TECHNICAL RESOURCES CENTER - P.C.C.
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REPORT ON

FOUNDATION INVESTIGATION

PROPOSED ROCK AND EARTH FILL EMBANKMENT ACROSS TRINIDAD ARM OF GATUN LAKE, PANAMA CANAL ZONE

FOR

TUDOR ENGINEERING CO.

SAN FRANCISCO, CALIFORNIA

SHANNON AND WILSON, INC.

SOIL MECHANICS AND FOUNDATION ENGINEERS

SEATTLE, WASHINGTON

MARCH 1963

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SECTION I

INTRODUCTION AND SUMMARY

1-1. AUTHORIZATION.

This report is submitted in accordance with the terms and provisions of the Contract for Engineering Services, between Tudor Engineering Company and Shannon and Wilson, Inc., dated 21 August 1962.

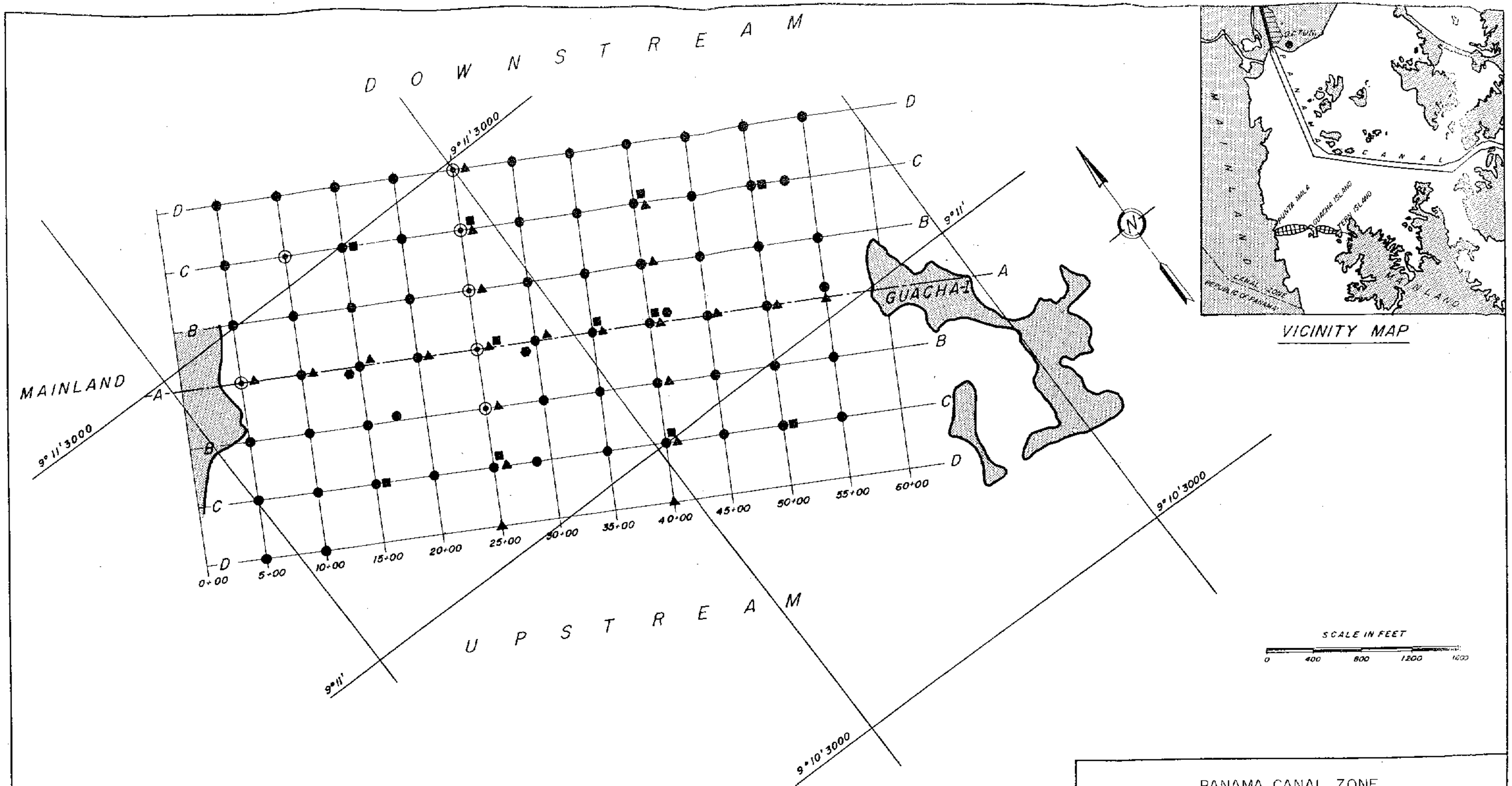
1-2. INTRODUCTION.

This report presents the results of a foundation investigation conducted at the site of the main section of the earth and rock fill dam proposed to be constructed across the Trinidad Arm of Gatun Lake, near Escobal on the Atlantic side of the Isthmus of Panama. The proposed dam will extend from Punta Mala on the west shore of Gatun Lake to Guacha Island, thence to Tern Island and from Tern Island to the mainland on the east shore of Gatun Lake. (See Vicinity Map, Figure 1-1.) The main portion of the Dam at which the foundation investigation was undertaken is that which extends from near Punta Mala to Guacha Island.

The purpose of the foundation investigation was to explore and evaluate the shear strength and consolidation properties of the foundation materials in order than an earth and rock fill dam section could be designed with an adequate factor of safety against failure. None of the foundation explorations treated in this report deals with foundations for proposed concrete structures such as locks and spillways.

1-3. SUMMARY.

The foundation material underlying the site of the proposed Dam consists predominantly of normally consolidated organic clay interbedded with layers



LEGEND:

- ● PROBING.
- ⊙ PROBING AND FIELD VANE SHEAR TEST.
- ▲ CLASSIFICATION BORING. UNDISTURBED SAMPLES FOR FIELD LABORATORY TESTING AND CLASSIFICATION.
- ■ UNDISTURBED SAMPLE BORING. SAMPLES SENT TO SEATTLE FOR LABORATORY TESTING.
- ● EXISTING BORING. BORING MADE BY PANAMA CANAL COMPANY.

PANAMA CANAL ZONE
PROPOSED DAM ACROSS TRINIDAD ARM OF GATUN LAKE

PLAN OF SITE EXPLORATIONS BETWEEN MAINLAND AND GUACHA IS.

W-62-147
FEBRUARY, 1963

Shannon and Wilson
Soil mechanics and foundation engineers
Seattle

FIG. 1-1

and seams of fibrous peat. The clay and peat materials comprise the principal constituents of the Atlantic Muck formation. This formation exists over most of the site and forms a thick mantle of sediment over more competent residual soil and bedrock materials. These sediments are relatively weak and highly compressible and they will, consequently, control the stability and resultant settlement of the proposed Dam.

Several engineering studies were undertaken to investigate the rate of gain in shear strength due to consolidation of the foundation materials and the probable settlements at various sections of the Dam. Results of these studies indicate that a moderate gain in foundation shear strength will occur near the base of the fill during the construction operations. Since values of in-situ shear strength are low, this strength gain becomes significant in designing the the dam section. The settlement study revealed that substantial settlements will continue for the design life of the Dam and that periodic additions to the rolled-filled portion of the dam, therefore, will probably be required to maintain the design crest elevation.

We recommend that various measuring devices and gages be installed to permit measurement of pore water pressures as well as vertical and horizontal displacements occurring from application of embankment loads. These devices will permit confirmation of some of the assumptions and evaluations included in this report and will also serve to indicate areas of potential foundation instability. In addition, studies should be initiated during the first two years of construction to determine the physical properties and characteristics of the typical fill materials to be used in the Dam embankment. Knowledge of such properties and characteristics as grading, saturated unit weight, particle angularity and hardness are particularly important for fill material evaluation.

It is further recommended that, because of the exceptionally weak nature of the foundation materials, provision be made for frequent inspection and review of construction progress and problems by highly qualified consultants.

SECTION II

SITE DESCRIPTION AND FIELD EXPLORATIONS

2-1. SITE DESCRIPTION.

The site under consideration for the proposed dam crosses the Trinidad Arm of Gatun Lake between the mainland near Punta Mala and Guacha Island a distance of approximately 6000 feet. The area of investigation included a strip 3000 feet wide bisected by the axis and extending the entire length of the axis.

The lake in this area occupies a broad, very flat-floored valley having walls which slope inward toward the center of the valley at grades ranging from 30 to 40 percent. Water depths are in the order of from 70 to 80 feet over about 80 percent of the site. The valley floor is at an average elevation of about +10 feet Precise Level Datum.

Prior to construction of Gatun Dam and the subsequent filling of Gatun Lake, the area was heavily vegetated. Inasmuch as no attempt was made to clear the lake bottom prior to filling, much of the previously exposed vegetation remains. Many of the trees and stumps have been knocked down by recent dragging operations, however many are still standing in the inaccessible areas. It is estimated that the present cover averages one sizeable tree for each 90,000 square feet of area.

2-2. FIELD EXPLORATIONS.

Field explorations were conducted to determine the characteristics and types of subsurface materials in the Mainland to Guacha reach of the proposed damsite. A preliminary geophysical exploration had been performed in February 1962 by Geo-Recon, Inc., of Seattle, Washington, using seismic and electrical resistivity techniques. Subsequent to these preliminary investigations, the detailed field exploration program presented herein was conducted. It included

a series of 77 cone penetrometer borings, 24 classification borings, 11 undisturbed borings, and 6 field vane shear test borings. (See Figure 1-1 for boring locations.)

Sampling and field explorations were performed from three barge-mounted drill rigs provided and operated by the Core Drilling Unit of the Dredging Division of the Panama Canal Company. All field operations were supervised by qualified inspectors provided by Shannon & Wilson, Inc.

Marker buoys placed by the Surveying Division of the Panama Canal Company provided the initial locations for each boring. Final locations were determined using sextant methods and are estimated to be accurate to within ten feet.

Water depths at all borings were measured using lead lines. Daily hydrographic reports obtained from the Meteorlogical and Hydrographic Branch of the Panama Canal Company were used to convert depths from lake level to Precise Level Datum elevations.

Details of field exploration procedures are given in Appendix B of this report.

2-3. GEOPHYSICAL EXPLORATIONS.

The preliminary subsurface data were obtained using refraction seismic and electrical resistivity surveys along the axis of the proposed damsite. These surveys were performed by Geo-Recon, Inc., of Seattle, Washington, and were completed on 19 February 1962. Four borings were made by the Panama Canal Company to correlate the geophysical data with specific material types. The results of these geophysical surveys, which aided in planning the subsequent exploration program, are contained in Appendix A of the June 1962 report titled "Study and Report on Increasing the Water Supply of the Panama Canal" by Tudor Engineering Company.

2-4. CONE PENETROMETER TESTS.

A series of 77 cone penetrometer borings was made to determine relative consistencies of the subsurface overburden materials. The cone penetrometer consisted of a three-inch diameter, 65-degree steel cone welded on a two-foot

section of N-rod. The penetrometer was driven by a hammer weighing approximately 140 pounds using a 30-inch drop.

Two methods were used in performing the cone penetrometer tests: (1) Continuous drive without casing and (2) a case-and-drive method, both limited to a predetermined maximum number of blows per foot as a practical control. The second method eliminated excessive skin friction on the drive rods, but tended to give abnormally low blow counts at the start of each ten foot drive, probably due to casing overwash. Terminal blow counts in both methods ranged from about 40 to 60 blows per foot and 33 borings and from about 100 to 120 blows per foot at the remaining borings.

2-5. CLASSIFICATION BORINGS.

