailings are generally found in the beach area between the crest of the embankment and the nearest point of ponded water. The sand tailings are generally silty fine sands that are loose to medium dense, and often containing thin seams or lenses of clayey silt (slimes) materials. The sands are relatively well-drained with permeabilities ranging from 4×10^{-5} to 4×10^{-3} centimeters per second (40 to 4,000 feet per year) based on laboratory percolation tests performed on relatively undisturbed samples obtained during the field exploration program. The average in-place dry density as determined from the relatively undisturbed samples was 96.8 pounds per cubic foot with an average field moisture content of 20.1 percent.

DRAINED STRENGTH

To evaluate steady-state seepage and end-of-construction conditions in the sand tailings, when the pore water pressures in the well-drained sands are known, the effective stress parameters of $\phi = 37^{\circ}$ and $c = 0$, as developed in the consolidated-undrained and consolidated-drained triaxial tests and drained direct shear tests, were used (Plates B-1B and B-1C).

UNDRAINED STRENGTH

The undrained strength properties of the sand tailings used to evaluate the seismic loading conditions would normally be selected on total stress, undrained strength properties obtained from consolidated-undrained triaxial test. However, due to the development of negative pore water pressures during triaxial shearing, the Mohr circle total stress envelope exceeds that of the effective stress envelope (Plate B-1A through B-1C). To be conservative the effective strength parameters of $\phi = 37^{o}$ and c = 0 were used for the end-of-construction and seismic loading conditions.

SLIME TAILINGS

GENERAL SOIL PROPERTIES

Slimes are the fine fractions of the tailings produced and are generally clayey silt or sometimes, silty clay. Slimes tailings are normally deposited in the center of the tailings pond where standing water exists; however, they are also often encountered as thin seams or lenses in the sand tailings due to temporary ponding of the discharged tailings that were trapped in the depressions on the beach area. Depending upon the magnitude of the consolidation pressure (overburden) that has been imposed on the slimes deposits, the slimes may exist in a form ranging from an unconsolidated slurry at the surface to a stiff, highly consolidated clayey silt at depth. The average in-place dry density of the consolidated slimes is 84.7 pounds per cubic foot with an average field moisture content of 37.4 percent. The slimes materials are relatively impervious with permeability ranging from 2×10^{-7} to 4×10^{-6} centimeters per second (0.2 to 4 feet per year). These values are based on percolation tests performed on relatively undisturbed samples obtained from slime layers that had been subjected to various consolidation pressures. Results of the gradation analyses have shown that the slimes generally contain a minimum of 90 percent by weight of particles finer than the No. 200 sieve. The material may possess low to high plasticity characteristics.

Due to the nature of the depositional process, the slimes material in the tailings pond is gradually consolidated with time at a continually increasing magnitude of pressure and, therefore, may be termed as a normally consolidated material. At some surface areas, where the slimes have been undisturbed for long periods of time, desication (the process of soil drainage through surface evaporation) has occurred and therefore such slimes have become overconsolidated to a degree.

DRAINED STRENGTH

Steady-state seepage conditions were analyzed using the effective stress parameters obtained from the consolidated-undrained triaxial compression tests performed on slime tailings samples (Plate B-2A). This condition assumes that sufficient time has ellapsed to dissipate excess pore water pressures due to construction of additional embankment raises. Effective stress parameters of $\phi = 31^{\circ}$ and c = 0 as developed in the consolidated-undrained triaxial tests were used.

UNDRAINED STRENGTH

In Sections A-A, C-C and D-D, where horizontal slimes tailings layers underlie the embankment from toe to crest, the undrained shear strength (S_u) of the soil was used in end-of-construction and pseudo-static stability analyses. The undrained shear strength of the slimes is related directly to the original overburden pressure. By correlating the undrained shear strength and the initial consolidation pressure of the laboratory consolidated-undrained triaxial compression tests in which the test specimens were normally consolidated, (that is S_u vs. α_c), an average straight line relationship between the two values was obtained. This relationship may be equated to the slope of the line shown on Plate B-2B whose angle with the horizontal (α) is 21° (that is, the relationship $S_u/\sigma_c = \tan\alpha$). The above values represent the undrained shear for the slimes encountered in Sections A-A and E-E as well as in Section C-C, where the slimes layers underlie the tailing pond area only.

The undrained shear strength of the slimes material, directly underlying the embankment area along the western edge of the tailings pond (see Section C-C), is represented by unconsolidated- undrained shear tests performed on samples obtained from Boring A-14. Although the shear strength values will vary with degree of consolidation, a conservative equivalent cohesive value of c = 1300 psf. The results of the undrained shear strength test data is shown below:

TABLE B2

UNCONSOLIDATED-UNDRAINED TRIAXIAL TEST RESULTS
FROM BORING A-14

Depth of Sample	Dry Density, PSF	Moisture Content, %	Undrained Shear Strength, Su, PSF
2.0 feet	99.9	21.8	1985
3.5 feet	94.5	26.7	1376
5.0 feet	98.5	26.9	1401
8.5 feet	103.9	23.2	2638

It has been assumed the tailings pond level behind the embankment rises at a rate sufficiently slow that no excessive pore pressures will develop from rapid pond build-up.

