ATTACHMENT D

"General Report"

Geotechnic, Hydrologic and Hydraulic Evaluation

of Sedimentation Structures

GENERAL REPORT

Geotechnic, Hydrologic and Hydraulic Evaluation of Sedimentation Structures

Kayenta and Black Mesa Mines

Navajo County, Arizona

for

PEABODY COAL COMPANY

Registered Professional Engineer (Civil)
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ARIZONA, U.S.A.

Dames & Moore
10139-011-22

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1.0 INTRODUCTION

Inspections, field investigations, laboratory testing and engineering analyses have recently been performed by Dames & Moore to evaluate compliance of sedimentation structures at Peabody Coal Company's Kayenta and Black Mesa Coal Mines in Navajo County, Arizona, with the performance standards for sedimentation structures set forth in the Office of Surface Mining (OSM) Indian land regulations, 30 CFR, Chapter VII, Parts 780 and 816. This General Report presents a summary of assumptions, data and methodologies that were used in our evaluations and is intended to serve as a companion document to the individual inspection reports that have been prepared for each of the sedimentation structures.

The locations of the sedimentation structures are shown on Drawing No. 85405.

2.0 GEOLOGY OF THE KAYENTA-BLACK MESA COAL MINES AREA

2.1 GENERAL

In the area surrounding the Kayenta-Black Mesa Coal Mines, several formations of the late Cretaceous Mancos Shale and Mesa Verde Group crop out (Figure 2-1). From the oldest, the Mancos Shale, the Toreva Formation, the Wepo Formation and the Yale Point Sandstone are exposed at the surface. Quaternary alluvium is found in the washes throughout the area. These outcrops are described below.

As shown on Figure 2-1, there are several folds in the area and the strata dip gently throughout most of Black Mesa Basin. No major faults have been mapped in the area.

2.2 MANCOS SHALE

The Mancos Shale (Km) is a marine shale that crops out in areas highly eroded by washes in the central portions of the basin and around the margins of the basin (Figure 2-1). The Mancos is composed of silt, clay, and very fine-grained sand. It varies in color from light to dark gray and is yellowish gray in areas where it has a high sand content. Thinly-bedded, fine-grained sandstones occur in several zones. Beds of bentonitic clay up to 3 feet thick occur in several horizons. All of the sediments in the Mancos of the Black Mesa area are well sorted, weakly cemented, and have flat, very thin bedding. The formation generally weathers to a fairly gentle slope (Page and Repenning, 1958; Cooley and others, 1969).

2.3 MESA VERDE GROUP

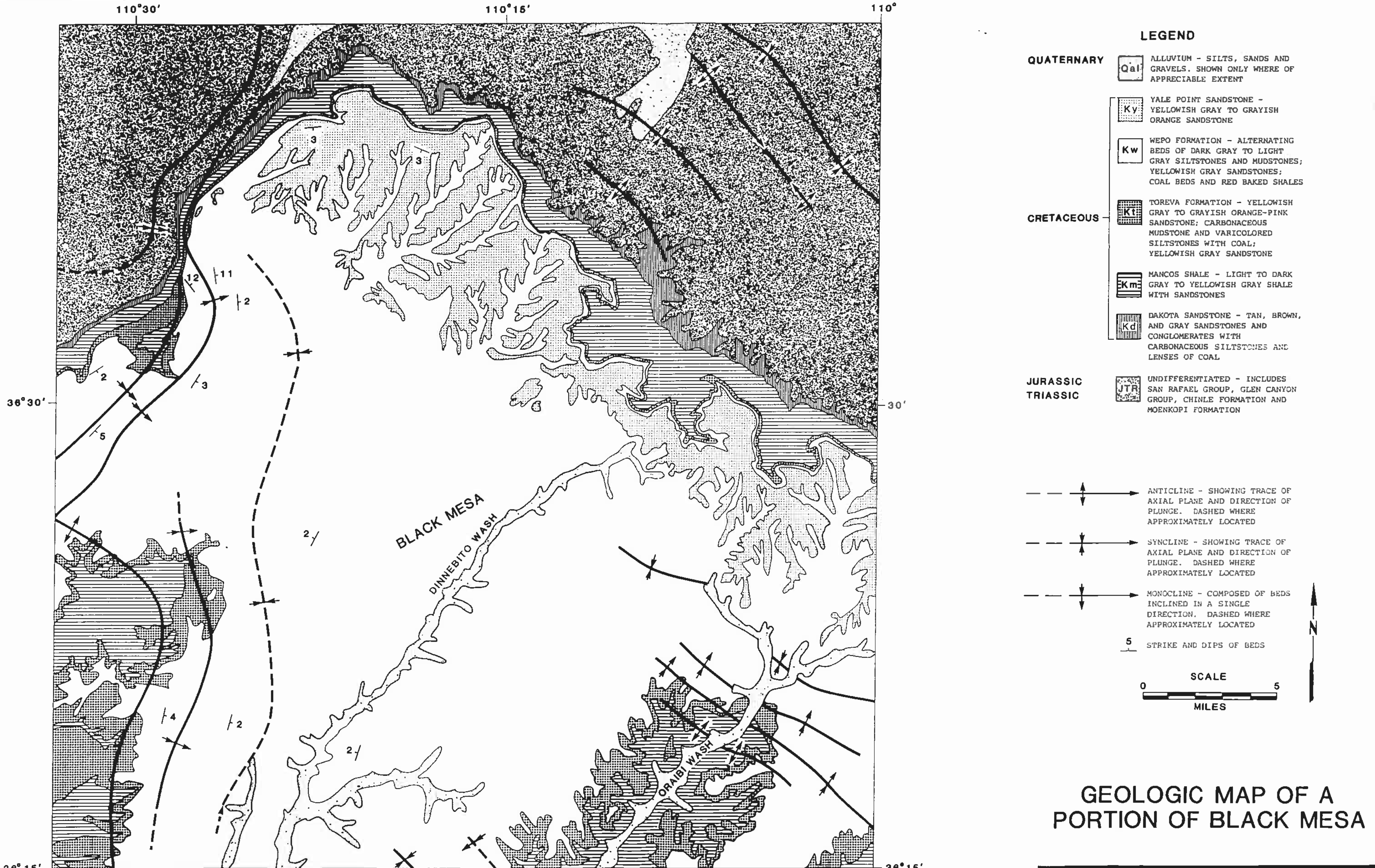
2.3.1 Toreva Formation

The Toreva Formation (Kt) overlies and intertongues with the Mancos Shale. This formation crops out in areas highly eroded by washes in the central portion of the basin and around the periphery of the basin (Figure 2-1). The Toreva has been subdivided into three members: a lower sandstone member, a middle carbonaceous member, and an upper sandstone member (Page and Repenning, 1958).

The lower sandstone member is light brown to pale yellowish gray, fine to medium-grained quartz sandstone with mica as an accessory mineral. Several mudstone units occur in the lower part of the section. Also, fine-grained sandstones are evident. The upper part of the lower sandstone member is fine- to medium-grained with no mudstones present. These sandstones are composed of several sets of crossbeds. The lower sandstone member of the Toreva forms vertical, blocky cliffs.

The middle carbonaceous member of the Toreva consists of an alternation of flat and thinly-bedded carbonaceous mudstone, varicolored siltstones with coal, and thick lenses of yellowish gray fine- to coarse-grained poorly sorted, cross-bedded quartz sandstone.

The upper member is a yellowish gray to grayish orange-pink cross-bedded sandstone composed of fine to very coarse-grained, poorly sorted quartz sand. Toward the north this upper sandstone member includes an



REFERENCE: GEOLOGIC MAP MODIFIED FROM COOLEY AND OTHERS, 1969

GEOLOGIC MAP OF A PORTION OF BLACK MESA

BY **Dames & Moore**

Figure 2-1

additional unit of coal, carbonaceous shale, and sandstone and is capped by medium- to fine-grained sandstone.

2.3.2 Wepo Formation

The Wepo Formation (Kw) is the predominant outcrop in the northern part of the basin and throughout the mine area (Figure 2-1). This formation is in gradational contact with and overlies the Toreva Formation. The Wepo Formation is a thick series of layered siltstone, mudstone, sandstone and coal. The siltstone and mudstone units vary in color from dark olive-gray to light olive-brown to medium light gray. The bedding is generally flat, laminated to very thin; cross bedding is occasionally apparent in some sandy horizons but it is often masked by the shaley weathering of these units. The siltstone-mudstone unit is mostly carbonaceous with some sandstone lenses and sandy zones.

The sandstone portion of the Wepo Formation is cross-bedded and usually has a yellowish-gray color. The sandstones vary from weakly cemented, very argillaceous units which weather to slopes, to strongly cemented, cliff-forming units. Some of the thicker sandstone units are partly conglomeritic. Iron-rich concretions, mud pellets, silty lenses, and carbonized plant remains are also common.

Siltstone units are common within the major sandstone units of the Wepo Formation. Coal beds occur within these siltstone layers. Also typical of the formation in this portion are hard baked shales which are the result from the burned coal. These baked layers vary from yellowish red to terracotta to dark reddish brown in color. Locally these layers are termed scoria.

2.3.3 Yale Point Sandstone

The Yale Point Sandstone (Ky) overlies and intertongues with the Wepo Formation. It crops out in the northeastern portion and around most of the margin of Black Mesa Basin (Figure 2-1) forming spectacular vertical cliffs. No younger consolidated sediments overlie the Yale Point Sandstone, so its upper limit is the surface of recent erosion.

This sandstone is yellowish gray and weathers to a grayish orange. It is composed of coarse- to medium-grained subrounded to subangular clear quartz. The formation has lenticular bedding and is cross-bedded. There are occasional silty units which weather to the ledges and minor slopes on the cliff face. In areas where the Yale Point Sandstone intertongues with the Wepo Formation, the outcrop has much more of a ledge appearance instead of the cliff-forming pattern. This is due to the increase in fine-grained layers. Minor amounts of coal are also present in these intertongued, ledge and slope-forming units.

2.4 QUATERNARY AND RECENT DEPOSITS

The Quaternary and recent unconsolidated materials were derived from the weathering of the surface formation. A veneer of residual soil mantles all but the steepest slopes and cliffs. These soils, transported as slope wash, increase in thickness on the lower portions of the slopes and contribute to the alluvium in the washes.

On the slopes, residual soils reflect the character of the parent bedrock. Shales and mudstones have weathered to clayey and silty soils of low to medium plasticity. Soils derived from sandstone consist of silty fine sands, generally with no plasticity. More resistant bedrock fragments are included in the soils as gravel- to cobble-size material.

The alluvial soils are predominantly very fine to coarse sands (SP, GP) with varying amounts of gravel, derived from the weathering of the surface formation and transport as alluvium in the washes. The alluvial soils are generally susceptible to collapse.

2.5 SEISMICITY

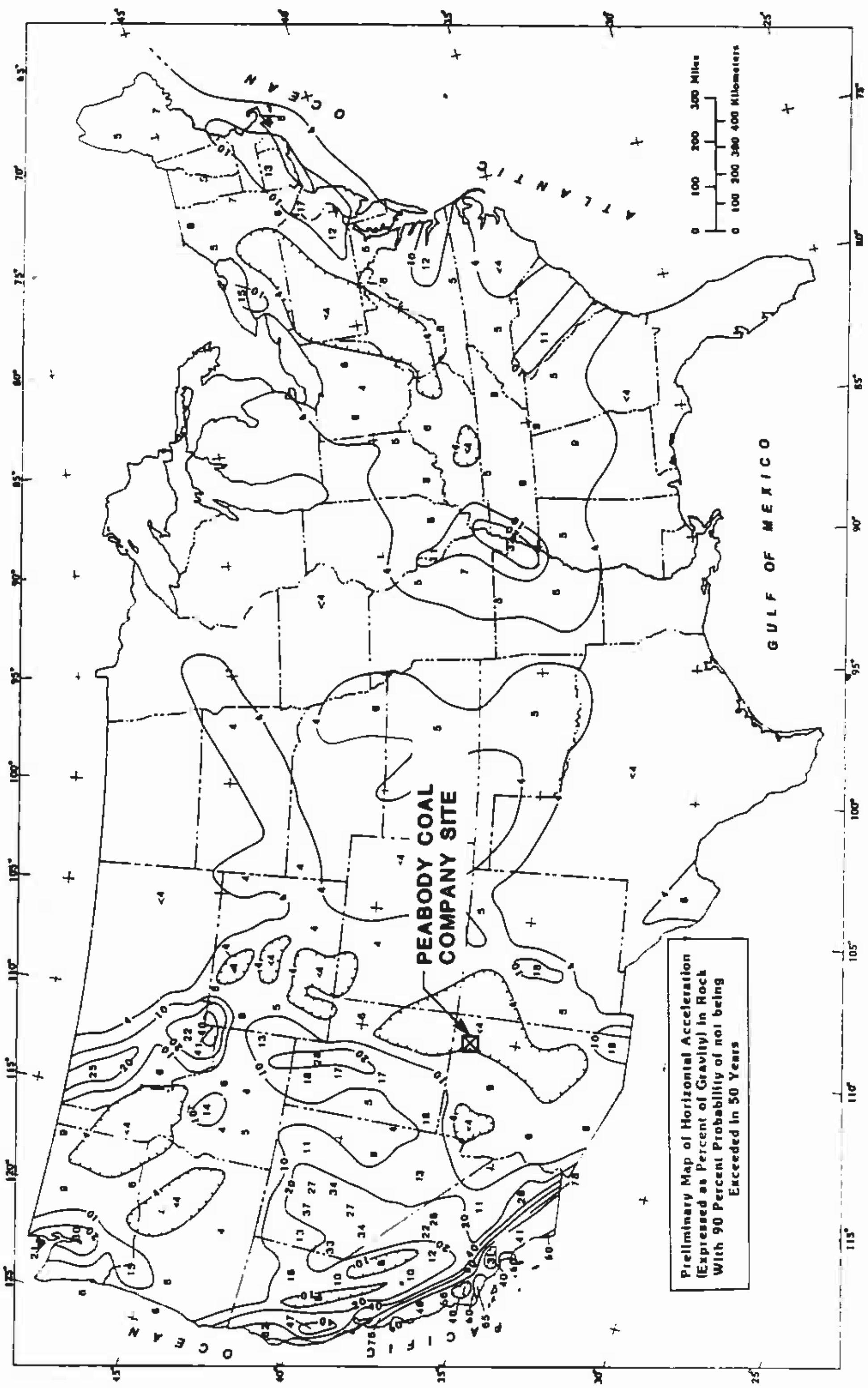
2.5.1 Historic Activity

Historic seismic activity in Arizona has been moderate in some areas to virtually nonexistent in others. Of the recorded epicenters within Arizona, very few have had a magnitude greater than 5.0 on the Richter scale. The strongest events have occurred in north-central and northwestern Arizona, in a northwest-trending zone, and in the southwest part of the state. Historic seismic activity in northeastern Arizona and the Four Corners area has been quite limited. Earthquakes of note in the area include one with a Richter magnitude of 5.75 which occurred in 1959 near

Fredonia, Arizona approximately 130 miles to the west of Kayenta and two minor earthquakes 20 to 30 miles southwest of Kayenta. No recent faults are known to occur within the site area (Dubois, 1979; Sumner, 1976).

2.5.2 Earthquake Probability

Studies by Algermissen and others (1982) indicate that the central part of Colorado Plateau has significantly less earthquake activity than at the margins. For the site area, Algermissen and others (1982) have estimated a horizontal acceleration of less than 0.04g in rock with a 90 percent probability of not being exceeded in 50 years (Figure 2-2). A horizontal acceleration of 0.04g is therefore considered appropriate for use in evaluating the stability of the sedimentation structures under earthquake loading conditions.



ALGERMISSEN AND PERKINS SEISMICITY

REFERENCE: ALGERMISSEN AND OTHERS (1962)

BY Dames & Moore

Figure 2-2

3.0 GEOTECHNICS

3.1 GENERAL

Field inspection of the sedimentation structures was conducted by a senior geotechnical engineer whose training and experience qualified him to recognize specific signs of structural instability and other hazardous conditions by visual observation. A general data sheet and visual inspection checklist was developed specifically for this evaluation and used by the engineer for each structure inspected.

One hundred and fourteen structures were inspected by Dames & Moore's engineers during this evaluation and eight structures were selected for detailed field explorations. These eight structures were selected to include the complete range of embankment material types and foundation material types represented by the 114 structures that were inspected. The remaining existing ponds consists of ponds to be reclaimed, MSHA-size structures, ponds incised which do not have embankments or ponds which already have been approved under the 30 CFR's which were designed and inspected by other engineers and which can be found in other portions of Chapter 6 (see Table 4B). The 114 structures inspected by Dames & Moore are well distributed around the permit area and are representative of the soils conditions and site conditions encountered during sedimentation pond construction. The explorations on the eight structures consisted of drilling borings into and through the embankments and recovering representative samples of the soil and rock encountered for testing (see Table 3-3). Details of the selection, explorations, and laboratory testing are described in subsequent sections of this report.

Stability analyses were then conducted to evaluate the factors of safety against slope failure of the sedimentation structures. Physical characteristics and strength parameters used in the stability analyses were derived from laboratory test data, a review of data from other reports for structures at the mine site and published literature. Stability analyses were performed using the STABL2 computer program.

3.2 INSPECTION PROCEDURES

The procedures for a typical embankment inspection began with locating available topographic plans, design files, construction records and previous inspection reports pertinent to the structure. These records were reviewed for consistency, i.e., whether elevations on the topographic maps agreed with actual design and construction grades, and whether design slopes and grades concurred with similar values in the construction records and subsequent inspection reports. Any discrepancies disclosed at this stage were discussed with Peabody Coal Company staff and surveys were initiated, if necessary, to determine the existing site topography. The details of design and construction of the structure were entered on the checklist, and copies of applicable plans were made for field checking.

With the checklist and copies of the drawings as reference, the geotechnical engineer measured the crest width, crest length, height and slope angles of the embankment to verify as-built parameters. Measurements were made with a 100-foot tape, a 6-foot folding rule and a hand-held clinometer. Additional information was sketched or noted on the drawings, including existing riprap protection, location of channels, instrumentation, observation wells, pipes and evidence of distress (including cracks, slumps and seepage).

After direct measurements were completed and entered on the checklist, the inspection continued with recording of the visual features and conditions of the structure. This portion of the inspection was subjective,

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relying on the experience and judgment of the geotechnical engineer to quantify the features and conditions without making direct measurements. The checklist served as a prompt with specific headings for features and conditions that were either present or absent. Such experience-based assessments were made of erosion, riprap size, percent coverage of vegetation, seepage rates, and characteristics of foundation and embankment materials.

3.3 MATERIAL CLASSIFICATIONS

The materials that constitute the embankment fills were classified into the following three main soil groups:

1) Residual sandstone soils (SM, SP, GP) consisting of mottled tan to reddish brown silty fine to medium sands with varying amounts of sandstone fragments.

2) Residual shale soils (SM, ML, GM) consisting of mottled light to dark brown fine sandy silts and silts with some clay and with fragments of shale.

3) Alluvial soils (SP, GP) consisting of brown very fine to coarse sands with some silt, clay and gravel.

The soil and rock materials that constitute the embankment foundations vary from bedrock to residual or alluvial soils derived from the parent bedrock. These materials were classified into the following groups:

1) Sandstone, tan to reddish brown; usually highly weathered and fractured.

2) Shale, light brown to brown to gray; usually highly weathered and flaking.

3) Residual sandstone soils.

4) Residual shale soils.

5) Alluvium, brown very fine to coarse sand with varying amounts of gravel in lenticular bands.

3.4 EMBANKMENT CATEGORIES

OSM, in agreement Peabody Coal Company, is allowing sedimentation structures to be grouped into appropriate categories to expedite the geotechnical evaluation of the structures (OSM, 1985a). We concur that this approach is sound since the majority of the structures are similar in size, design and construction.

The criteria for grouping of the sedimentation structures into categories included is based on the type of soil or rock material in the embankments and the foundations of these structures. These criteria were selected because the engineering properties of the embankment and foundation materials are the principal factors contributing to the stability of the structures. Table 3-1 lists the categories that were selected to represent the sedimentation structures.

It was noted that a number of embankments consist of a mixture of residual sandstone, residual shale and alluvial soils. For purposes of categorization, the embankment soil type was classified in accordance to the material which predominated in the embankment. A few of the structures inspected did not fall into the categories listed above. Of these, two are small concrete walls that act as dams and spillways combined, and three are internal impoundments that are made of mine spoil and control sediment runoff into a temporary landfill. These few special cases have been evaluated individually.

Table 3-1

CATEGORIES OF STRUCTURES BASED ON
EMBANKMENT AND FOUNDATION MATERIALS

Category	Embankment Soil Type	Foundation Type
A-1	Residual Sandstone Soil (SM)	Residual Sandstone Soil (SM)
A-2	Residual Sandstone Soil (SM)	Residual Shale Soil (ML)
A-3	Residual Sandstone Soil (SM)	Alluvial Soil (SP)
A-4	Residual Sandstone Soil (SM)	Shale Bedrock
A-5	Residual Sandstone Soil (SM)	Sandstone Bedrock
B-1	Residual Shale Soil (ML)	Residual Shale Soil (ML)
B-2	Residual Shale Soil (ML)	Residual Sandstone Soil (SM)
B-3	Residual Shale Soil (ML)	Alluvial Soil (SP)
B-4	Residual Shale Soil (ML)	Shale Bedrock
B-5	Residual Shale Soil (ML)	Sandstone Bedrock
C-1	Alluvial Soil (SP)	Alluvial Soil (SP)

In addition to grouping the structures based on the engineering properties of the embankment and foundation materials, structures were also categorized according to topographic setting as shown in Table 3-2.

Table 3-2

CATEGORIES OF STRUCTURES BASED ON TOPOGRAPHIC SETTING

Structure Type	Description
Cross-Valley	A single embankment that completely crosses a valley or drainage channel.
Side-hill	An embankment that lies along the side of a hill or valley, consisting of a main embankment with small sections that tie back into the hill at both ends.
Roadway	An embankment, similar to either a side-hill or cross-valley in setting, where the crest also serves as a roadway.
In-wash	An embankment that is located entirely in a wash or drainage channel and makes up at least three sides of the structure.
Incised	A sedimentation structure that has no embankment, i.e. a totally below-grade structure with the excavated material used as fill in a nearby embankment or a depression in a reclaimed area.

This topographic categorization has only a minor impact on the stability of the structure; however, some design considerations influence the overall performance of the structure. For example, the slopes of the in-wash embankments need to be riprapped on the flow side of the wash, and roadway embankments are usually wider at the crest than normal and have CMP spillways rather than open channels.

3.5 FIELD EXPLORATIONS

As mentioned previously, field explorations were conducted to investigate the embankment and foundation materials of eight sedimentation structures. The eight structures were selected to include all of the embankment material and foundation material types included on the project. Further, because of their size, setting or perceived deficiencies based on the field inspection, these eight structures were considered to be representative of the least stable of the 114 structures that were inspected. The selected embankments are listed in Table 3-3.

Table 3-3

SEDIMENTATION STRUCTURES SELECTED FOR FIELD EXPLORATION

Sedimentation Structure	Category	Embankment Material	Foundation Material
J3-E	A-1	R. Sandstone Soils	R. Sandstone Soils
J7-I	A-5	R. Sandstone Soils	Sandstone
J16-J	A-1	R. Sandstone Soils	R. Sandstone Soils
J28-C	B-1	R. Shale Soils	R. Shale Soils
N1-AC	B-1	R. Shale Soils	R. Shale Soils
N1-O	C-1	Alluvial Soils	Alluvium
N10-D	B-3	R. Shale Soils	Alluvium
N14-O	B-3	R. Shale Soils	Alluvium

The field explorations consisted of drilling borings at selected locations on the embankments of the sedimentation structures. A total of 21 borings, ranging in depth from 17 to 47 feet, were drilled with a Mobile B-61 drill rig using 6.25-inch-diameter hollow stem augers. The drill rig was operated by the Jim Winnek Drilling Company. The drilling program was directed by a Dames & Moore geotechnical engineer who logged the borings as they were drilled and assisted in obtaining samples of the soils encountered.

Subsurface materials encountered in the 21 borings included soils classified, according to the Unified Soil Classification system, as GP, SP, SM, SC, ML and CL. The cohesionless materials were generally medium dense to dense. Recovered materials showing some cohesion generally fell into one of two groups; one was a dense to very dense soil. The other was a soft to medium dense soil; however, this soil type usually contained gravel sized fragments of sandstone or shale stone.

Borings were sampled at 5-foot vertical intervals. Samples were recovered using a 2.42 inch inside diameter drive sampler of the type shown on Figure 3-1. The sampler was driven with a 140-pound hammer falling 30 inches per blow. The number of blows to drive the sampler each 6-inch interval was recorded and provided an indication of the relative density of the materials sampled. In addition to the drive samples, the cuttings from the augers were inspected and random samples of cuttings were also recovered. All samples were returned to the Dames & Moore laboratory for additional classification and testing.

This type of soil sampling device was chosen for two reasons: first, we have successfully recovered samples with this sampler on other projects with similar and worse soils conditions. Secondly, with the interbedded granular materials, a core retaining device is necessary to keep the soil sample in the sampler during recovery. As shown in Figure 3-1, the modified Sprague & Henwood sampler can be filled with a thin-wall tube; however, this sampling method is primarily for soft, cohesive soils. Due to the dense, granular and non-uniform nature of the encountered soils, pushing a thin-wall or "Shelby" tube sampler was not considered feasible.

The sampler shown in Figure 3-1 is similar to the Dames & Moore sampler. The difference is that the Dames & Moore sampler has a 3.25-inch outside diameter; whereas the Modified Sprague & Henwood's outside diameter is 3.0 inches. The internal components of the two samplers are interchangeable. This sampler is a sophisticated piece of sampling equipment which has been refined by 30 years of use and improvements. This sampler has been used to obtain relatively undisturbed samples on numerous water and tailings dams throughout North America, and our experience has provided a high level of confidence in the representative nature of the samples obtained with this equipment. The sampler's relatively large diameter provides a sample with a lesser percentage of disturbed material than with smaller samplers. An evaluation of the sample disturbance for the Modified Sprague & Henwood sampler, solely on the basis of inside to outside diameter ratio, is deceptive because of the sharp-edged and gently-tapered cutting bit used on the sampler.

Typically the effects of sample disturbance are minimized for all laboratory strength testing by re-consolidating the test samples under confining pressures which simulate the in situ pressures in the embankment prior to testing. Sample disturbance is a problem which is common, in varying degrees, to all sampling equipment and procedures. The procedure described above minimizes disturbance effects.

The sampler shown in Figure 3-1 has been used to investigate the soil conditions in literally hundreds of dams in North America over the past 30 years. The recovered soil samples were then tested to develop shear strength values for use in the dam design. Final dam designs have then been subject to approval by county, state and Federal agencies including U.S. Soil Conservation Service, U.S. Department of Energy, U.S. Environmental Protection Agency and U.S. Nuclear Regulatory Commission. For the Sebastian Martin-Black Mesa flood control dam project in New Mexico, conducted for the U.S. Soil Conservation Service, we were requested to drill and sample the foundation soil using the Modified Sprague & Henwood sampler, a 6-inch diameter Pitcher sampler, and a 3-foot diameter bucket auger. The borings for each of the three sampling methods were drilled in adjacent locations and comparisons were made of the recovered samples. Results of this project re-affirmed our confidence in the quality of recovered sample using the Modified Sprague & Henwood drive sampler.

The locations of the borings are shown on Figures 3-2A through 3-2H, and the Log of Borings are presented on Figures 3-3A through 3-3U. The materials are identified on the basis of the Unified Soil Classification System presented on Figure 3-4A. The Key to Log of Borings is presented on Figure 3-4B.

3.6 LABORATORY TESTING

Selected samples of the soils encountered were tested in our laboratory to aid in identification and classification and to determine the engineering properties. Testing was completed to evaluate moisture content, dry density, grain size distribution, soil plasticity, specific gravity, consolidation and shear strength. The laboratory testing data are presented on Figures 3-5A through 3-5D, 3-7A and 3-7B, 3-10 and on the Log of Borings, Figures 3-3A through 3-3U.

3.6.1 Moisture Content and Dry Density

The moisture content and dry density of recovered samples were determined as an aid to classification of the soils and estimation of engineering properties. The moisture content was determined in accordance with ASTM D 2216 test procedures. The results of the moisture content and density determinations are presented on the Logs of Borings, Figures 3-3A through U.

3.6.2 Grain Size Distribution

The particle size distribution of representative samples was determined by passing a specimen of soil through a nested set of standard sieves. The test was completed in accordance with ASTM D 422 procedures. The test results are presented on Figures 3-5A through D.

3.6.3 Atterberg Limits

As an aid to classifying the soils, the liquid and plastic limits of representative samples were determined in accordance with ASTM D 4318 procedures. The results of the plastic and liquid limit determinations are presented on the Logs of Borings, Figures 3-3A through U.

3.6.4 Specific Gravity

The specific gravity of selected soil samples was determined to provide information for the engineering analyses. The specific gravity was determined in accordance with ASTM C 854 procedures. Results of specific gravity testing are presented on Table 3-4.