Classification borings were performed at 500 foot intervals along the Axis and along cross sections at stations 25+00 and 40+00. The sampling interval in these borings was approximately every five feet, with the boring depth generally extending ten feet below the overburden materials. Undisturbed sampling methods were employed using three-inch O.D. Shelby tubing in an Osterberg piston sampler and in a standard fixed-piston sampler. In cases where the materials were too firm to sample using undisturbed sampling methods, both a split-spoon drive sampler (Standard Penetration Test) and an M-series NX core barrel were used. All undisturbed samples were classified, and waxed in their Shelby tubes, immediately after detachment from the sampler.

2-6. UNDISTURBED BORINGS.

Eleven undisturbed borings were made to provide undisturbed samples for testing in the Shannon & Wilson, Inc. laboratory in Seattle, Washington. Representative boring locations were chosen over the dam site area; three on the Axis, two along station 40+00, two along station 25+00, two along station 15+00, and two along station 50+00. One undisturbed boring was made in the early stages of the field exploration program at station 25+00 on the Axis. The remaining boring locations were selected after review and evaluation of the available cone penetrometer and classification boring data. The borings were made using the same sampling techniques used in the classification borings

except sampling was done with both the 3-inch and 5-inch Osterberg piston samplers. All undisturbed borings extended approximately 50 feet below the mudline except at station 25+00 on the Axis where the boring extended to bedrock.

2-7. FIELD VANE SHEAR TESTS.

Six field vane shear borings were made to determine the in-place shear strength of the soft overburden materials. The forces required to rotate the vane apparatus for both the undisturbed and remolded soil conditions were measured through the use of steel vanes inserted into the soil. Bearings placed approximately every twenty feet were used to minimize friction created by the vane rods which extended from lake level down through the casing to the bottom of the boring. In each case the force required to overcome friction in the bearing system was measured and subtracted from the total force required to turn the vanes in soil.

An attempt was made to use a strain-controlled apparatus (0.1 degree rotation per seond) but was unsuccessful due to gear assembly break down caused by wave action induced by canal traffic.

Some difficulty was also encountered when apparent metal fatigue caused some vanes to shear-off in the hole after several tests. These problems minimized the number of field vane shear tests performed.

SECTION III

GEOLOGY

3-1. SUBSURFACE GEOLOGY.

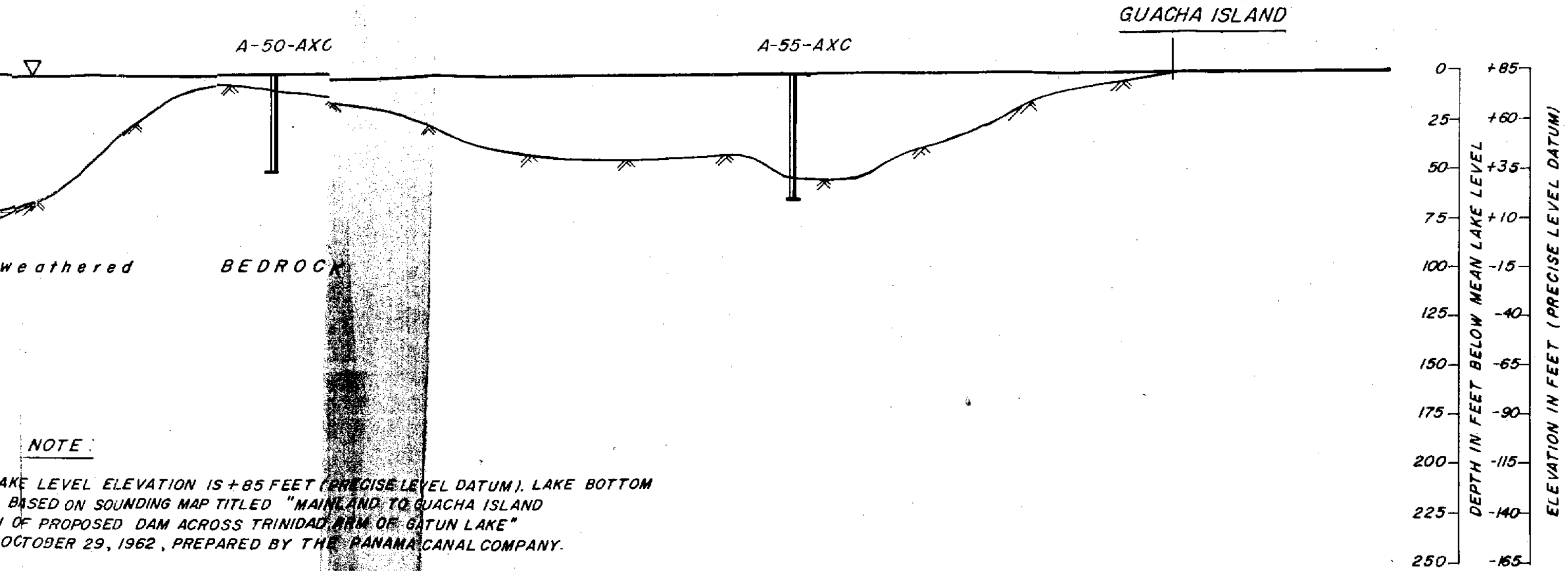
The geology within the damsite area is relatively simple insofar as its bearing on the proposed dam is concerned, and in general relects the result of marine and fluvial erosion and deposition. Basically, soft, horizontally stratified sedimentary overburden materials, generally designated Atlantic Muck, overlie an old erosional surface formed on what originally was hard, cemented sedimentary bedrock materials.

3-2. ATLANTIC MUCK FORMATION.

Since the Atlantic Muck formation was deposited over an old irregular erosional surface, the thicknesses of the muck materials are extremely variable but generally are thickest in the old meandering stream channel and thinnest on the old valley walls. The maximum thickness of Atlantic Muck formation noted in the investigation was about 160 feet measured from the lake bottom. Bedrock outcrops above about elevation +15 feet (P.L.D.).

Generalizing, the Atlantic Muck formation consists of very soft to medium soft organic clay, usually with some silt, which is horizontally stratified with lenses and layers of peat, silt, and sand. Detailed lateral continuity within the Atlantic Muck formation is limited; however, if materials are grouped by general types and relative consistencies, they can easily be correlated from boring to boring. (See Figures 3-1, 3-2, and 3-3.)

The environmental conditions which existed during deposition of the Atlantic Muck formation were estuarine in nature and as a result, both marine and non-marine sediments were deposited. Deposition took place slowly allowing clay, peat, sand, and silt layers to develop over the existing residual clay beds, resulting in a predominantly clay formation containing interbedded zones and layers of peat, sand, and silt. The clays designated soft to medium



NOTE:

AKE LEVEL ELEVATION IS +85 FEET (PRECISE LEVEL DATUM). LAKE BOTTOM
BASED ON SOUNDING MAP TITLED "MAINLAND TO GUACHA ISLAND
OF PROPOSED DAM ACROSS TRINIDAD ARM OF GATUN LAKE"
OCTOBER 29, 1962, PREPARED BY THE PANAMA CANAL COMPANY.

LEGEND:

AXC – DRILL HOLE LOCATION AND DESIGNATION
A – AXIS
45 – STATION 45+00, REFER TO PLAN, FIGURE 1-1
AX – AXIS
C – CLASSIFICATION BORING.
—— APPROXIMATE CONTACT — — — — INFERRED CONTACT

PROPOSED DAM - TRINIDAD ARM
GATUN LAKE, PANAMA CANAL ZONE

GEOLOGIC CROSS SECTION
AXIS

W-62-147
FEBRUARY, 1963

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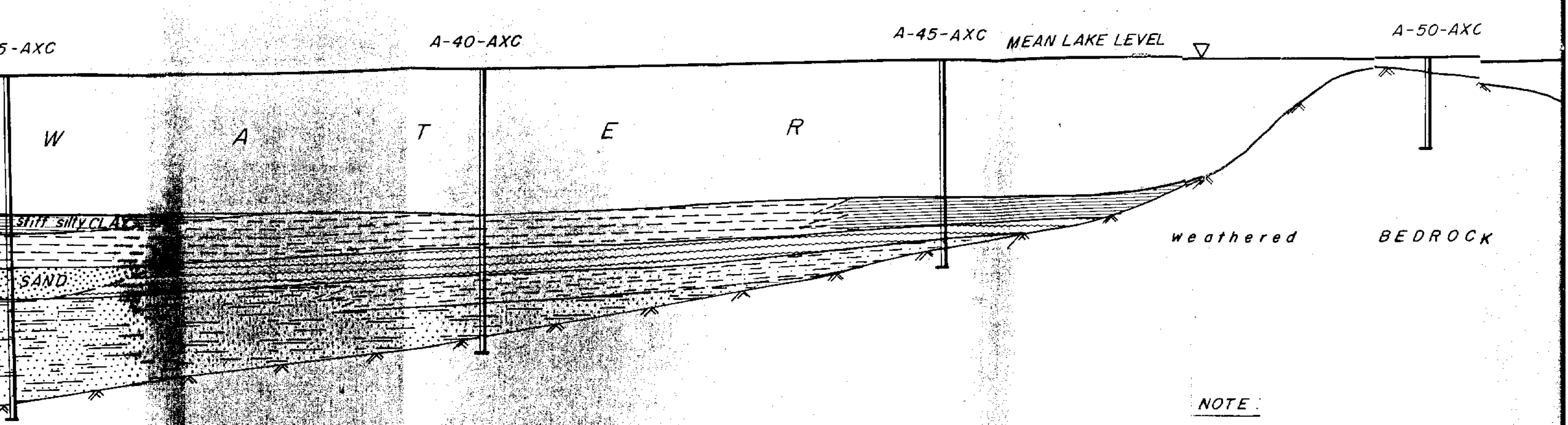
Soil Mechanics and Foundation Engineers
Seattle, Washington

FIG. 3-1



NOTE :

MEAN LAKE LEVEL ELEVATION IS + 85 FEET (PRECISE LE
PROFILE BASED ON SOUNDING MAP TITLED "MAINLAND TO
PORTION OF PROPOSED DAM ACROSS TRINIDAD ARM OF G
DATED OCTOBER 29, 1962, PREPARED BY THE PANAMA

LEGEND :

A-45AXC — DRILL HOLE LOCATION AND DESIGNATION

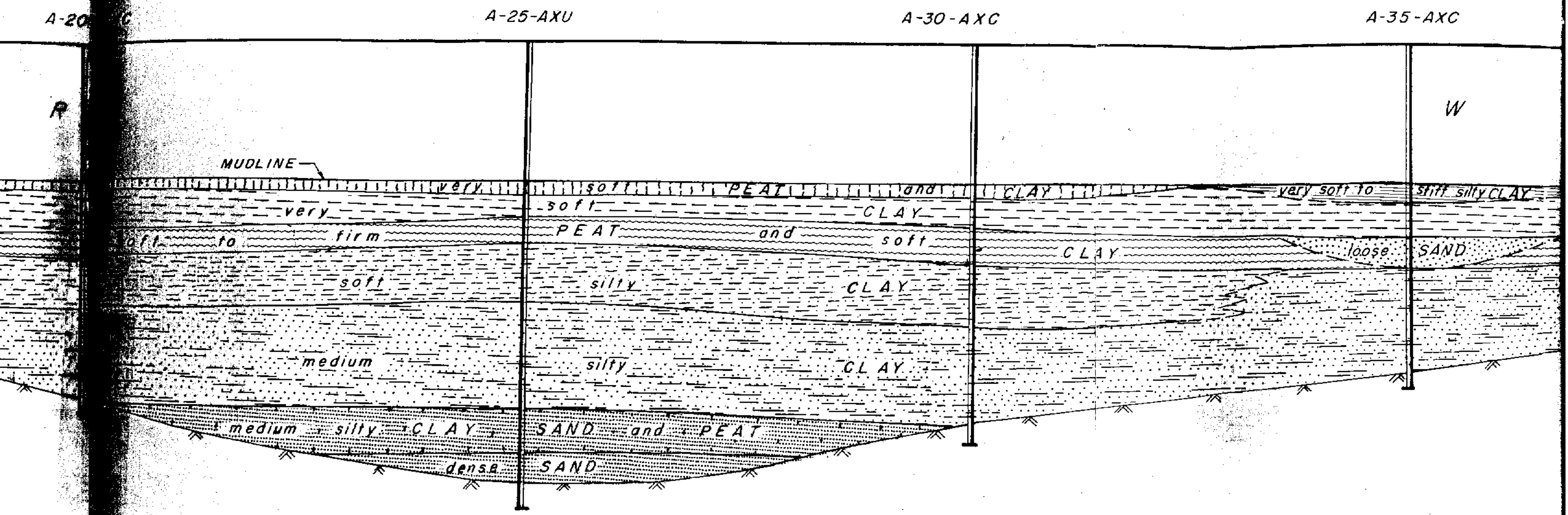
A — AXIS

45 — STATION 45+00, REFER TO PLAN, FIGU

AX — AXIS

C — CLASSIFICATION BORING.