NATURAL RED SANDY SOILS

GENERAL SOIL PROPERTIES

The natural occurring, red sandy soils are generally the foundation material of the tailings pond-embankment system. As previously described, these soils are predominantly sand with varying amounts of silt and gravel. The gradation of the natural soils are highly variable and cobbles and boulders are known to be sometimes present. In its natural state, some of these soils appear to be weakly cemented, as can be inferred from the results of laboratory unconsoolidated-undrained triaxial compression tests. Saturation of the natural materials tends to destroy some of the cementation. The natural sandy soils may be considered relatively permeable to water, with permeability ranging from 8 x 10^{-5} to 2 x 10^{-3} centimeters per second (80 to 2,000 feet per year) based on percolation tests performed on relatively undisturbed samples taken from the exploratory borings. The samples taken have an average dry density of 108.8 pounds per cubic foot with an average in-place moisture of 11.9 percent.

DRAINED STRENGTH

Based on the results of laboratory direct shear tests,as shown on Plate B-3, the natural red sandy soils have an average effective, or drained, angle of internal friction, ø, of 40°. The test specimens were saturated prior to the tests and the cohesion value of the material appears to be near zero, indicating that any natural cementation was destroyed by the saturation process. For conservatism, values of $\phi = 36^{\circ}$ and $c = 0$ were used to represent the drained shear strength parameters of the in-situ material, assuming that any natural cementation in the foundation material will be completely destroyed as a result of the existence of the tailings pond-embankment system. This value of ø may be used as the shear strength parameter of the natural red sandy soils for analyzing slope stability of the tailings pond-embankment system under drained (steady state seepage and end of construction conditions).

UNDRAINED STRENGTH

For undrained conditions, that is, under earthquake loadings, the undrained (total stress) shear strength parameters for the natural red sands are assumed the same as that of for drained conditions. This assumption was used because with the generally dense state of the in-situ material, small positive, or even negative pore pressures would occur under earthquake loading conditions and, hence, the same or higher undrained shear strength parameter would result.

RECOMPACTED NATURAL RED SANDY SOILS AND SAND TAILINGS

DRAINED STRENGTH

The natural red sandy soils and the sand tailings have been used and proposed planning indicates they will be continued to be used to build the starter dikes and the supplemental dikes that comprise the outer portion of the main tailings pond embankments as well as the entire section of the recently completed western embankment. Based on direct shear tests performed on samples of the red sandy soils compacted to 90 percent of the maximum dry density as determined by the American Society of Testing Materials (ASTM) D-698 Method of compaction, the remolded natural materials exhibit effective (drained), parameters of $\phi = 32.5^{o}$ and c = 0 (Plate B-4). Although the western embankment has been constructed using 95 percent compaction criteria and all future embankment additions will be constructed to 95 percent compaction, for conservatism the values given above have been selected to represent the shear strength parameters of recompacted materials used in the slope stability analyses under steady state seepage and end-of-construction loading conditions.

UNDRAINED STRENGTH

Undrained loading conditions, such as earthquake within a saturated zone was not used since almost all the recompacted materials are above the water table. Thus, shear strength parameters for the non-earthquake case were used for cases under psuedo-static seismic loading condition.

946.7 (REV. 6-...)