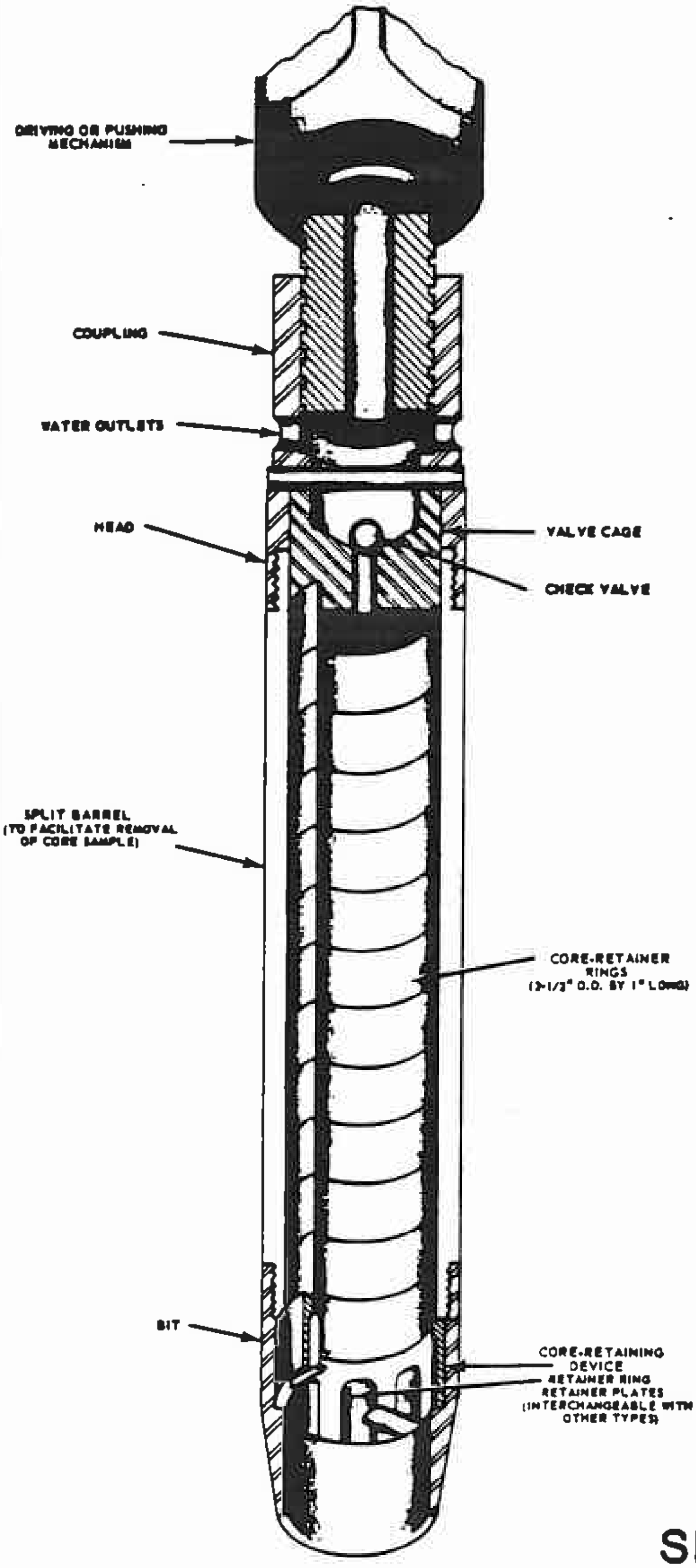
Table 3-4

RESULTS OF SPECIFIC GRAVITY TESTING

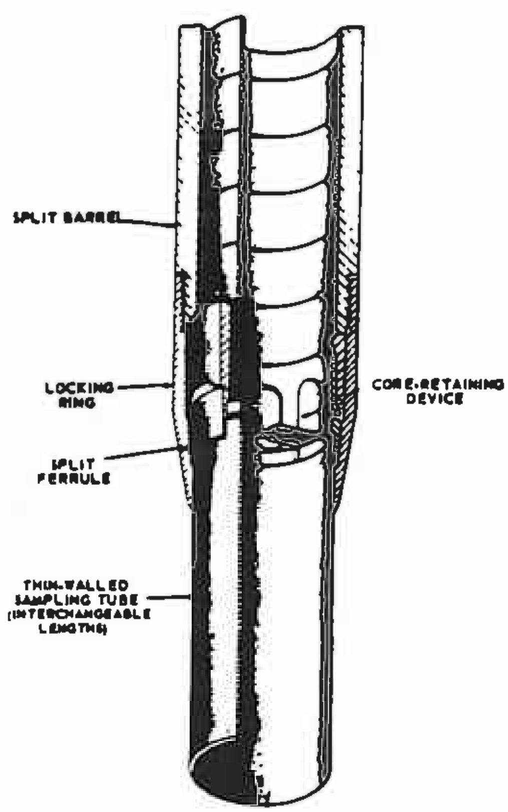
Sedimentation Structure	Sample Depth (ft)	Soil Type	Specific Gravity
J3-E1	8	SM-ML	2.58
J7-I2	8	SM	2.59
N1-O1	5.5	ML	2.54
N10-C	Surface	SP	2.64
N10-D1	23	SP	2.62
N10-E	Surface	SP	2.56

3.6.5 Direct Shear Tests

Direct shear tests were performed to evaluate the shear strength of representative samples of alluvial soils. Samples were loaded vertically (normal to the ends of the sample) and the shearing force was applied horizontally in the form of a constant rate of deflection. The test results are presented on Figures 3-3A through U, and the method of completing the tests is described on Figure 3-6.