APPROXIMATE CONTACT — — — INF

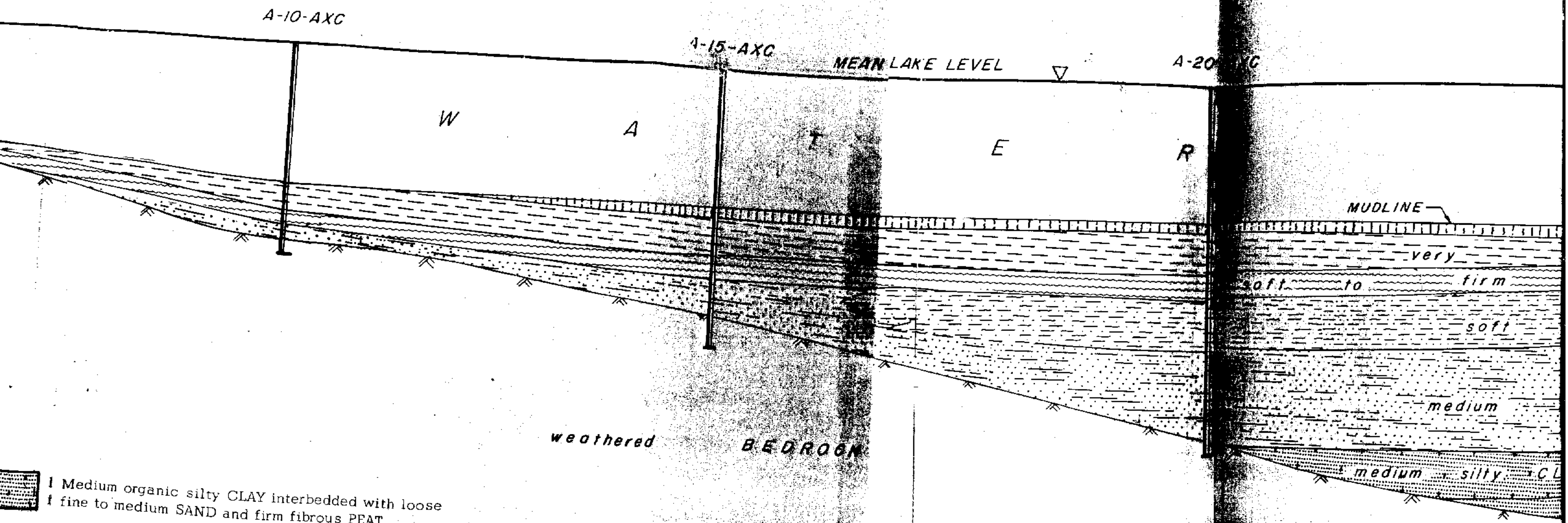


GEOLOGIC CROSS SECTION

AXIS

SCALE:

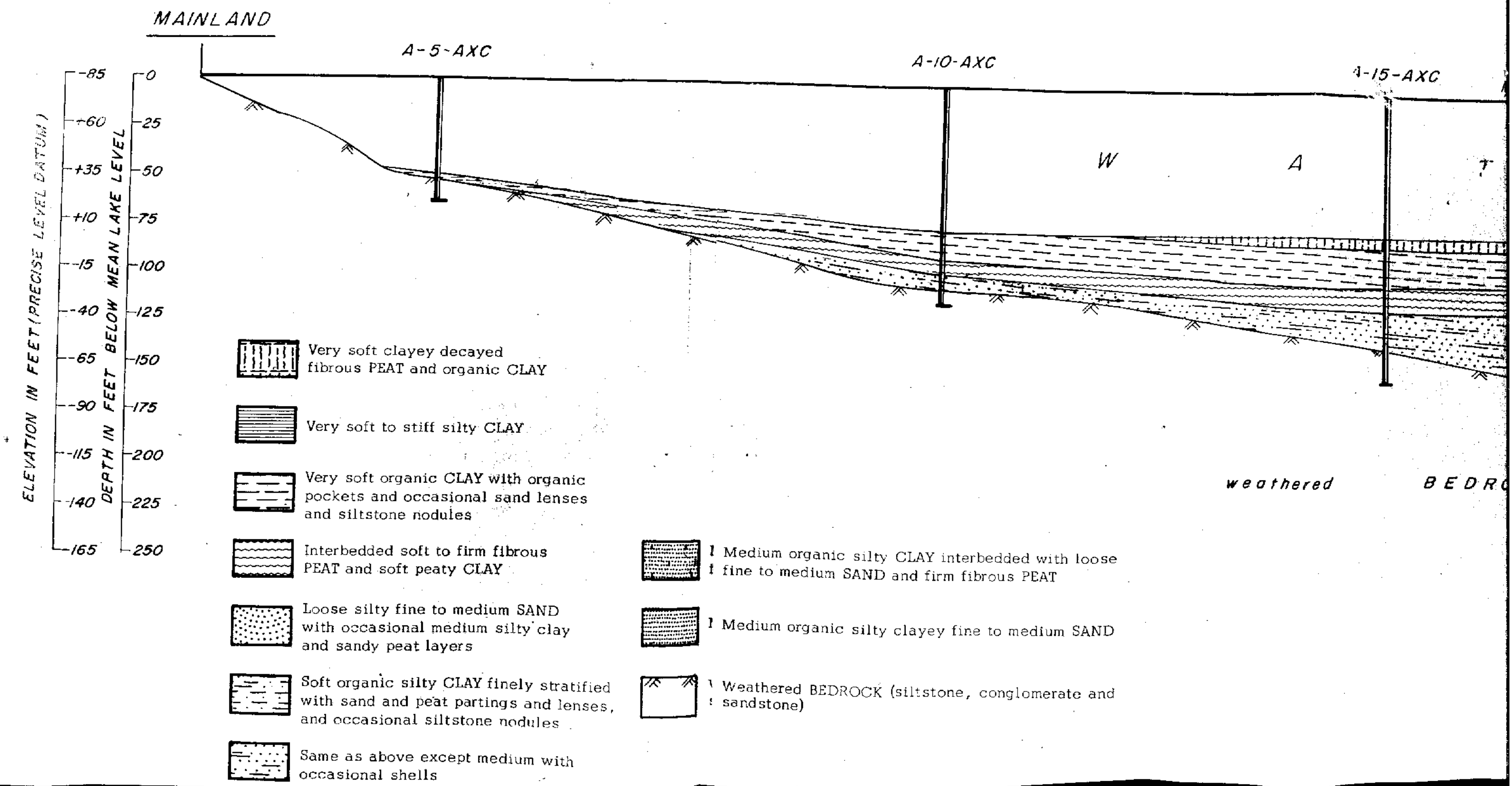
Horizontal 1" – 100'
Vertical 1" – 50'