BY [signature] DATE ________
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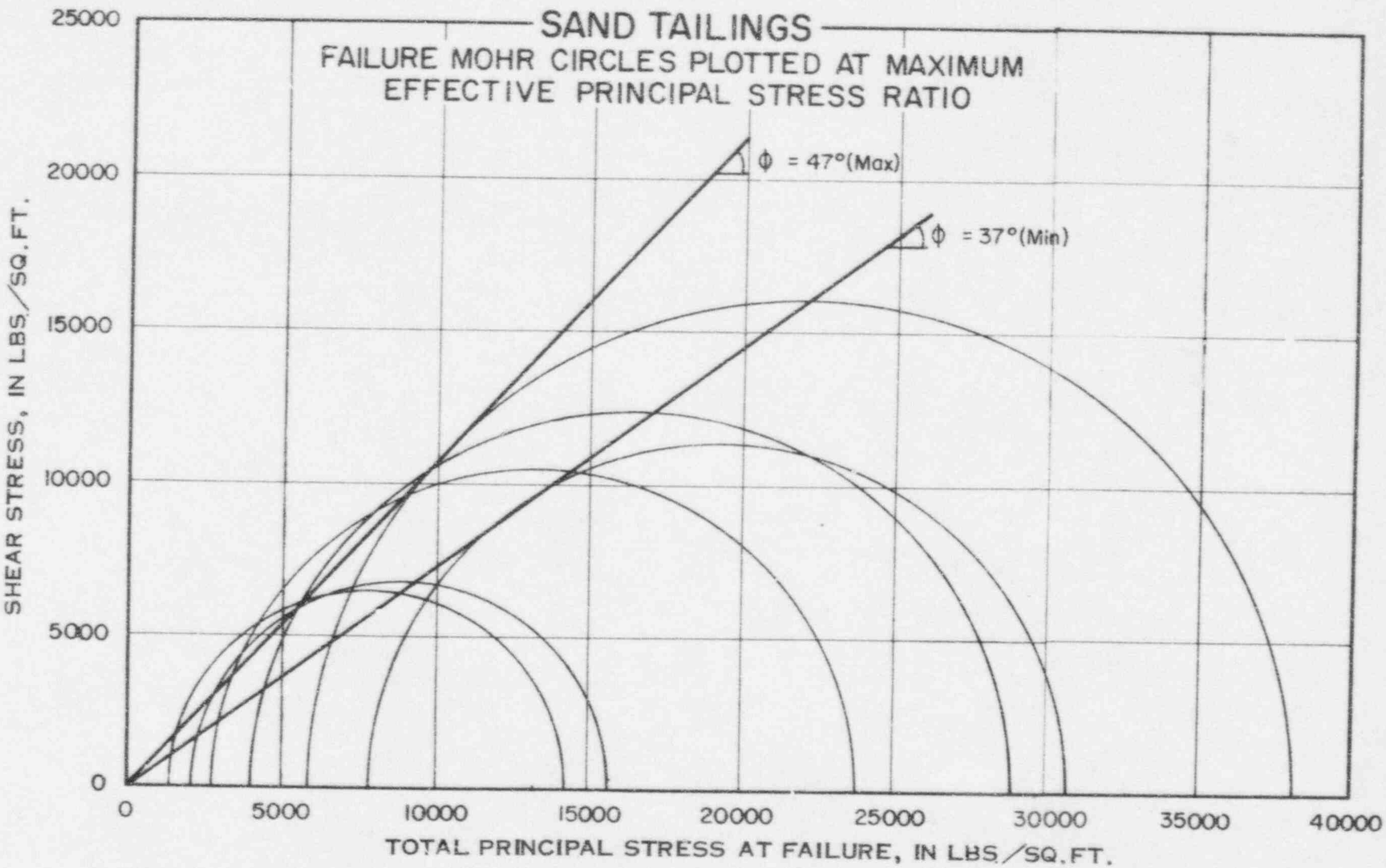

RESULTS OF CONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TESTS (SAND TAILINGS) TOTAL STRESS

DAMES & MOORE

PLATE B-1A

946.7 (REV. ...)