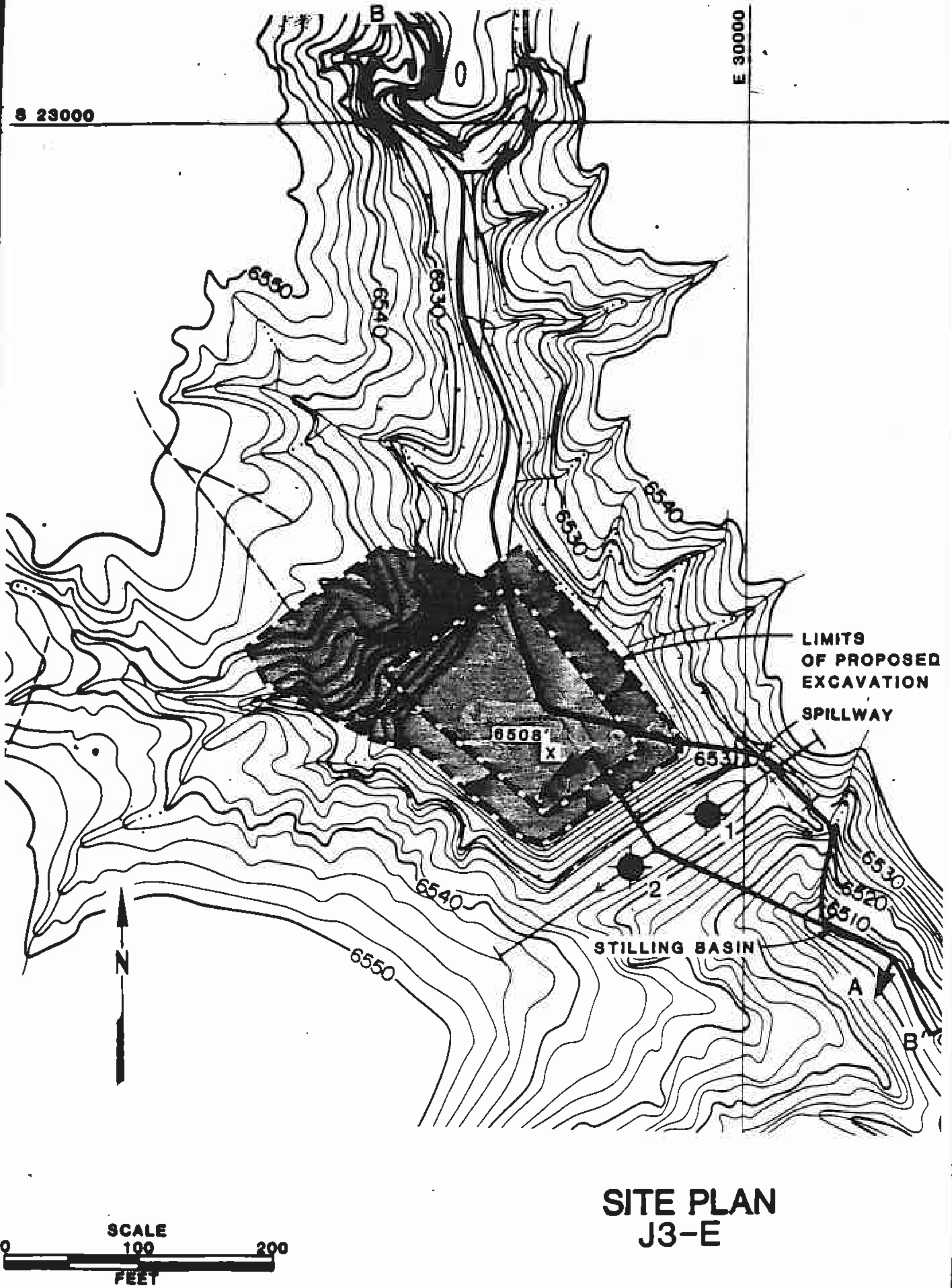
ALTERNATE ATTACHMENTS



MODIFIED SPRAGUE & HENWOOD SAMPLER

BY Dames & Moore

Figure 3-1

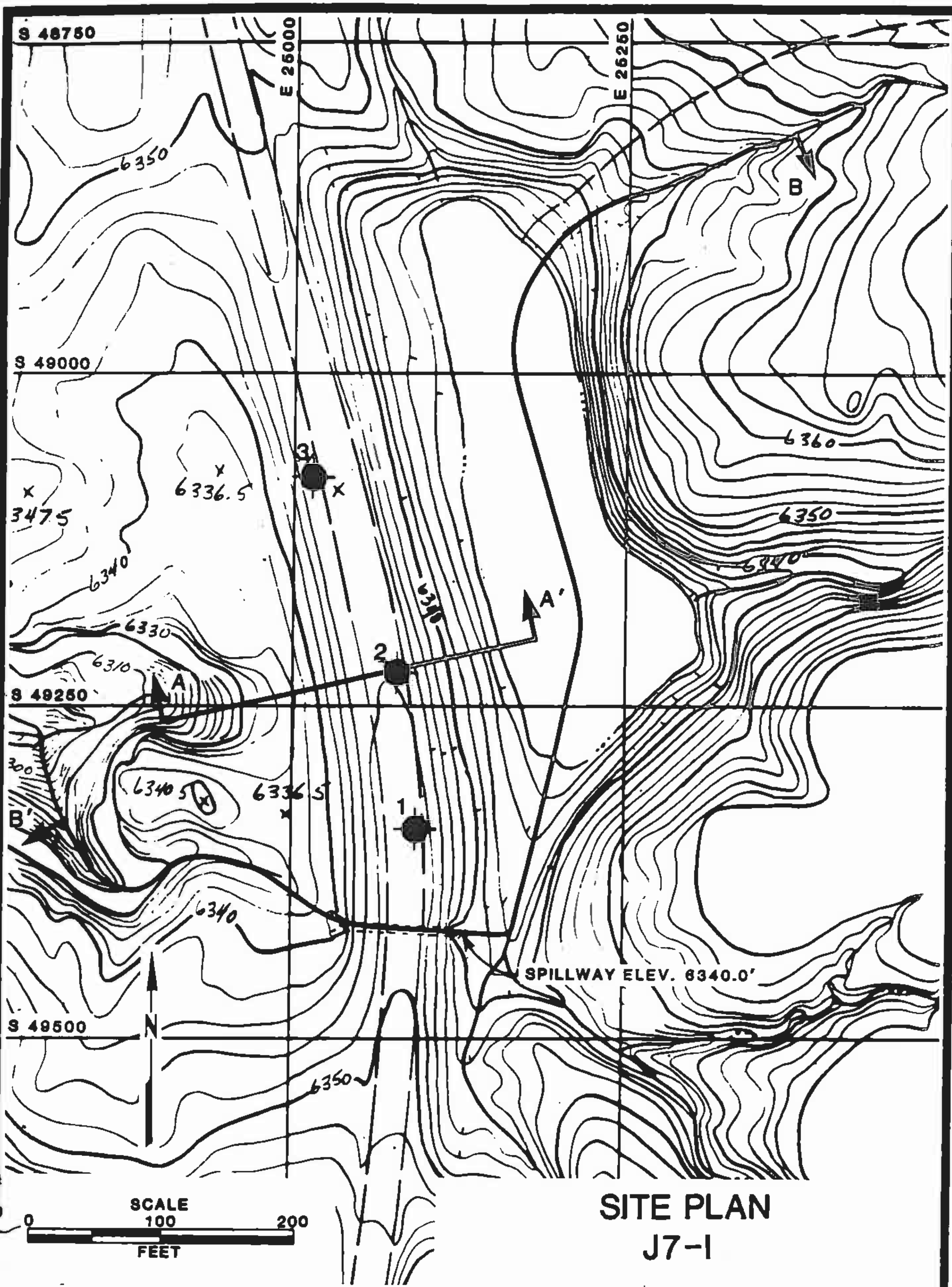


LOCATION OF DAMES & MOORE BORING

SITE PLAN
J3-E

BY Dames & Moore

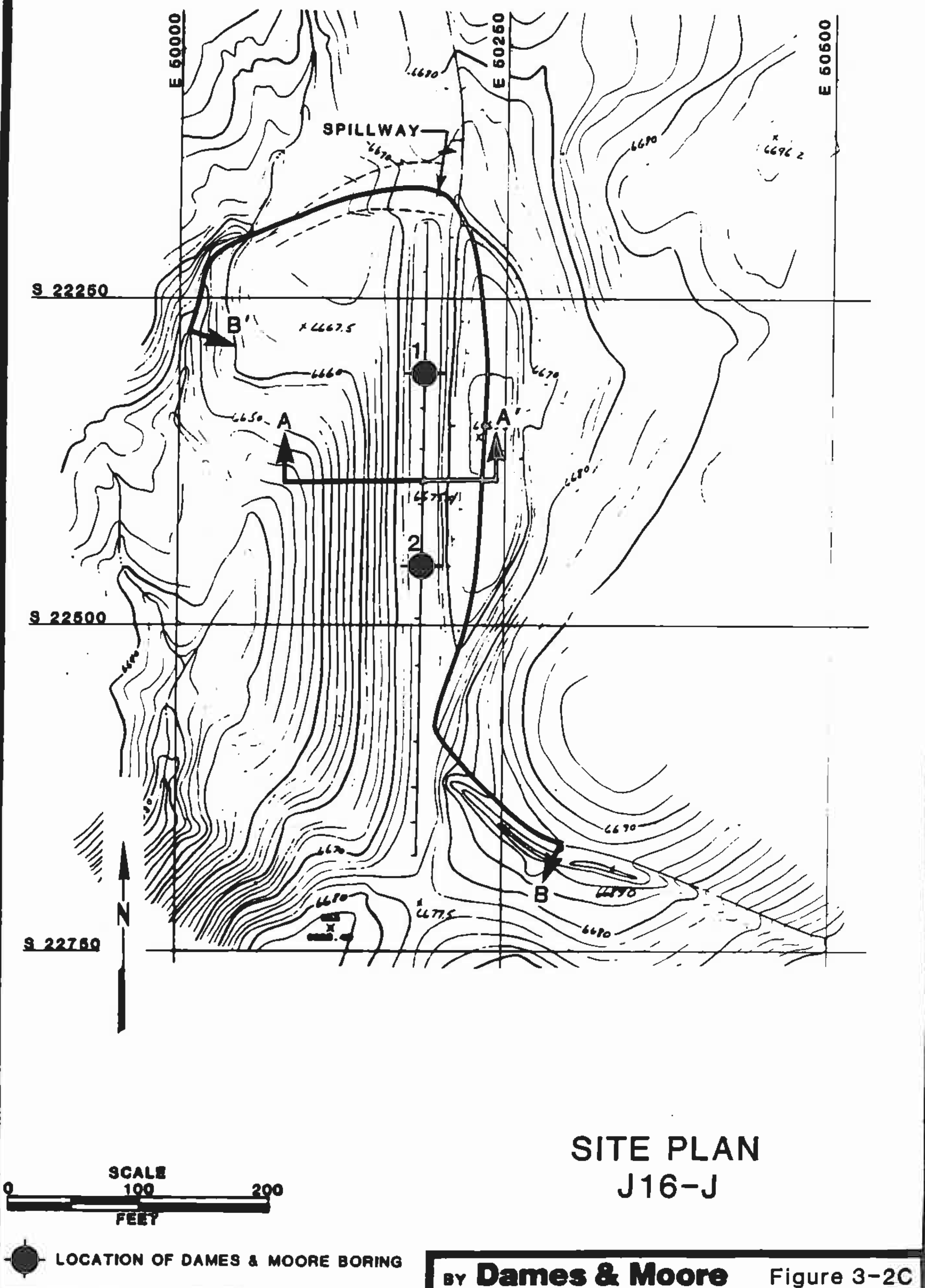
Figure 3-2A



SITE PLAN
J7-I

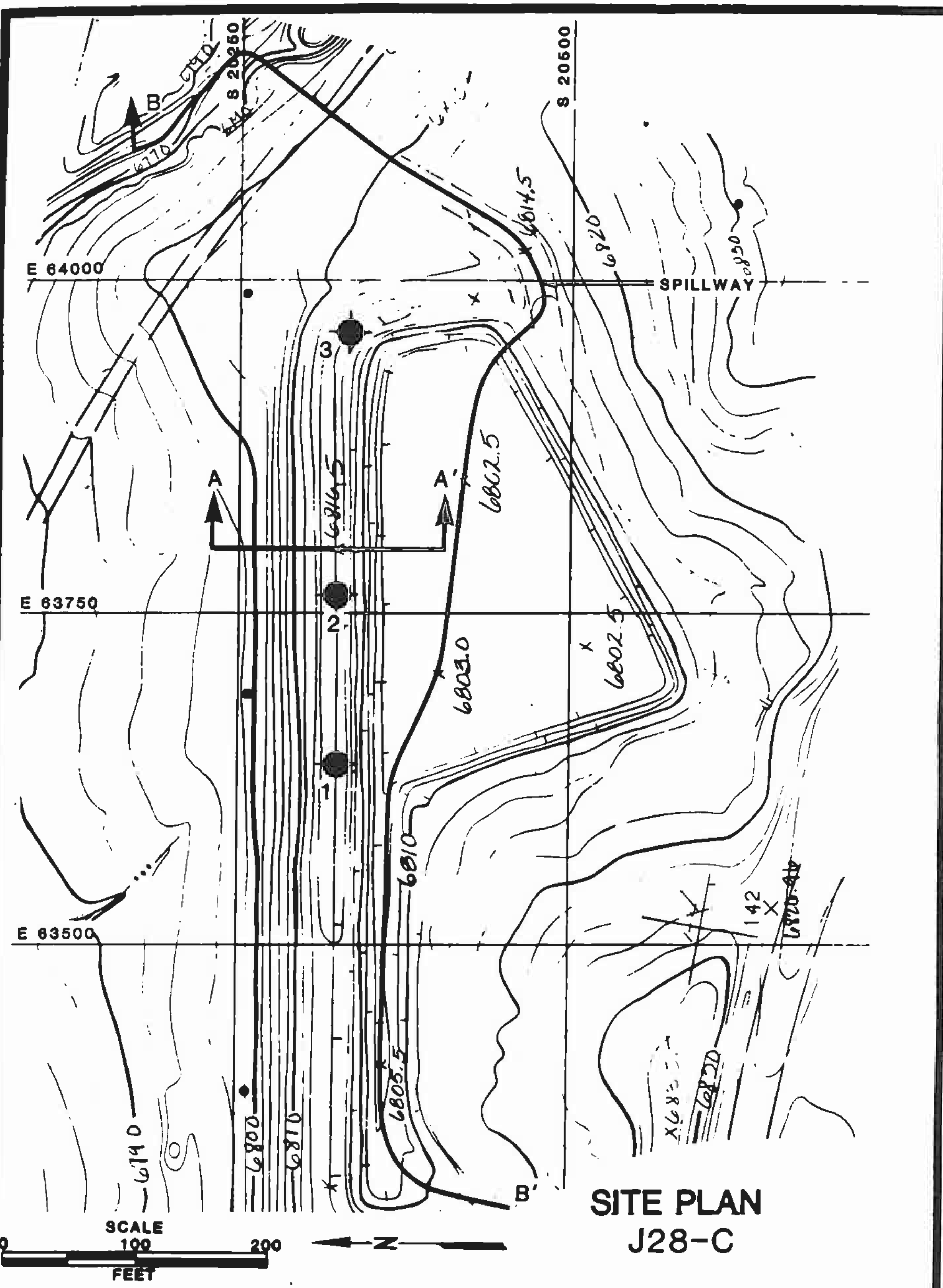
LOCATION OF DAMES & MOORE BORING

BY **Dames & Moore** Figure 3-2B



SITE PLAN
J16-J

BY **Dames & Moore** Figure 3-2C

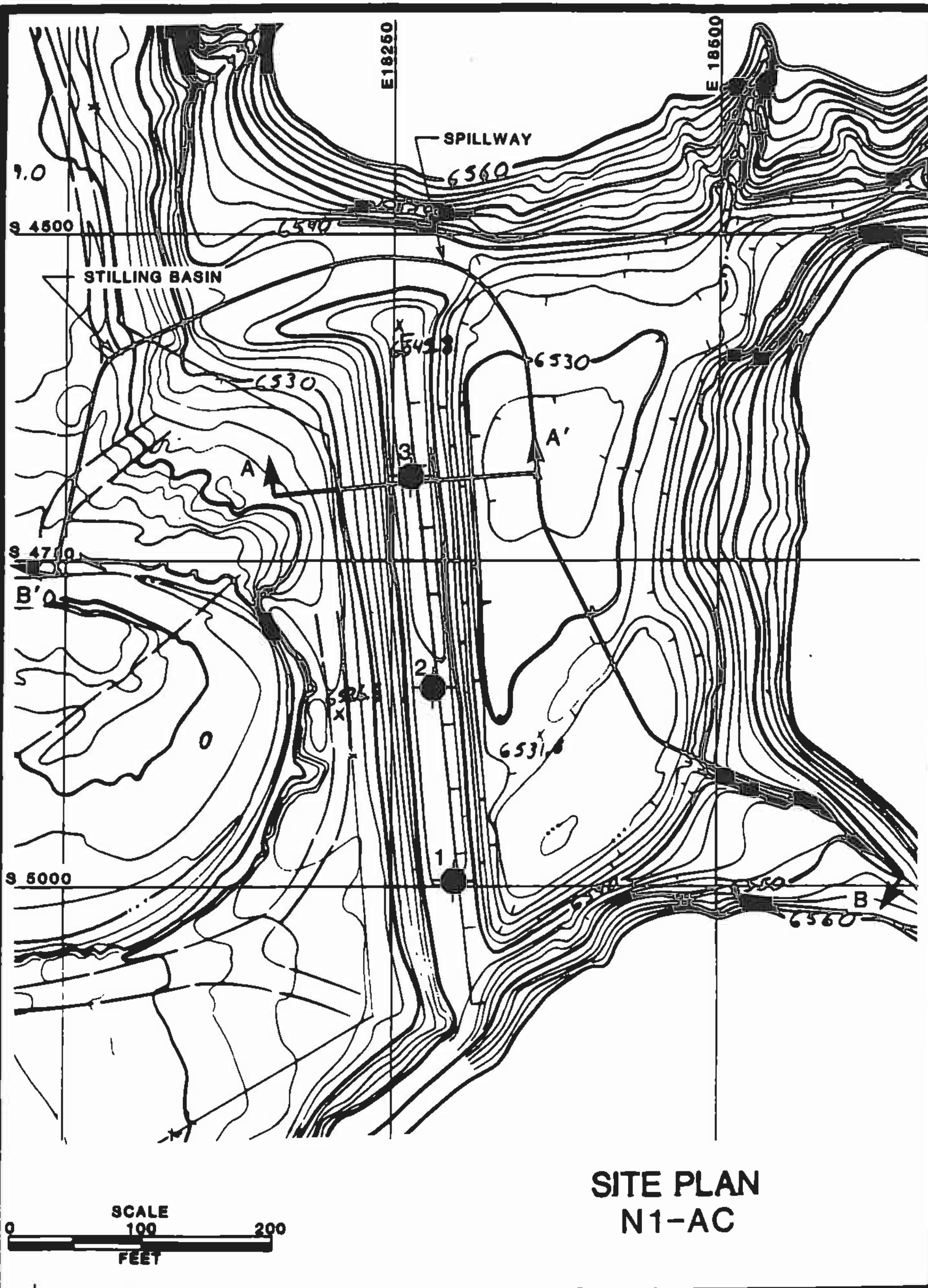


SITE PLAN
J28-C

LOCATION OF DAMES & MOORE BORING

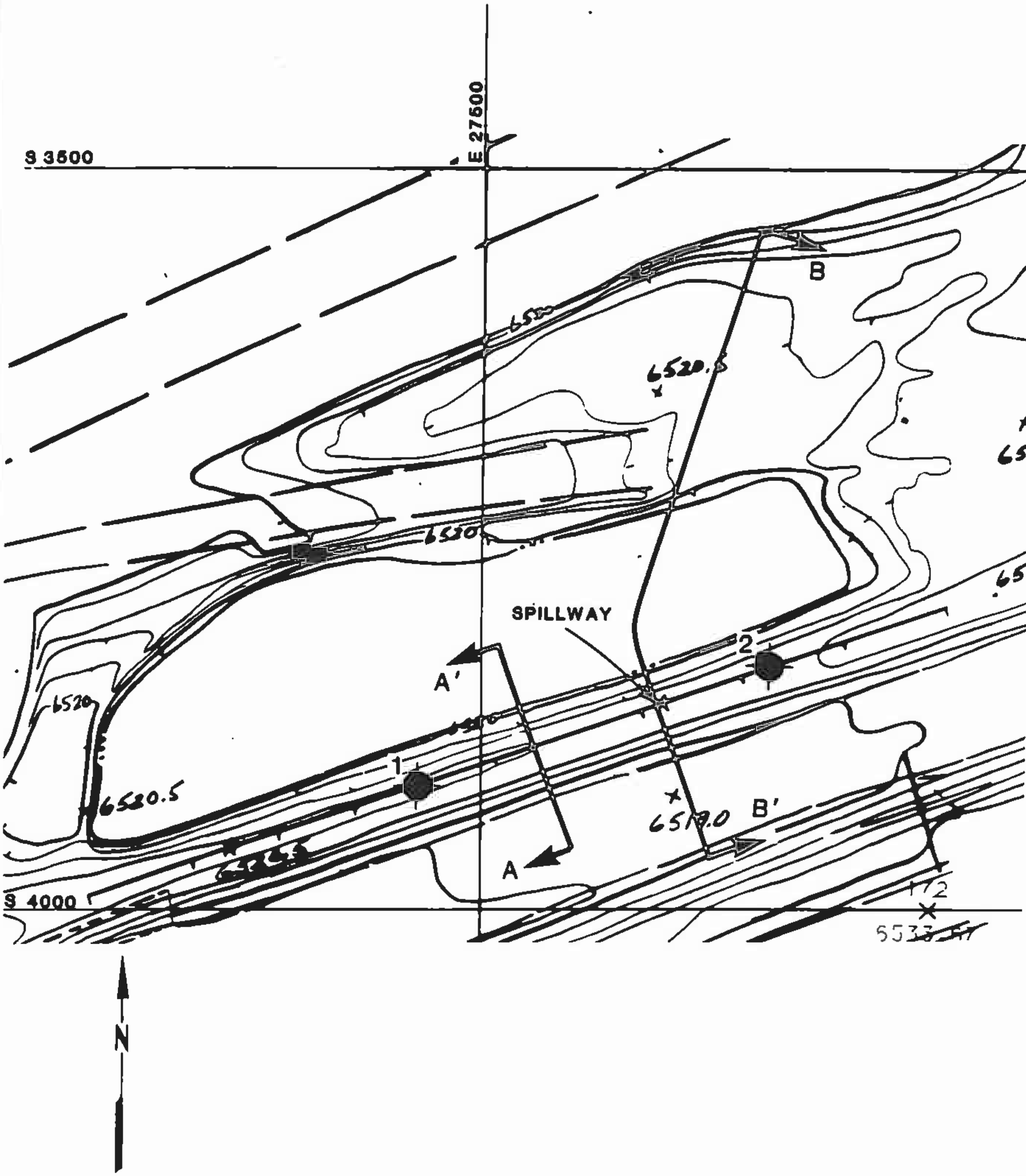
BY **Dames & Moore**

Figure 3-2D



SITE PLAN
N1-AC

BY **Dames & Moore** Figure 3-2E

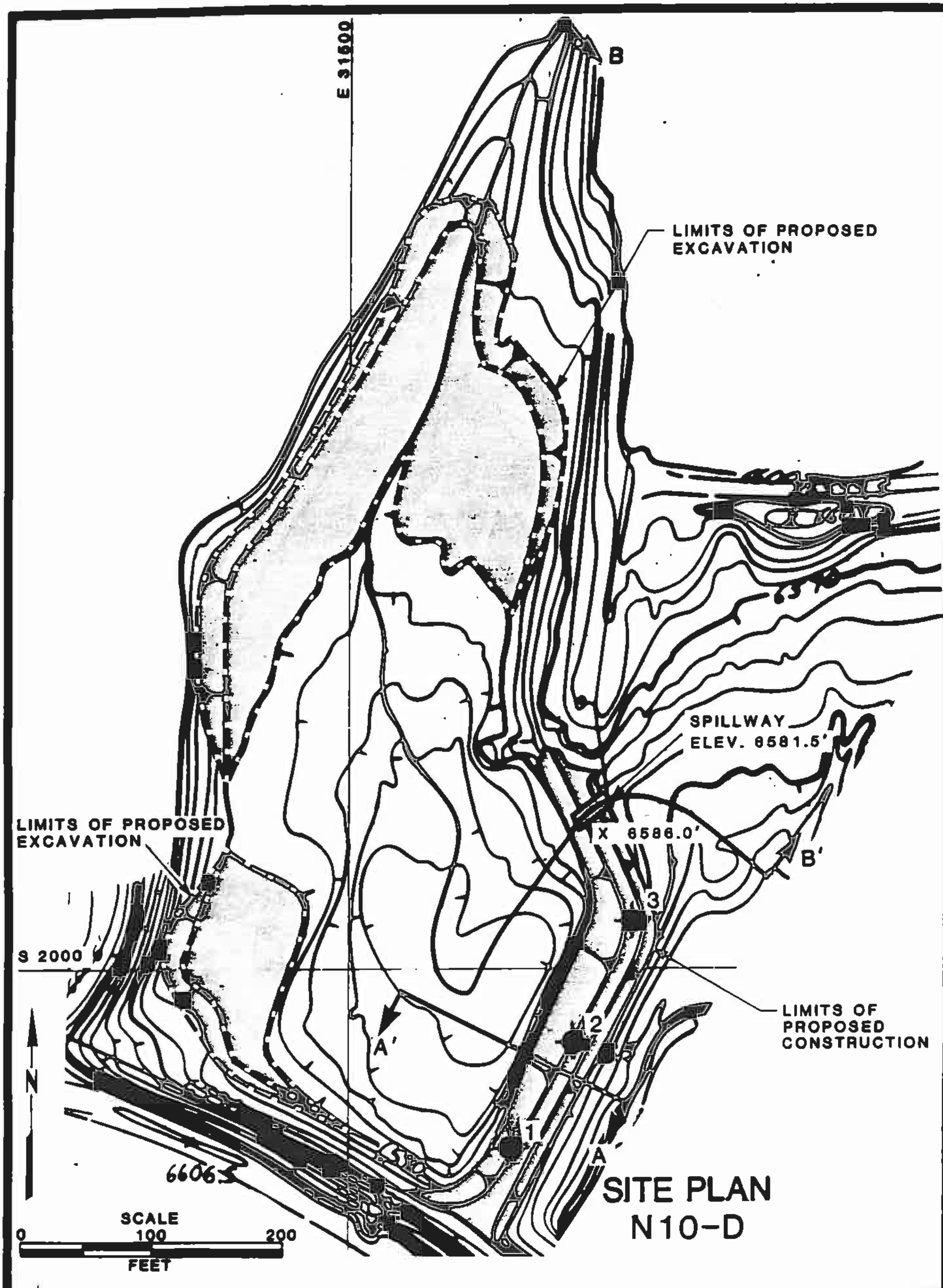


SCALE
0 100 200
FEET

LOCATION OF DAMES & MOORE BORING

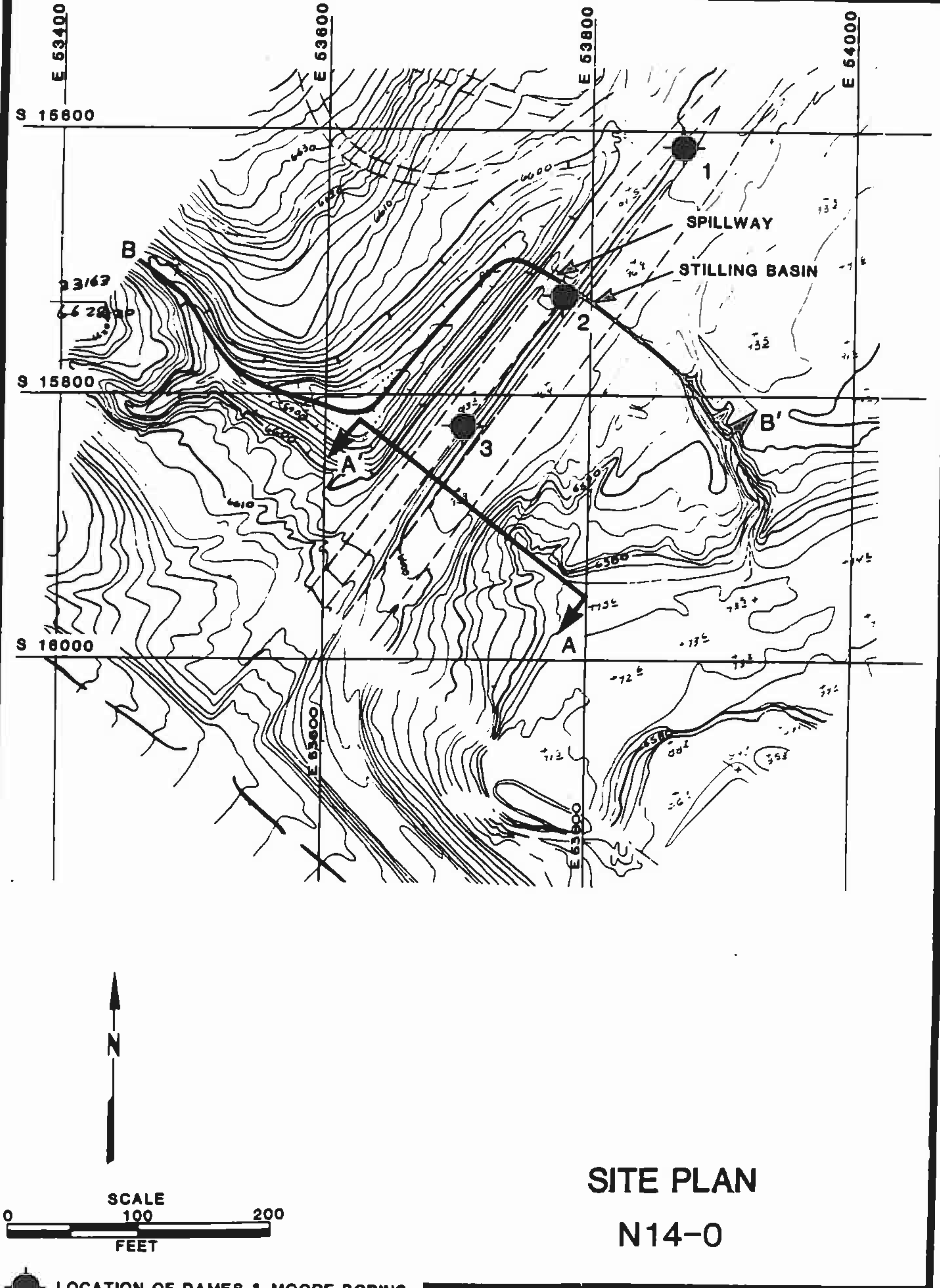
SITE PLAN
N1-O

BY **Dames & Moore** Figure 3-2F



SITE PLAN
N10-D

BY Dames & Moore Figure 3-2G



SITE PLAN

N14-0

BY **Dames & Moore** Figure 3-2H

BORING J3-E1

SURFACE ELEVATION: 6536.3 FEET
PCC COORDINATES
S 23517
E 29967

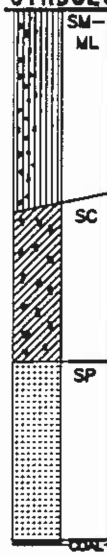
LABORATORY TEST DATA

DEPTH (FT)	TESTS REPORTED ELSEWHERE	ATTERBERG LIMITS: LIQUID LIMIT (%)	ATTERBERG LIMITS: PLASTICITY INDEX (%)	STRENGTH TEST DATA: TYPE OF TEST	STRENGTH TEST DATA: NORMAL OR CONFINING PRESSURE (PSF)	STRENGTH TEST DATA: SHEAR STRENGTH (PSF)	STRENGTH TEST DATA: DEVIATOR STRESS (PSF)	MOISTURE CONTENT (%)	DRY DENSITY (PCF)
0–5								10.4	115.3
5–10	G								
10–15	MA	22.9	7.3					13.6	115.8

Depth scale: 0, 5, 10, 15, 20, 25, 30, 35, 40, 45, 50, 55, 60, 65, 70, 75, 80

BLOWS/FT.	SAMPLES
75	▯
52	▯
21	▯
80	▯
21	▯

SYMBOLS - DESCRIPTION



SM-ML MOTTLED BROWN AND GRAY SILTY FINE SAND, CLAYEY AND SANDY SILT WITH FRAGMENTS OF SANDSTONE (DENSE)

MEDIUM DENSE

SC CLAYEY FINE SAND WITH FRAGMENTS OF SANDSTONE

LOOSE

FILL

SP SANDSTONE BOULDER (DENSE)

BROWN FINE SAND WITH GRAVEL AND FRAGMENTS OF SANDSTONE

BORING TERMINATED AT 27.5 FEET ON 9-30-85.
NO GROUNDWATER ENCOUNTERED.

LOG OF BORING

BY **Dames & Moore** Figure 3-3A

BORING J3-E2

SURFACE ELEVATION: 6535.9 FEET
PCC COORDINATES
S 23548
E 29918

LABORATORY TEST DATA

Depth (ft)	Tests Reported Elsewhere	Atterberg Limits: Liquid Limit (%)	Atterberg Limits: Plasticity Index (%)	Strength Test Data: Type of Test	Strength Test Data: Normal or Confining Pressure (PSF)	Strength Test Data: Shear Strength (PSF)	Strength Test Data: Deviator Stress (PSF)	Moisture Content (%)	Dry Density (PCF)	Blows/Ft.
0										
5										70
10								11.6	118.5	92
15								11.8	114.7	53
20								17.3	111.9	50/4"
25										
30										
35										
40										
45										
50										
55										
60										
65										
70										
75										
80										

SAMPLES

SYMBOLS	DESCRIPTION
SM-ML	MOTTLED BROWN AND GRAY SILTY FINE SAND, CLAYEY AND SANDY SILT WITH FRAGMENTS OF SANDSTONE (DENSE)
	DENSE
	MEDIUM DENSE
SM	BROWN SILTY FINE SAND WITH FRAGMENTS OF SANDSTONE (DENSE)
SHALE	

FILL

BORING TERMINATED AT 22.5 FEET ON 9/30/85.
NO GROUNDWATER ENCOUNTERED.

LOG OF BORING

BY **Dames & Moore** Figure 3-3B

BORING J7-I1

SURFACE ELEVATION: 8348.4 FEET
PCC COORDINATES
S 49344
E 25096

LABORATORY TEST DATA

Depth	TESTS REPORTED ELSEWHERE	ATTERBERG LIMITS: LIQUID LIMIT (%)	ATTERBERG LIMITS: PLASTICITY INDEX (%)	STRENGTH TEST DATA: TYPE OF TEST	STRENGTH TEST DATA: NORMAL OR CONFINING PRESSURE (PSF)	STRENGTH TEST DATA: SHEAR STRENGTH (PSF)	STRENGTH TEST DATA: DEVIATOR STRESS (PSF)	MOISTURE CONTENT (%)	DRY DENSITY (PCF)
0									
5									
10									
15									
20									
25									
30									
35									
40									
45									
50									
55									
60									
65									
70									
75									
80									

BLOWS/FT.	SYMBOLS	DESCRIPTION
	SM	MOTTLED GRAY AND BROWN SILTY FINE SAND WITH SANDSTONE FRAGMENTS (VERY DENSE) — FILL
54/6"		
	SHALE	MOTTLED OLIVE BROWN WEATHERED SHALE (VERY DENSE)
75/8"	COAL	
	SS	LIGHT GRAY SANDSTONE (VERY DENSE)
50/2"		

BORING TERMINATED AT 17.5 FEET ON 9/25/85.
NO GROUNDWATER ENCOUNTERED.

LOG OF BORING

BY Dames & Moore

Figure 3-3C

BORING J7-12

SURFACE ELEVATION: 8347.3 FEET
PCC COORDINATES
S 49228
E 25080

LABORATORY TEST DATA

Depth (ft)	Tests Reported Elsewhere	Atterberg Limits: Liquid Limit (%)	Atterberg Limits: Plasticity Index (%)	Strength Test Data: Type of Test	Strength Test Data: Normal or Confining Pressure (PSF)	Strength Test Data: Shear Strength (PSF)	Strength Test Data: Deviator Stress (PSF)	Moisture Content (%)	Dry Density (PCF)	Blows/Ft.	Samples
0–5		26.3	9.4					14.1	113.3	80	▯
5–10	G							12.1	97.9	49	▯
10–15								17.5	111.1	52	▯
15–20											
20–25											
25–30											
30–35											
35–40											
40–45											
45–50											
50–55											
55–60											
60–65											
65–70											
70–75											
75–80											

Symbols	Description
ML	MOTTLED DARK BROWN SANDY SILT WITH FRAGMENTS OF SANDSTONE, TRACE CLAY (MEDIUM DENSE)
SM	BROWN SILTY SAND WITH FRAGMENTS OF SANDSTONE (MEDIUM DENSE)
SS	LIGHT GRAY SANDSTONE (VERY DENSE)

BORING TERMINATED AT 17.6 FEET ON 9/25/85.
NO GROUNDWATER ENCOUNTERED.

LOG OF BORING

BY **Dames & Moore** Figure 3-3D

BORING J7-13

SURFACE ELEVATION: 8348.7 FEET
PCC COORDINATES
S 49078
E 25014

LABORATORY TEST DATA

Depth	Tests Reported Elsewhere	Atterberg Limits: Liquid Limit (%)	Atterberg Limits: Plasticity Index (%)	Strength Test Data: Type of Test	Strength Test Data: Normal or Confining Pressure (PSF)	Strength Test Data: Shear Strength (PSF)	Strength Test Data: Deviator Stress (PSF)	Moisture Content (%)	Dry Density (PCF)	Blows/Ft.	Symbols	Description
0											SM	DARK BROWN SILTY FINE TO COARSE SAND WITH FRAGMENTS OF BLACK SANDSTONE, TRACE CLAY (VERY DENSE) (FILL)
5										50/5"		
10										103/9"		COLOR CHANGES TO MOTTLED BROWN (VERY DENSE)
15								12.5	103.7	50/5"		VERY DENSE
										50/1"	SS	LIGHT GRAY SANDSTONE
20												
25												
30												
35												
40												
45												
50												
55												
60												
65												
70												
75												
80												

BORING TERMINATED AT 17.2 FEET ON 9/25/85.
NO GROUNDWATER ENCOUNTERED.

LOG OF BORING

BY **Dames & Moore** Figure 3-3E

BORING J16-J1

SURFACE ELEVATION: 8875.9 FEET
PCC COORDINATES
S 22309
E 50189

LABORATORY TEST DATA

Depth (ft)	Tests reported elsewhere	Atterberg limits: Liquid limit (%)	Atterberg limits: Plasticity index (%)	Strength test data: Type of test	Strength test data: Normal or confining pressure (PSF)	Strength test data: Shear strength (PSF)	Strength test data: Deviator stress (PSF)	Moisture content (%)	Dry density (PCF)	Blows/ft.	Samples
0–5								10.7	122.2	83/ 11"	■
5–10								7.1	102.2	62	■
10–15										95/ 11"	■
15–20											
20–25											
25–30											
30–35											
35–40											
40–45											
45–50											
50–55											
55–60											
60–65											
65–70											
70–75											
75–80											

SYMBOLS / DESCRIPTION

SM — MOTTLED GRAY AND OLIVE BROWN SILTY FINE SAND (DENSE)

BROWN SILTY FINE SAND (MEDIUM DENSE)

ML — BROWN FINE SANDY SILT WITH TRACE OF CLAY AND SHALE FRAGMENTS

SHALE — BROWN SHALE (DENSE)

COAL

FILL

BORING TERMINATED AT 17.5 FEET ON 9/27/85.
NO GROUNDWATER ENCOUNTERED.

LOG OF BORING

BY **Dames & Moore** Figure 3-3F

BORING J16-J2

SURFACE ELEVATION: 6681.9 FEET
PCC COORDINATES
S 22458
E 50181

LABORATORY TEST DATA

Depth (ft)	Tests Reported Elsewhere	Atterberg Limits: Liquid Limit (%)	Atterberg Limits: Plasticity Index (%)	Strength Test Data: Type of Test	Strength Test Data: Normal or Confining Pressure (PSF)	Strength Test Data: Shear Strength (PSF)	Strength Test Data: Deviator Stress (PSF)	Moisture Content (%)	Dry Density (PCF)	Blows/Ft.
0										
5								11.7	117.1	65
10		29.1	3.3					11.4	104.5	77/ 6"
										98
15										
								21.9	94.3	50
20										
25										
30										
35										
40										
45										
50										
55										
60										
65										
70										
75										
80										

SAMPLES

SYMBOLS	DESCRIPTION
SM	BROWN SILTY FINE SAND (DENSE)
ML	BROWN FINE SANDY SILT
SM	MOTTLED BLACK, BROWN SILTY SAND WITH FRAGMENTS OF SHALE (DENSE)
SHALE	GRAY SHALE
COAL	COAL (MEDIUM DENSE)

FILL

BORING TERMINATED AT 18.0 FEET ON 9/27/85.
NO GROUNDWATER ENCOUNTERED.

LOG OF BORING

BY Dames & Moore Figure 3-3G

BORING J28-C1

SURFACE ELEVATION: 6816.0 FEET
PCC COORDINATES
S 20321
E 63636

LABORATORY TEST DATA

Depth (ft)	Tests Reported Elsewhere	Atterberg Limits: Liquid Limit (%)	Atterberg Limits: Plasticity Index (%)	Strength Test Data: Type of Test	Strength Test Data: Normal or Confining Pressure (PSF)	Strength Test Data: Shear Strength (PSF)	Strength Test Data: Deviator Stress (PSF)	Moisture Content (%)	Dry Density (PCF)	Blows/Ft.
0		27.1	9.4					7.9	99.5	64
5										29
10	MA			TXCUPP				15.2	110.7	31
15										27
20								132.9	34.7	72
25										
30										
35										
40										
45										
50										
55										
60										
65										
70										
75										
80										

BLOWS/FT. SAMPLES SYMBOLS DESCRIPTION

ML MOTTLED TAN AND BROWN FINE SANDY SILT WITH TRACE OF CLAY (MEDIUM DENSE)

MEDIUM DENSE

MEDIUM DENSE

MEDIUM DENSE

FILL

COAL

ML GRAY CLAYEY SILT (DENSE)

SHALE MOTTLED BROWN SHALE

BORING TERMINATED AT 24.0' ON 9/18/85.
NO GROUNDWATER ENCOUNTERED.

LOG OF BORING

BY **Dames & Moore** Figure 3-3H

BORING J28-C2

SURFACE ELEVATION: 6815.0 FEET
PCC COORDINATES
S 20323
E 63763

LABORATORY TEST DATA

Depth (ft)	Tests Reported Elsewhere	Atterberg Limits: Liquid Limit (%)	Atterberg Limits: Plasticity Index (%)	Strength Test Data: Type of Test	Strength Test Data: Normal or Confining Pressure (PSF)	Strength Test Data: Shear Strength (PSF)	Strength Test Data: Deviator Stress (PSF)	Moisture Content (%)	Dry Density (PCF)	Blows/Ft.
0										
5										44
10				TXCUPP				16.5	108.4	25
15				DS-UU	1500	1520		16.7	193.0	55
20		27.4	10.9	TXCUPP				18.2	107.4	28
25		36.3	16.1					18.3	112.2	129
30										
35										
40										
45										
50										
55										
60										
65										
70										
75										
80										

SAMPLES

SYMBOLS	DESCRIPTION
ML	LIGHT BROWN FINE SANDY SILT WITH TRACE OF CLAY (MEDIUM DENSE)
	COLOR CHANGES TO MOTTLED BROWN (MEDIUM DENSE)
CL	DARK BROWN SILTY CLAY (MEDIUM DENSE)
	MEDIUM DENSE
	COLOR CHANGES TO MOTTLED OLIVE BROWN
SHALE	BROWN SHALE FRAGMENTS, BLACK COAL AND GRAY SHALE (VERY DENSE)

BORING TERMINATED AT 27.5 FEET ON 9/19/85.
NO GROUNDWATER ENCOUNTERED.

LOG OF BORING

BY **Dames & Moore** Figure 3-31

BORING J28-C3

SURFACE ELEVATION: 8810.0 FEET
PCC COORDINATES
S 20332
E 83982

LABORATORY TEST DATA

Depth (ft)	Tests Reported Elsewhere	Atterberg Limits: Liquid Limit (%)	Atterberg Limits: Plasticity Index (%)	Strength Test Data: Type of Test	Strength Test Data: Normal or Confining Pressure (PSF)	Strength Test Data: Shear Strength (PSF)	Strength Test Data: Deviator Stress (PSF)	Moisture Content (%)	Dry Density (PCF)	Blows/Ft.
4								8.1	107.9	63
9				DS-UU	1000	2920		14.9	117.4	62
14		30.5	13.3					16.4	102.1	11
19								10.9	117.7	100/5"

Depth scale: 0, 5, 10, 15, 20, 25, 30, 35, 40, 45, 50, 55, 60, 65, 70, 75, 80

Symbols	Description
ML	MOTTLED OLIVE BROWN FINE SANDY SILT WITH SOME CLAY (DENSE)
	COLOR CHANGES TO DARK BROWN (DENSE)
SC	LIGHT BROWN CLAYEY FINE SAND (LOOSE)
SM	MOTTLED BROWN FINE SAND WITH TRACE OF CLAY (VERY DENSE)
SS	BROWN SANDSTONE

FILL

BORING TERMINATED AT 22.5 FEET ON 9/19/85.
NO GROUNDWATER ENCOUNTERED.

LOG OF BORING

BY Dames & Moore

Figure 3-3J

BORING N1-AC1

SURFACE ELEVATION: 6541.5 FEET
PCC COORDINATES
S 4994
E 15298

LABORATORY TEST DATA

Depth (ft)	Tests Reported Elsewhere	Atterberg Limits: Liquid Limit (%)	Atterberg Limits: Plasticity Index (%)	Strength Test Data: Type of Test	Strength Test Data: Normal or Confining Pressure (PSF)	Strength Test Data: Shear Strength (PSF)	Strength Test Data: Deviator Stress (PSF)	Moisture Content (%)	Dry Density (PCF)	Blows/Ft.	Samples	Symbols	Description
0												SM	MOTTLED BROWN SILTY FINE SAND WITH FRAGMENTS OF SANDSTONE (MEDIUM DENSE) — FILL
5								9.3	118.3	54	▮		
10		26.5	5.4							61	▮		GRADES WITH SOME CLAY (DENSE)
15								6.1	97.4	15	▮		GRADES WITH LESS CLAY (LOOSE)
20										40	▮		
25		24.0	7.7					10.6	97.7	37	▮		GRADES WITH SOME CLAY AND WHITE STREAKS (MEDIUM DENSE)
30													
35													
40													
45													
50													
55													
60													
65													
70													
75													
80													

BORING TERMINATED AT 28.5 FEET ON 9/24/85.
NO GROUNDWATER ENCOUNTERED.

LOG OF BORING

BY **Dames & Moore**

Figure 3-3K

BORING N1-AC2

SURFACE ELEVATION: 8543.3 FEET
PCC COORDINATES
S 4849
E 15281

Depth (ft)	LABORATORY TEST DATA: Tests Reported Elsewhere	Atterberg Limits: Liquid Limit (%)	Atterberg Limits: Plasticity Index (%)	Strength Test Data: Type of Test	Strength Test Data: Normal or Confining Pressure (PSF)	Strength Test Data: Shear Strength (PSF)	Strength Test Data: Deviator Stress (PSF)	Moisture Content (%)	Dry Density (PCF)	Blows/Ft.	Samples	Symbols	Description
0												SM	MOTTLED BROWN SILTY FINE SAND WITH FRAGMENTS OF SANDSTONE (VERY DENSE) — FILL
5		24.1	7.6					10.0	124.7	104/10"	▮		
10										83	▮		DENSE
15								10.8	120.6	76	▮		WITH SOME CLAY (DENSE)
17												SP	BROWN VERY FINE TO FINE SAND (MEDIUM DENSE)
20										43	▮		
23												SM	BROWN SILTY FINE SAND WITH SOME CLAY (MEDIUM DENSE)
25		27.8	9.7					7.7	93.0	54	▮		
30								7.2	102.0	60	▮		MEDIUM DENSE
35													
40													
45													
50													
55													
60													
65													
70													
75													
80													

BORING TERMINATED AT 30.5 FEET ON 9/24/85.
NO GROUNDWATER ENCOUNTERED.

LOG OF BORING

BY Dames & Moore

Figure 3-3L

BORING N1-AC3

SURFACE ELEVATION: 8544.5 FEET
PCC COORDINATES
S 4688
E 18266

LABORATORY TEST DATA

TESTS REPORTED ELSEWHERE	ATTERBERG LIMITS: LIQUID LIMIT (%)	ATTERBERG LIMITS: PLASTICITY INDEX (%)	STRENGTH TEST DATA: TYPE OF TEST	STRENGTH TEST DATA: NORMAL OR CONFINING PRESSURE (PSF)	STRENGTH TEST DATA: SHEAR STRENGTH (PSF)	STRENGTH TEST DATA: DEVIATOR STRESS (PSF)	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	BLOWS/FT.
							10.2	123.8	81
									79
							16.7	115.8	82
									70
MA	23.6	5.3					13.6	119.5	66
									39

Depth (ft): 0 5 10 15 20 25 30 35 40 45 50 55 60 65 70 75 80

SAMPLES

SYMBOLS	DESCRIPTION
ML-CL	MOTTLED BROWN SILTY FINE SAND WITH FRAGMENTS OF SANDSTONE AND SOME CLAY (DENSE)
	GRADES WITH LESS CLAY (DENSE)
	DENSE
	GRADES TO INCLUDE FINE SANDY SILT POCKETS (DENSE)
ML	DARK BROWN FINE SANDY SILT (DENSE)
	DENSE
SP	VERY FINE TO FINE SAND WITH FRAGMENTS OF SANDSTONE

FILL

BORING TERMINATED AT 29.0 FEET ON 9/24/85.
NO GROUNDWATER ENCOUNTERED.

LOG OF BORING

BY Dames & Moore Figure 3-3M

BORING N1-01

SURFACE ELEVATION: 6528.1 FEET
PCC COORDINATES
S 3918
E 27459

LABORATORY TEST DATA

Depth (ft)	Tests Reported Elsewhere	Atterberg Limits: Liquid Limit (%)	Atterberg Limits: Plasticity Index (%)	Strength Test Data: Type of Test	Strength Test Data: Normal or Confining Pressure (PSF)	Strength Test Data: Shear Strength (PSF)	Strength Test Data: Deviator Stress (PSF)	Moisture Content (%)	Dry Density (PCF)	Blows/Ft.
5	G							11.5	116.4	92/11"
10								8.0	104.0	89
15										23
25								20.2	108.1	75/9"

Depth scale: 0, 5, 10, 15, 20, 25, 30, 35, 40, 45, 50, 55, 60, 65, 70, 75, 80

SYMBOLS	DESCRIPTION
ML	MOTTLED OLIVE BROWN FINE SANDY SILT WITH FRAGMENTS OF SHALE (VERY DENSE) — FILL
SP	BROWN FINE TO COARSE SAND WITH GRAVEL (DENSE)
	BROWN FINE TO COARSE SAND WITH GRAVEL (LOOSE)
	GRADING LESS GRAVEL
ML	BLACK GRAY SILT
SHALE	DARK GRAY SHALE (DENSE)

BORING TERMINATED AT 25.5 FEET ON 9/27/85.
GROUNDWATER ENCOUNTERED AT 15.0 FEET ON 9/27/85.

LOG OF BORING

BY Dames & Moore Figure 3-3N

BORING N1-02

SURFACE ELEVATION: 6526.0 FEET
PCC COORDINATES
S 3839
E 27896

LABORATORY TEST DATA

TESTS REPORTED ELSEWHERE	ATTERBERG LIMITS: LIQUID LIMIT (%)	ATTERBERG LIMITS: PLASTICITY INDEX (%)	STRENGTH TEST DATA: TYPE OF TEST	STRENGTH TEST DATA: NORMAL OR CONFINING PRESSURE (PSF)	STRENGTH TEST DATA: SHEAR STRENGTH (PSF)	STRENGTH TEST DATA: DEVIATOR STRESS (PSF)	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	BLOWS/FT.
							7.6	116.9	72
							12.9	113.1	46
							4.8	100.5	30
							19.7	105.9	30
									72
									50/1"

Depth scale (feet): 0, 5, 10, 15, 20, 25, 30, 35, 40, 45, 50, 55, 60, 65, 70, 75, 80

BLOWS/FT. · SAMPLES · SYMBOLS · DESCRIPTION

SYMBOLS	DESCRIPTION
ML	MOTTLED OLIVE BROWN FINE SANDY SILT WITH FRAGMENTS OF SHALE (DENSE)
	DENSE
SP	BROWN FINE SAND WITH TRACES OF COAL (MEDIUM DENSE)
SM-SP	BROWN SILTY FINE SAND AND FINE TO COARSE SAND WITH GRAVEL (MEDIUM DENSE)
	(DENSE)
	GRADING LESS GRAVEL
ML	DARK GRAY SILT
	VERY DENSE
COAL	

FILL

BORING TERMINATED AT 36.1 FEET ON 9/27/85.
GROUNDWATER ENCOUNTERED AT 17.0 FEET ON 9/27/85.

LOG OF BORING

BY Dames & Moore

Figure 3-30

BORING N10-D1

SURFACE ELEVATION: 6584.8 FEET
PCC COORDINATES
S 2138
E 31628

LABORATORY TEST DATA

Depth (ft)	Tests Reported Elsewhere	Atterberg Limits: Liquid Limit (%)	Atterberg Limits: Plasticity Index (%)	Strength Test Data: Type of Test	Strength Test Data: Normal or Confining Pressure (PSF)	Strength Test Data: Shear Strength (PSF)	Strength Test Data: Deviator Stress (PSF)	Moisture Content (%)	Dry Density (PCF)	Blows/Ft.
5	Cu			DS-UU	500	2200		9.6	120.0	58
10		17.8	0.1	TXCUPP	–	–		10.5	119.4	50
15		19.2	0.1	TXCUPP	–	–				43
20		25.1	4.3	DS-UU	5000	3840		10.2	101.2	11
25	G MA,C			DS-UU	2000	1400		10.0	110.6	27
30										60

Depth scale: 0, 5, 10, 15, 20, 25, 30, 35, 40, 45, 50, 55, 60, 65, 70, 75, 80

SYMBOLS / DESCRIPTION

Symbol	Description
SM	BROWN SILTY FINE SANDS (MEDIUM DENSE)
	MEDIUM DENSE
	MEDIUM DENSE
SP	LIGHT BROWN FINE SAND WITH SOME SILT (LOOSE)
	BROWN SAND WITH SOME SILT AND A TRACE OF GRAVEL (MEDIUM DENSE)
SS	LIGHT GREY SANDSTONE (DENSE)

FILL

BORING TERMINATED AT 29.0 FEET ON 9/23/85.
GROUNDWATER ENCOUNTERED AT 26.0 FEET ON 9/23/85.