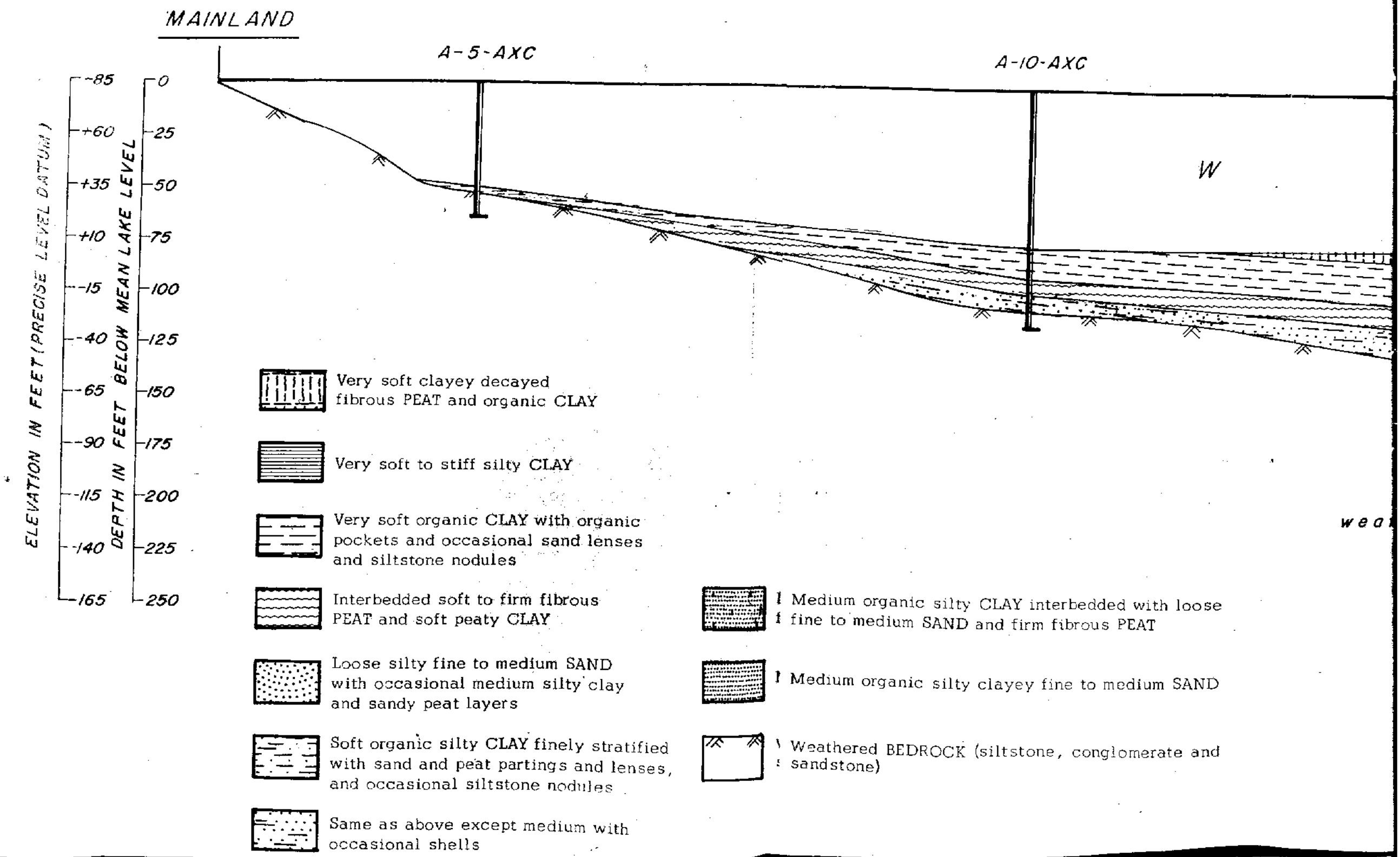


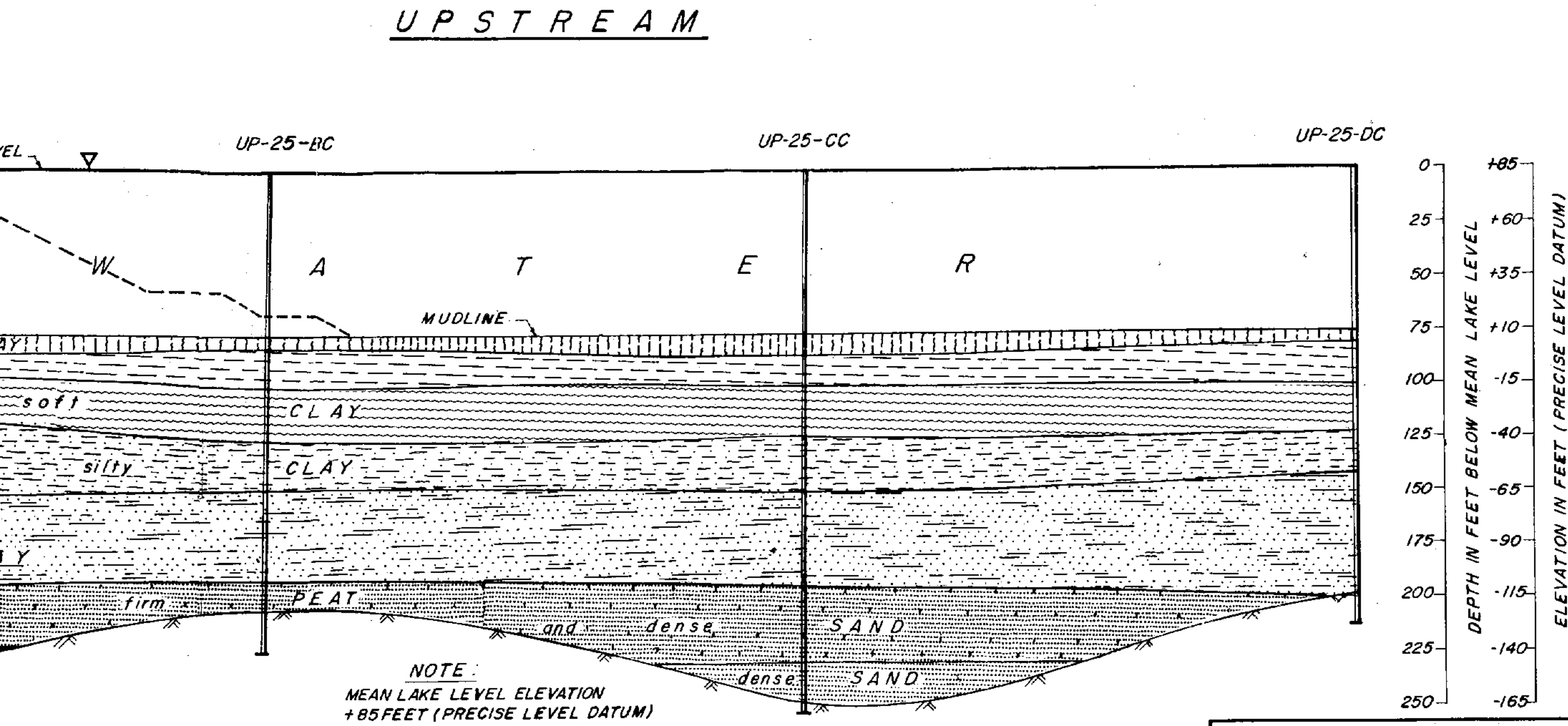
Medium organic silty CLAY interbedded with loose fine to medium SAND and firm fibrous PEAT

Medium organic silty clayey fine to medium SAND

Weathered BEDROCK (siltstone, conglomerate and sandstone)


MAINLAND
A-5-AXC
A-10-AXC
A-15-AXC
W
A
T
weathered
BEDRO
ELEVATION IN FEET (PRECISE LEVEL DATUM)
DEPTH IN FEET BELOW MEAN LAKE LEVEL
-85
+60
+35
+10
-15
-40
-65
-90
-115
-140
-165
0
25
50
75
100
125
150
175
200
225
250
Very soft clayey decayed fibrous PEAT and organic CLAY
Very soft to stiff silty CLAY
Very soft organic CLAY with organic pockets and occasional sand lenses and siltstone nodules
Interbedded soft to firm fibrous PEAT and soft peaty CLAY
Loose silty fine to medium SAND with occasional medium silty clay and sandy peat layers
Soft organic silty CLAY finely stratified with sand and peat partings and lenses, and occasional siltstone nodules
Same as above except medium with occasional shells
Medium organic silty CLAY interbedded with loose fine to medium SAND and firm fibrous PEAT
Medium organic silty clayey fine to medium SAND
Weathered BEDROCK (siltstone, conglomerate and sandstone)


MAINLAND
A-5-AXC
A-10-AXC
W
ELEVATION IN FEET (PRECISE LEVEL DATUM)
-85
+60
+35
+10
-15
-40
-65
-90
-115
-140
-165
DEPTH IN FEET BELOW MEAN LAKE LEVEL
0
25
50
75
100
125
150
175
200
225
250
Very soft clayey decayed fibrous PEAT and organic CLAY
Very soft to stiff silty CLAY
Very soft organic CLAY with organic pockets and occasional sand lenses and siltstone nodules
Interbedded soft to firm fibrous PEAT and soft peaty CLAY
Loose silty fine to medium SAND with occasional medium silty clay and sandy peat layers
Soft organic silty CLAY finely stratified with sand and peat partings and lenses, and occasional siltstone nodules
Same as above except medium with occasional shells
Medium organic silty CLAY interbedded with loose fine to medium SAND and firm fibrous PEAT
Medium organic silty clayey fine to medium SAND
Weathered BEDROCK (siltstone, conglomerate and sandstone)



NOTE:
MEAN LAKE LEVEL ELEVATION
+85 FEET (PRECISE LEVEL DATUM)

LEGEND:

DN-25-BC – DRILL HOLE LOCATION AND DESIGNATION
DN – DOWNSTREAM
25 – STATION 25+00, REFER TO PLAN FIGURE I-I
B – GRID LINE B, REFER TO PLAN, FIGURE I-I
C – CLASSIFICATION BORING
– APPROXIMATE CONTACT

PROPOSED DAM-TRINIDAD ARM
GATUN LAKE, PANAMA CANAL ZONE

GEOLOGIC CROSS SECTION
STATION 25+00
W-62-147
FEBRUARY, 1963

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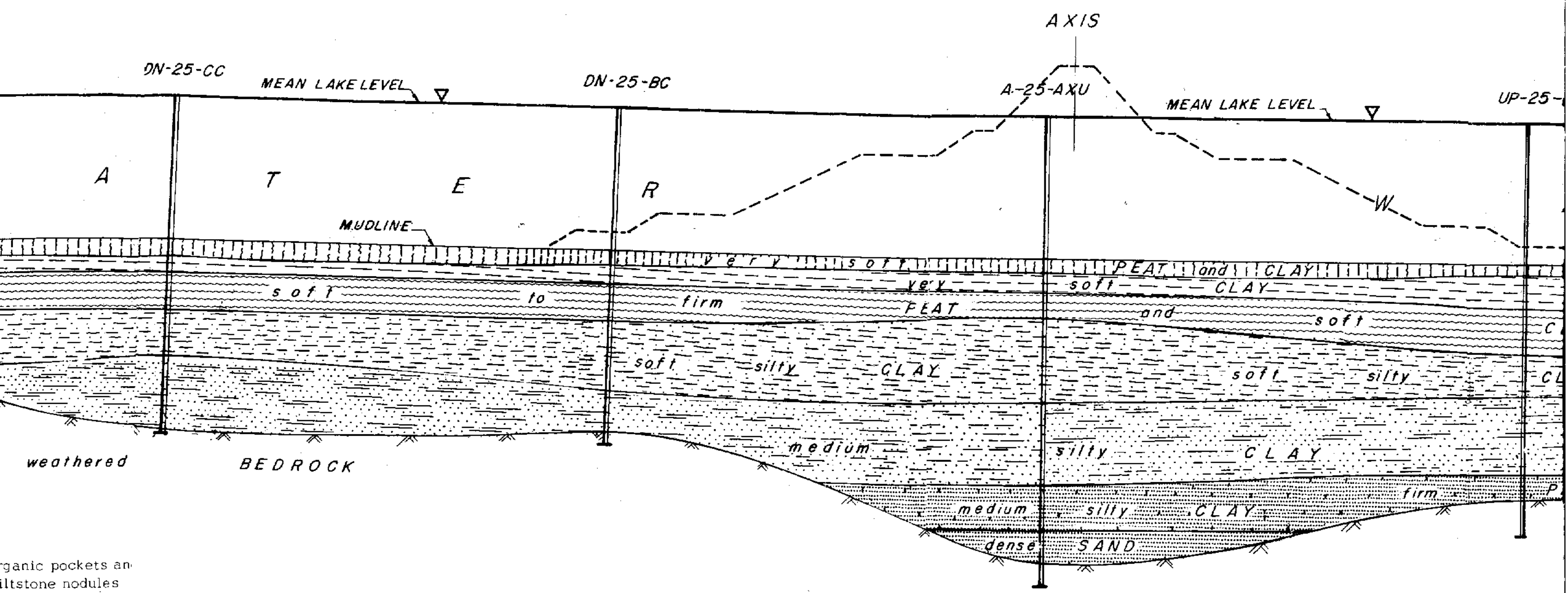
Soil Mechanics and Foundation Engineers
Seattle, Washington

FIG. 3-2



rganic pockets an
iltstone nodules

s PEAT and soft
and lenses

stratified with sand
tone nodules, ancasional shells

Medium organic silty CLAY interbedded with loose fine to medium SAND and firm fibrous PEAT

Medium organic silty clayey fine to medium SAND with some gravel

Weathered bedrock (siltstone, conglomerate, claystone, and basalt)