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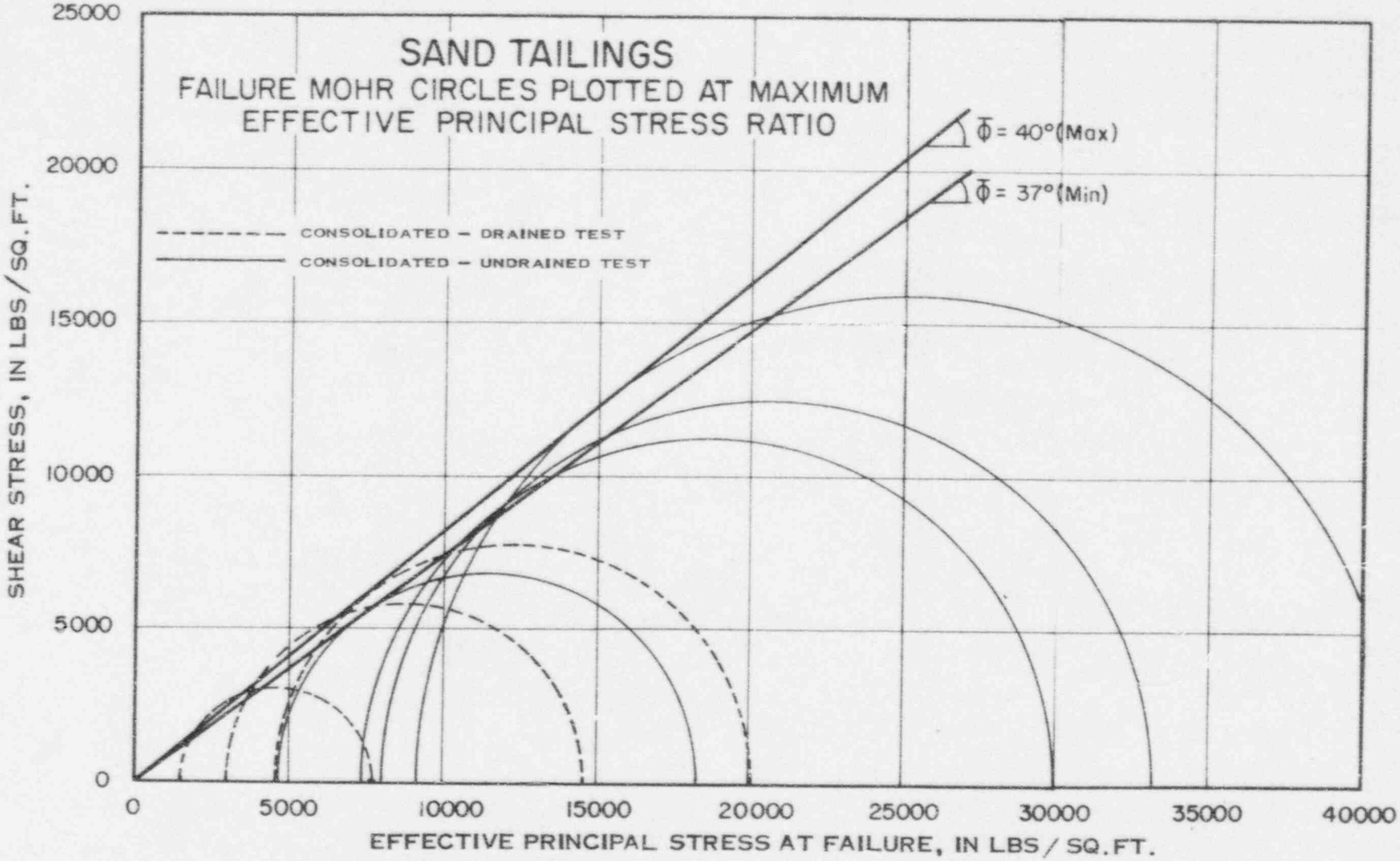

RESULTS OF TRIAXIAL COMPRESSION TESTS
(SAND TAILINGS) - EFFECTIVE STRESS

PLATE B-1B

DAMES & MOORE

946.7 (REV. ...)

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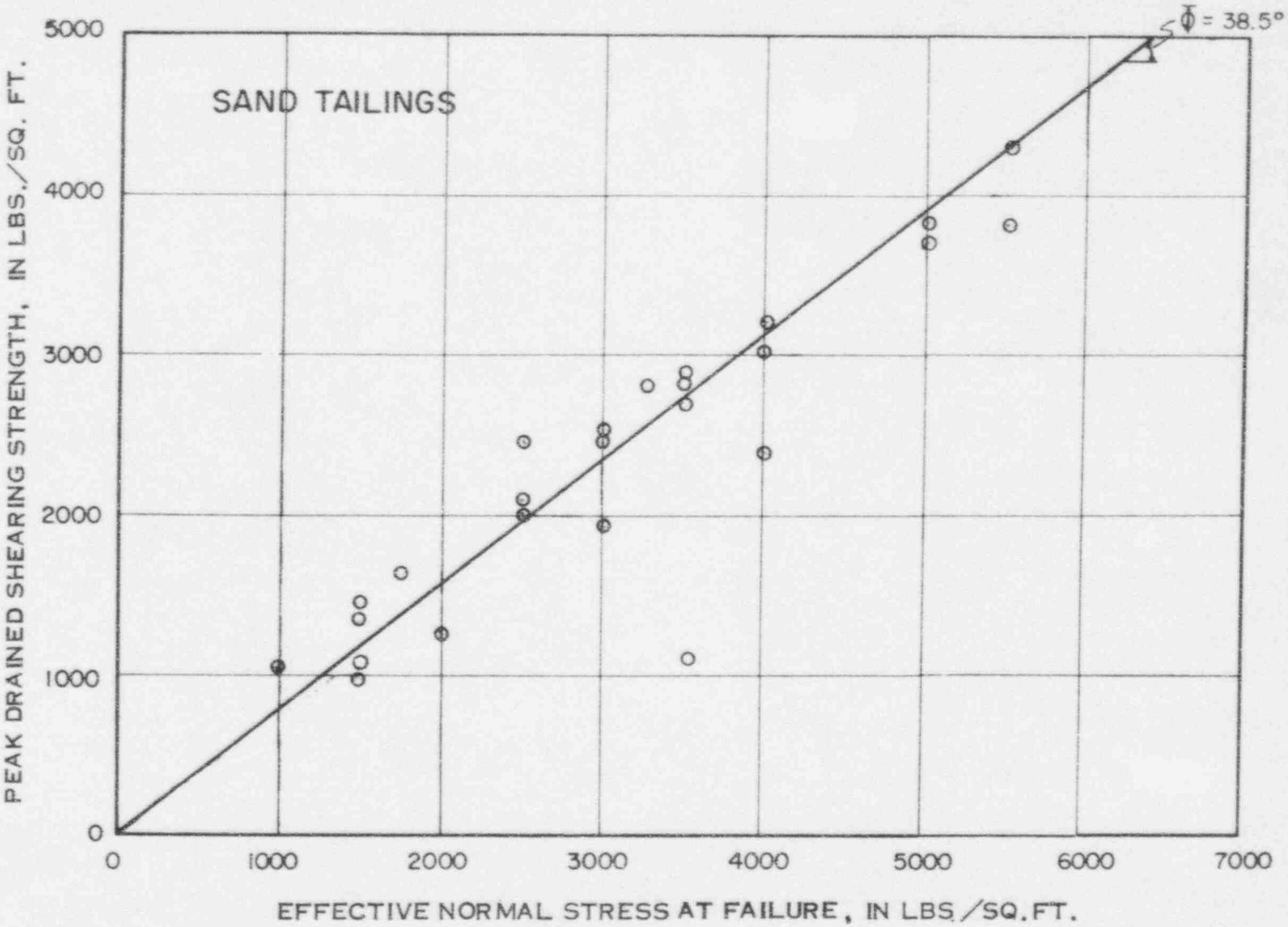