LOG OF BORING

BY Dames & Moore

Figure 3-3P

BORING N10-D2

SURFACE ELEVATION: 6585.5 FEET
PCC COORDINATES
S 2059
E 31669

LABORATORY TEST DATA

Depth (ft)	Tests Reported Elsewhere	Atterberg Limits: Liquid Limit (%)	Atterberg Limits: Plasticity Index (%)	Strength Test Data: Type of Test	Strength Test Data: Normal or Confining Pressure (PSF)	Strength Test Data: Shear Strength (PSF)	Strength Test Data: Deviator Stress (PSF)	Moisture Content (%)	Dry Density (PCF)	Blows/Ft.
4				DS-UU	500	1800		6.9	110.5	60
9		20.4	2.9					11.3	119.7	64
14										56
19										24
24				DS-UU	3000	3240		8.1	111.5	48
29								12.2	110.3	95/8"

SYMBOLS / DESCRIPTION

- 0 ft — SM — BROWN SILTY FINE SANDS WITH TRACE OF CLAY (DENSE)
  - DENSE
  - MEDIUM DENSE
  - GRADES WITH RED SANDSTONE FRAGMENTS (LOOSE)
  - FILL (0 ft to about 20 ft)
- About 20 ft — SP — BROWN SAND WITH GRAVEL (MEDIUM DENSE)
- About 28.5 ft — SHALE — GRAY SHALE SLIGHTLY WEATHERED

BORING TERMINATED AT 29.2 FEET ON 9/23/85.
NO GROUNDWATER ENCOUNTERED.

LOG OF BORING

BY **Dames & Moore** Figure 3-3Q

BORING N10-D3

SURFACE ELEVATION: 6583.9 FEET
PCC COORDINATES
S 1966
E 31718

LABORATORY TEST DATA

Depth (ft)	Tests Reported Elsewhere	Atterberg Limits: Liquid Limit (%)	Atterberg Limits: Plasticity Index (%)	Strength Test Data: Type of Test	Strength Test Data: Normal or Confining Pressure (PSF)	Strength Test Data: Shear Strength (PSF)	Strength Test Data: Deviator Stress (PSF)	Moisture Content (%)	Dry Density (PCF)	Blows/Ft.
5–10								10.4	82.7	16
10–15										18
15–20										8
20–25	MA			DS-UU	2000	2560		9.0	112.8	22
20–25								15.1	117.1	
25–30										90/8"

Depth scale: 0, 5, 10, 15, 20, 25, 30, 35, 40, 45, 50, 55, 60, 65, 70, 75, 80

SYMBOLS	DESCRIPTION
SM	LIGHT BROWN SILTY VERY FINE SAND WITH SOME CLAY (LOOSE)
	LOOSE
	VERY LOOSE
ML	DARK BROWN SANDY SILT
GP	BROWN SANDY GRAVEL (MEDIUM DENSE)
SS	LIGHT GRAY SANDSTONE (VERY DENSE)

FILL

BORING TERMINATED AT 27.5 FEET ON 9/23/85.
GROUNDWATER ENCOUNTERED AT 25.2 FEET ON 9/23/85.

LOG OF BORING

BY Dames & Moore

Figure 3-3R

BORING N14-01

SURFACE ELEVATION: 6604.7 FEET
PCC COORDINATES
S 15614
E 53870

LABORATORY TEST DATA

Depth (ft)	Tests Reported Elsewhere	Atterberg Limits: Liquid Limit (%)	Atterberg Limits: Plasticity Index (%)	Strength Test Data: Type of Test	Strength Test Data: Normal or Confining Pressure (PSF)	Strength Test Data: Shear Strength (PSF)	Strength Test Data: Deviator Stress (PSF)	Moisture Content (%)	Dry Density (PCF)	Blows/Ft.
0–5								6.8	105.2	101
5–10								8.0	96.5	23
10–15										30
15–20								8.0	100.9	38
20–25										32
25–30										56
30–35										56
35–40										
40–45										
45–50										
50–55										
55–60										
60–65										
65–70										
70–75										
75–80										

SYMBOLS / DESCRIPTION

SM — BROWN SILTY FINE SAND WITH SOME SMALL FRAGMENTS OF SANDSTONE (DENSE)

FILL

MEDIUM DENSE

MEDIUM DENSE

BROWN WITH TAN STREAKS SILTY FINE SAND (MEDIUM DENSE)

BORING TERMINATED AT 31.5 FEET ON 9/19/85.
NO GROUNDWATER ENCOUNTERED.

LOG OF BORING

BY Dames & Moore

Figure 3-3S

BORING N14-02

SURFACE ELEVATION: 6604.6 FEET
PCC COORDINATES
S 15728
E 53781

LABORATORY TEST DATA

Depth (ft)	Tests Reported Elsewhere	Atterberg Limits: Liquid Limit (%)	Atterberg Limits: Plasticity Index (%)	Strength Test Data: Type of Test	Strength Test Data: Normal or Confining Pressure (PSF)	Strength Test Data: Shear Strength (PSF)	Strength Test Data: Deviator Stress (PSF)	Moisture Content (%)	Dry Density (PCF)	Blows/Ft.	Symbol	Description
0											SM	BROWN SILTY FINE SAND WITH SOME FRAGMENTS OF SANDSTONE (MEDIUM DENSE) — FILL
5								11.1	114.6	25		
10	MA							7.0	90.1	22	ML	BROWN SANDY CLAYEY SILT (MEDIUM DENSE)
15								6.4	93.4	28	CL	BROWN SANDY CLAY (MEDIUM DENSE)
20		25.9	9.4					8.8	96.4	65		GRADES TO DENSE
25										27	ML	BROWN WITH TAN STREAKS, CLAYEY SILT (MEDIUM DENSE)
30								10.8	105.2	52	SM	BROWN WITH TAN STREAKS, SILTY FINE SAND (MEDIUM DENSE)

BORING TERMINATED AT 32.5 FEET ON 9/19/85.
NO GROUNDWATER ENCOUNTERED.

Depth scale (ft): 0, 5, 10, 15, 20, 25, 30, 35, 40, 45, 50, 55, 60, 65, 70, 75, 80

LOG OF BORING

BY Dames & Moore

Figure 3-3T

BORING N14-03

SURFACE ELEVATION: 6604.9 FEET
PCC COORDINATES
S 15823
E 53706

LABORATORY TEST DATA

Depth (ft)	Tests Reported Elsewhere	Atterberg Limits: Liquid Limit (%)	Atterberg Limits: Plasticity Index (%)	Strength Test Data: Type of Test	Strength Test Data: Normal or Confining Pressure (PSF)	Strength Test Data: Shear Strength (PSF)	Strength Test Data: Deviator Stress (PSF)	Moisture Content (%)	Dry Density (PCF)	Blows/Ft.	Samples
0											
5								10.9	113.5	64	▯
10								6.6	107.9	62	▯
15										23	▯
20								3.9	100.0	38	▯
25				DS-UU	3000	3040		6.5	97.5	46	▯
30	MA	29.3	11.4					5.3	102.9	67	▯
35											
40										36	▯
45										100/ 0.25"	
50											
55											
60											
65											
70											
75											
80											

SYMBOLS / DESCRIPTION

SM — BROWN SILTY FINE SAND WITH SOME SMALL FRAGMENTS OF SANDSTONE (DENSE)

FILL

DENSE

MEDIUM DENSE

MEDIUM DENSE

CL — MOTTLED OLIVE BROWN AND TAN SANDY CLAY WITH FRAGMENTS OF SANDSTONE (DENSE)

SM — BROWN SILTY SAND (MEDIUM DENSE)

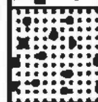
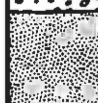
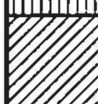
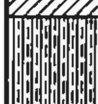
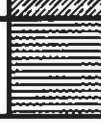
SANDSTONE

BORING TERMINATED AT 47.0 FEET ON 9/20/85.
NO GROUNDWATER ENCOUNTERED.

LOG OF BORING

BY Dames & Moore

Figure 3-3U

MAJOR DIVISIONS			GRAPHIC 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

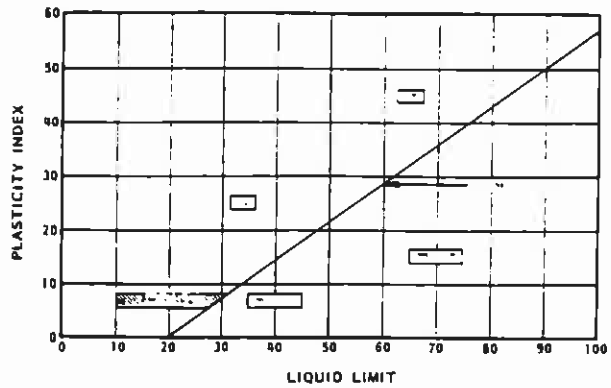
NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

UNIFIED SOIL CLASSIFICATION SYSTEM

BY **Dames & Moore** Figure 3-4A

SYMBOL	TYPE OF TEST
M	MOISTURE
QD	QUICK MD TEST BASED ON ASSUMED SPECIFIC GRAVITY
MD	MOISTURE-DENSITY
CD	CHUNK DENSITY ON BULK SAMPLE
RD	RELATIVE DENSITY
COMP	COMPACTION CURVE
CI	CALIFORNIA IMPACT
CC	COMPACTED CORE
G	SPECIFIC GRAVITY
pH	HYDROGEN ION CONCENTRATION
MA	MECHANICAL ANALYSIS *
SA	SIEVE ANALYSIS (+200 ONLY)
HA	HYDROMETER ANALYSIS (-200 ONLY)
AL	ATTERBERG LIMITS (LL & PL)
SL	SHRINKAGE LIMIT
FS	FREE SWELL
SS	SHRINK-SWELL
EXP	EXPANSION
C (COL)	CONSOLIDATION (COLLAPSE)
VC	VIBRATING CONSOLIDATION
P	PERMEABILITY
FP	FIELD PERMEABILITY
UC	UNCONFINED COMPRESSION
TXUU	TRIAXIAL COMPRESSION TEST 1. UNCONSOLIDATED-UNDRAINED
TXCU	2. CONSOLIDATED-UNDRAINED
TXCUM	3. CU/MULTIPHASE**
TXCUPP	4. CU/WITH PORE PRESSURE MEASUREMENTS
TXCD	5. CONSOLIDATED-DRAINED
DS/UU	DIRECT SHEAR TEST 1. UNCONSOLIDATED-UNDRAINED
DS/CU	2. CONSOLIDATED-UNDRAINED
DS/CD	3. CONSOLIDATED-DRAINED
DS/CD/M◇	4. CD/MULTIPHASE**
LV	TORVANE SHEAR (LAB VANE SHEAR)

* INCLUDES COMPLETE ANALYSIS, SIEVING AND HYDROMETER
** SERIES OF TESTS RUN ON SAMPLE



PLASTICITY CHART

■ INDICATES DEPTH OF AUGER CUTTINGS SAMPLE

■ INDICATES DEPTH OF UNDISTURBED SAMPLE

☒ INDICATES DEPTH OF DISTURBED SAMPLE

□ INDICATES DEPTH OF SAMPLING ATTEMPT WITH NO RECOVERY

◪ INDICATES DEPTH OF STANDARD PENETRATION TEST

□ INDICATES DEPTH OF STANDARD PENETRATION TEST WITH NO RECOVERY

70% I 5% INDICATES DEPTH AND LENGTH OF CORE RUN

RQD (ROCK QUALITY DETERMINATION) PERCENT OF THE TOTAL CORE RUN HAVING AN UNFRACTURED LENGTH OF 4" OR MORE

PERCENT OF CORE RUN RECOVERED

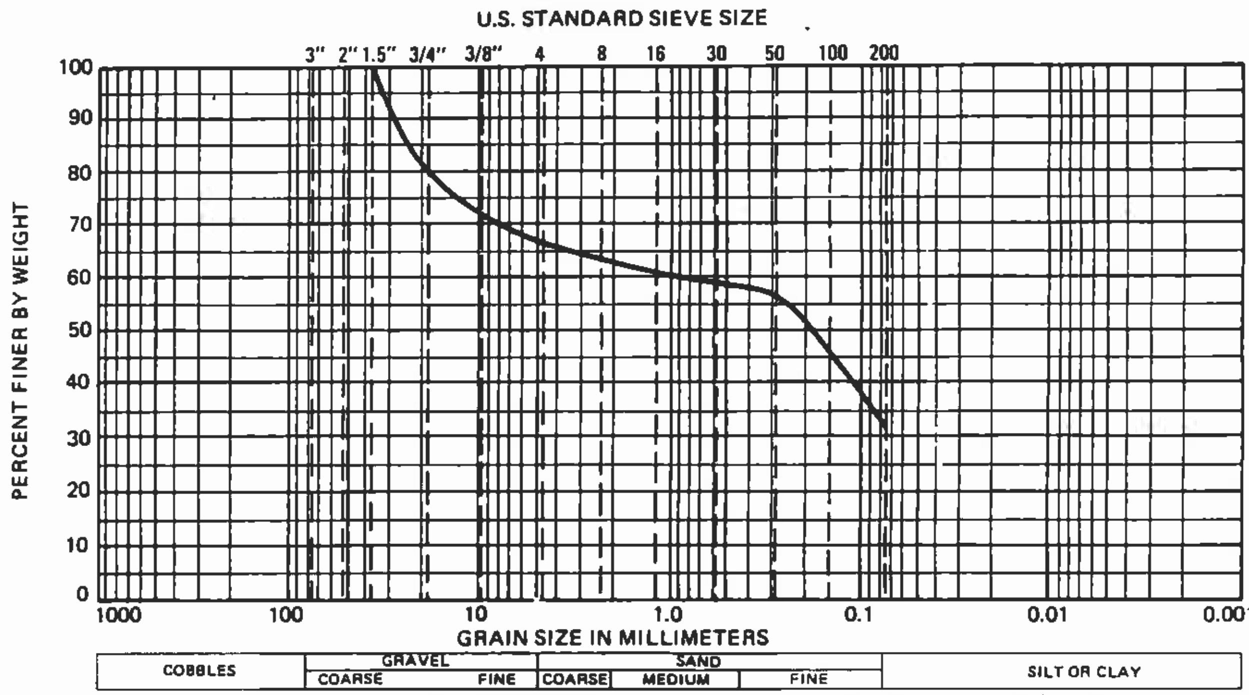
◘ INDICATES DEPTH OF FIELD VANE SHEAR TEST

NOTE
UNLESS OTHERWISE NOTED SAMPLING RESISTANCE IS MEASURED IN BLOWS PER FOOT REQUIRED TO DRIVE SAMPLER 12-INCHES AFTER SAMPLER HAS BEEN SEATED 6-INCHES. A 140-POUND HAMMER, FREE FALLING A DISTANCE OF 30 INCHES IS USED TO DRIVE THE SAMPLER.

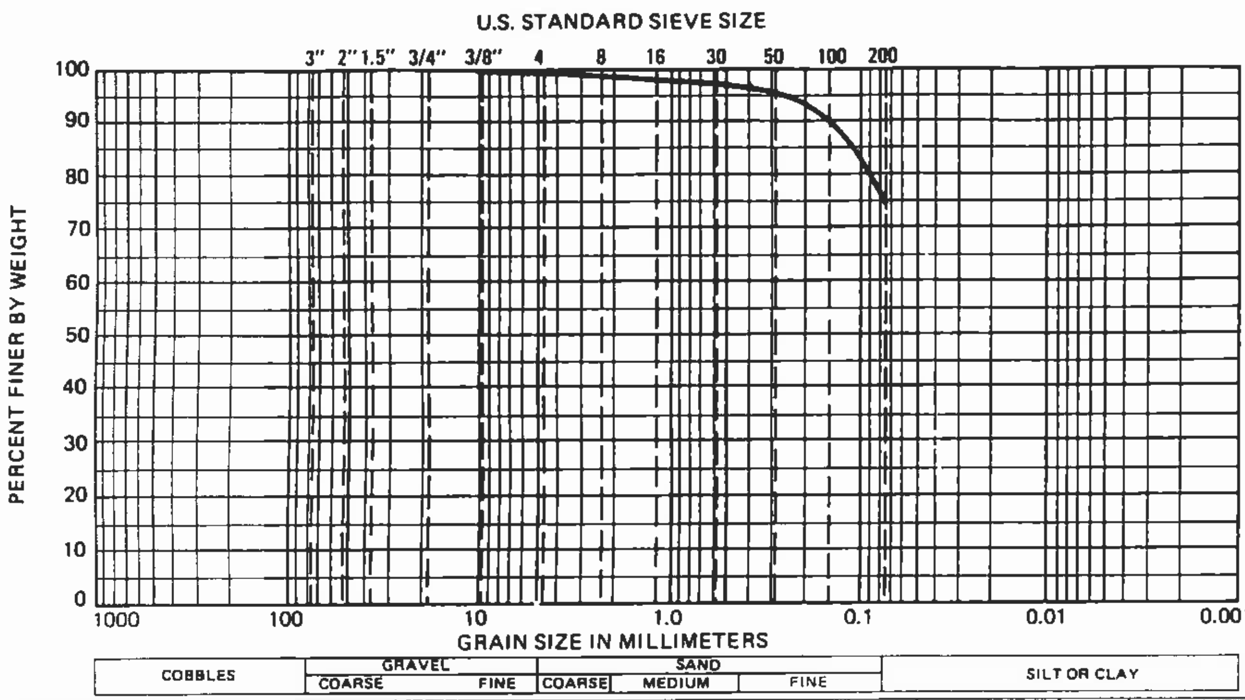
KEY TO SAMPLES

KEY TO LOG OF BORINGS

BY **Dames & Moore** Figure 3-4B



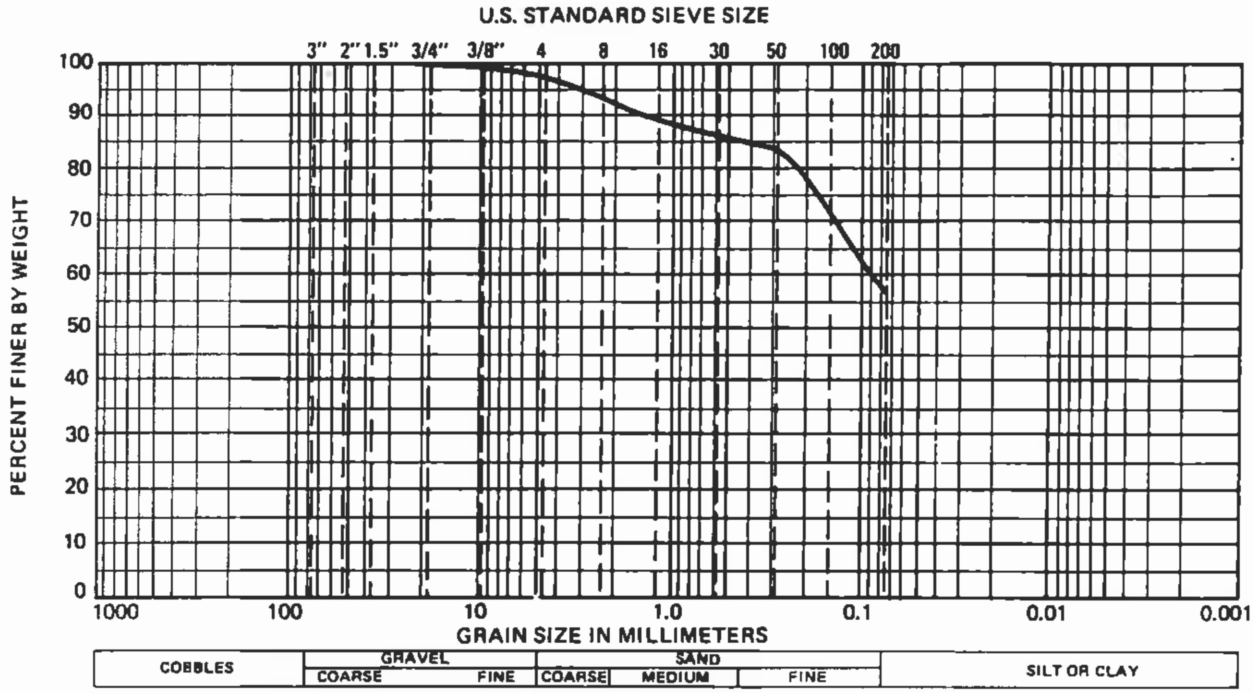
J3-E1	DEPTH		CLASSIFICATION	NAT. W C	LL	PL	PI	
SAMPLE 3	13.0'-13.5'	SC	BROWN CLAYEY SAND WITH SOME GRAVEL	11.9	22.9	15.6	7.3	



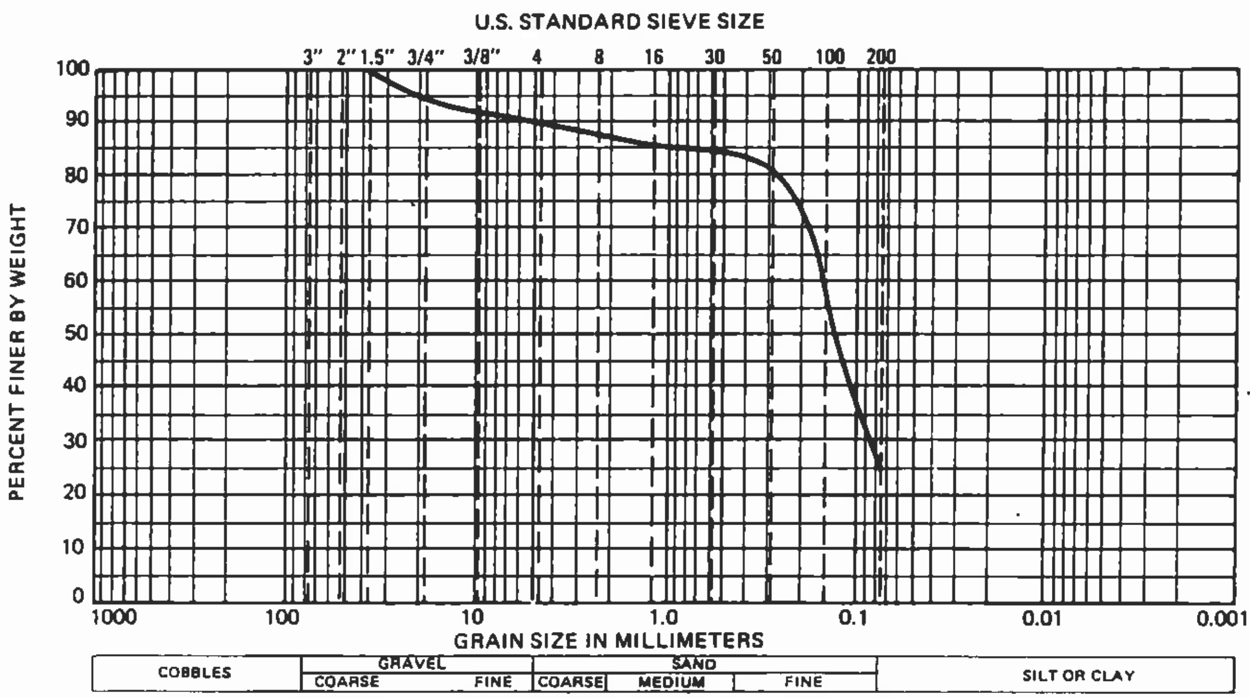
J28-C1	DEPTH		CLASSIFICATION	NAT. W C	LL	PL	PI	
SAMPLE 1	3.0'	ML	MOTTLED TAN & BROWN FINE SANDY SILT WITH TRACE CLAY	8.3	27.1	17.7	9.4	

GRADATION CURVES

BY **Dames & Moore** Figure 3-5A



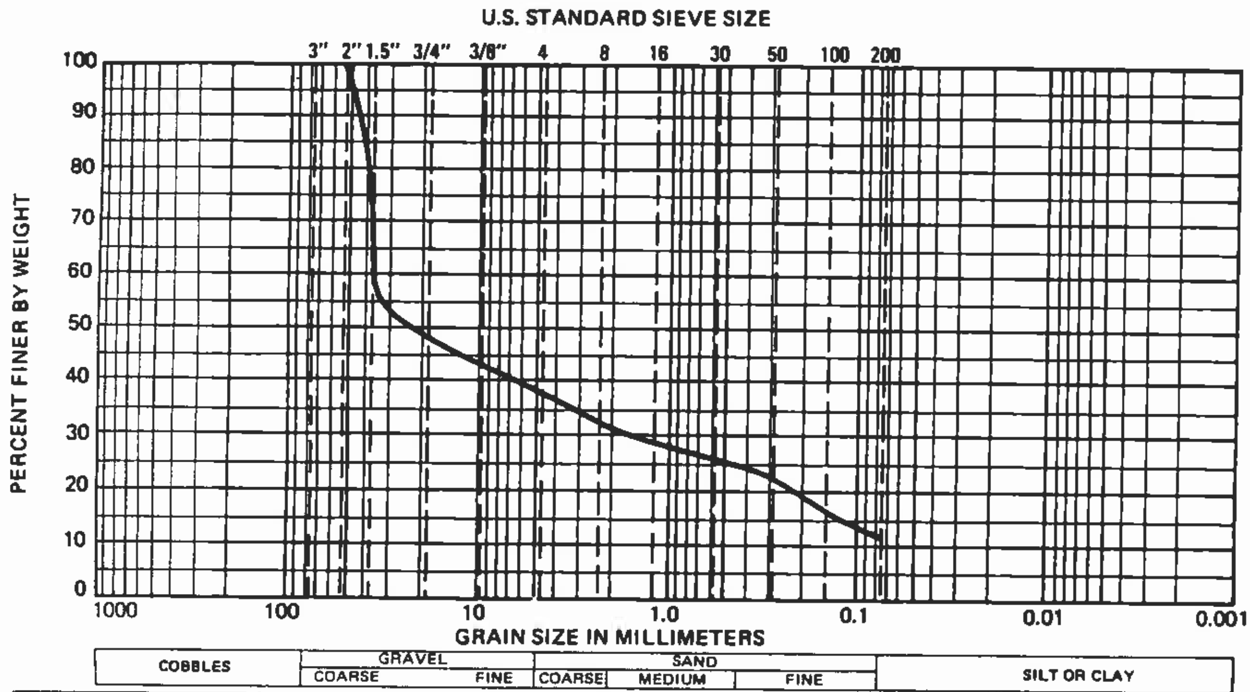
N1-AC3	DEPTH		CLASSIFICATION	NAT. W C	LL	PL	PI	
SAMPLE 5	23.0'- 23.5'	ML	DARK BROWN FINE SANDY SILT	13.2	23.6	18.3	5.3	



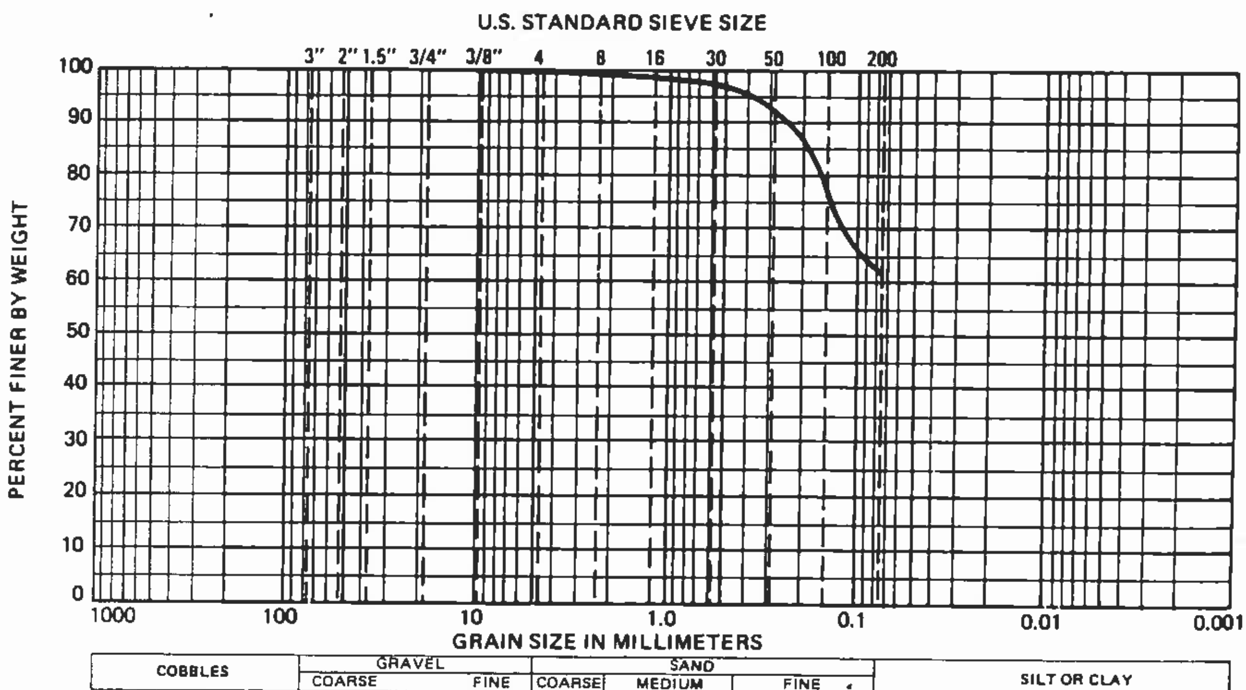
N10-01	DEPTH		CLASSIFICATION	NAT. W C	LL	PL	PI	
SAMPLE 5	23.0'	SP	BROWN SAND W/SOME SILT AND A TRACE GRAVEL	12.5	-	-	-	

GRADATION CURVES

BY **Dames & Moore** Figure 3-5B



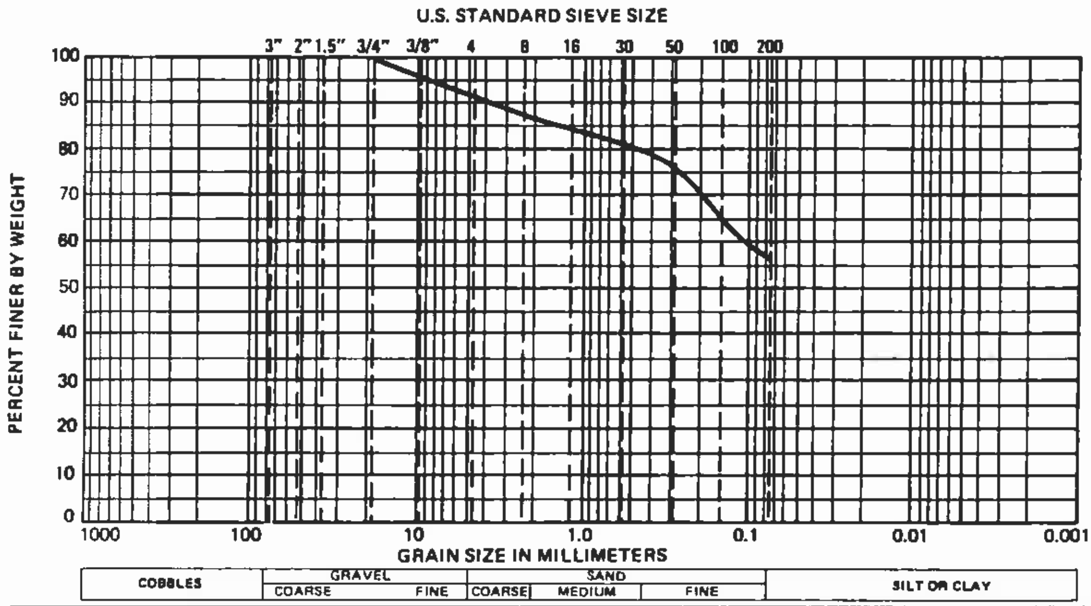
N10-03	DEPTH	CLASSIFICATION		NAT. W C	LL	PL	PI	
SAMPLE 4	20.5'	GP	BROWN SANDY GRAVEL	9.6	-	-	-	



N14-02	DEPTH	CLASSIFICATION		NAT. W C	LL	PL	PI	
SAMPLE 2	11.5'-12.0'	ML	BROWN SANDY CLAYEY SILT	6.9	-	-	-	

GRADATION CURVES

BY **Dames & Moore** Figure 3-5C



N14-03	DEPTH		CLASSIFICATION	NAT. W C	LL	PL	PI	
SAMPLE 6	29.5'-30.0'	CL	BROWN AND TAN SANDY CLAY W/SOME SILT	8.5	29.3	17.9	11.4	

GRADATION CURVES

BY **Dames & Moore** Figure 3-5D

3.6.1 Moisture Content and Dry Density

The moisture content and dry density of recovered samples were determined as an aid to classification of the soils and estimation of engineering properties. The moisture content was determined in accordance with ASTM D 2216 test procedures. The results of the moisture content and density determinations are presented on the Logs of Borings, Figures 3-3A through U.