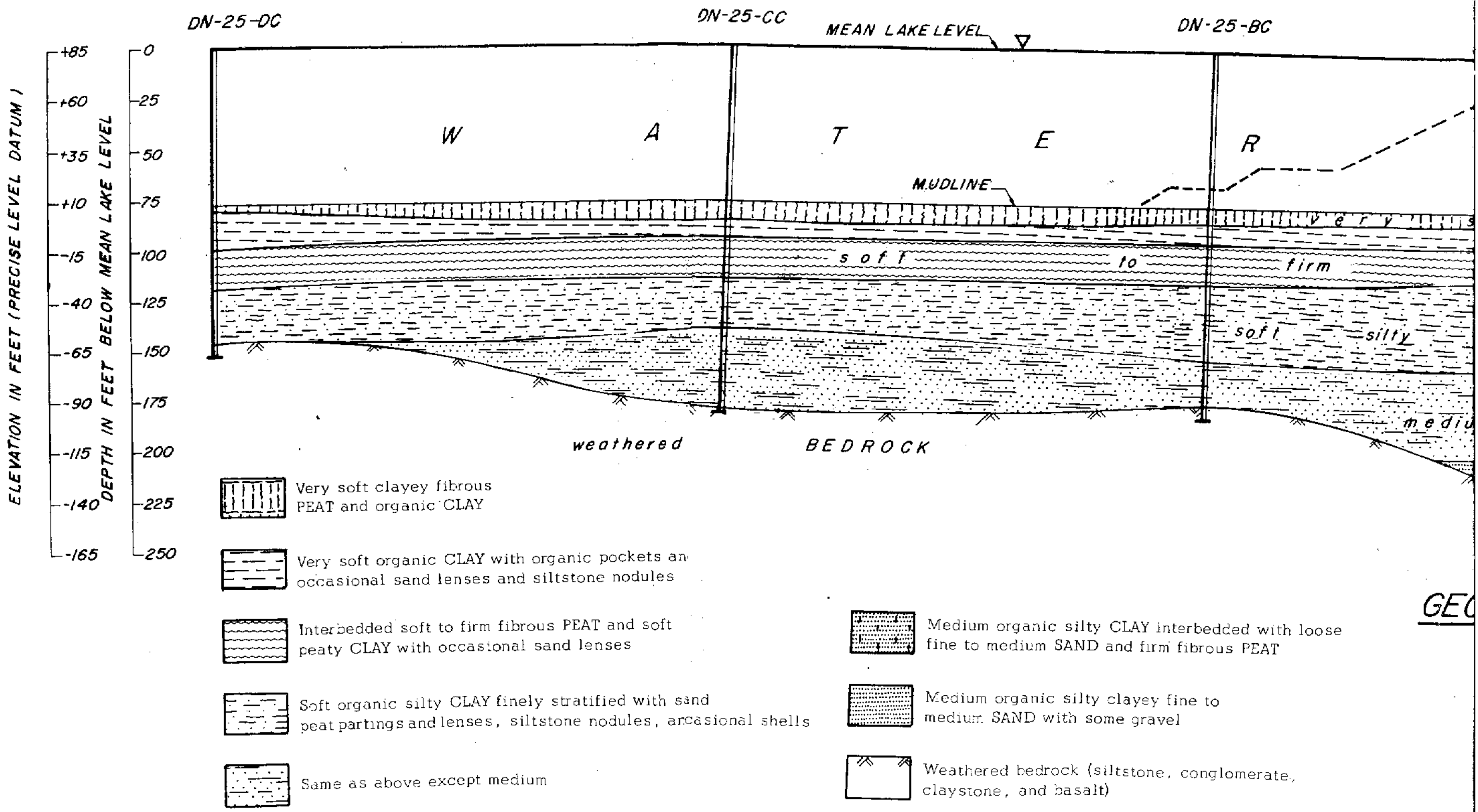
GEOLOGIC CROSS SECTION
STATION 25+00

SCALE

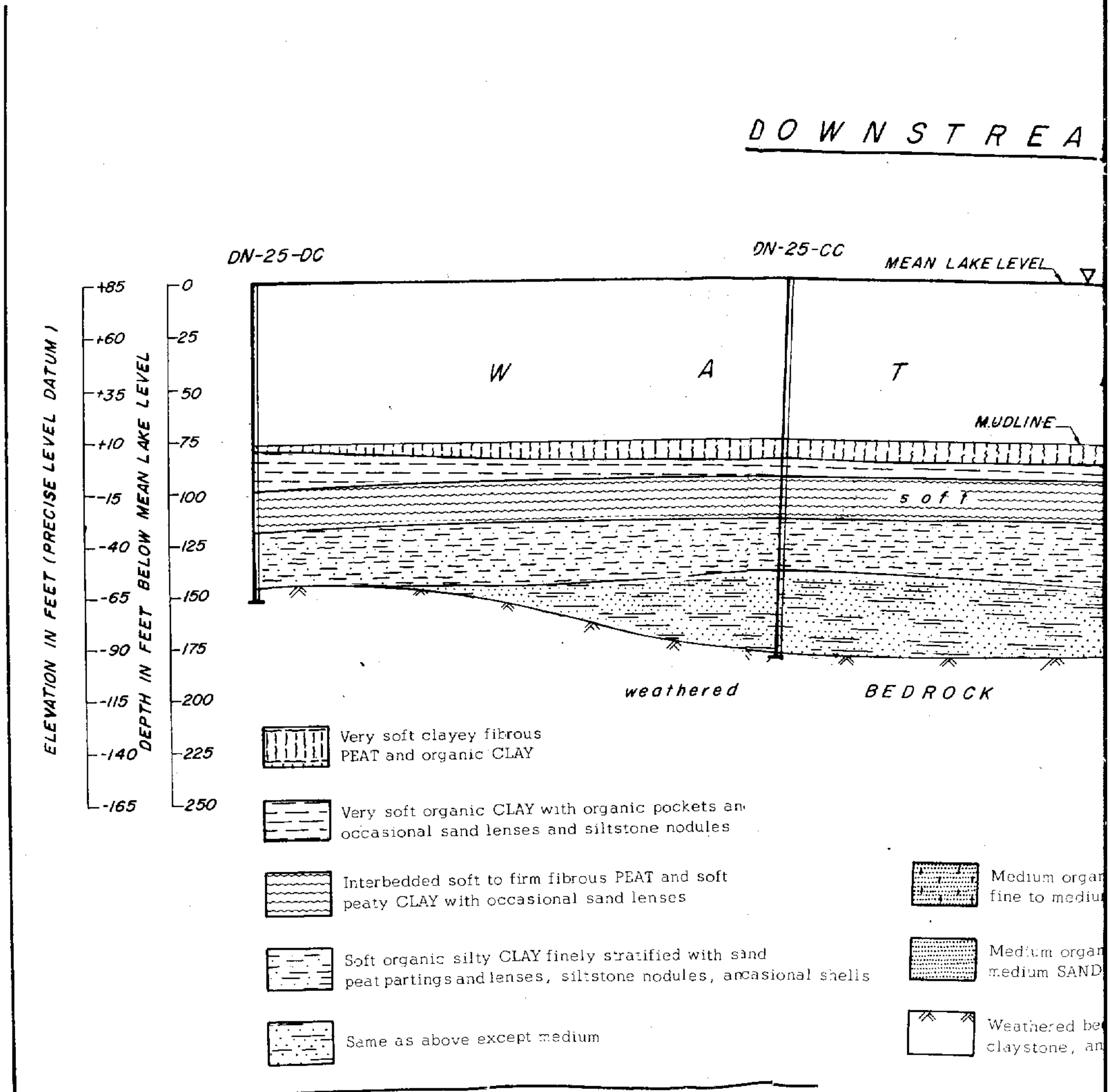
Horizontal 1" – 100'
Vertical 1" – 50'

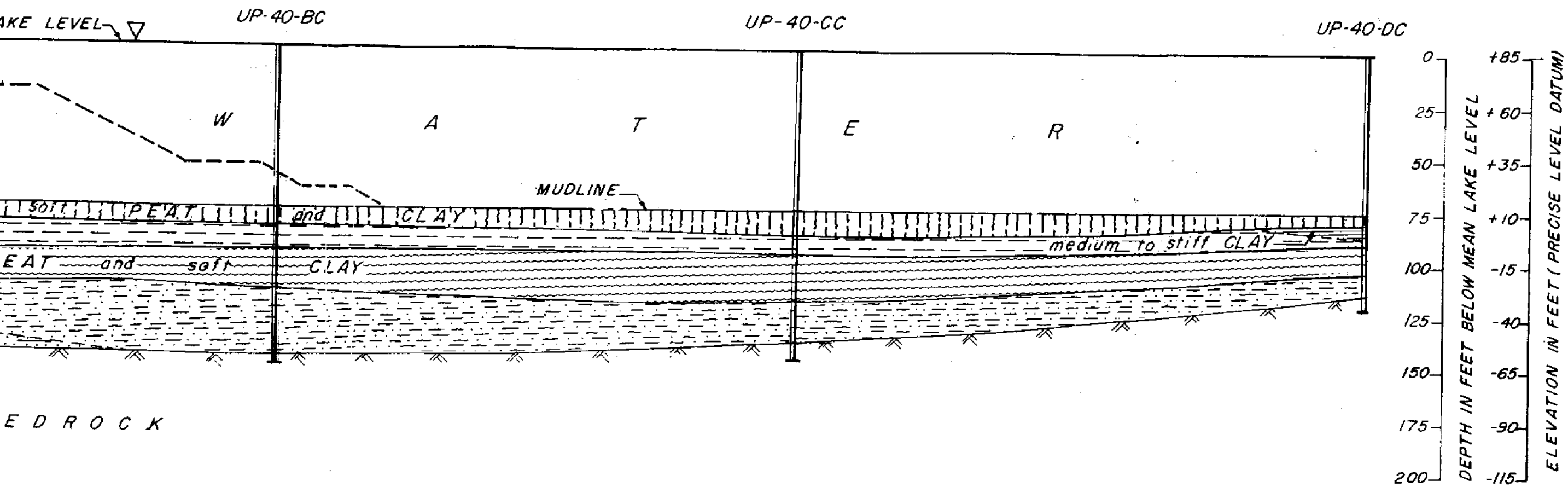
DN-25-

DOWNSTREAM



GEO


DOWNSTREA
DN-25-DC
DN-25-CC
MEAN LAKE LEVEL
ELEVATION IN FEET (PRECISE LEVEL DATUM)
+85
+60
+35
+10
-15
-40
-65
-90
-115
-140
-165
DEPTH IN FEET BELOW MEAN LAKE LEVEL
0
25
50
75
100
125
150
175
200
225
250
W A T
MUDLINE
soft
weathered BEDROCK
Very soft clayey fibrous PEAT and organic CLAY
Very soft organic CLAY with organic pockets an occasional sand lenses and siltstone nodules
Interbedded soft to firm fibrous PEAT and soft peaty CLAY with occasional sand lenses
Soft organic silty CLAY finely stratified with sand peat partings and lenses, siltstone nodules, ancasional shells
Same as above except medium
Medium organ fine to mediu
Medium organ medium SAND
Weathered be claystone, an



NOTE

MEAN LAKE LEVEL ELEVATION + 85 FEET
(PRECISE LEVEL DATUM)

LEGEND

DN-40-BC – DRILL HOLE LOCATION AND DESIGNATION
DN – DOWNSTREAM
40 – STATION 40+00 REFER TO PLAN, FIGURE 1-1
B – GRID LINE B, REFER TO PLAN, FIGURE 1-1
C – CLASSIFICATION BORING
——— APPROXIMATE CONTACT - - - - - - INFERRED CONTACT