RESULTS OF DIRECT SHEAR TESTS
(SAND TAILINGS) - EFFECTIVE STRESS

DAMES & MOORE

PLATE B-1C

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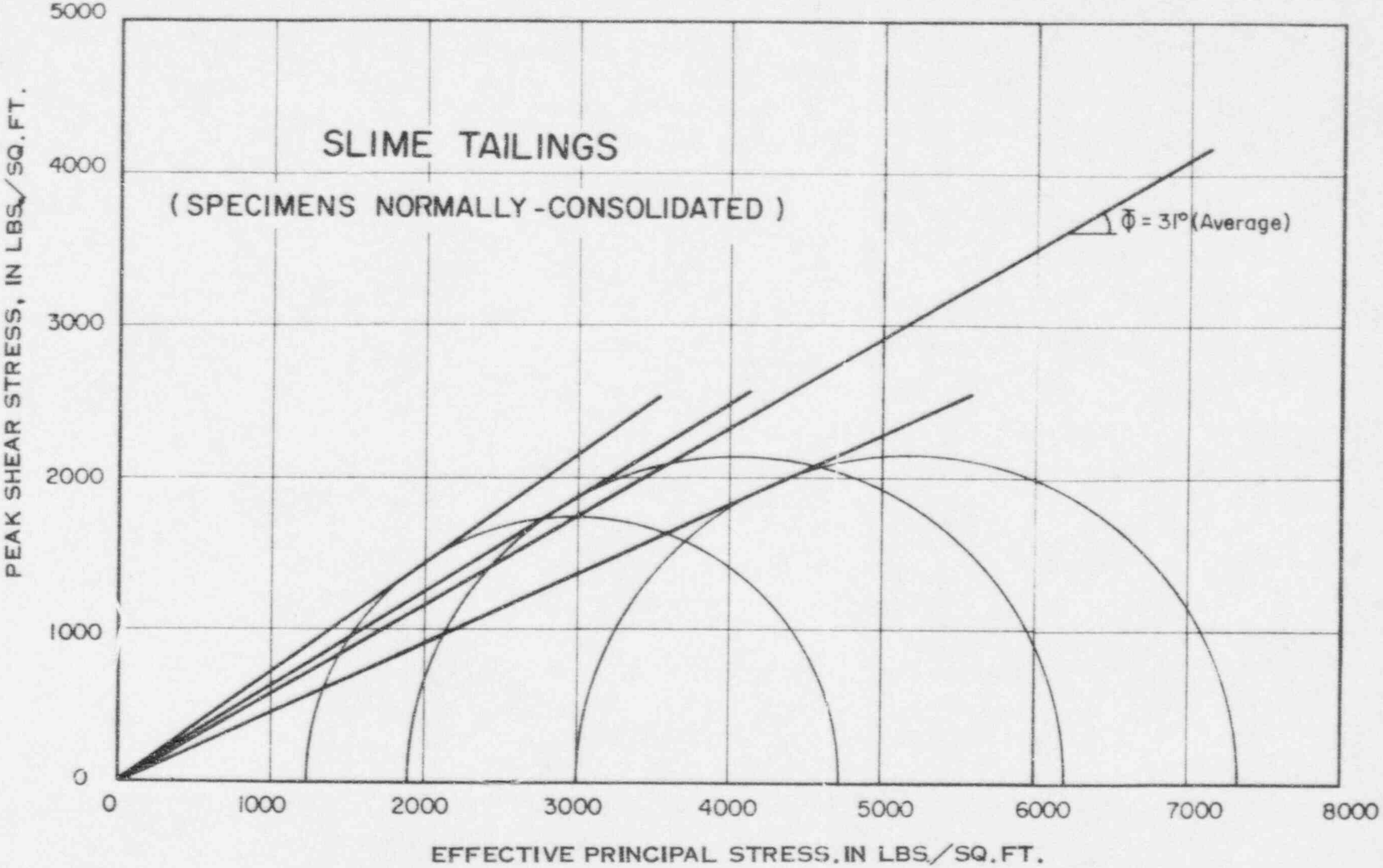
BY ________ DATE ________