3.6.2 Grain Size Distribution

The particle size distribution of representative samples was determined by passing a specimen of soil through a nested set of standard sieves. The test was completed in accordance with ASTM D 422 procedures. The test results are presented on Figures 3-5A through D.

3.6.3 Atterberg Limits

As an aid to classifying the soils, the liquid and plastic limits of representative samples were determined in accordance with ASTM D 4318 procedures. The results of the plastic and liquid limit determinations are presented on the Logs of Borings, Figures 3-3A through U.

3.6.4 Specific Gravity

The specific gravity of selected soil samples was determined to provide information for the engineering analyses. The specific gravity was determined in accordance with ASTM C 854 procedures. Results of specific gravity testing are presented on Table 3-4.

Table 3-4

RESULTS OF SPECIFIC GRAVITY TESTING

Sedimentation Structure	Sample Depth (ft)	Soil Type	Specific Gravity
J3-E1	8	SM-ML	2.58
N7-I2	8	SM	2.59
N1-O1	5.5	ML	2.54
N10-C	Surface	SP	2.64
N10-E	Surface	SP	2.56

3.6.5 Direct Shear Tests

Direct shear tests were performed to evaluate the shear strength of representative samples of alluvial soils. Samples were loaded vertically (normal to the ends of the sample) and the shearing force was applied horizontally in the form of a constant rate of deflection. The test results are presented on Figures 3-3A through U, and the method of completing the tests is described on Figure 3-6.

Method Of Performing Direct Shear and Friction Tests

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 APPARATUS WITH ELECTRONIC RECORDER

Each sample is tested in a split sample holder, two and one-half inches in diameter and one inch high. Undisturbed samples of in-place soils are extruded from 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 one-inch length of the sample is tested in direct single 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 upper sample holder in a direction perpendicular to the axis of the sample. Transverse movement of the lower sample holder is prevented.

The shearing failure is accomplished by applying to the upper sample holder a constant rate of deflection. The shearing load and the deflections in both the axial and transverse directions are recorded and plotted. The shearing strength of the soils 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 lower sample holder 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 soil over the friction material surface.

3.6.6 Triaxial Compression Test

Triaxial consolidated undrained compression tests with pore pressure measurements were completed to evaluate shear strength of residual sandstone and residual shale soils under simulated loading conditions similar to those expected in the field. Samples were subjected first to an all-round confining pressure and allowed to consolidate. A shearing force was then applied vertically in the form of a constant rate of deflection. Measurements taken during a test define successive stress states within the sample and can be plotted as points on a stress path. The test results are presented on Figures 3-7A through 3-7B. A general description of the test procedure is presented on Figure 3-8.

3.6.7 Consolidation/Collapse Test

A consolidation test was performed on a representative sample extracted from borings to provide information on the settlement characteristics of the soil. The test was performed in the manner described on Figure 3-9. In addition, in order to evaluate the collapse potential of the soils, the sample was loaded to a specific consolidation pressure at the field moisture content. Once the consolidation process was completed at the field moisture content, the sample was saturated and allowed to consolidate further. The collapse potential of the sample was then evaluated based on the additional consolidation that occurred during saturation. The results of the test are presented on Figure 3-10.

3.7 ENGINEERING PROPERTIES OF MATERIALS

A registered engineer specializing in soil mechanics inspected the embankment and foundation soils of each of the sedimentation structures. A comparison was then made of the structures relative slope stability and soil strengths based on the engineer's judgement and experience. Of all the sedimentation structures inspected, the eight that were sampled and listed in Table 3-3 are considered representative of the least stable. The engineering properties of the materials encountered in the sedimentation structures were determined by laboratory testing or assumed based on experience and available literature.

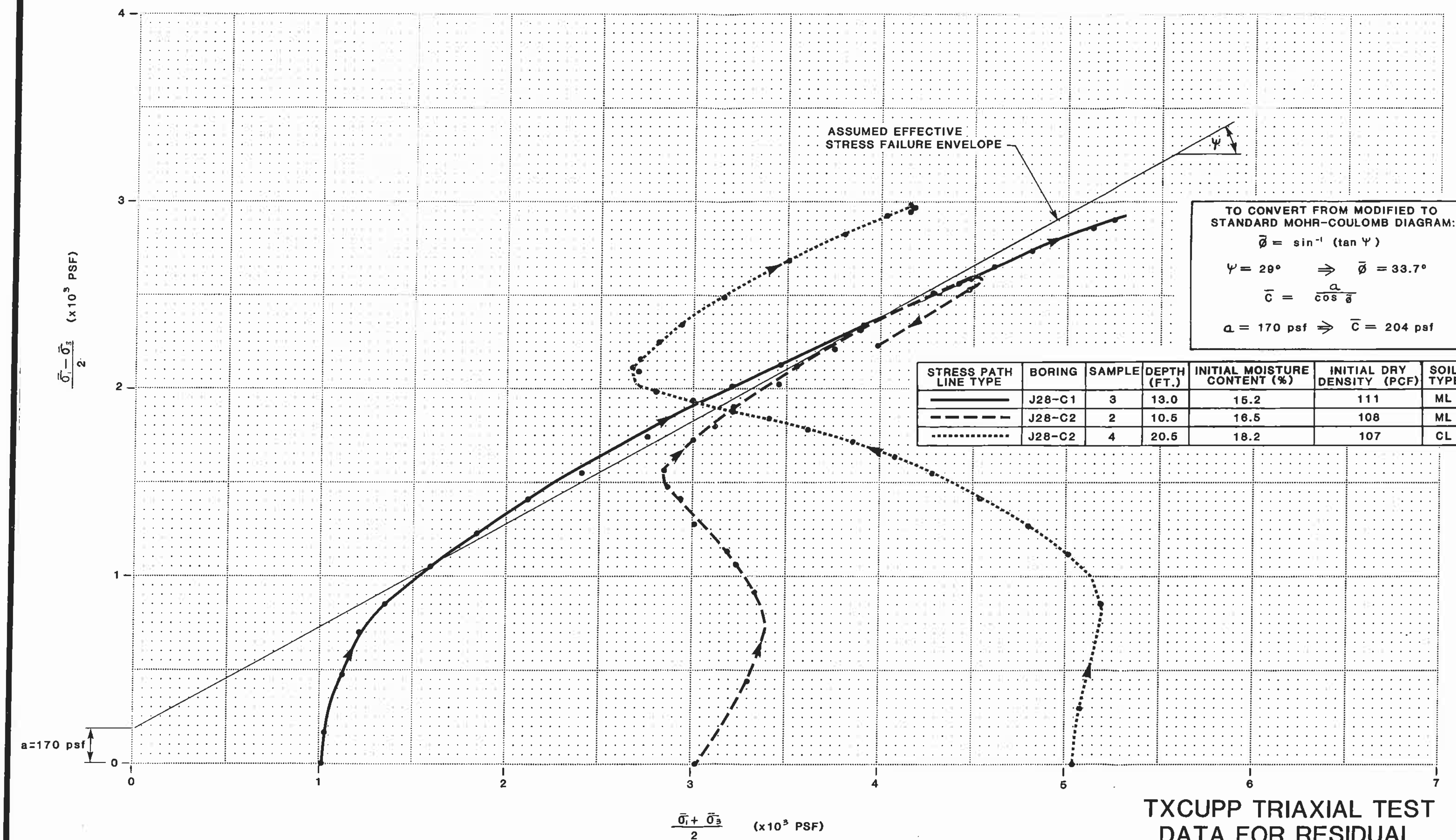
Laboratory test results show that the engineering properties of the soils derived from weathering of sandstone and shale are similar, although they tend to be classified differently. The material classifications for the three soil types have previously been described in Section 3.3 of the General Report. The residual sandstone soils are silty fine to medium sands (SP,SM,GP). The residual shale soils are fine sandy silts and silts with some clay (SM,ML,CL,GP). The alluvial soils are very fine to coarse sands (SP, GP) with varying amounts of gravel. Average dry and saturated densities of 118.3 and 129 pounds per cubic foot, respectively, were selected for the three soil types. Shear strength parameters for the three observed embankment and foundation soil types were developed in a series of triaxial shear (residual standstone and shale soils) and direct shear (alluvial soils) strength tests on representative soil samples recovered from the eight structures described above. Laboratory tests were not performed on the sandstone and shale bedrock; their strength parameters were assumed based on published literature and our experience. The strength parameters selected for the soil and rock encountered at the site are listed in Table 3-5.

Table 3-5

EFFECTIVE STRESS STRENGTH PARAMETERS

Soil or Rock Type	Friction Angle (degrees)	Cohesion (psf)
Residual Sandstone Soils	36	0
Residual Shale Soils	33	200
Alluvial Soils	36	0
Sandstone Bedrock	25	20,000
Shale Bedrock	25	20,000

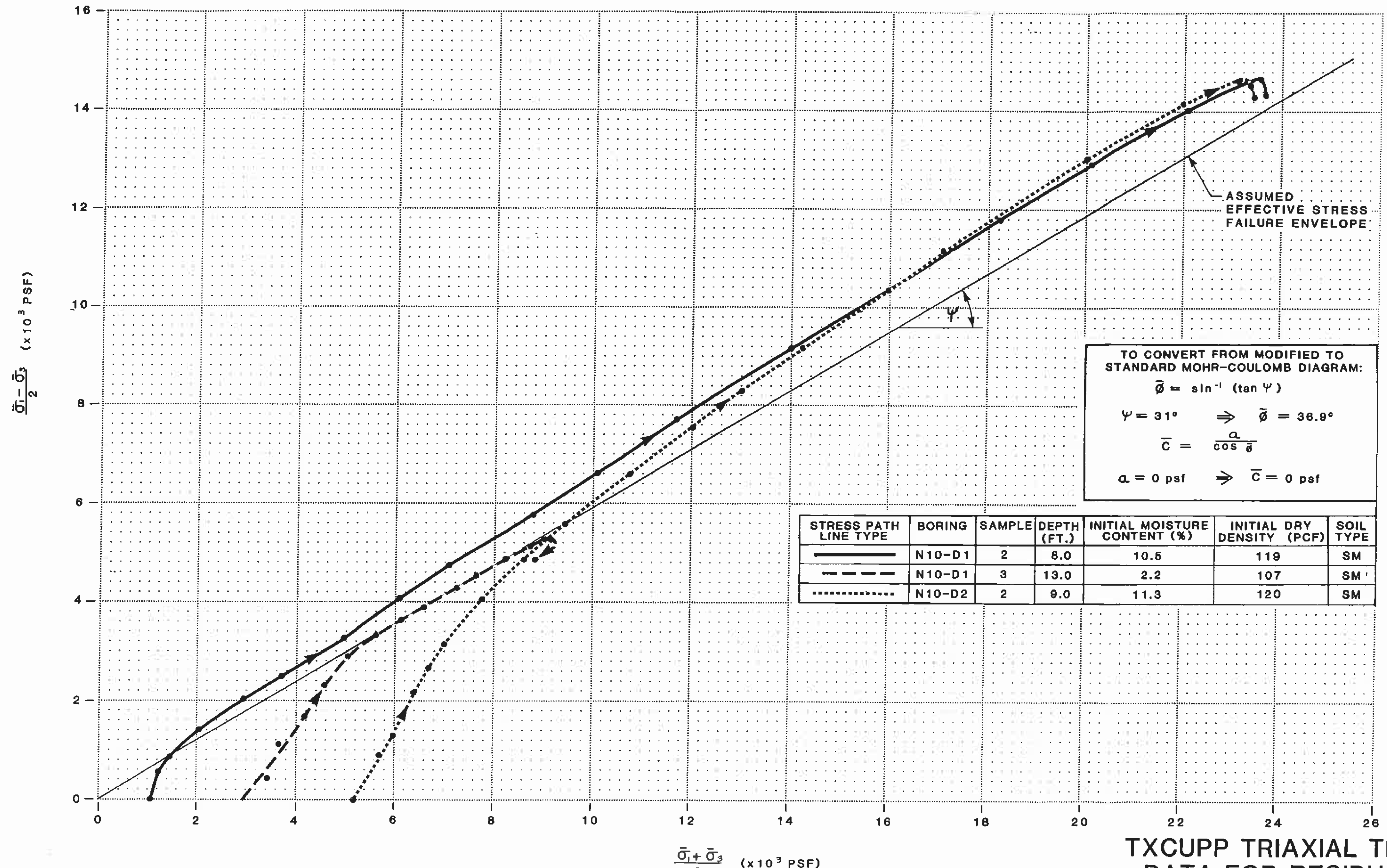
(INTENTIONALLY BLANK)



STRESS PATH LINE TYPE	BORING	SAMPLE	DEPTH (FT.)	INITIAL MOISTURE CONTENT (%)	INITIAL DRY DENSITY (PCF)	SOIL TYPE
———	J28-C1	3	13.0	15.2	111	ML
– – – –	J28-C2	2	10.5	16.5	108	ML
··········	J28-C2	4	20.5	18.2	107	CL

TXCUPP TRIAXIAL TEST
DATA FOR RESIDUAL
SHALESTONE SOILS

BY **Dames & Moore** Figure 3-7A



STRESS PATH LINE TYPE	BORING	SAMPLE	DEPTH (FT.)	INITIAL MOISTURE CONTENT (%)	INITIAL DRY DENSITY (PCF)	SOIL TYPE
———— (solid)	N10-D1	2	8.0	10.5	119	SM
— — — (dashed)	N10-D1	3	13.0	2.2	107	SM'
········ (dotted)	N10-D2	2	9.0	11.3	120	SM

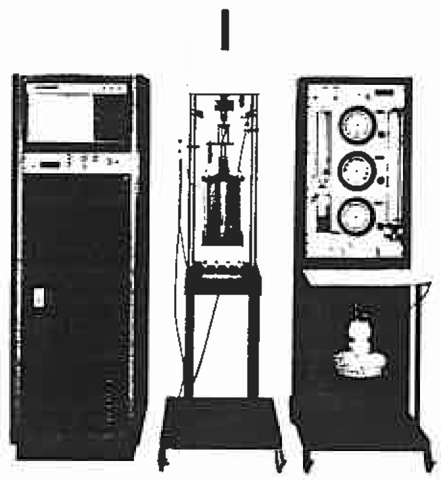
TXCUPP TRIAXIAL TEST DATA FOR RESIDUAL SANDSTONE SOILS

BY **Dames & Moore** Figure 3-7B

Methods of Performing Unconfined Compression and Triaxial Compression Tests

THE SHEARING STRENGTHS OF SOILS ARE DETERMINED FROM THE RESULTS OF UNCONFINED COMPRESSION AND TRIAXIAL COMPRESSION TESTS. IN TRIAXIAL COMPRESSION TESTS THE TEST METHOD AND THE MAGNITUDE OF THE CONFINING PRESSURE ARE CHOSEN TO SIMULATE ANTICIPATED FIELD CONDITIONS.

UNCONFINED COMPRESSION AND TRIAXIAL COMPRESSION TESTS ARE PERFORMED ON UNDISTURBED OR REMOLDED SAMPLES OF SOIL APPROXIMATELY SIX INCHES IN LENGTH AND TWO AND ONE-HALF INCHES IN DIAMETER. THE TESTS ARE RUN EITHER STRAIN-CONTROLLED OR STRESS-CONTROLLED. IN A STRAIN-CONTROLLED TEST THE SAMPLE IS SUBJECTED TO A CONSTANT RATE OF DEFLECTION AND THE RESULTING STRESSES ARE RECORDED. IN A STRESS-CONTROLLED TEST THE SAMPLE IS SUBJECTED TO EQUAL INCREMENTS OF LOAD WITH EACH INCREMENT BEING MAINTAINED UNTIL AN EQUILIBRIUM CONDITION WITH RESPECT TO STRAIN IS ACHIEVED.



TRIAXIAL COMPRESSION TEST UNIT

YIELD, PEAK, OR ULTIMATE STRESSES ARE DETERMINED FROM THE STRESS-STRAIN PLOT FOR EACH SAMPLE AND THE PRINCIPAL STRESSES ARE EVALUATED. THE PRINCIPAL STRESSES ARE PLOTTED ON A MOHR'S CIRCLE DIAGRAM TO DETERMINE THE SHEARING STRENGTH OF THE SOIL TYPE BEING TESTED.

UNCONFINED COMPRESSION TESTS CAN BE PERFORMED ONLY ON SAMPLES WITH SUFFICIENT COHESION SO THAT THE SOIL WILL STAND AS AN UNSUPPORTED CYLINDER. THESE TESTS MAY BE RUN AT NATURAL MOISTURE CONTENT OR ON ARTIFICIALLY SATURATED SOILS.

IN A TRIAXIAL COMPRESSION TEST THE SAMPLE IS ENCASED IN A RUBBER MEMBRANE, PLACED IN A TEST CHAMBER, AND SUBJECTED TO A CONFINING PRESSURE THROUGHOUT THE DURATION OF THE TEST. NORMALLY, THIS CONFINING PRESSURE IS MAINTAINED AT A CONSTANT LEVEL, ALTHOUGH FOR SPECIAL TESTS IT MAY BE VARIED IN RELATION TO THE MEASURED STRESSES. TRIAXIAL COMPRESSION TESTS MAY BE RUN ON SOILS AT FIELD MOISTURE CONTENT OR ON ARTIFICIALLY SATURATED SAMPLES. THE TESTS ARE PERFORMED IN ONE OF THE FOLLOWING WAYS:

> UNCONSOLIDATED-UNDRAINED: THE CONFINING PRESSURE IS IMPOSED ON THE SAMPLE AT THE START OF THE TEST. NO DRAINAGE IS PERMITTED AND THE STRESSES WHICH ARE MEASURED REPRESENT THE SUM OF THE INTERGRANULAR STRESSES AND PORE WATER PRESSURES.

> CONSOLIDATED-UNDRAINED: THE SAMPLE IS ALLOWED TO CONSOLIDATE FULLY UNDER THE APPLIED CONFINING PRESSURE PRIOR TO THE START OF THE TEST. THE VOLUME CHANGE IS DETERMINED BY MEASURING THE WATER AND/OR AIR EXPELLED DURING CONSOLIDATION. NO DRAINAGE IS PERMITTED DURING THE TEST AND THE STRESSES WHICH ARE MEASURED ARE THE SAME AS FOR THE UNCONSOLIDATED-UNDRAINED TEST.

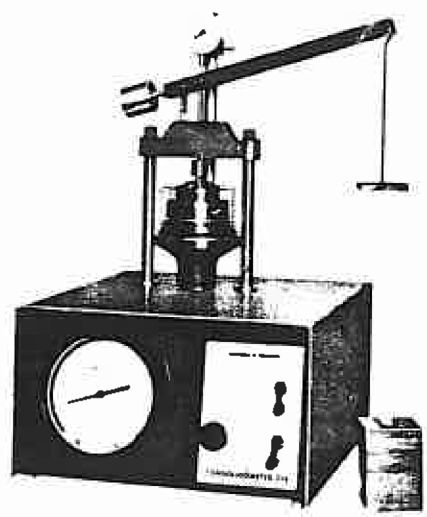
> DRAINED: THE INTERGRANULAR STRESSES IN A SAMPLE MAY BE MEASURED BY PERFORMING A DRAINED, OR SLOW, TEST. IN THIS TEST THE SAMPLE IS FULLY SATURATED AND CONSOLIDATED PRIOR TO THE START OF THE TEST. DURING THE TEST, DRAINAGE IS PERMITTED AND THE TEST IS PERFORMED AT A SLOW ENOUGH RATE TO PREVENT THE BUILDUP OF PORE WATER PRESSURES. THE RESULTING STRESSES WHICH ARE MEASURED REPRESENT ONLY THE INTERGRANULAR STRESSES. THESE TESTS ARE USUALLY PERFORMED ON SAMPLES OF GENERALLY NON-COHESIVE SOILS, ALTHOUGH THE TEST PROCEDURE IS APPLICABLE TO COHESIVE SOILS IF A SUFFICIENTLY SLOW TEST RATE IS USED.

AN ALTERNATE MEANS OF OBTAINING THE DATA RESULTING FROM THE DRAINED TEST IS TO PERFORM AN UNDRAINED TEST IN WHICH SPECIAL EQUIPMENT IS USED TO MEASURE THE PORE WATER PRESSURES. THE DIFFERENCES BETWEEN THE TOTAL STRESSES AND THE PORE WATER PRESSURES MEASURED ARE THE INTERGRANULAR STRESSES.

Method Of Performing Consolidation Tests

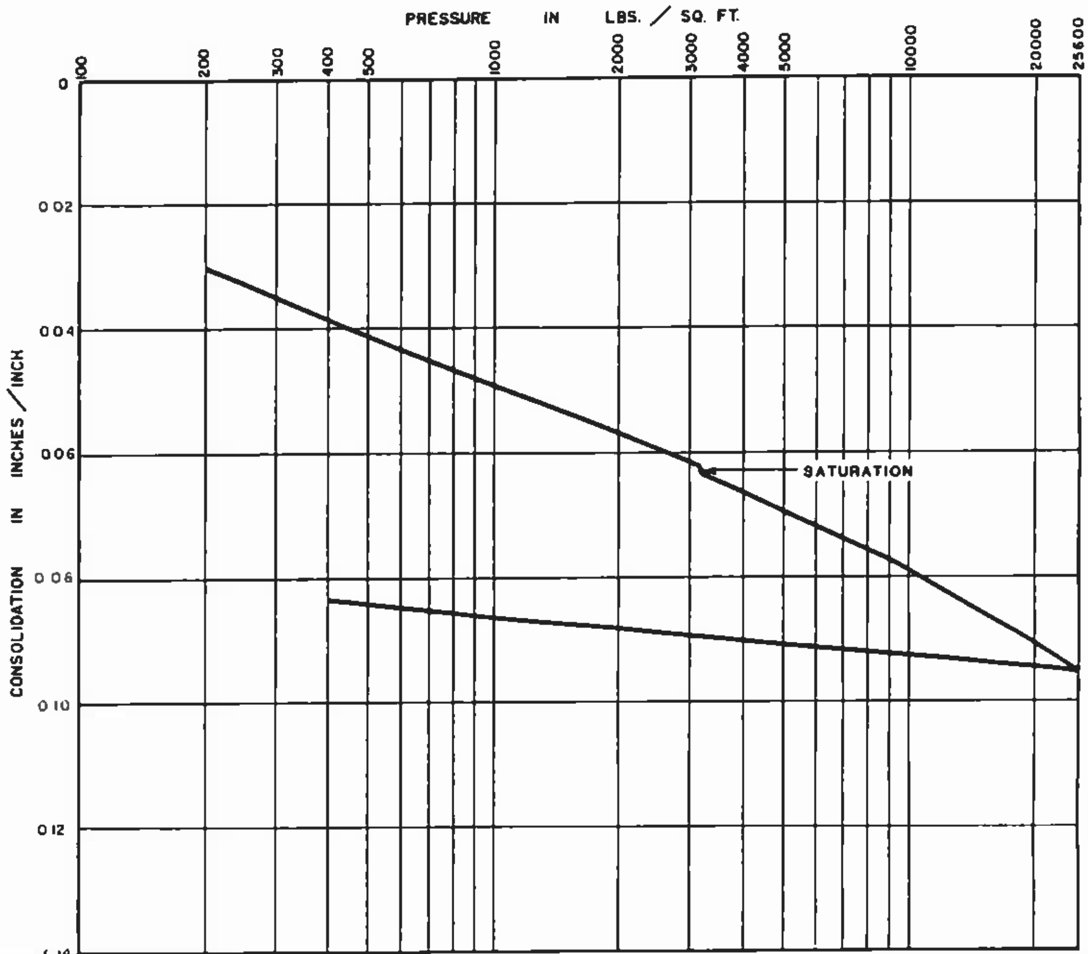
Consolidation tests are performed to evaluate the volume changes of soils subjected to increased loads. Time-consolidation and pressure-consolidation curves may be plotted from the data obtained in the tests. Engineering analyses based on these curves permit estimates to be made of the probable magnitude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



BORING NUMBER		SOIL TYPE	SYMBOL	MOISTURE CONTENT IN PERCENT		DRY DENSITY IN pcf		KEY
NO.	DEPTH			BEFORE	AFTER	BEFORE	AFTER	
N10-D1	23'	BROWN SAND WITH SOME SILT AND TRACE OF GRAVEL	SP	15.5	19.5	99.6	110.6	

CONSOLIDATION TEST DATA

BY **Dames & Moore** Figure 3-10

3.8 STABILITY ANALYSES

3.8.1 Stability Requirements

Regulation 30 CFR Part 816.49 (a,3) states that "impoundments shall have a minimum static safety factor of 1.5 for the normal pool with steady seepage saturation conditions, and a seismic safety factor of at least 1.2". Strict interpretation of this requirement leads to the conclusion that embankments need only satisfy the stability requirements under normal (or spillway-level) pool conditions; however, from a slope stability standpoint, it was assumed that the critical condition for the upstream slope will be when the impoundment is empty (no restraining water force). However, as there can never be a restraining water force on the downstream slope, the critical condition for the downstream slope will be under normal pool conditions.

The stability of upstream embankment slopes was evaluated under empty (termed "end-of-construction") and normal pool conditions to reflect reasonable operating conditions and regulatory requirements; downstream slope stability was evaluated only under the more critical normal pool conditions.

Consideration of the stability of embankments composed of low-cohesion materials generally leads to a need to distinguish potential serious failures (often termed "deep-seated") from surficial, shallow sloughs which can be treated merely as a maintenance problem. Through discussions with OSM staff (OSM, 1985b), it was determined that OSM considers a failure surface greater than 5 feet in depth (measured vertically) on either the upstream or downstream slope a failure; any slump or sloughing less than 5 feet in depth can be considered as a maintenance problem.

3.8.2 Description of Analyses

The stability analyses performed used the effective stress shear strength parameters shown in Table 3-5. These parameters were developed from shear strength testing results on representative soil samples recovered from the eight sedimentation structures identified in Table 3-3. We have previously stated that the structures, identified in Table 3-3, are considered representative of the least stable of the inspected structures and therefore would be considered "worst case" examples in terms of slope stability. In analyzing structures and soils exhibiting the least stability or "worst case" condition, it is considered that the remaining structures will have factors of safety equal to or greater than the analyzed structures.

The stability of the upstream and downstream slopes of the sedimentation structures was analyzed by computer using the STABL2 program, which is capable of analyzing both circular and non-circular failure surfaces. In this set of analyses, the Modified Bishop Method of Slices was used to evaluate the stability of circular failure surfaces.

Earthquake loading conditions were considered in the stability analyses by use of conventional pseudostatic techniques and an applied horizontal acceleration of 0.04 g. For purposes of consistency, safety factors under seismic conditions have been reported for the same "critical" surface identified under static conditions. In all cases, the required slope was controlled by static, rather than seismic, considerations.

Under normal pool, steady state seepage conditions, the phreatic line within an embankment (which defines the boundary between saturated and unsaturated material) would exit above the toe of the downstream slope since the structures are homogenous embankments with no internal drains. A generally-accepted technique (Casagrande, 1937) was used to estimate the

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exit point of the phreatic line on the downstream slope under normal pool, steady state seepage conditions. In the end-of-construction situation, the entire embankment was considered to be unsaturated.

A maximum embankment height of 30 feet was chosen for the study, based on the 20-foot maximum height for OSM-regulated structures (defined as the difference between the original upstream toe and spillway crest elevations) and an assumed maximum height difference of 10 feet between the spillway and embankment crests. A minimum required slope of 2.5:1 (horizontal to vertical) for downstream slopes was chosen based on experience, long-term stability considerations, and maintenance considerations. Similarly, minimum required upstream slopes of 1.5:1 for 10-foot- high embankments, 1.75:1 for 15-foot-high embankments, and 2.0:1 for 20-foot and higher embankments were chosen.

The embankment model used in our stability analyses included embankment heights of 10, 15, 20, and 30 feet and a uniform crest width of 10 feet. The embankment height was defined as the difference in elevation of the upstream toe and embankment crest. The slope of the foundation under the embankment was assumed to be 5 percent. In the few instances where the foundation slopes were found to be greater than 5 percent, stability results and required slopes have been reported based on a higher height category; thus, in these few cases, the upstream and downstream slopes fall into different height categories.

The results of the stability analyses for the categories of structures at various embankment heights are presented in Table 3-6. The required slope, shown for each category and embankment height, is defined as the minimum slope required to satisfy the safety factor requirements discussed previously. In certain cases, when the stability analyses suggested allowable slopes steeper than the limiting values described above, the limiting value is shown.