PROPOSED DAM TRINIDAD ARM
GATUN LAKE, PANAMA CANAL ZONE

GEOLOGIC CROSS SECTION
STATION 40+00

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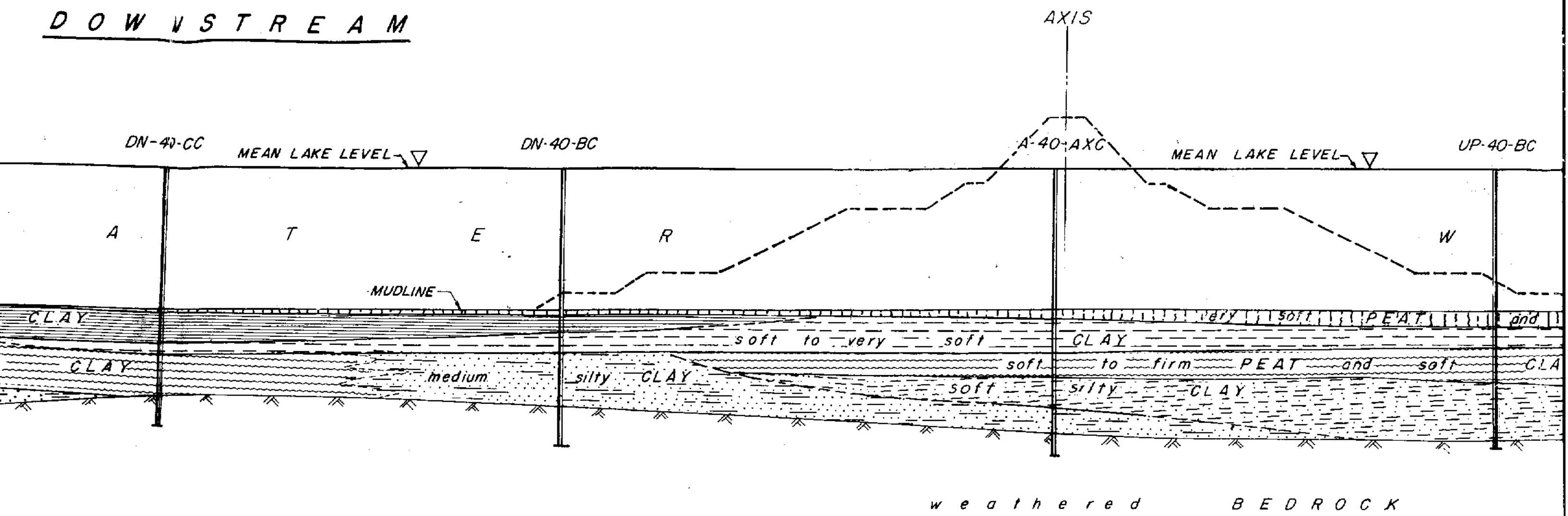
Soil Mechanics and Foundation Engineers

FIG 3-3



Soft organic silty CLAY finely stratified with sand and peat partings and lenses, and occasional siltstone nodules

Same as above except medium

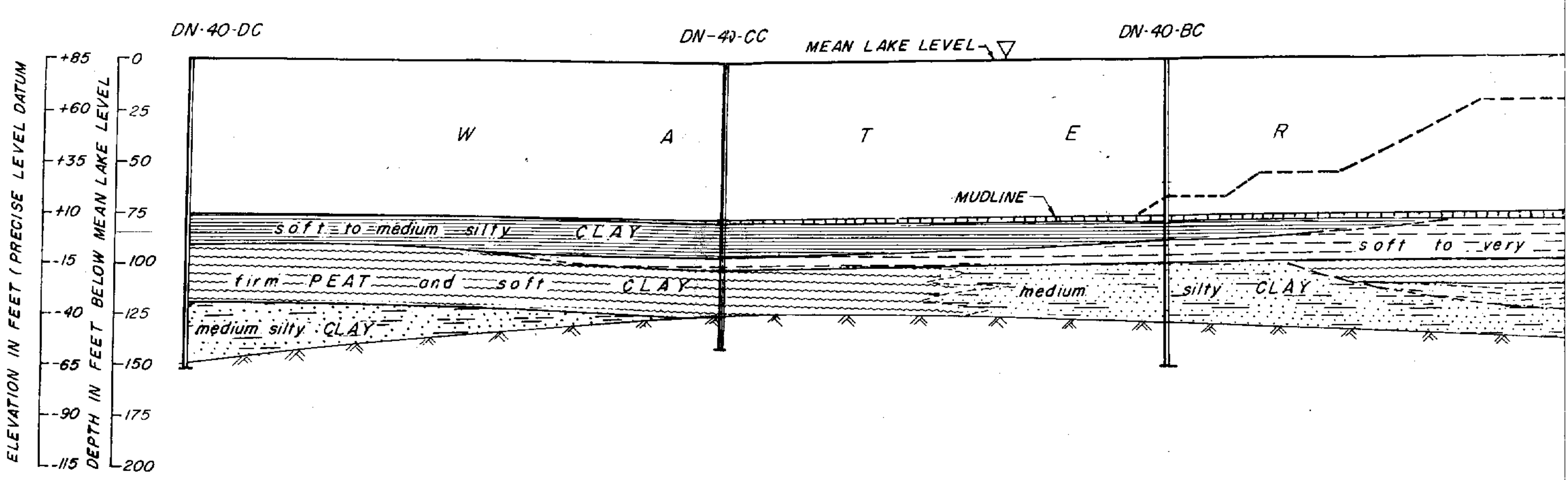
Weathered BEDROCK (siltstone and sandstone)

GEOLOGIC CROSS SECTION
STATION 40+00

SCALE

Horizontal : 1" – 100'
Vertical : 1" – 50'

DOWNSTREAM



Very soft fibrous PEAT, organic accumulations, and organic CLAY

Soft to medium organic very fine sandy silty CLAY with occasional grass roots

Very soft to soft organic CLAY with organic pockets and occasional sand lenses and siltstone nodules

Interbedded soft peaty CLAY and firm fibrous PEAT with layers of silty fine sand

Soft organic silty CLAY finely stratified with sand and peat partings and lenses, and occasional siltstone nodules

Same as above except medium

Weathered BEDROCK (siltstone and sandstone)

GEOLO

S

are generally finely laminated with fine sand, silt, and organic partings. The photographs shown on Figures 3-4 and 3-5, which were taken of representative samples of the Atlantic Muck formation, illustrate material composition and development of stratification with depth.

The relative softness of the Atlantic Muck sediments is apparently a function of the thickness of the overlying materials with the consistencies ranging from very soft near the mudline to stiff at greater depths.

In the eastern portion of the site a medium to stiff layer of organic clay overlies the typically soft materials of the Atlantic Muck formation. This firmer layer is believed to be the result of desiccation.

Since creation of Gatun Lake, there has been little change in the character of the Atlantic Muck materials. The cover of water has been responsible both for preventing further weathering of the sediments and for deposition of a thin layer of algal material over the entire lake bottom.

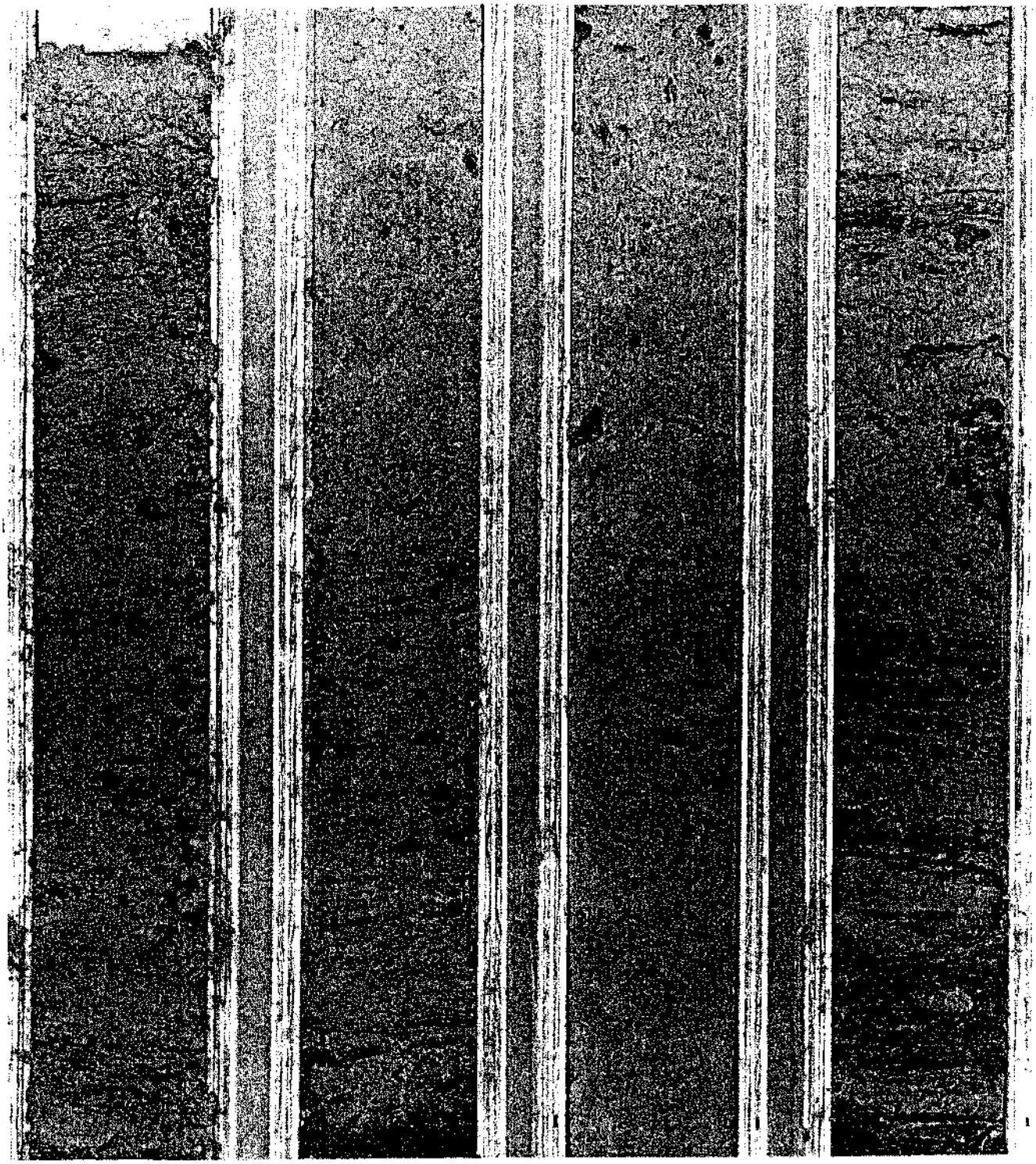
3-3. BEDROCK.

In general the bedrock consists of siltstone, sandstone, conglomerate, claystone, and a trace of basalt. Most of these have been thoroughly weathered prior to deposition of the Atlantic Muck formation to form highly colored (mostly red), lateritic, very stiff clay with only the relict textures remaining to indicate the rock's original form. Strengths of these materials are much greater than those of the Atlantic Muck sediments.

According to the results of the geophysical survey the bedrock between Guacha Island and the Mainland near Punta Mala has not been subjected to major structural deformation.