RESULTS OF CONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TESTS
(SLIME TAILINGS) - EFFECTIVE STRESS

DAMES & MOORE

PLATE B-2A

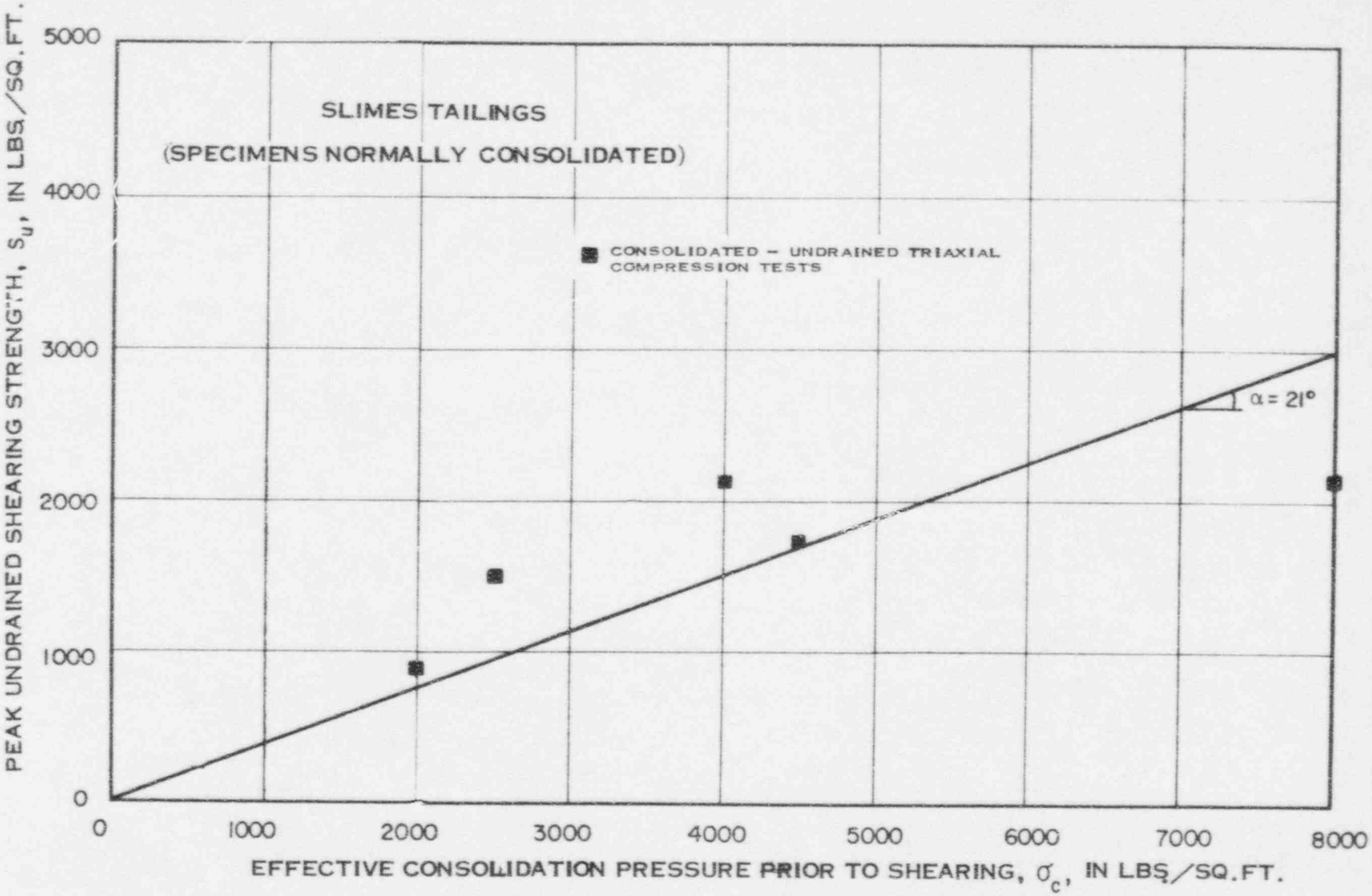
946.7 (REV. [illegible])