Table 3-6

RESULTS OF STABILITY ANALYSES

		Upstream Slope					Downstream Slope		
Category	Height of Embankment (ft)	Required Slope (:1)	End of Construction Factor of Safety Static	End of Construction Factor of Safety Seismic	Long-Term Steady State Seepage Factor of Safety Static	Long-Term Steady State Seepage Factor of Safety Seismic	Required Slope (:1)	Long-Term Steady State Seepage Factor of Safety Static	Long-Term Steady State Seepage Factor of Safety Seismic
A1-A5 and C1	30	2.00	1.50	1.37	1.72	1.40	4.25	1.54	1.32
	20	2.00	1.54	1.29	1.63	1.41	4.00	1.52	1.3
	15	1.75	1.51	1.38	1.63	1.42	3.25	1.51	1.32
	10	1.50	1.53	1.39	1.71	1.50	2.50	1.52	1.35
B1-B5	30	2.00	2.00	1.82	3.04	2.52	2.50	1.54	1.38
	20	2.00	2.45	2.21	3.43	2.84	2.50	1.91	1.71
	15	1.75	2.49	2.27	3.50	2.95	2.50	2.33	2.09
	10	1.50	2.76	2.51	3.84	3.28	2.50	2.80	2.49

3.8.3 Application of Stability Results

The results of the stability analyses were applied to the existing sedimentation structure embankments in the following manner. Each structure was classified according to its embankment and foundation materials and then the existing slopes were compared to the safe slopes determined by the stability analyses. If the existing slope(s) was steeper than that deemed necessary by the stability study to meet the minimum standard, a recommendation to flatten the slope(s) was included in the remedial compliance plan.

Safe slopes for future sedimentation structures may be selected with the use of Table 3-6. The identification of embankment and foundation materials for the new structure will place it in one of the categories listed in Tables 3-1 and 3-6.

4.0 HYDROLOGY

4.1 GENERAL

For each sedimentation structure, the relationship between rainfall and runoff was determined through a hydrologic analysis of the tributary drainage area. Unit hydrographs were developed for each structure based on the characteristics of the tributary drainage area. Precipitation depths developed for the mine site were combined with the unit hydrographs to determine the inflow hydrograph for each structure. A computer program was used to develop the inflow hydrographs and determine the storage and spillway capacity requirements at each structure.

4.2 CHOICE OF DESIGN STORM

The storm events used for designing spillway capacity and storage capacity of sedimentation structures are specified in OSM regulation 30 CFR 816.46. This regulation requires that each sedimentation structure or series of structures have sufficient capacity to contain runoff from the 10-year, 24-hour storm. Each structure must also have a spillway with sufficient capacity to safely pass runoff from the 25-year, 6-hour storm.

A conservative approach has been used to design spillways for structures located in series along the same water course. When the combined active storage capacity of a particular structure and all upstream structures exceeds 20 acre-feet, the 100-year storm was used to design the spillway for that structure. When the combined active storage capacity is less than 20 acre-feet, the 25-year, 6-hour storm was used. This approach is not a regulation or policy that is applicable to future structure designs. The approach will be evaluated by Peabody Coal Company on a case by case basis to determine its applicability for future designs.

Several sedimentation structures have been designed without spillways. In these cases the structure has been sized to contain the runoff from the Probable Maximum Precipitation (PMP) event producing the largest runoff volume. (This procedure was presented and agreed upon at a meeting on August 27, 1985 attended by personnel from OSM, Peabody Coal Company, and Dames & Moore.)

The following sections describes the methods used to determine the rainfall and runoff associated with each design storm.

4.3 PRECIPITATION

Precipitation depths for the 10-year, 25-year and 100-year storms were developed using procedures and data published in the National Oceanic and Atmospheric Administration Atlas 2, (NOAA, 1973). Table 4-1 shows the precipitation frequency-depth-duration data developed for the Kayenta and Black Mesa Mines.

Table 4-1

PRECIPITATION FREQUENCY - DEPTH - DURATION
KAYENTA AND BLACK MESA MINES, ARIZONA

Duration	Precipitation (inches)		
	10-Year Storm	25-Year Storm	100-Year Storm
5 min	0.35	0.42	0.56
10 min	0.54	0.65	0.86
15 min	0.68	0.83	1.09
30 min	0.95	1.15	1.52
1 h	1.20	1.45	1.92
2 h	1.34	1.60	2.08
3 h	1.43	1.71	2.19
6 h	1.60	1.90	2.40
12 h	1.80	2.20	2.75
24 h	2.10	2.50	3.05

PMP depths were calculated using procedures from Hydrometeorological Report No. 49 of the National Weather Service (1977). Precipitation depths were developed for both the general storm and the local storm. August proved to be the month with the greatest general storm precipitation depth. The precipitation depths for each storm are summarized in Table 4-2.

Table 4-2

PROBABLE MAXIMUM PRECIPITATION
KAYENTA AND BLACK MESA MINES, ARIZONA

General Storm - August		Local Storm	
Duration (hr)	Precipitation Depth (in)	Duration (hr)	Precipitation Depth (in)
6	4.7	0.25	5.4
12	6.2	0.5	6.5
18	7.3	0.75	6.9
24	8.0	1	7.3
48	10.2	2	8.0
72	11.1	3	8.4
		4	8.6
		5	8.7
		6	8.8

4.4 RUNOFF

4.4.1 General

The inflow hydrograph for each sedimentation structure was calculated using the computer program HEC-1 Flood Hydrograph Package developed by the U.S. Army Corps of Engineers (1981). HEC-1 provides several unit hydrograph methods for modeling the hydrologic response of a watershed. It includes procedures to account for rainfall-depth-duration, precipitation losses, and unit hydrograph shape. Hydrographs can be combined and routed through single sedimentation structures or a network of several structures.

The tributary drainage area for many structures includes local depressions that will trap some part of the surface runoff. The effect of these depressions is to reduce the runoff volume and peak flowrate reaching the sediment structure. These local depressions have been ignored in the analysis of each structure. This is a conservative procedure that may result in a slight overestimate of the inflow to each structure.

Synthetic storms for each storm frequency were developed by HEC-1 using the depth-duration data from Tables 4-1 and 4-2. A triangular precipitation distribution was constructed such that the depth specified for the duration occurred during the central part of the storm. This distribution is referred to as a balanced storm.

Interception and infiltration losses were calculated using the Soil Conservation Service (SCS) curve number method (SCS, 1972). Each tributary watershed was assigned a curve number describing the drainage characteristics of the watershed. Values throughout the mine ranged from 60 to 94 where the lower value corresponds to the lowest runoff rate and the higher value to the highest runoff rate. Since the SCS method gives total precipitation excess for a storm, HEC-1 calculates the incremental excess for each time period in the hydrograph analysis as the difference between the accumulated excess at the end of the current time period and the accumulated excess at the end of the previous period.

The initial precipitation abstraction was calculated by HEC-1 using the formula:

$$IA = 0.2\ \frac{(1000 - 10(CN))}{(CN)}$$

Where CN = the SCS curve number
IA = the initial abstraction in inches.

A synthetic unit hydrograph for each structure was developed by HEC-1 using the SCS dimensionless unit hydrograph shown in Figure 4-1. The time to peak and the peak flow for the unit hydrograph were calculated based on a single parameter, lag time. Lag time is defined as the time between the center of mass of rainfall excess and the peak of the unit hydrograph. The time to peak is calculated using

$$Tp = 0.5\ (t) + LAG$$

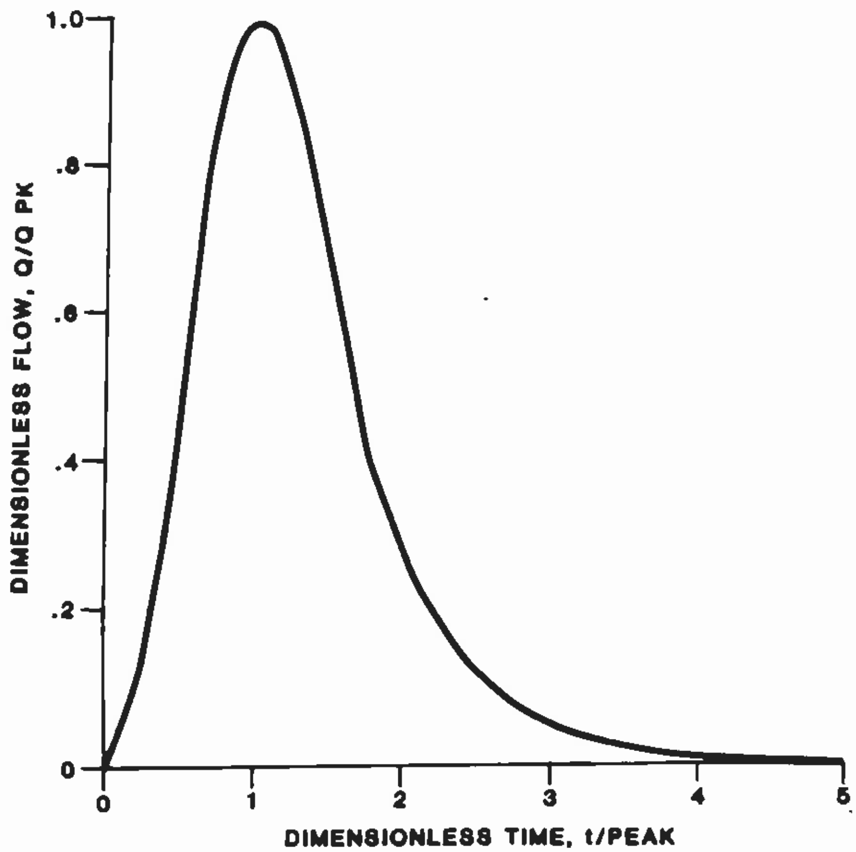
Where T_p = time to peak,
t = the storm duration
LAG = the lag time.

The peak flow of this unit hydrograph is calculated using

$$Qp = 484\ (AREA)/Tp$$

Where Qp = peak flow in cfs
AREA = the drainage area in square miles
484 = units conversion.

The synthetic storm, precipitation losses, and synthetic unit hydrograph were used by HEC-1 to calculate the inflow hydrograph to each sedimentation structure. From the above discussion, it is apparent that the HEC-1 model requires the SCS curve number, lag time, and drainage area for the watershed draining into each sedimentation pond. These parameters were developed for each sedimentation structure using the following procedures.



SCS DIMENSIONLESS
UNIT HYDROGRAPH

BY **Dames & Moore** Figure 4-1

4.4.2 Curve Numbers

SCS curve numbers were estimated for each tributary drainage area based on the cover type, percent vegetation cover, hydrologic conditions and hydrologic soil type. Several sources were used to obtain this data:

1. Cover Type -- Aerial photographs of the mine site were used to identify the existing cover type. Maps delineating the proposed mining plan were used to identify areas that will be disturbed by mining. Three general categories of cover type were used: reclaimed, undisturbed and disturbed. Further sub-classifications were made in each category as shown in Table 4-3.

   The cover type (and the tributary drainage area) for some structures will vary throughout the life of the structure as mining and subsequent reclamation occurs. For these cases, the worst condition was assumed for the hydrologic analysis. Usually the worst condition is the maximum disturbed area at the end of the mining activity and just prior to the start of land reclamation.

2. Percent Vegetation Cover -- The percent of the ground surface covered by vegetation in undisturbed areas was estimated from field inspections.

3. Hydrologic Conditions -- The hydrologic condition was directly related to the percent vegetation cover as shown in Table 4-3.

4. Hydrologic Soil Type -- Soil survey maps (Espey, Huston & Assoc., 1980; Intermountain Soils, Inc., 1985) provided the basis for determining hydrologic soil type. Tables 4-4 and 4-5 show the soil type for each soil series name.

The above data were collected and compiled for each tributary drainage area. Cover types and hydrologic soil types were delineated on topographic maps showing the drainage area contributing to each structure. A curve number was assigned to each distinct hydrologic region of the watershed, using the values in Table 4-3. An overall curve number for the watershed was derived by calculating a watershed weighted average, based on the relative acreage of each distinct hydrologic region.

Table 4-3

SCS CURVE NUMBERS
KAYENTA AND BLACK MESA MINES, ARIZONA

Cover Type	Vegetation Cover	Hydrologic Conditions	Hydrologic Soil Type B	C	D
Reclaimed Areas (Herbaceous)					
Pre-Law (1977)		poor	--	87	--
Post-Law (1977) Contoured		fair	--	81	--
Undisturbed Areas					
Pinion-Juniper					
Poor Conditions	0-30%	poor	75	85	89
Average Mine Conditions	35%	--	65	78	83
Fair Conditions	30-70%	fair	58	73	80
Sagebrush-Grass					
Poor Conditions	0-30%	poor	67	80	85
Average Mine Conditions	30%	--	60	73	79
Fair Conditions	30-70%	fair	51	63	70
Disturbed Areas					
Paved w/open ditches (including right-of-way)		--	89	92	93
Gravel roads (including right-of-way)		--	85	89	91
Dirt roads (including right-of-way)		--	82	87	89
Newly graded areas or bare ground		--	86	91	94

Sources: Revised SCS Technical Release No. 55.
Communication with Colorado and Arizona SCS State Hydrologist (8-5-85).

Table 4-4

HYDROLOGIC SOIL TYPES
BLACK MESA AND KAYENTA MINES, ARIZONA

Hydrologic Soil Type	Map Symbol*	Map Unit Name
D	1	Zyme very channery loam, 0 to 8 percent slopes
D	2	Zyme very channery loam, 8 to 30 percent slopes
D	3	Zyme-Travessilla complex, 15 to 30 percent slopes
D	4	Zyme-Travessilla complex, 8 to 15 percent slopes
B	5	Cahonavery fine sandy loam, 0 to 3 percent slopes
B	6	Begay loam, 0 to 3 percent slopes
B	7	Las Lucas sandy clay loam, 0 to 8 percent slopes
B	8	Las Lucas sandy clay loam, severely eroded, 0 to 8 percent slopes
D	9	Travessilla gravelly fine sandy loam, 0 to 8 percent slopes
D	10	Travessilla gravelly fine sandy loam, 8 to 15 percent slopes
D	11	Travessilla gravelly fine sandy loam, 15 to 30 percent slopes
C	20	Zyme-Cahona-Dulce association, 0 to 30 percent slopes
C	21	Zyme-Las Lucas complex, 0 to 15 percent slopes

Table 4-4 (Continued)

Hydrologic Soil Type	Map Symbol*	Map Unit Name
C	22	Zyme-Las Lucas-Dulce association, 0 to 30 percent slopes
D	23	Zyme-Dulce complex, severely eroded, 0 to 30 percent slopes
D	24	Zyme-Dulce association, 8 to 30 percent slopes
D	25	Zyme-Dulce-Las Lucas association, 0 to 30 percent slopes
C	26	Cahona-Zyme association, 0 to 30 percent
B	27	Begay-Las Lucas association, 0 to 8 percent slopes
C	28	Las Lucas-Zyme-Dulce complex, 0 to 8 percent slopes
D	29	Dulce gravelly find sandy loam, 0 to 30 percent slopes
D	30	Dulce-Zyme association, 15 to 30 percent slopes
C	31	Dulce-Cahona association, 0 to 30 percent slopes
C	32	Dulce-Las Lucas association, 0 to 15 percent slopes
D	33	Dulce-Las Lucas-Zyme association, 8 to 30 percent slopes
D	34	Pits and dumps

Table 4-4 (Continued)

Hydrologic Soil Type	Map Symbol*	Map Unit Name
D	35	Torriorthents, reclaimed
B	36	San Mateo silt loam, 0 to 8 percent slopes

*Map symbol refers to symbols in Espey, Huston & Assoc., 1980
Sources: Espey, Huston & Assoc., Soil Survey, 1980
Intermountain Soils Inc., 1985

Table 4-5

HYDROLOGIC SOIL GROUP
BLACK MESA AND KAYENTA MINES, ARIZONA

Soil Series	Hydrologic Group
Begay	B
Bond	D
Cahona	B
Chilton	B
Dulce	D
Las Lucas	B
Oelop	B
Pulpit	B
San Mateo	B
Sharps	B
Travessilla	D
Zyme	D
Soil A	B
Soil B	B

Source: Intermountain Soils Inc., 1985

4.4.3 Drainage Area

The tributary drainage area for each sediment structure was measured on 1 inch equals 400 feet topographic maps supplied by Peabody Coal Company (Drawing No. 84500, Sheets 1 to 26 of 26).

In some cases, mining will cause the drainage area to change during the life of the sediment structure. When the pit moves into the watershed, runoff is intercepted by the pit and diverted away from the structure. In these cases a conservative procedure was used; the structure was analyzed for the largest anticipated drainage area that will contribute runoff to the structure. This condition usually occurs at the start of mining and again during the reclamation period.

4.4.4 Time of Concentration and Lag Time

The runoff time of concentration was calculated using the following equation (USBR, 1977):

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

Where: L = length of longest water course in miles
H = watershed elevation difference in feet
T_c = time of concentration in hours

The lag time was calculated as 60 percent of the time of concentration (SCS, 1972).

5.0 HYDRAULIC ANALYSIS

5.1 GENERAL

The hydraulic analysis of each sedimentation structure was completed using the computer program HEC-1 (USACE, 1981). The inflow hydrograph was routed through the structure to determine the peak stage and peak outflow. The Modified Puls method was used for storage routing (USACE, 1981; Linsley and Franzini, 1972).

The storage capacity of each structure was analyzed assuming the structure to be empty at the start of the storm. The existing storage capacity-elevation relationship was used in the routing analysis. As the hydrograph was routed into the structure, any unused storage between the peak stage and the spillway elevation was assumed available for sediment storage. The available storage divided by the calculated annual sediment inflow rate gives the number of years of sediment storage life for the structure. When the structure has less than one year of sediment storage remaining, it will need excavation or other modifications to restore its capacity for containing precipitation runoff and the continuing sediment inflow.

The spillway capacity of each structure was analyzed assuming the structure to be full of water to the spillway elevation at the start of the storm. This is a conservative assumption that allows for the possibility of several large storms occurring prior to the spillway design storm. The existing storage capacity-elevation curve and spillway dimensions were used in the routing. The peak stage during the storm was compared with the embankment crest elevation to determine if adequate freeboard was available to safely pass the storm through the spillway.

If the hydraulic analysis showed that the structure's storage capacity or spillway capacity was inadequate, the structure was redesigned to correct the deficiency and the routing analysis repeated to assure that the redesigned structure could meet the storage and spillway capacity requirements.

Special analysis procedures were used to analyze structures in series on the same watercourse. The procedures varied depending on whether or not the combined active storage capacity of the structures exceeded 20 acre-feet.

Storage capacity for structures in series was analyzed using the 10-year, 24-hour storm. The storm was routed through each structure to determine whether or not the storage capacity was adequate. In some cases the upstream structure could not contain the storm and contributed excess flow to the downstream structure. Analyzing the two structures together showed whether or not the combined storage capacity was adequate to contain the storm.

Spillway capacity for structures in series was analyzed using the 25-year storm in cases where the combined storage capacity of the structures was less than 20 acre-feet. The storm was routed through both structures similar to the analysis for storage capacity.

In cases where the combined storage capacity of the structures was greater than 20 acre-feet, the 100-year storm was used to analyze the spillway capacity for the downstream structure and the 25-year storm was used for the upstream structure. Each structure was in turn analyzed neglecting the other structure. The downstream structure used the combined watershed area to calculate the 100-year storm runoff. This required a reevaluation of the hydrologic parameters for the combined watershed.

5.2 STORAGE CAPACITY

The storage capacity of the sedimentation structure was determined using the most current topographic information supplied by Peabody Coal Company (Plate 1 in each sedimentation structure inspection or design report). This included 1 inch equals 100 feet scale maps and surveyed elevations for the bottom of the structure, spillway, and embankment crest. Areas within contours on the topographic maps were planimetered and cumulative storage volumes calculated by the average-end area method. These volumes are presented on the volume-elevation curves in each sedimentation structure report (Plate 3 in each report).

5.3 SEDIMENT INFLOW

5.3.1 General

The sediment inflow rate for each structure was calculated in order to determine the number of years before sediment accumulation reduces the storage capacity to a point where the 10-year storm cannot be contained.

The sediment inflow rates for sheet flow and rill erosion were calculated using the SCS Universal Soil Loss Equation (USLE) (SCS, 1976). This method predicts the annual soil loss from a drainage basin using the equation:

$$A = (R)(K)(LS)(C)(P)$$

Where: A = estimated annual soil loss in tons per acre
R = rainfall factor
K = soil erodibility factor
LS = length and slope steepness factor
C = plant cover factor
P = erosion control factor

The annual soil loss in tons per acre was converted to a sediment inflow rate for each structure using the equation:

$$SI = (A)(DA)(SDR)(94)/192,400$$

Where: SI = sediment inflow rate in acre-feet per year
A = soil loss in tons per acre per year from the USLE
DA = drainage basin area in acres
SDR = sediment delivery ratio
94 = sediment unit weight in pounds per cubic foot
192,400 = units conversion factor

The sediment delivery ratio for Black Mesa and Kayenta Mines was conservatively estimated as:

SDR = 0.95 for drainage basins less than 100 acres
SDR = 0.90 for drainage basins greater than 100 acres

This conservative estimate recognizes that some sediment will be deposited in small local depressions prior to reaching the sedimentation structure and that some channel erosion may occur which is not predicted by the USLE. Sediment delivery ratios reported in the literature are often as low as 50 percent and therefore the assumptions used here are very conservative.

The average sediment unit weight was estimated at 94 pounds per cubic foot based on samples collected by Peabody Coal Company.

Data for calculating the annual soil loss rate were obtained from tables and figures contained in Conservation Planning Note No. 11 - Arizona (SCS, 1976), field inspections, and measurements made on topographic maps and aerial photographs of the mine.

5.3.2 Rainfall Factor

Conservation Planning Note No. 11 (SCS, 1976) gives average annual values of the rainfall factor (R) for Arizona. Values from the figure include the effects of snow fall where applicable. For Black Mesa and Kayenta Mines an R value of 40 was used.

5.3.3 Soil Erodibility Factor

The soil survey for the mine (Espey, Huston & Assoc., 1980; Intermountain Soils, Inc., 1985) and Conservation Planning Note No. 11 (SCS, 1976) were used to determine the soil erodibility factor. Tables 5-1 and 5-2 show the values for each soil type. The drainage area for each structure was subdivided according to soil type and a weighted average K value was determined based on relative areas.

Table 5-1

SOIL ERODIBILITY FACTORS
BLACK MESA AND KAYENTA MINES, ARIZONA

Soil Series	"K" Factor
Begay	0.43
Bond	0.43
Cahona	0.49
Chilton	0.13
Dulce	0.13
Las Lucas	0.28
Oelop	0.37
Pulpit	0.49
San Mateo	0.37
Sharps	0.49
Travessilla	0.12
Zyme	0.22
Soil A	0.04
Soil B	0.04

Source: Intermountain Soils Inc., 1985

Table 5-2

SOIL ERODIBILITY FACTORS
BLACK MESA AND KAYENTA MINES, ARIZONA

Map Symbol*	Percent of Area: Zyme	Travessilla	Cahona	Begay	Las Lucas	San Mateo	Rock	Weighted K
20	40	25	25	--	--	--	10	0.24
21	65	--	--	--	30	--	5	0.27
22	60	15	--	--	15	--	10	0.22
23	75	15	--	--	--	--	10	0.18
24	45	45	--	--	--	--	10	0.16
25	55	25	--	--	15	--	5	0.22
26	45	--	45	--	--	--	10	0.32
27	--	--	--	65	20	--	15	0.36
28	30	20	--	--	40	--	10	0.20
29	--	85	--	--	--	--	15	0.11
30	60	--	--	--	--	--	10	0.14
31	--	50	40	--	--	--	10	0.26
32	--	60	--	--	30	--	5	0.22
33	20	50	--	--	25	--	5	0.22
34 - Pits	--	--	--	--	--	--	--	0.22
35 - Reclaimed		--	--	--	--	--	--	0.42
36	--	--	--	--	--	90	10	0.33
K Values	0.22	0.13	0.49	0.43	0.43	0.43	0.37	

*Refers to symbols used in Espey, Huston & Assoc., 1980
Sources: Espey, Huston & Assoc. Soil Survey, 1980.
Intermountain Soils Inc., 1985.

5.3.4 Length and Steepness Factor

The length and steepness factor was determined using tables and figures in Conservation Planning Note No. 11 (SCS, 1976). The slope length in feet and slope in percent were measured on 1" = 400' scale topographic maps (Drawing No. 85400 Sheets 1 to 26 of 26). An area weighting was used to calculate a weighted factor for each drainage basin.

5.3.5 Cover Factor

The cover factor was calculated using data from Conservation Planning Note No. 11 (SCS, 1976). Portions of that data assumed applicable to the mine site are reproduced in Table 5-3.

Table 5-3

COVER FACTOR
BLACK MESA AND KAYENTA MINES, ARIZONA

Type and Height of Raised Canopy[1]	Canopy Cover[2] %	Type[3]	Percent Ground Cover				
			10	20	40	60	80
Reclaimed (no appreciable canopy)			0.45	0.24	0.15	0.09	0.043
Sagebrush-Grass (0.5m fall height)	25	W	0.36	0.20	0.13	0.082	0.041
	50	W	0.26	0.16	0.11	0.075	0.039
	75	W	0.17	0.12	0.09	0.067	0.038
Pinion-Juniper (2m fall height)	25	W	0.40	0.22	0.14	0.085	0.042
	50	W	0.34	0.19	0.13	0.081	0.041
	75	W	0.28	0.17	0.12	0.077	0.040
Disturbed Area			-------------1.00-------------				

*Source: Conservation Planning Note No. 11 (SCS, 1976)

1 Average fall height of waterdrops from canopy to soil surface.

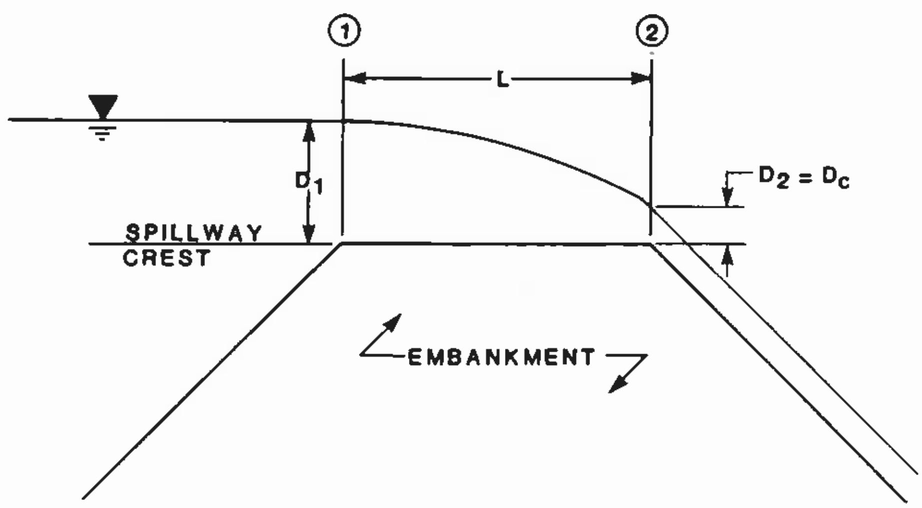
2 Portion of surface area that would be hidden from view by canopy in a vertical position.

3 W = cover at surface is mostly broadleaf herbaceous plants with little lateral-root network near the surface and/or undecayed residue.

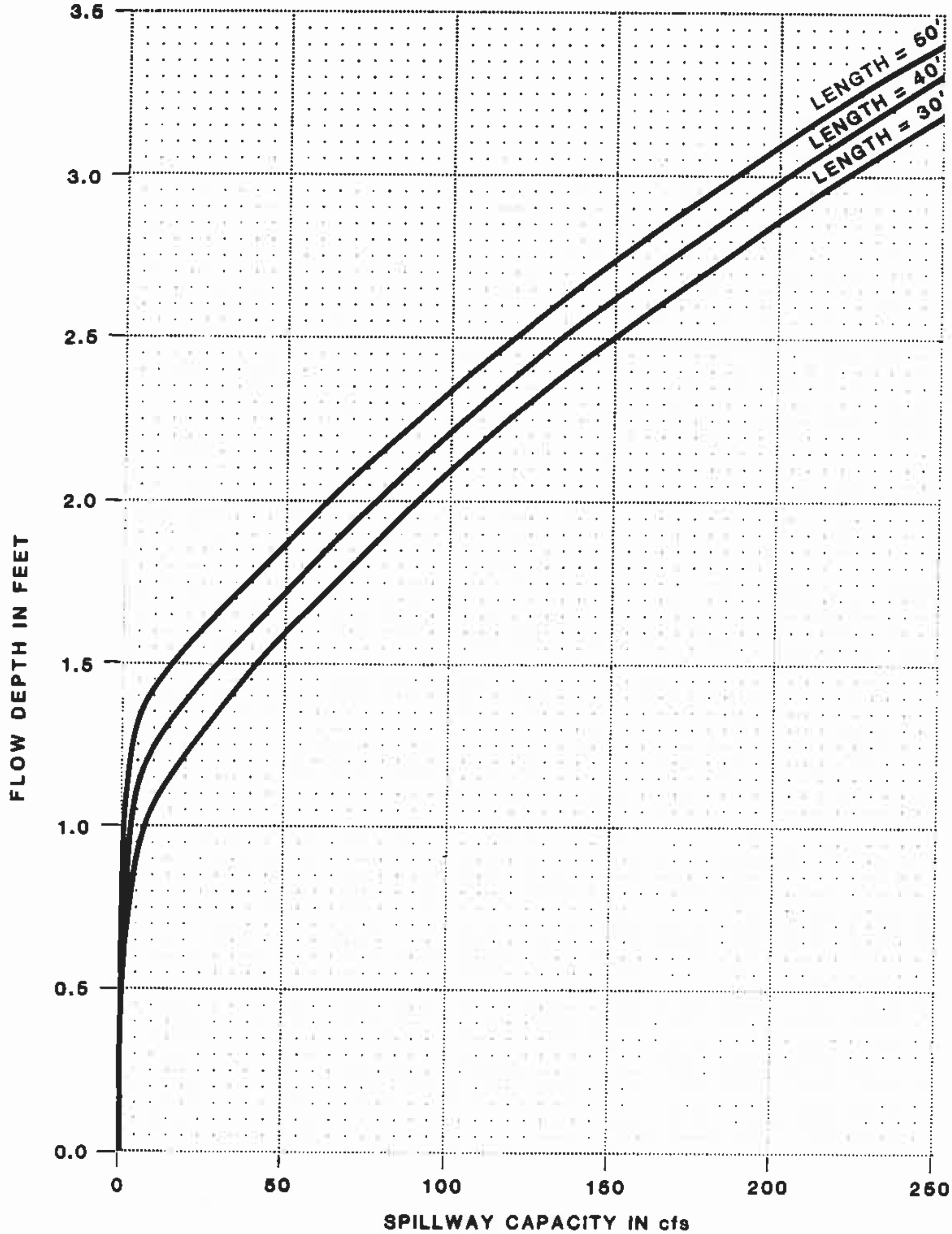
5.4 SPILLWAY CAPACITY

Most sedimentation structures on the mine site have a trapezoidal open channel spillway. Some structures have a CMP spillway. Spillway capacity curves for the open channel spillways were prepared for typical standard dimensions. Figures 5-1 through 5-12 show the calculated capacity for widths ranging from 15 feet to 100 feet, lengths ranging from 30 feet to 50 feet and Manning's "n" values of 0.035 and 0.040.