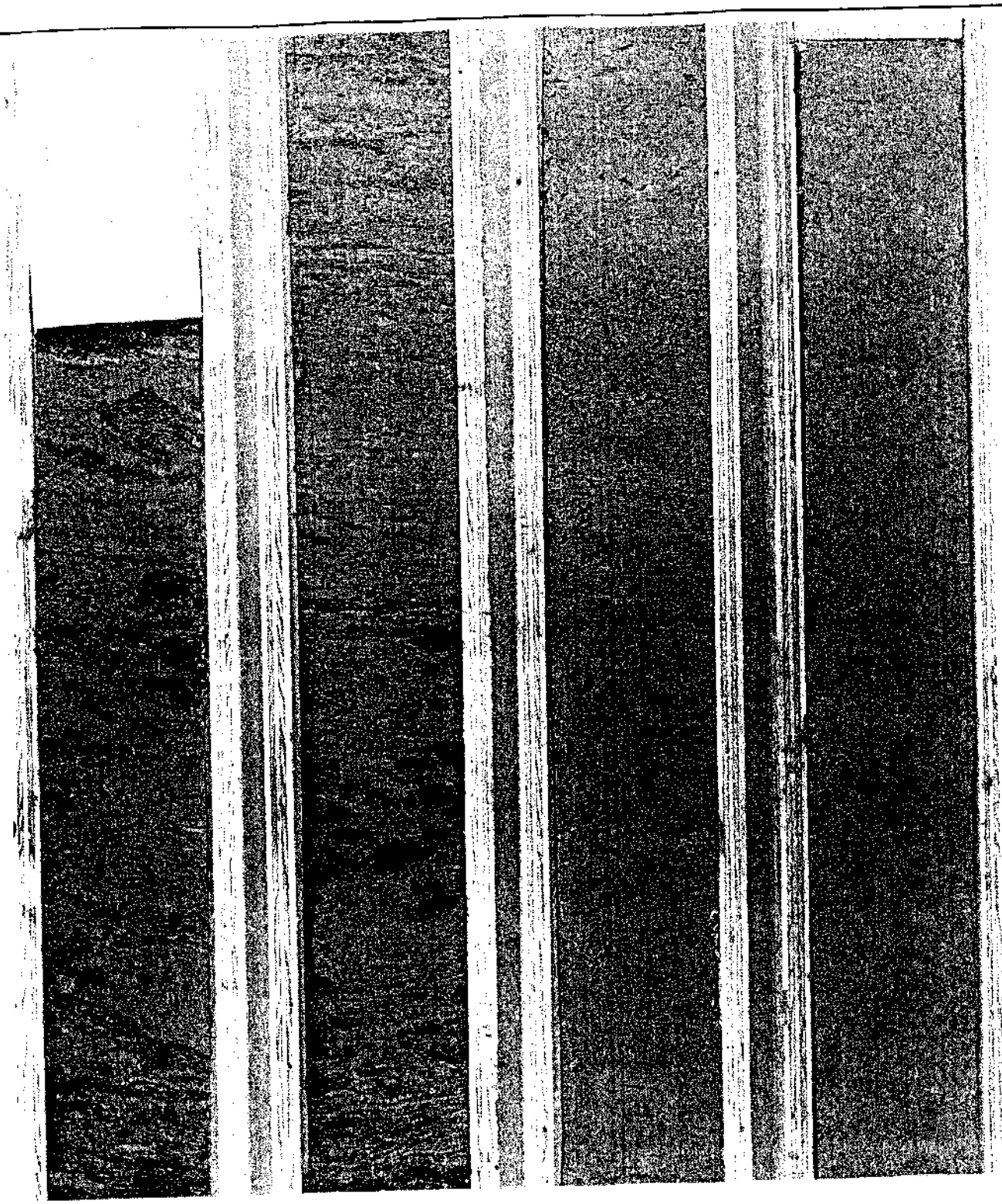
3-4. SOIL TYPES AND DISTRIBUTION.

The columnar section, Figure 3-6, summarizes the soil types and their distribution within the site area.



S-3	S-5	S-7	S-9
79.0'-81.5'	85.0'-87.5'	91.0'-93.5	97.0'-99.5'
Decayed PEAT	Peaty organic CLAY	Organic CLAY with peat inclusions	Organic CLAY interbedded with peat

Figure 3-4. Photograph of samples S-3, S-5, S-7, and S-9, Drill Hole DN-25-CU



S-11	S-13	S-15	S-17
103.0'-105.5'	109.0'-111.5'	118.0'-120.5'	124.0'-126.5'
Organic CLAY interbedded with PEAT	Organic CLAY with peat laminations	Organic CLAY with silt pockets	Organic CLAY with siltstone nodules

Figure 3-5. Photograph of samples S-11, S-13, S-15, and S-17, Drill Hole DN-25-CU

DEPTH FROM LAKE BOTTOM IN FEET	FORMATION	COLUMNAR SECTION	MAXIMUM THICKNESS IN FEET	SOIL TYPE AND DISTRIBUTION
10	ATLANTIC MUCK		10	Nearly liquid to very soft, sometimes clayey, decayed, fibrous PEAT. Overlies almost entire site in varying thicknesses.
20		very soft / medium stiff	15	Very soft organic CLAY with numerous organic pockets and occasional peat layers. Extends over almost entire site. In eastern portion of site becomes soft to stiff clay.
30, 40			25	Interbedded firm fibrous PEAT and soft organic silty CLAY. Peat is commonly clayey and clay is commonly peaty. These beds extend over almost the entire site. A layer of sand with peat and clay 20 feet thick was encountered at boring DH A-35-AXC in the same stratigraphic position as the peat and clay beds.
50, 60, 70		soft	25	Soft organic silty CLAY with frequent layers, lenses, and partings of peat, silt and sand. Occasional layers of hard, flat siltstone nodules occur throughout. Zone is relatively continuous over the entire site, grading into medium material with the same composition near the edges of the proposed damsite abutments.
80, 90, 100, 110, 120		medium	50	Medium organic silty CLAY with same composition as overlying material. Occurs over entire site area, thinning-out at the site edges. Occasional shell beds occur in both this material and the overlying material.
130, 140, 150, 160		BEDROCK	40	Medium silty clayey fine to medium SAND which is interbedded with organic silty clay and peat in the top 25 feet. Becomes occasionally gravelly in the lower 15 feet. This material occupies the old stream channel and is the oldest portion of the Atlantic Muck.
170			Unknown	Weathered Bedrock. Outcrops above elevation +15 feet (PLD).

NOTE:

The intent of this graphical presentation is to summarize the material types and their stratigraphic relationships within the site area.

PROPOSED DAM - TRINIDAD ARM
GATUN LAKE, PANAMA CANAL ZONE

COLUMNAR SECTION (SOIL TYPES AND DISTRIBUTION)

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SHANNON & WILSON
SOIL MECHANICS & FOUNDATION ENGINEERS

FIG. 3-6

material would give adequate protection and provide the necessary drainage as the muck consolidates under the fill load as it is placed.

The mat serves a two fold purpose in that (1) it allows drainage as the foundation is loaded and (2) it will prevent the rupture of the foundation caused by the dumping of large boulders in the Zone 2 & 3 fill. The test fill constructed earlier demonstrated that large boulders would penetrate the foundation and cause loss of foundation strength.

2. Penetration. Studies and computations made during the course of investigations indicate that a rock 8 inches in diameter dropped through 70 feet of water will penetrate the foundation material to a depth of approximately 6 inches. A rock 4 inches in diameter will penetrate the foundation material approximately 2 inches. Larger rocks in the order of 4 feet in diameter would penetrate the foundation to a depth of 9 to 10 feet. This depth of penetration is borne out by inspection of the test fill constructed by the Panama Canal Company. The underwater inspection was made by a soils engineer and geologist of the Panama Canal Company.

Figure No. 2 shows curves prepared by plotting penetration and velocity versus size of material for a fall through 70 feet of water. The penetration was determined by computing the kinetic energy of a mass with submerged unit weight of 90 pcf falling through the water, and determining the displacement of the foundation material required to dissipate the energy, using a shear

3-5. ATLANTIC MUCK FORMATION.

The thickness and extent of the Atlantic Muck formation was originally controlled by the erosional surface upon which it was deposited. The Atlantic Muck formation was not detected in this foundation investigation above approximately elevation +15 feet (P.L.D.).

Within the site area, the thickest section of Atlantic Muck was encountered at station 25+00 on the Axis and at station 25+00 on grid line "C", 1000 feet upstream. Borings at these locations intersected the old meandering stream channel which generally trends in a northerly direction, more or less perpendicular to the axis of the proposed damsite. The maximum thickness of Atlantic Muck along this channel is 160 feet. Since the original valley was "V-shaped," thicknesses of the Muck decrease in both directions from station 25+00 along the Axis.

All materials of the Atlantic Muck formation are mantled by a layer of peat varying in thickness from one to ten feet. The peat is nearly liquid to very soft, decayed to semi-decayed, and somewhat fibrous. Often the peat is clayey and interbedded with lenses of very soft organic clay.

Underlying the top layer of peat is a layer of very soft, light to dark gray, organic clay with numerous pockets of decayed organic material and with occasional pockets of clayey sand, yellow clayey silt, and siltstone nodules. This clay is fairly continuous over the damsite except in the eastern portion of the site where it becomes soft to stiff. It is believed that the zones of stiffer clay are the result of desiccation of the existing clay beds. This stiffer clay was penetrated in borings A-45-AXC, A-35-AXC, UP-40-DC, DN-40-BC, DN-40-CC, and DN-40-DC.

Underlying these clays is a nearly continuous stratum of interbedded peat and clay. The peat is usually firm, fibrous, and generally somewhat clayey. It parts readily along horizontal planes. The constituents of the peat are easily identified as leaves, twigs, stems, vines, and other organic debris and are quite fresh in appearance. The clay layers are very soft to soft, organic, and silty, with pockets, layers, and lenses of peat, and occasionally with sand partings and lenses, and siltstone nodules. A layer of silty clayey sand containing peat and clay was encountered at boring A-35-AXC, in the same

stratigraphic position as the peat and clay beds, representing a local change in mode of deposition.

The material underlying the clay and peat is soft to medium organic silty clay which is stratified with sand layers and partings, occasional peat layers and lenses, silt layers and siltstone nodules. This material can be separated into two distinct layers; one soft and one medium. The soft layer grades into medium toward the abutments of the proposed dam.

Underlying the soft to medium clays is a layer of medium sand with interbedded medium organic silty clay and firm fibrous peat materials. The sand is silty and clayey and overlies the sands and gravels of the old river channel.

The channel sands are dense, silty, and clayey and occasionally contain gravel. This sand is the oldest member of the Atlantic Muck formation.

3-6. BEDROCK.

Within the proposed damsite area, bedrock consists of sandstone, siltstone, conglomerate, claystone, and basalt. All, with the exception of those encountered in the old stream channel, have been affected by severe weathering. These materials have been decomposed by chemical weathering processes to such an extent that almost all the constituent minerals have been altered to clay. This has resulted in the formation of a stiff to very stiff layer of highly colored (primarily red) slightly sandy clay overlying the bedrock surface. The thickness of this clay is undetermined. All materials classified as residual clay or bedrock are superior in strength, in comparison with all other materials encountered in the foundation, and accordingly they are presumed not to be a factor in estimating stability of the earth and rock fill dam proposed for the site. They are not considered further in this report.

SECTION IV

LABORATORY TESTING

4-1. GENERAL.

The purpose of the laboratory testing program was to determine those physical characteristics of the foundation materials necessary for the design of the proposed dam. To supplement testing in the Shannon & Wilson, Inc. Seattle laboratory, and to minimize the cost of shipping soil samples, a field laboratory was set up near Gatun Landing in Gatun.

4-2. FIELD LABORATORY.

The field laboratory was operated by Shannon & Wilson laboratory technicians assisted by Panama Canal Company personnel. This laboratory was responsible for performing classification tests and unconfined compression tests on all samples from the classification borings. The procedures, practices, and equipment employed by the field laboratory in conducting the classification and unconfined compression tests are described in Appendix A.

4-3. SEATTLE LABORATORY.

The Seattle laboratory of Shannon & Wilson, Inc. was utilized to perform tests which required equipment not reasonably transportable to the field. The testing program included triaxial shear tests, laboratory vane shear tests, consolidation tests and special triaxial tests in addition to classification tests. Only those samples obtained from the undisturbed borings were shipped to Seattle for testing.

The program for field explorations included a provision for one boring to be made in the early stages of the foundation investigation to provide undisturbed samples for testing in the Seattle laboratory. The initial boring was located at

station 25+00 on the Axis after review of cone penetrometer test data revealed that this location contained comparatively weak foundation materials.

The undisturbed samples from drill hole A-25-AXU arrived in Seattle in October 1962 and a thorough laboratory testing program was initiated to investigate (1) the index properties of the clay and peat materials, (2) the "in-situ" shear strength of the foundation materials, and (3) the consolidation characteristics of the clay and peat materials with particular emphasis on time functions relating to the rate of consolidation.

Subsequent to the completion of the laboratory program for drill hole A-25-AXU, available field and laboratory data were reviewed to determine whether the properties of the foundation materials and subsurface conditions at this location were, in general, representative of other locations within the investigational area. This review along with the results of supplementary test data indicated that the properties of the clay and peat materials determined from the laboratory tests on samples from drill hole A-25-AXU were typical.