BY Pat Walter DATE

CHECKED BY [illegible] 9-29-77 FILE 05467-018 Atlas Minerals

REVISIONS

BY ________ DATE ________



RESULTS OF UNDRAINED SHEARING STRENGTH
OF NORMALLY - CONSOLIDATED SLIMES

AS A FUNCTION OF EFFECTIVE CONSOLIDATION PRESSURE PRIOR TO SHEARING

DAMES & MOORE

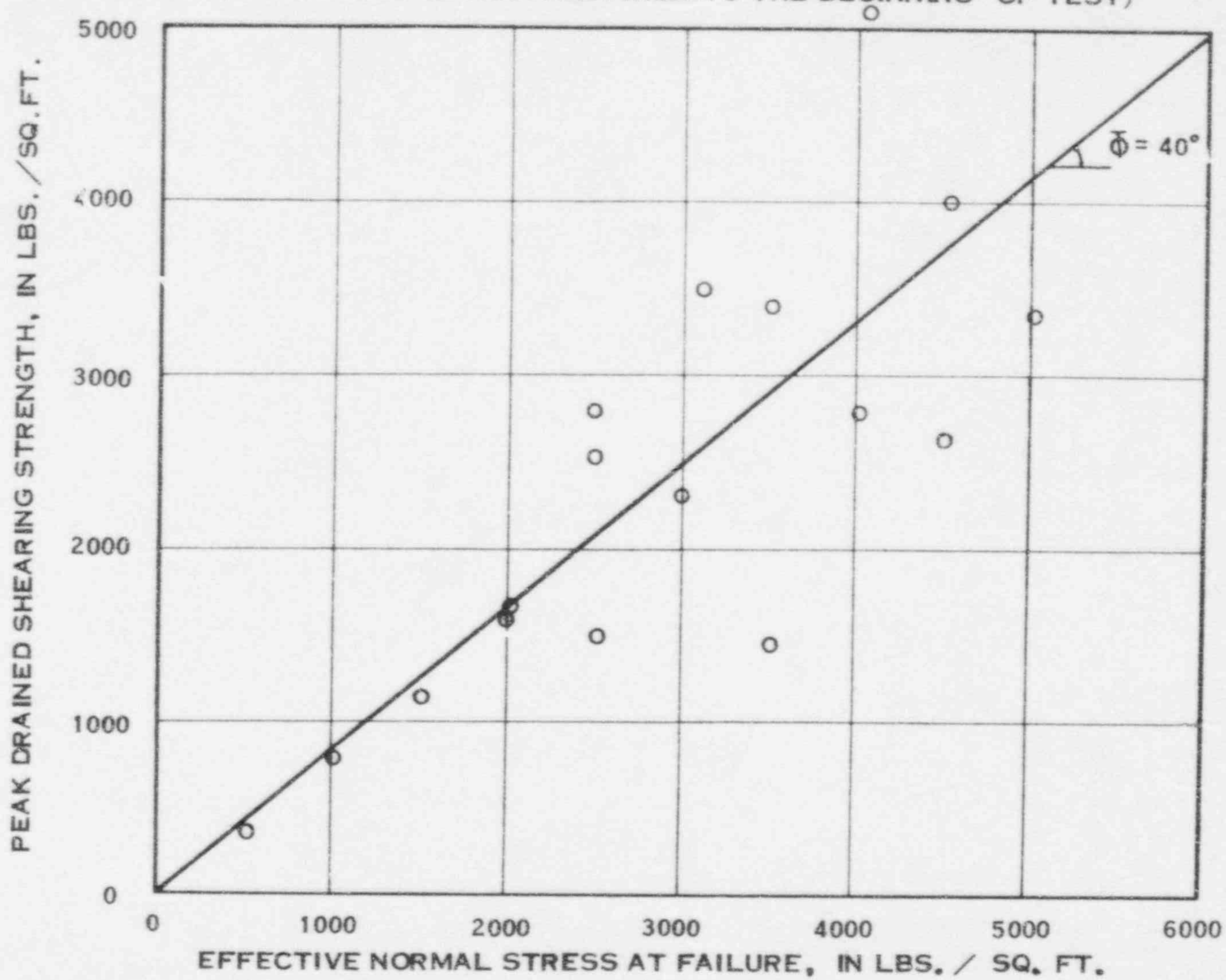
PLATE B-2B

946.7 (REV

BY ______ DATE ______
CHECKED BY ______ FILE ______

REVISIONS
BY ______ DATE ______

NATURAL RED SANDY FOUNDATION SOILS
(SPECIMENS SATURATED PRIOR TO THE BEGINNING OF TEST)