The open channel spillway capacity curves were developed from a hydraulic analysis of flow over a horizontal, trapezoidal shaped spillway crest illustrated below:



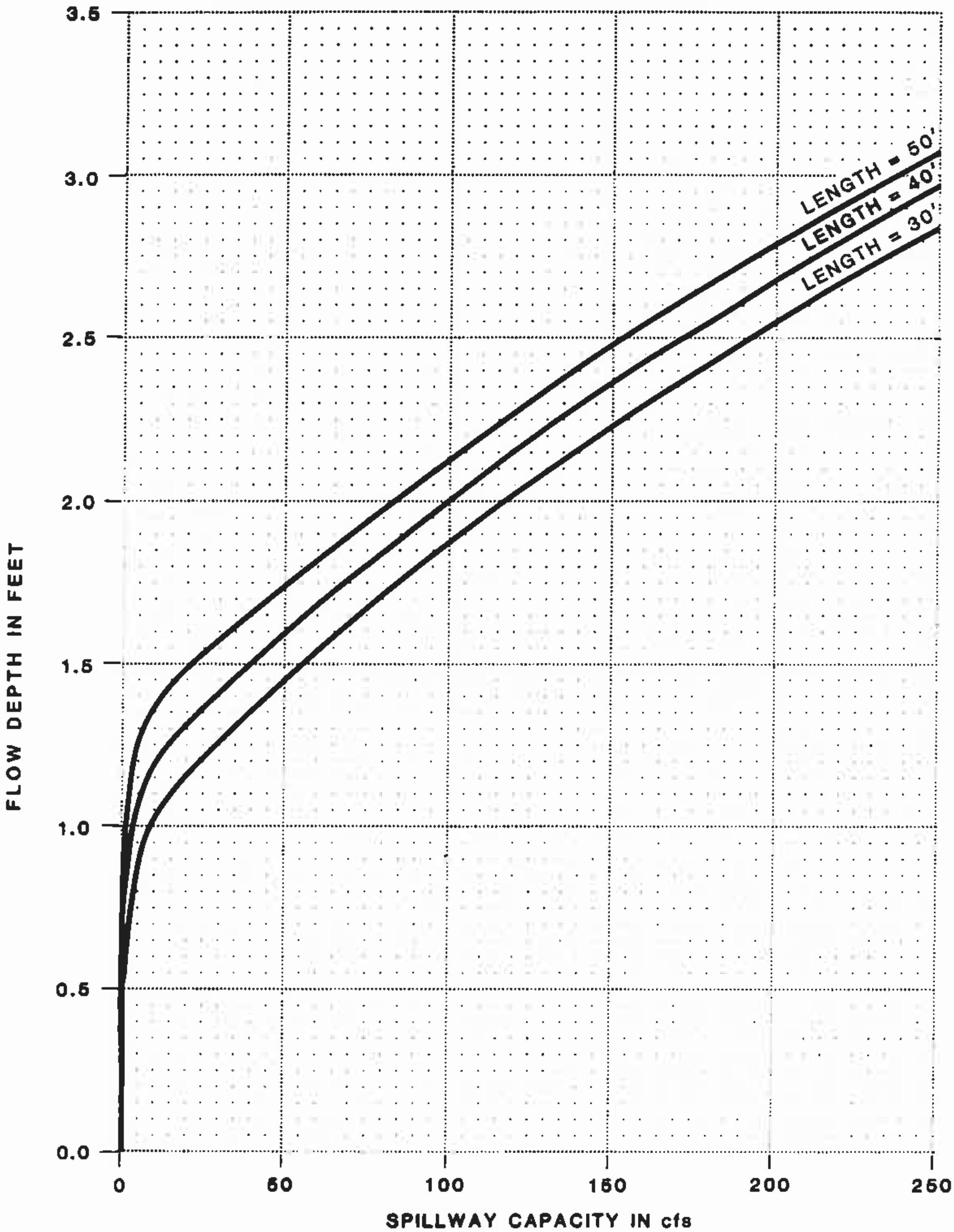
SPILLWAY PROFILE



TRAPAZOIDAL CHANNEL
SIDE SLOPE = 2H/1V
MANNINGS N = 0.040

SPILLWAY CAPACITY
15' WIDE CHANNEL

BY **Dames & Moore** Figure 5-1



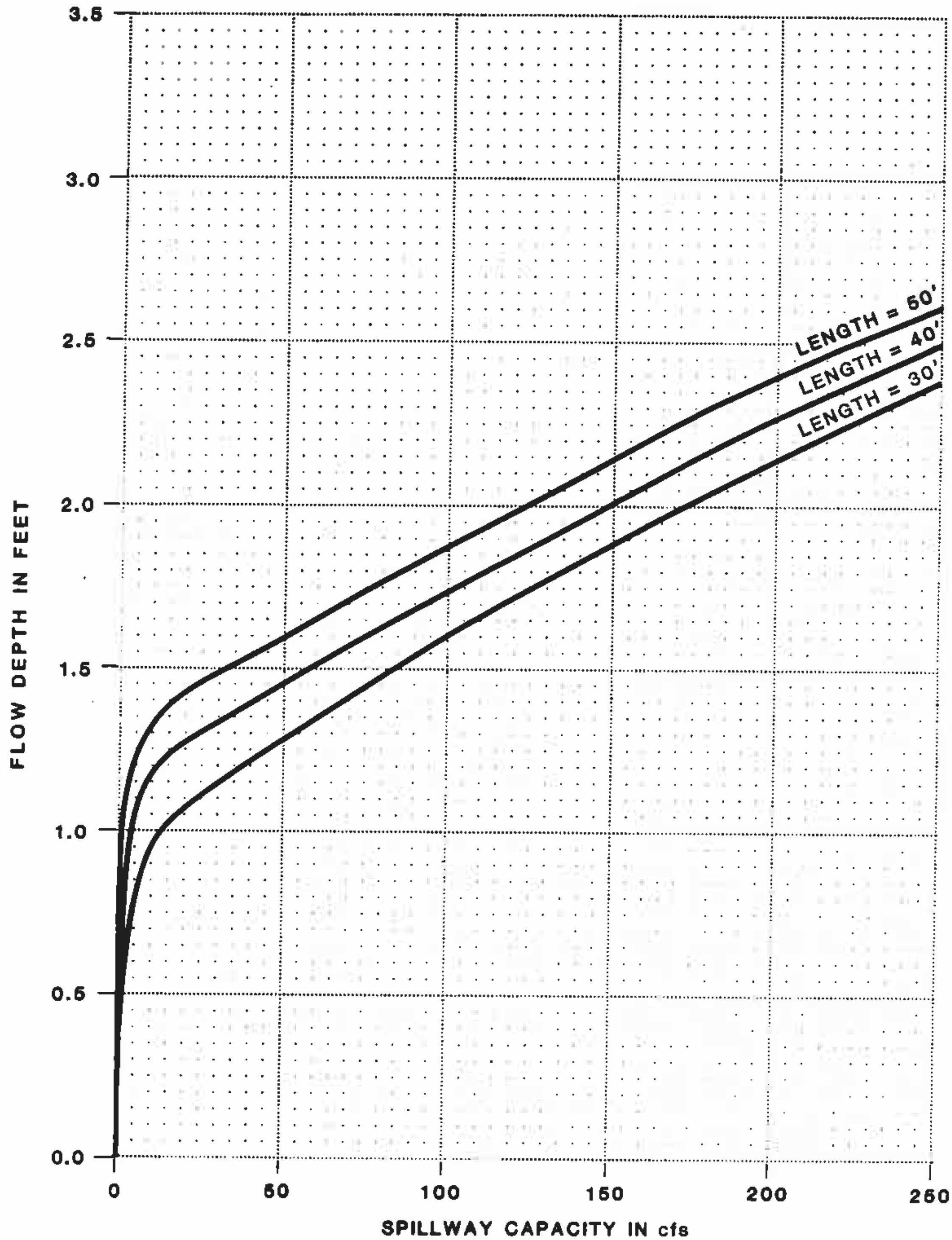
TRAPAZOIDAL CHANNEL

SIDE SLOPE = 2H/1V

MANNINGS N = 0.040

SPILLWAY CAPACITY
20' WIDE CHANNEL

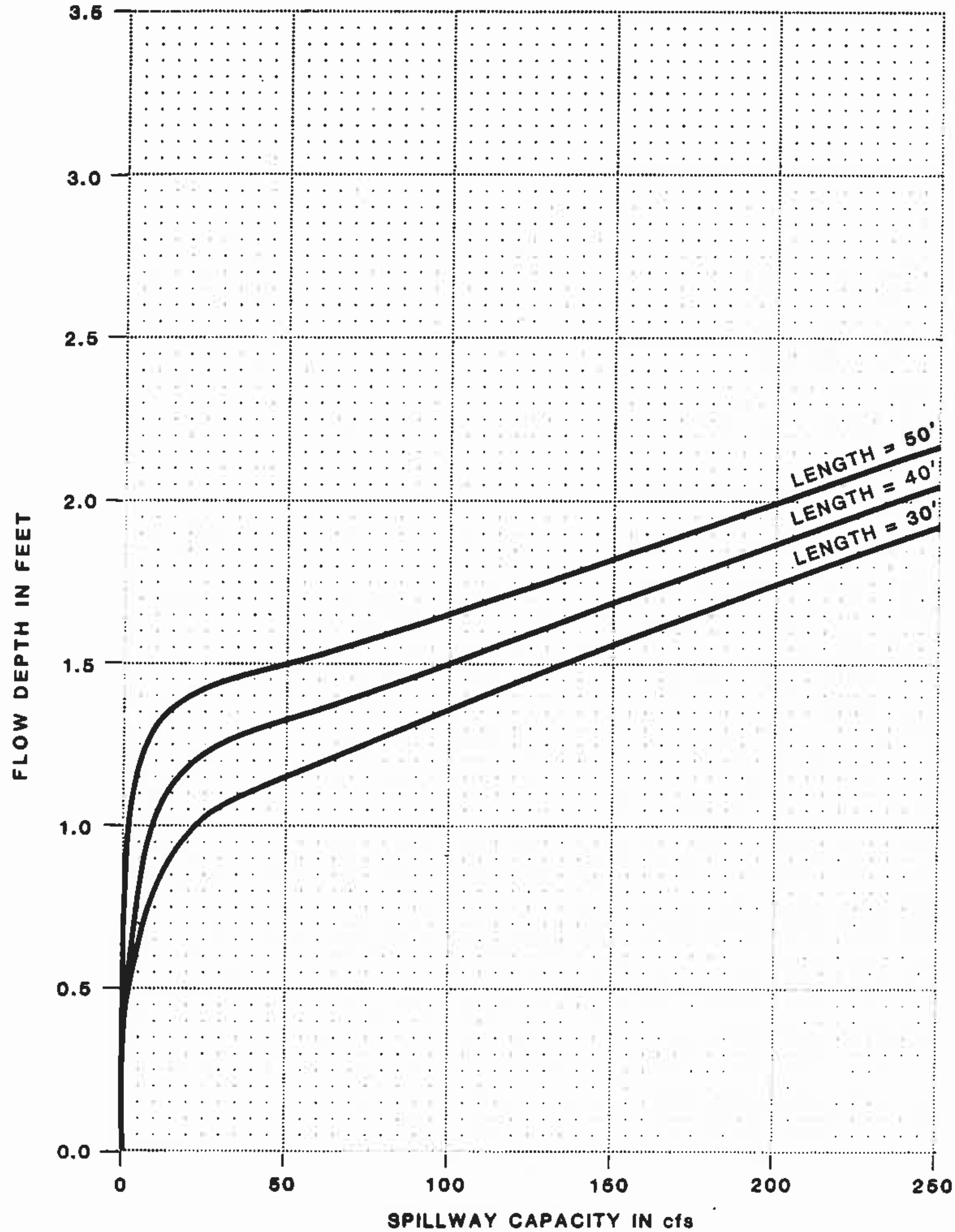
BY **Dames & Moore** Figure 5-2



TRAPAZOIDAL CHANNEL
SIDE SLOPE = 2H/1V
MANNINGS N = 0.040

SPILLWAY CAPACITY
30' WIDE CHANNEL

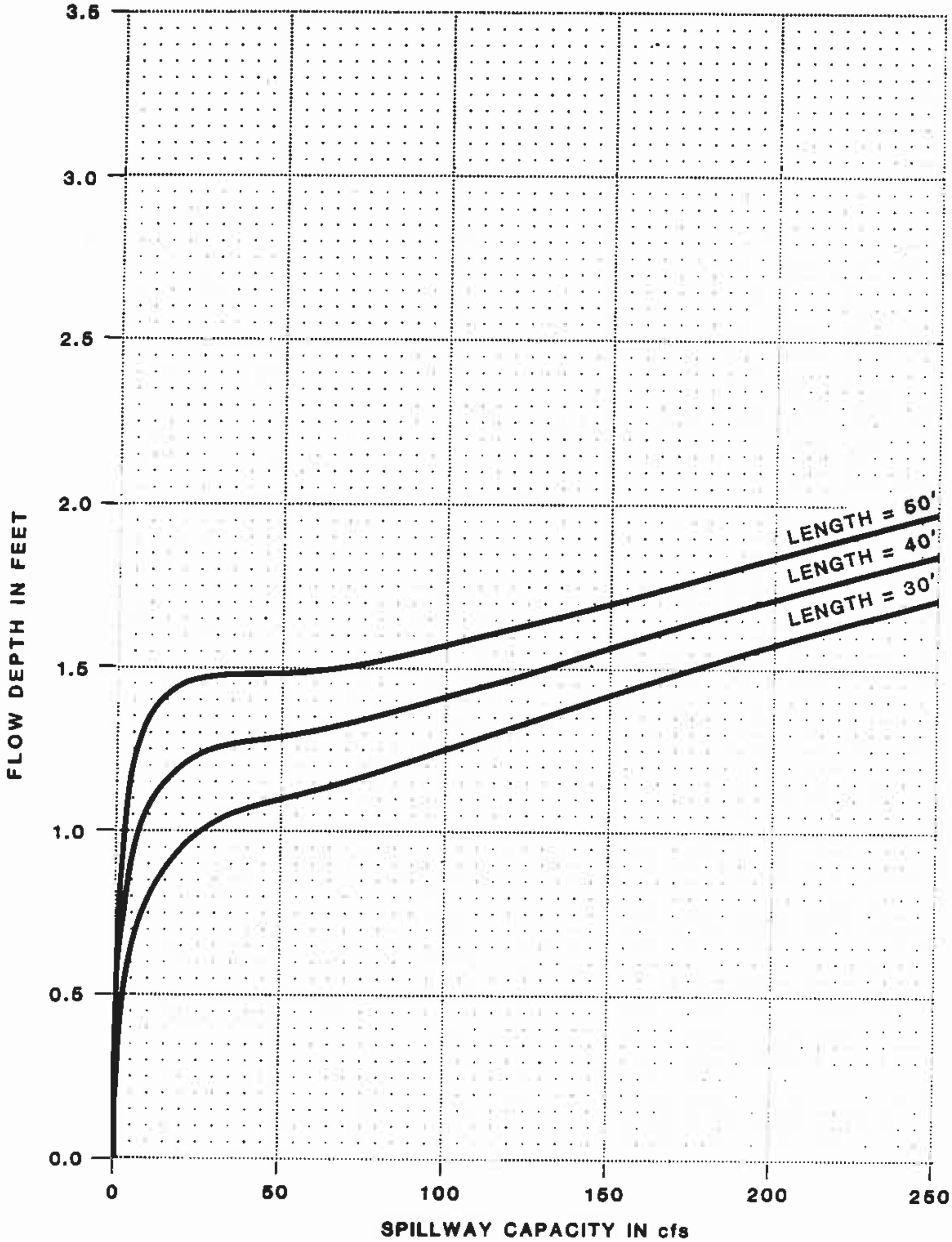
BY Dames & Moore Figure 5-3



TRAPAZOIDAL CHANNEL
SIDE SLOPE = 2H/1V
MANNINGS N = 0.040

SPILLWAY CAPACITY 50' WIDE CHANNEL

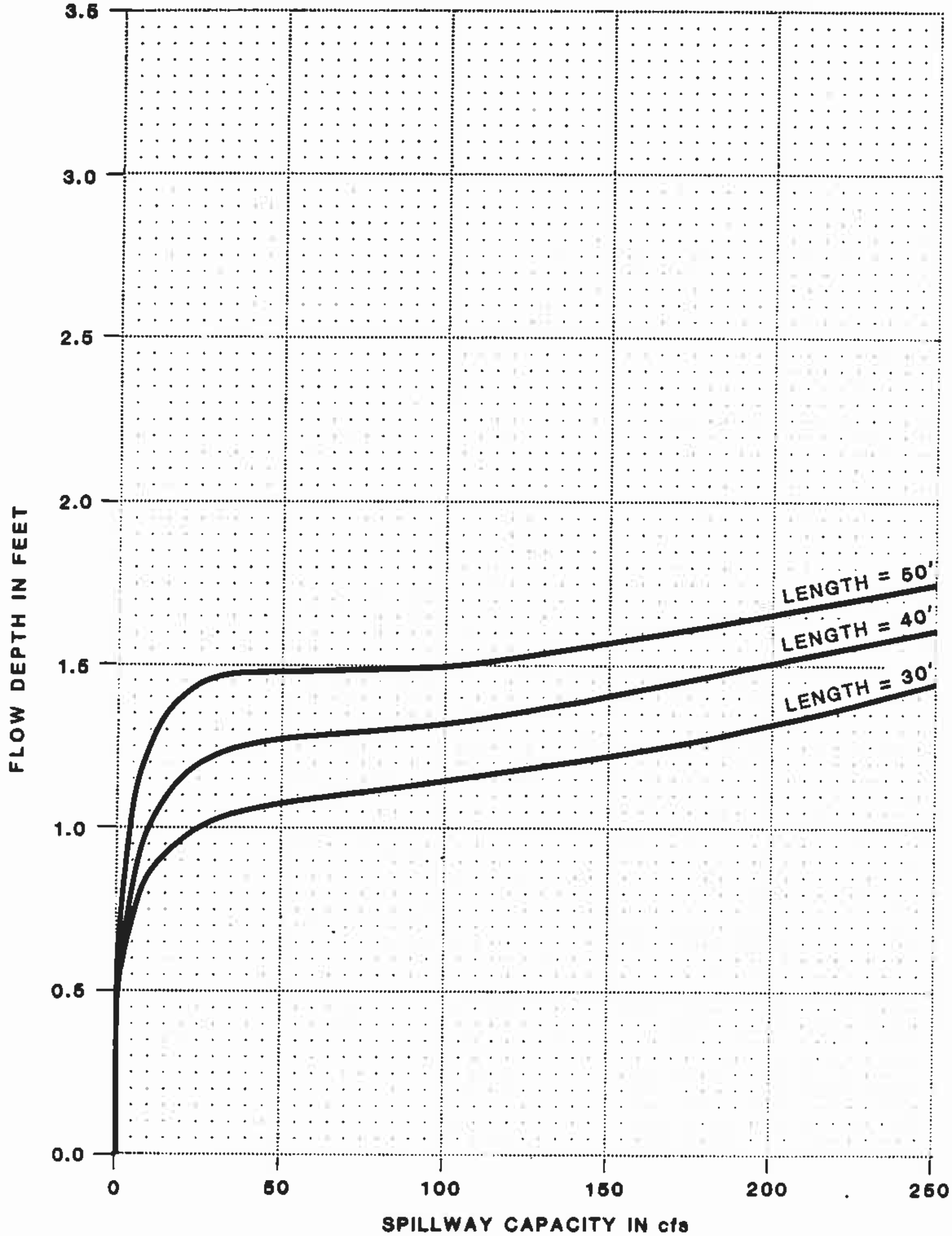
BY Dames & Moore Figure 5-4



TRAPAZOIDAL CHANNEL
SIDE SLOPE = 2H/1V
MANNINGS N = 0.040

SPILLWAY CAPACITY 65' WIDE CHANNEL

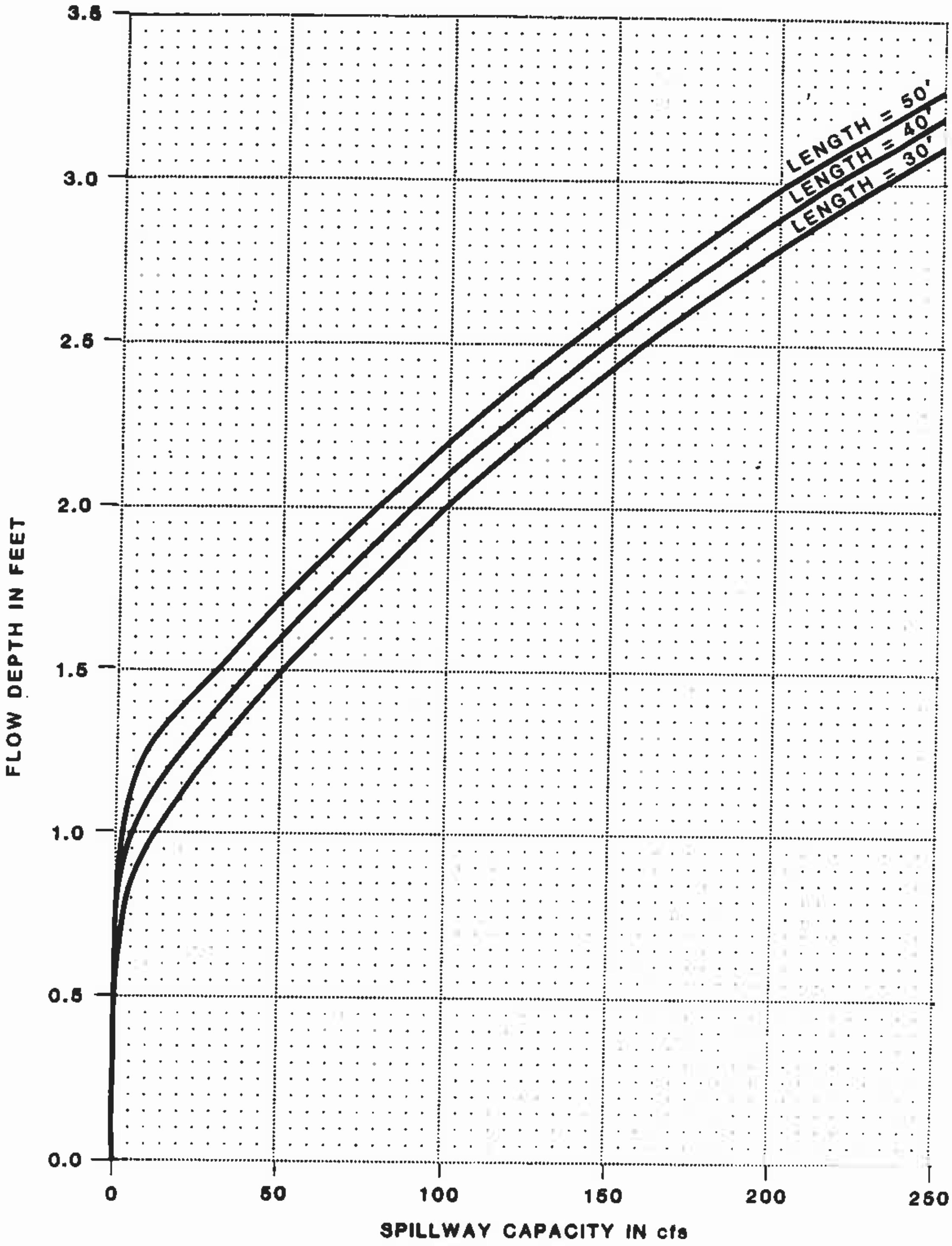
BY **Dames & Moore** Figure 5-5



TRAPAZOIDAL CHANNEL
SIDE SLOPE = 2H/1V
MANNINGS N = 0.040

SPILLWAY CAPACITY
100' WIDE CHANNEL

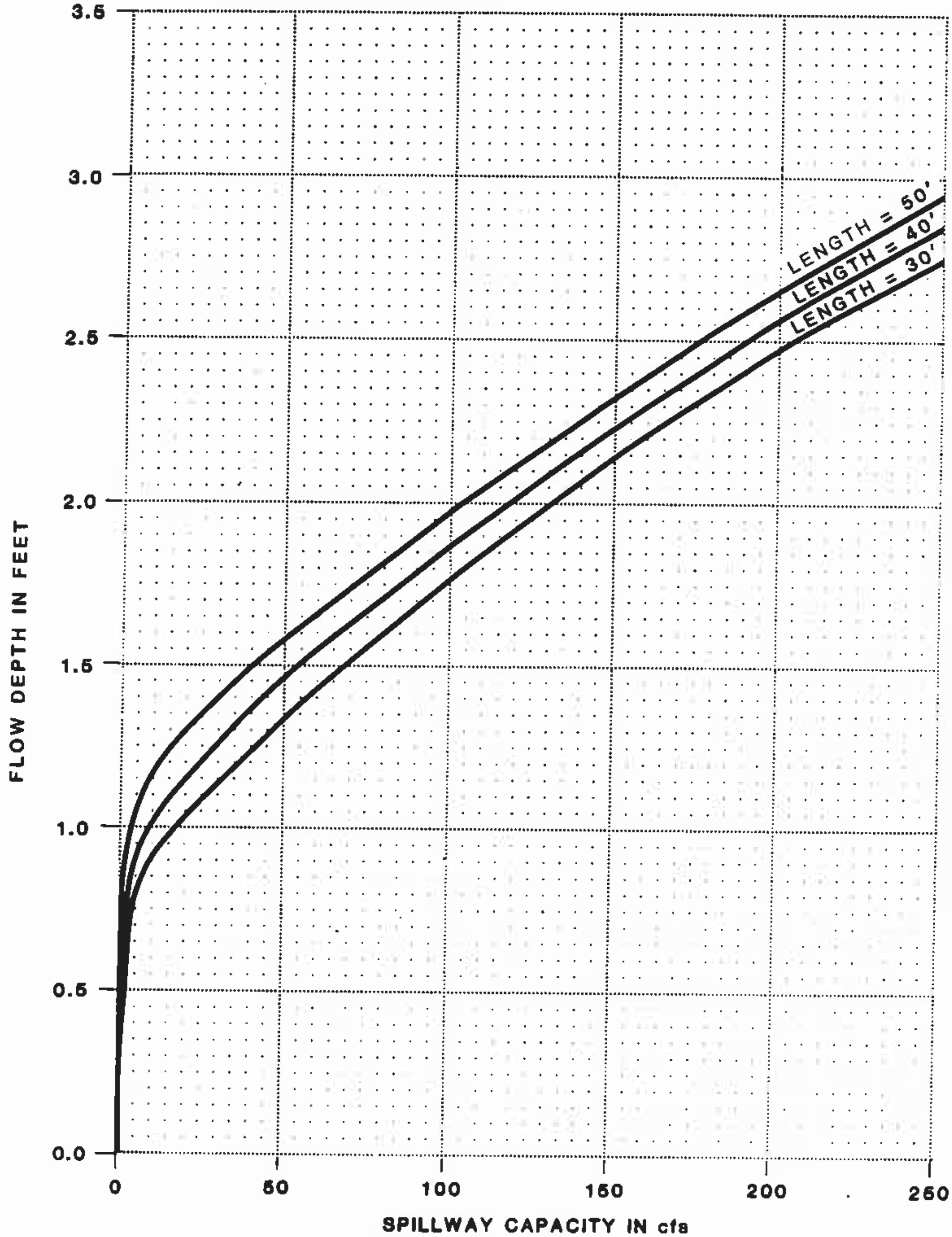
BY **Dames & Moore** Figure 5-6



TRAPAZOIDAL CHANNEL
SIDESLOPE = 2H/1V
MANNINGS N = 0.035

SPILLWAY CAPACITY
15' WIDE CHANNEL

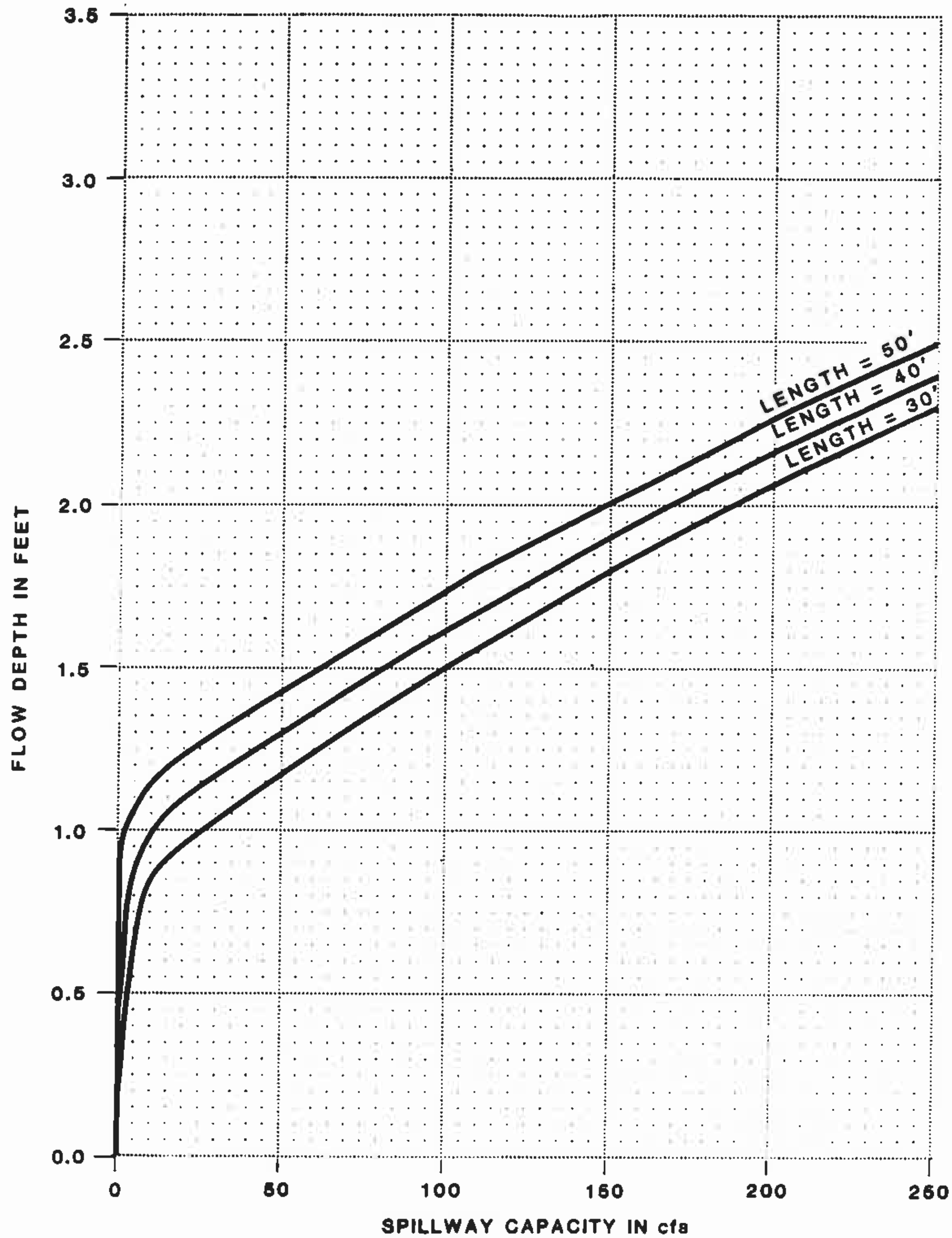
BY **Dames & Moore** Figure 5-7



TRAPAZOIDAL CHANNEL
SIDESLOPE = 2H/1V
MANNINGS N = 0.035

SPILLWAY CAPACITY
20' WIDE CHANNEL

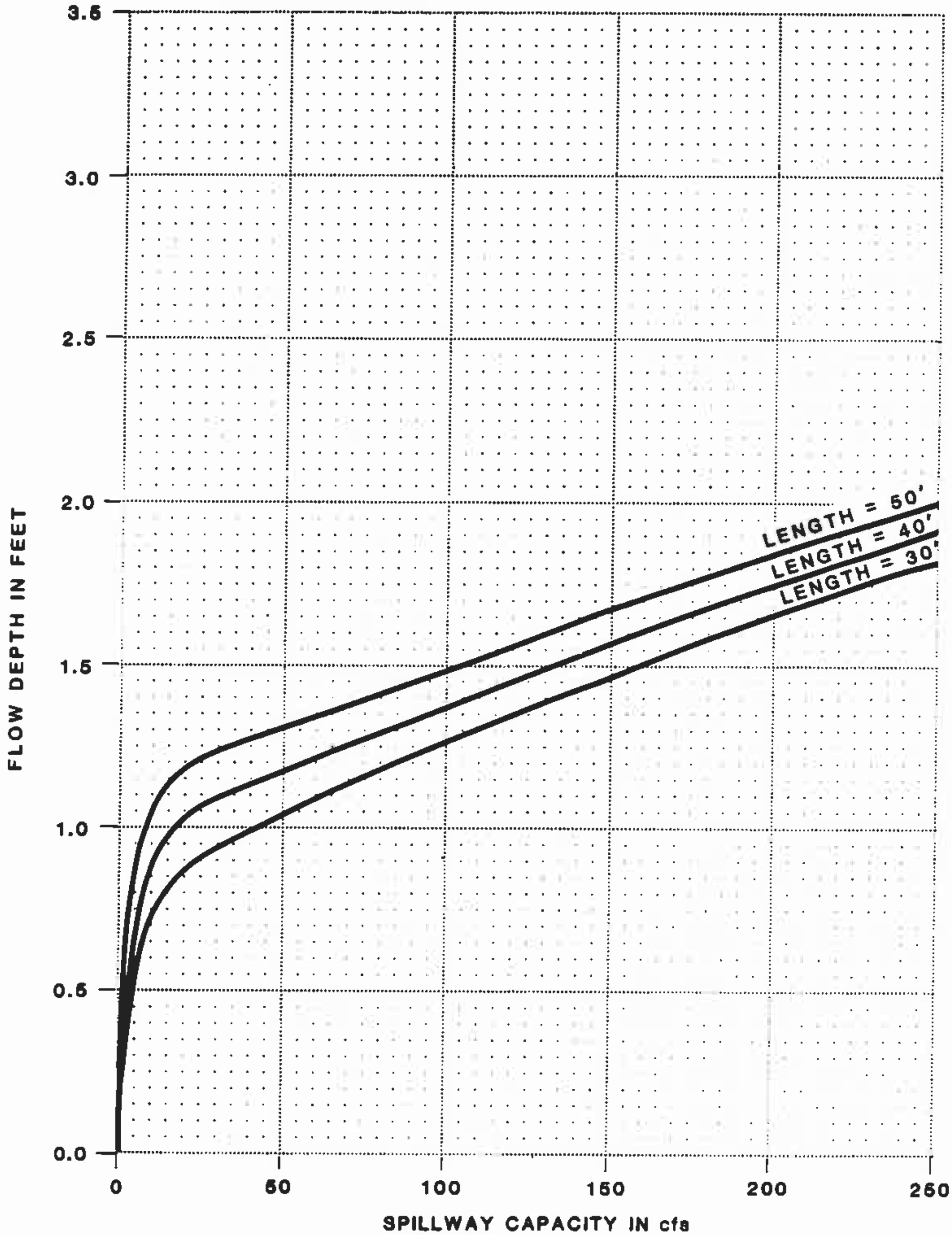
BY Dames & Moore Figure 5-8



TRAPAZOIDAL CHANNEL
SIDESLOPE = 2H/1V
MANNINGS N = 0.035

SPILLWAY CAPACITY 30' WIDE CHANNEL

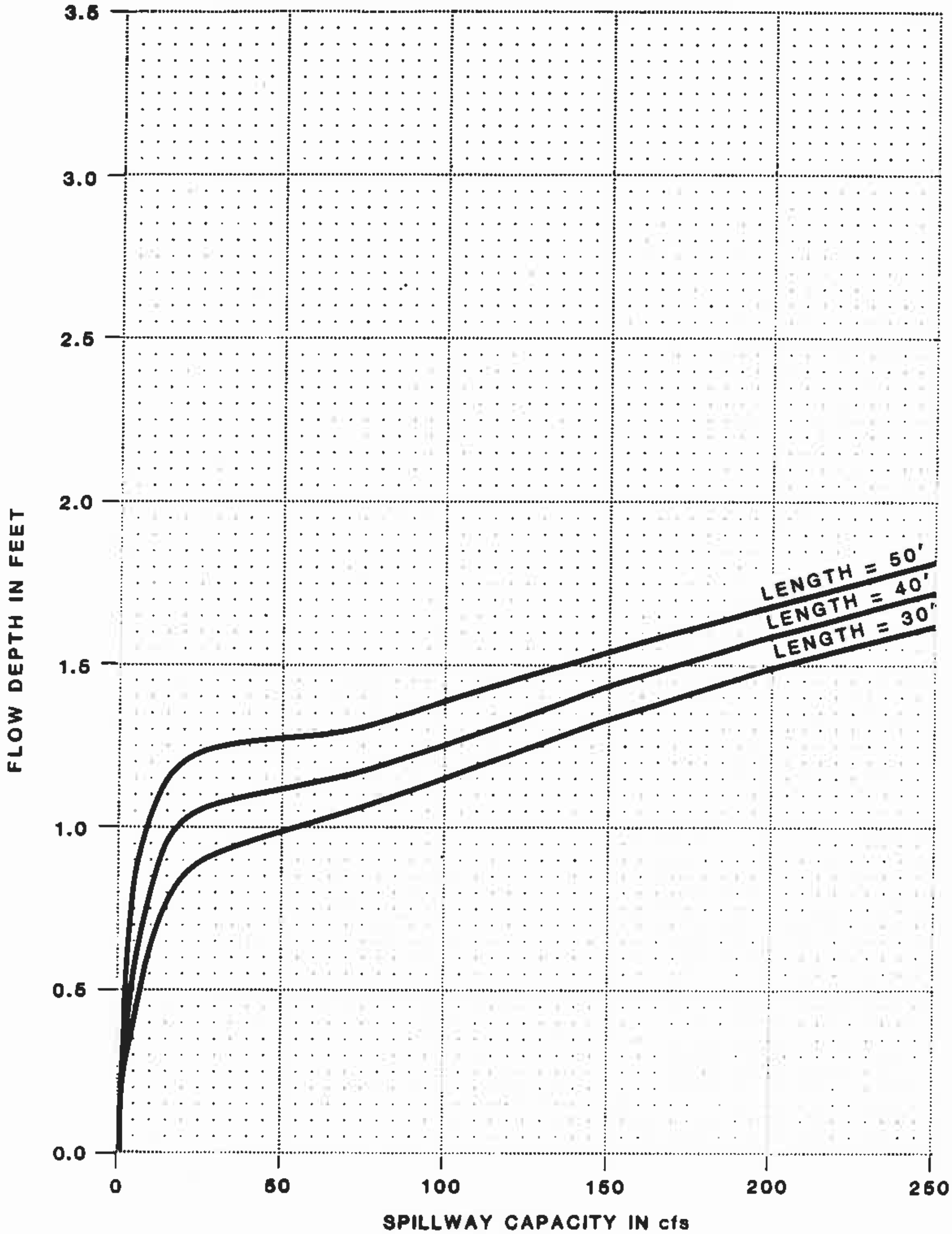
BY **Dames & Moore** Figure 5-9



TRAPAZOIDAL CHANNEL
SIDESLOPE = 2H/1V
MANNINGS N = 0.035

SPILLWAY CAPACITY 50' WIDE CHANNEL

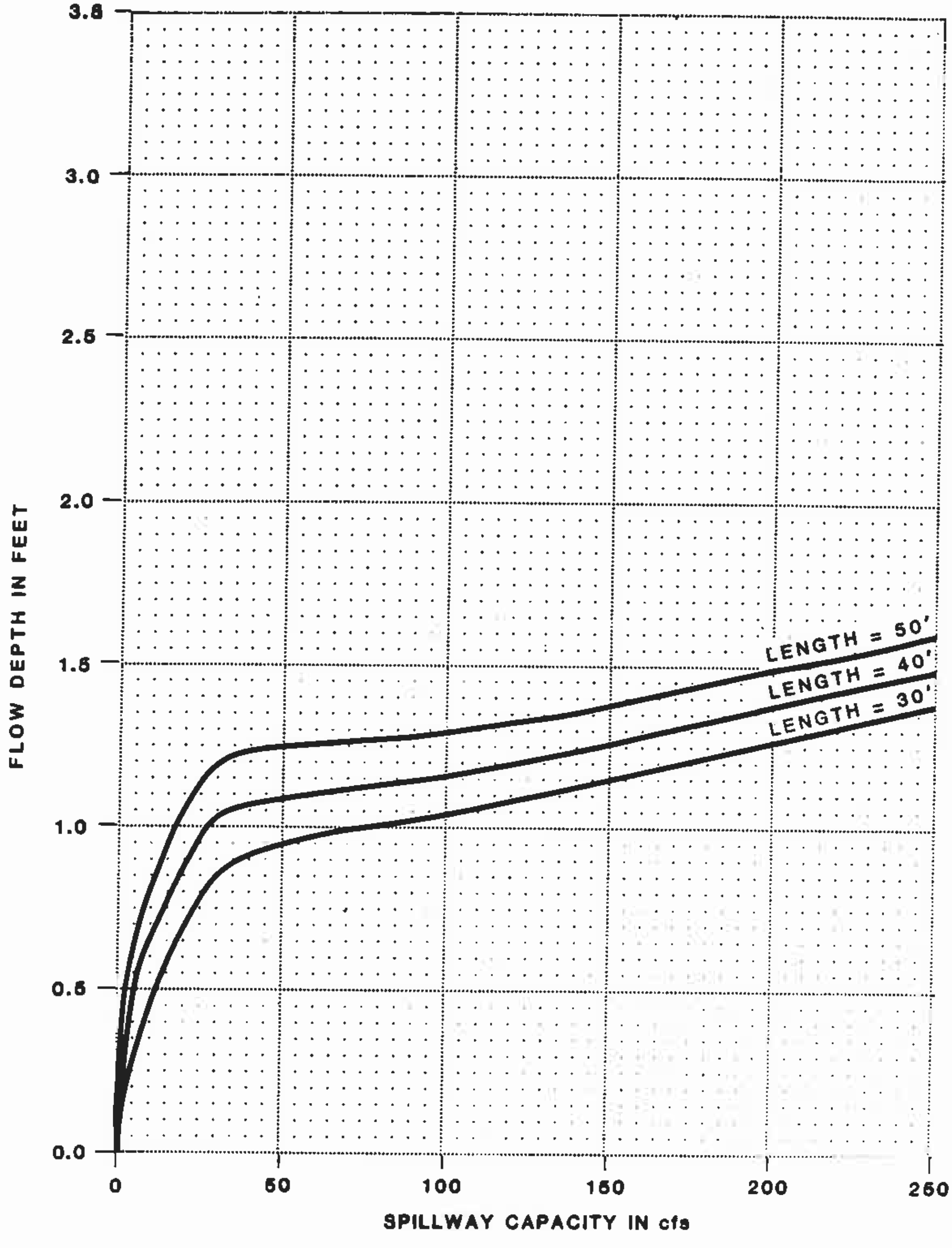
BY Dames & Moore Figure 5-10



TRAPAZOIDAL CHANNEL
SIDESLOPE = 2H/1V
MANNINGS N = 0.035

SPILLWAY CAPACITY
65' WIDE CHANNEL

BY **Dames & Moore** Figure 5-11



TRAPAZOIDAL CHANNEL
SIDESLOPE = 2H/1V
MANNINGS N = 0.035

SPILLWAY CAPACITY 100' WIDE CHANNEL

BY **Dames & Moore** Figure 5-12

At (2) the flow passes through critical depth, calculated using:

$$Q = \frac{5.671[(b + zD_c)D_c]^{1.5}}{[b + 2zD_c]^{0.5}}$$

Where Q = flow in cfs

b = trapezoidal channel bottom width in feet

z = channel side slope (H/V)

$D_c = D_2$ = critical depth in feet

At (1) the depth of flow was calculated using the energy equation, neglecting the energy head at 1 because the velocity is low.

$$D_2 = D_1 + \frac{(V_1)^2}{2g} + h_1$$

Where D = depth of flow in feet

V = flow velocity in fps

g = gravitational constant

h_1 = head loss in feet

The head loss in the spillway channel was calculated using Manning's equation for the average conditions in the channel:

$$h_1 = \frac{L\,n^2}{2.21}\left[\frac{V_1^{\,2}}{R_1^{\,1.33}} + \frac{V_2^{\,2}}{R_2^{\,1.33}}\right] x\ (0.5)$$

Where h_1 = head loss in feet

n = Mannings "n"

R = hydraulic radius in feet

V = velocity in fps

L = length of spillway in feet

Calculations using the above listed methods were selectively checked against charts developed by the Soil Conservation Service (SCS, 1968). The calculation methods indicated results that were more conservative than those obtained from the SCS charts. Spillway capacity curves for CMP spillways were calculated using standard hydraulic capacity charts (U.S. Department of Commerce, 1964).

6.0 HYDRAULIC DESIGN

6.1 GENERAL

Structures with inadequate spillway capacity or storage capacity were redesigned to bring them into compliance with the regulations. A spillway was considered inadequate if it could not pass the spillway design storm with a minimum of 1 foot between the maximum water surface and the embankment crest elevation. A spillway was also considered inadequate if it did not have erosion protection or a spillway outflow channel capable of safely carrying the spillway discharge to a natural channel downstream.

Storage capacity was considered inadequate if the 10-year, 24-hour storm and two years of sediment inflow could not be contained with no spillway discharge.

Remedial compliance plans to bring the spillway and/or storage capacity into compliance with the regulations were developed based on the best available topographic information. Conditions encountered during construction may make it impossible or impractical to carry out the modifications exactly as shown in the report for each sedimentation structure. For example, bedrock may be encountered in areas designated for excavation; or the actual topography may vary from the map. In these cases the recommended remedial compliance plan may need alteration in order to minimize construction costs and difficulties. Data from the hydrologic and hydraulic analyses provide the basis for revising the plan. In all cases the storage capacity must be adequate to contain the runoff and sediment inflow (2 years minimum) calculated in the hydrologic analysis. The spillway must have adequate capacity and freeboard to carry the spillway design flow calculated in the hydraulic analysis.

If modifications to the proposed plan are made, a new hydraulic analysis must be completed to determine outflow hydrographs and reservoir peak stage. Modifications to the proposed pond excavation, or the proposed spillway and outflow channel alignment or slope, will change the peak storage, flow rate and velocity. The results of the new analysis must be used to resize the spillway and/or storage capacity and the spillway erosion protection.