RESULTS OF DIRECT SHEAR TESTS
(IN-SITU RED SANDY SOIL)-EFFECTIVE STRESS

DAMES & MOORE

PLATE B-3

946.7 (REV. 6-

BY Smith DATE 2-13-78

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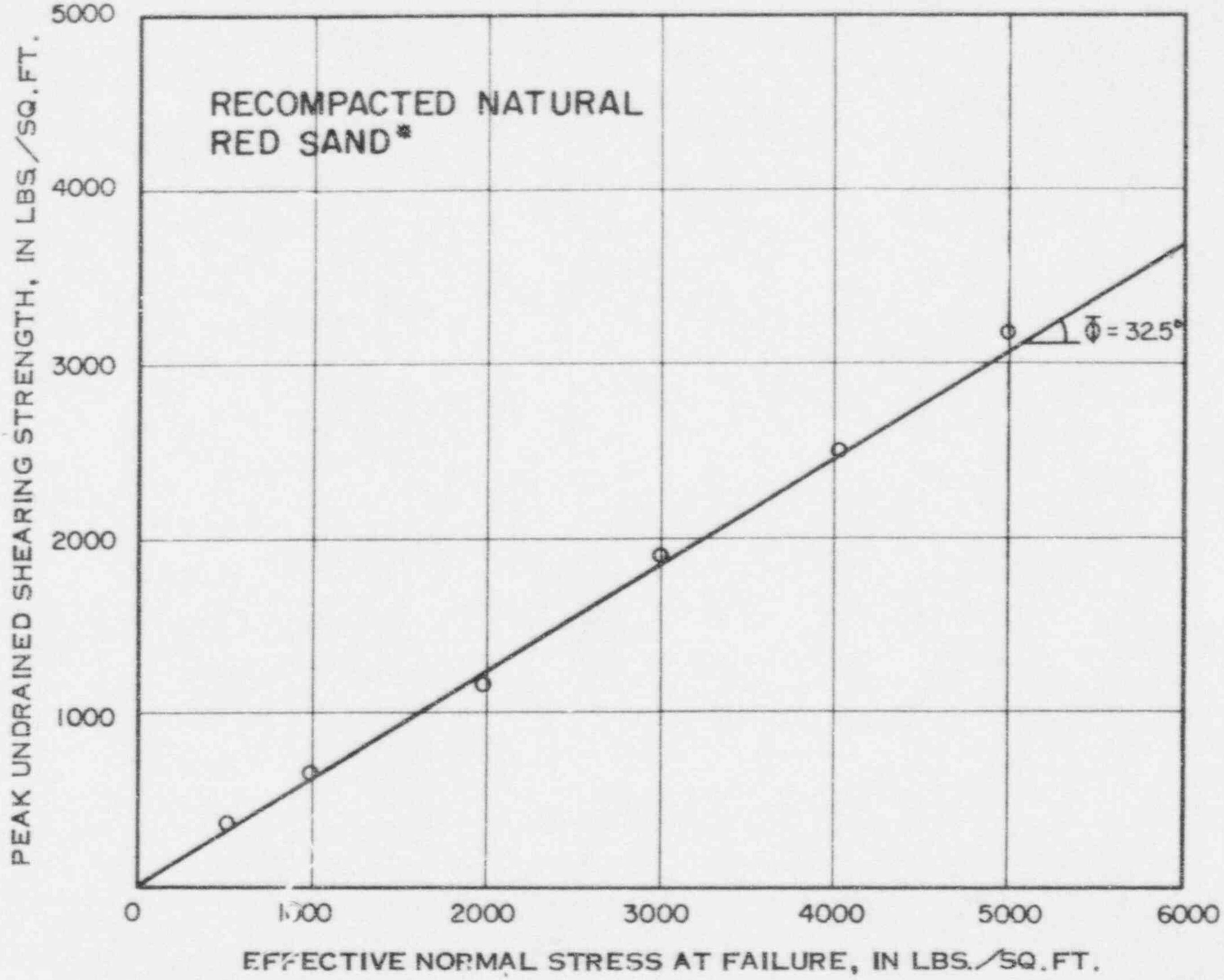
BY DATE

* SPECIMENS WERE RECOMPACTED TO AN AVERAGE OF 100.5 PCF (OR 90 PERCENT OF THE MAXIMUM DRY DENSITY AS DETERMINED IN ACCORDANCE WITH A.S.T.M. D-698 METHOD OF COMPACTION)



RESULTS OF DIRECT SHEAR TESTS (RECOMPACTED RED SANDY SOIL) - EFFECTIVE STRESS

DAMES & MOORE

PLATE B-4

APPENDIX C

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