Several types of remedial action were specified for sedimentation structures. The following sections describe general procedures and criteria used in preparing remedial compliance plans.

6.2 STORAGE CAPACITY

Plans for increasing storage capacity used a combination of excavating the impoundment and/or raising the spillway and embankment. Excavation was assumed at maximum slopes of 3H:1V. Embankment construction follows the stability requirements described in Section 3.9.

6.3 SPILLWAY CHANNEL

Trapezoidal spillway channels were sized to pass the design storm with a minimum freeboard of 1 foot. In all cases the channels are lined with geotextile and either riprap or gravel as shown on Figures 6-1 and 6-2. Discussions between OSM and Dames & Moore on November 8, 1985 led to the agreement that in cases where the calculated critical velocity for a gravel lined spillway was less than 4 feet per second (fps), gravel lining would be adequate to protect the channel from erosion. When the velocity exceeds 4 fps, riprap lining is required.

The flow velocity in the spillway channel varies from a minimum value at the upstream end to a maximum value at the downstream end. The flow passes through critical depth at the grade break between the horizontal spillway channel and the sloping outflow channel. The calculated critical velocity at this point was used to determine the type of lining (gravel or riprap) required to protect the spillway. Riprap lining was sized using the design chart on Figure 6-3. Gravel lining (3" maximum size, D_{50} = 2") was assumed stable up to a velocity of 4 fps.

6.4 OUTFLOW CHANNEL

Outflow channels were located to carry flow from the spillway to the natural channel below the toe of the embankment. Flow depth and velocity in the outflow channel were calculated using Manning's equation. The channels were assumed to have either riprap lining with a Manning's "n" of 0.040 or gravel lining with an "n" of 0.035 (USBR, 1977 and Chow, 1959).

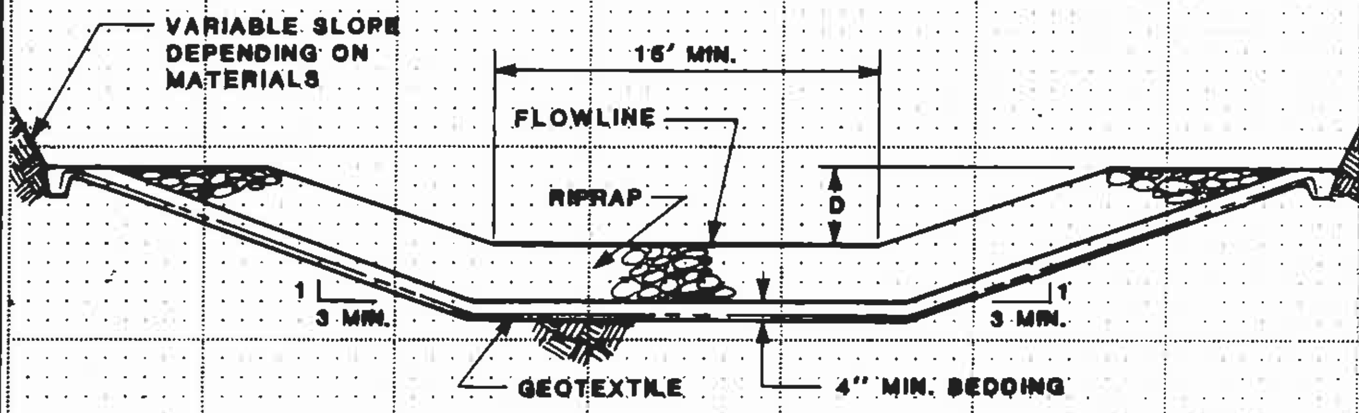
The discussions mentioned previously between OSM and Dames & Moore also led to the agreement that outflow channels with a calculated normal velocity less than 4 fps could be lined with geotextile and gravel to protect against erosion. If the calculated velocity exceeds 4 fps riprap protection is required. Figures 6-1 and 6-2 show typical riprap and gravel lined outflow channels.

In many cases the outflow channel flowline has several slope changes in order to conform to the natural topography. The steepest slope in the outflow channel produces the highest velocity for sizing the riprap or gravel protection. The flattest slope produces the deepest flow depth for sizing the channel depth.

Channel design depths were set equal to the calculated flow depth plus 1 foot. All design depths were rounded to the nearest 0.5 foot. This procedure gives a freeboard ranging from 0.75 to 1.25 feet.

6.5 STILLING BASIN

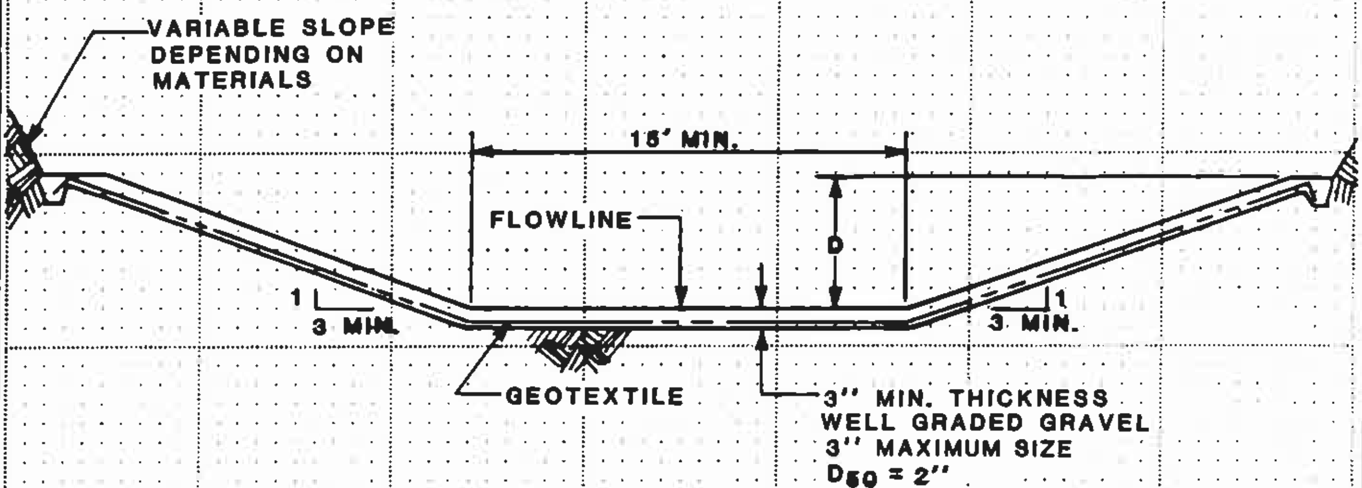
Stilling basins were designed for spillways where large discharges and high flow velocities may cause severe erosion at the end of the outflow channel. A hydraulic jump-type stilling basin lined with riprap was sized using procedures described in Design of Small Dams, (USBR, 1977). The conjugate depth for the hydraulic jump was estimated using Figure 268 in Design of Small Dams with an estimated head loss of 30 percent. The tailwater depth below the stilling basin was estimated using Manning's equation for a trapezoidal channel with dimensions similar to the outflow



TYPICAL
SPILLWAY AND
OUTFLOW CHANNEL
CROSS SECTION

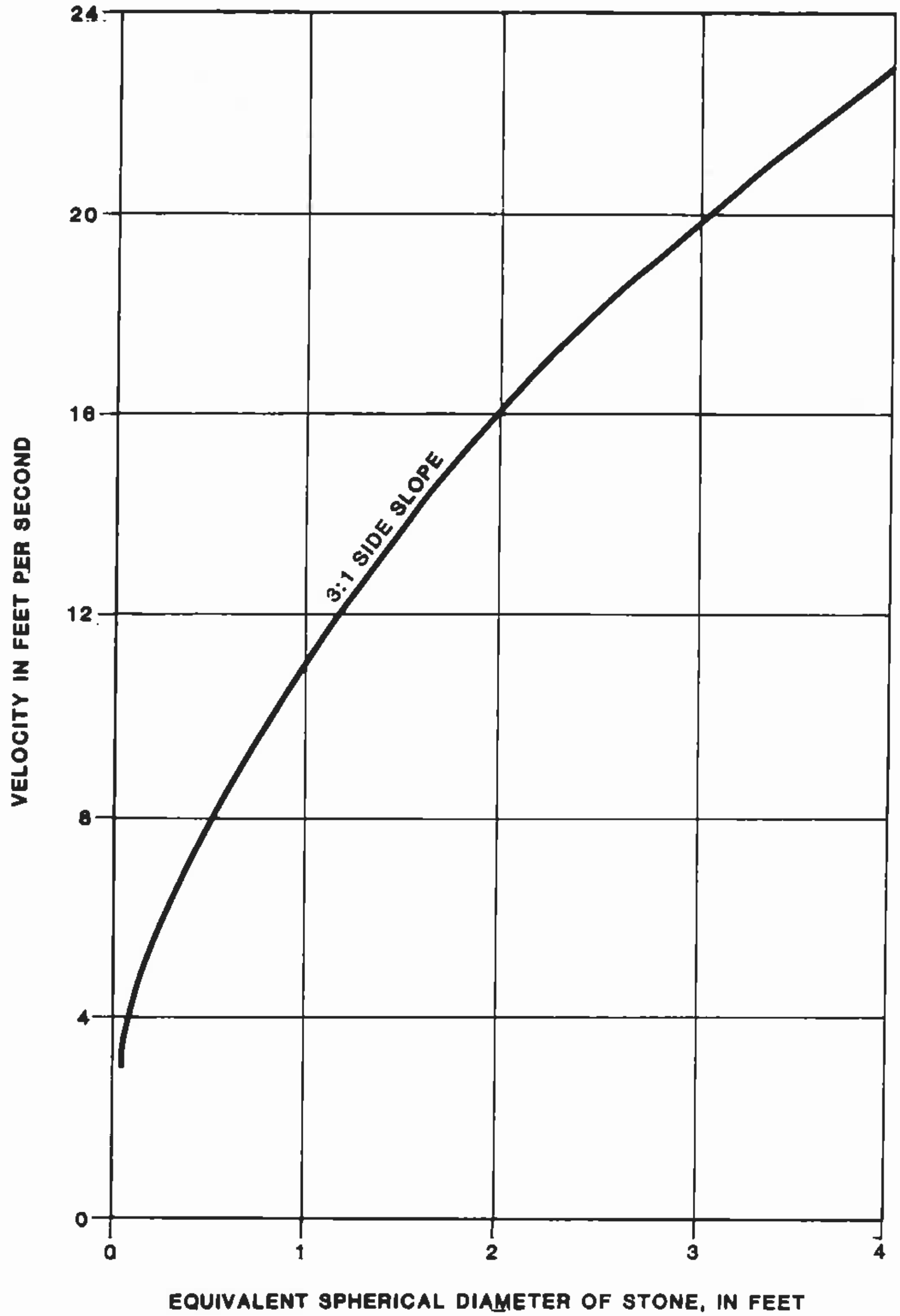
BY Dames & Moore

Figure 6-1



SPILLWAY AND
OUTFLOW CHANNEL
CROSS SECTION

BY Dames & Moore Figure 6-2



FOR STONE WEIGHING 165 LBS.
PER CU. FT.

RIPRAP DESIGN CHART

ADAPTED FROM REPORT OF
THE SUBCOMMITTEE ON SLOPE
PROTECTION, AMERICAN
SOCIETY OF CIVIL ENGINEERS
PROCEEDINGS, JUNE 1948

BY **Dames & Moore** Figure 6-3

channel. The length of the stilling basin was estimated based on research reported in Hydraulic Design of Spillways (USACE, 1965), where basin lengths of five times the hydraulic jump conjugate depth proved adequate. The depth of the stilling basin below the natural stream bed elevation was calculated by subtracting the tailwater depth from the hydraulic jump conjugate depth.

Riprap lining for the stilling basin was sized using the calculated velocity in the outflow channel leading to the stilling basin. The minimum height of riprap along the sidewalls of the stilling basin was set equal to the hydraulic jump conjugate depth plus freeboard. Freeboard was calculated using the following empirical equation from Design of Small Dams (USBR, 1977).

$$FB = 0.1\ (V + d_2)$$

Where FB = freeboard in feet

V = velocity of flow entering the basin in feet per second

d_2 = hydraulic jump conjugate depth in feet

Freeboard values were rounded to the nearest half foot.

Stilling basins were not designed for cases where: 1) the flow is very low or 2) the natural channel has a slope equal to or greater than the spillway outflow channel. In these cases, local erosion could occur where the outflow channel enters the natural channel at an angle to the natural flow direction. If the natural channel does not have natural armoring to protect against erosion, it will be necessary to provide riprap protection. The need for riprap and the location should be determine based on conditions encountered during construction. If the natural channel has a sandy bottom and sides, riprap will be required. If the natural channel consists of cobbles, riprap will not be required.

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