Further laboratory testing in Seattle on undisturbed samples from other locations was, therefore, selective and limited principally to tests not previously undertaken in the initial testing program. Tests were undertaken to (1) define the shear strength parameters, cohesion, c, and angle of shearing resistance, $\emptyset$, for clay and peat materials from consolidated-undrained ($\overline{R}$) triaxial tests, (2) examine the influence of the rate of axial strain in strength tests on values of shear strength, and (3) confirm in triaxial dissipation tests values of the coefficient of consolidation, C_v, obtained from consolidation test data. In addition to routine Atterberg Limit tests, special Atterberg Limit tests were undertaken to examine, in qualitative terms, the organic content of the foundation clays.

Specific test procedures and practices, employed in the Seattle laboratory in conducting the various tests are outlined in Appendix A along with various figures and tables presenting detailed test results.

The following section presents a summary of the properties of the clay and peat materials disclosed by the Seattle and field laboratory testing programs. These clay and peat materials comprise the Atlantic Muck formation. Since they constitute the "weakest" materials encountered in the foundation investigation,

the properties of the material in this formation will control the stability and settlement of the proposed earth and rock fill dam.

SECTION V

PROPERTIES OF FOUNDATION MATERIALS

5-1. INDEX PROPERTIES.

The classification tests performed in the laboratory were used to determine certain index properties of the foundation materials. Such index properties as natural water content, Atterberg Limits, unit weight, grain-size distribution and specific gravity were obtained utilizing standard laboratory tests. These index properties assisted in soil classification and, in addition were useful in forming general impressions of other material properties, such as shear strength, permeability and compressibility.

5-2. NATURAL WATER CONTENT.

The natural water content (w) of the foundation materials generally decreases with depth, reflecting a greater degree of consolidation with depth under increasing effective overburden pressures. Erratic variations in moisture content, occurring within short depth intervals below the mudline to about depth 50 feet, result principally from abrupt changes in organic content; an increase in organic content resulting in an increase in moisture content. The natural water contents of the peats range principally from 100 to 300 percent while the natural water contents of the organic clays vary primarily within the range of 70 to 90 percent.

5-3. ATTERBERG LIMITS.

Atterberg Limits were performed on clay and clayey peat materials. The limits are essentially uniform with depth except for wide fluctuations in values of liquid limit (LL) below the mudline to about depth 50 feet. These fluctuations were caused by variations in the amount of organic material in the limit samples. The values of liquid limit in this zone vary between 80 and 250 percent while below depth 50 feet the variation between liquid limit values ranges primarily

between 100 and 120 percent. The plastic limit (PL) of the foundation materials is uniform with depth, varying between 40 and 60 percent. The mean value of the plasticity index (PI) is in the order of 70 percent. The results of all Atterberg Limit tests are shown on the Plasticity Chart, Figure 5-1. The results plot both above and below the A-line within a section identifying highly compressible and relatively impermeable materials.

Plotted points from tests in any one boring did not show any significant arrangement or trend; tests on adjacent tubes would at times be widely separated and on opposite sides of the A-line.

Tests on clay which plot below the A-line indicate, as a general rule, the presence of organic material. Special Atterberg Limit tests were performed which indicate that most organic material in the clay is in the form of pockets, fragments, and lenses and is not intimately mixed with the individual clay particles. This suggests that the strength and compressibility of the clay will not be significantly influenced by its organic constituents.

5-4. WATER-PLASTICITY RATIO.

The relationship between the natural water content (w) and Atterberg Limit values can be expressed by the water-plasticity ratio which is defined as $B = \frac{W-PL}{LL-PL}$. As the water content increases from the plastic limit (PL) to the liquid limit (LL), the value of B increases from 0 to 1.0. Deposits which have water contents close to the liquid limit are usually much softer than those with moisture contents close to the plastic limit. The plot of water-plasticity ratio versus depth for drill hole A-25-AXU (see Figure 5-2) shows that close to the mudline, the material is in a nearly "fluid" or very soft state, and that the consistency of the material increases with depth. A consistent decrease in this ratio with depth from values close to 1 near the mudline is evident at most boring locations; the only exceptions being at borings where a desiccated clay layer exists near the mudline.

5-5. UNIT WEIGHT.

In general, the unit weight of the foundation material increases with depth. This trend is particularly noticeable in the deeper borings along

SECTION V

PROPERTIES OF FOUNDATION MATERIALS

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station 25+00. Unit weights of peat samples varied from about 60 to 75 pounds per cubic foot. Unit weights of clay samples were in the order of 80 to 95 pounds per cubic foot.

It is probable that the unit weights determined from laboratory measurements are lower than in-situ unit weights, due to expansion of the material as in-situ stresses were removed during the sampling operation. It is estimated that the values of unit weight presented in this report are from four to eight pounds per cubic foot lower than in-place values of unit weight.

5-6. HYDROMETER ANALYSIS.

The amount of clay size material (minus 2 microns) in typical specimens of clay was determined from the hydrometer analysis to be in the order of 40 percent. The "activity"[1] of the clay is equal to 1.5. This relatively high value of activity reflects the presence of organic matter in the clay specimens.

5-7. SPECIFIC GRAVITY.

Three specific gravity tests were conducted; two tests on organic clay and one test on peat. The specific gravity values of the organic clay specimens were determined to be 2.56 and 2.64. The specific gravity of the peat sample was determined to be 1.57.

5-8. CONSOLIDATION AND STRENGTH PROPERTIES.

Other laboratory tests in addition to classification tests, were performed to determine specific strength and consolidation parameters of clay and peat materials for use in the engineering studies. The triaxial compression test was utilized to obtain the shear strength parameters, cohesion, c, and angle of shearing resistance, Ø. Other strength tests, such as vane shear and compression tests were performed both in the field and laboratory to evaluate foundation in-situ shear strength and foundation material "sensitivity". Triaxial

---

1 $\text{activity} = \dfrac{\text{Plasticity Index}}{\%\ \text{minus}\ 2\mu}$

dissipation tests and consolidation tests were used to determine values of coefficient of consolidation, C_v.

5-9. COEFFICIENT OF CONSOLIDATION, C_v.

The coefficient of consolidation, C_v, was determined in both consolidation and triaxial dissipation tests for typical clay and peat materials. All but one of the consolidation tests were performed on samples from various depth in drill hole A-25-AXU. Triaxial dissipation tests and one consolidation test were conducted on material from sample S-1, drill hole UP-15-CU. The values of C_v from these two widely separated locations are in the same order of magnitude (see Figure 5-3), indicating that for engineering studies, values of C_v from samples in drill hole A-25-AXU may be considered representative of foundation materials across the dam site.

The values of C_v for peat and clay are in the order of 0.01 to 0.04 cm^2/min., indicating low material permeability and, consequently, very slow rates of material consolidation.

5-10. SHEAR STRENGTH PARAMETERS, c AND Ø.

Shear strength parameters in terms of total and effective stresses, cohesion, c and c', and angle of shearing resistance, Ø and Ø', were determined in consolidated-undrained (R) triaxial tests for both clay and fibrous peat materials. These parameters are shown on the Mohr's strength envelope plots constructed for the organic clay and fibrous peat materials. (See Figures 5-4 and 5-5.) The rate of axial strain adopted in these triaxial R tests was determined from special triaxial tests described in paragraph A-14 of Appendix A

5-11. SENSITIVITY.

The sensitivity[1] of the foundation materials was determined from laboratory vane shear tests conducted on samples from drill hole A-25-AXU. A plot of material "sensitivity" versus depth for this location is presented on Figure 5-2. These values are in agreement with values of sensitivity revealed by the

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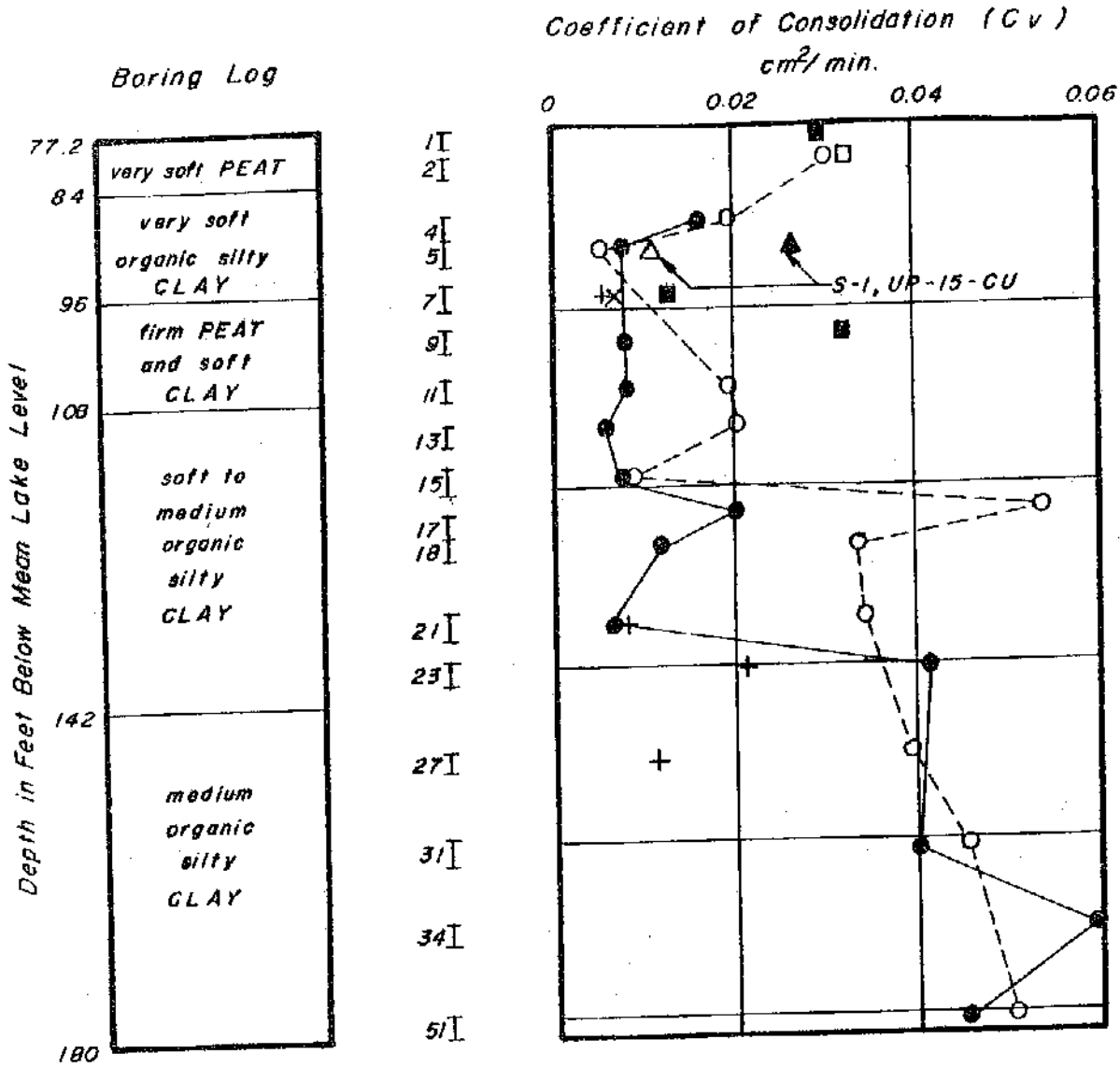
1 $\text{sensitivity} = \dfrac{\text{Undisturbed shear strength}}{\text{Remolded shear strength}}$



LEGEND

CONSOLIDATION PRESSURE

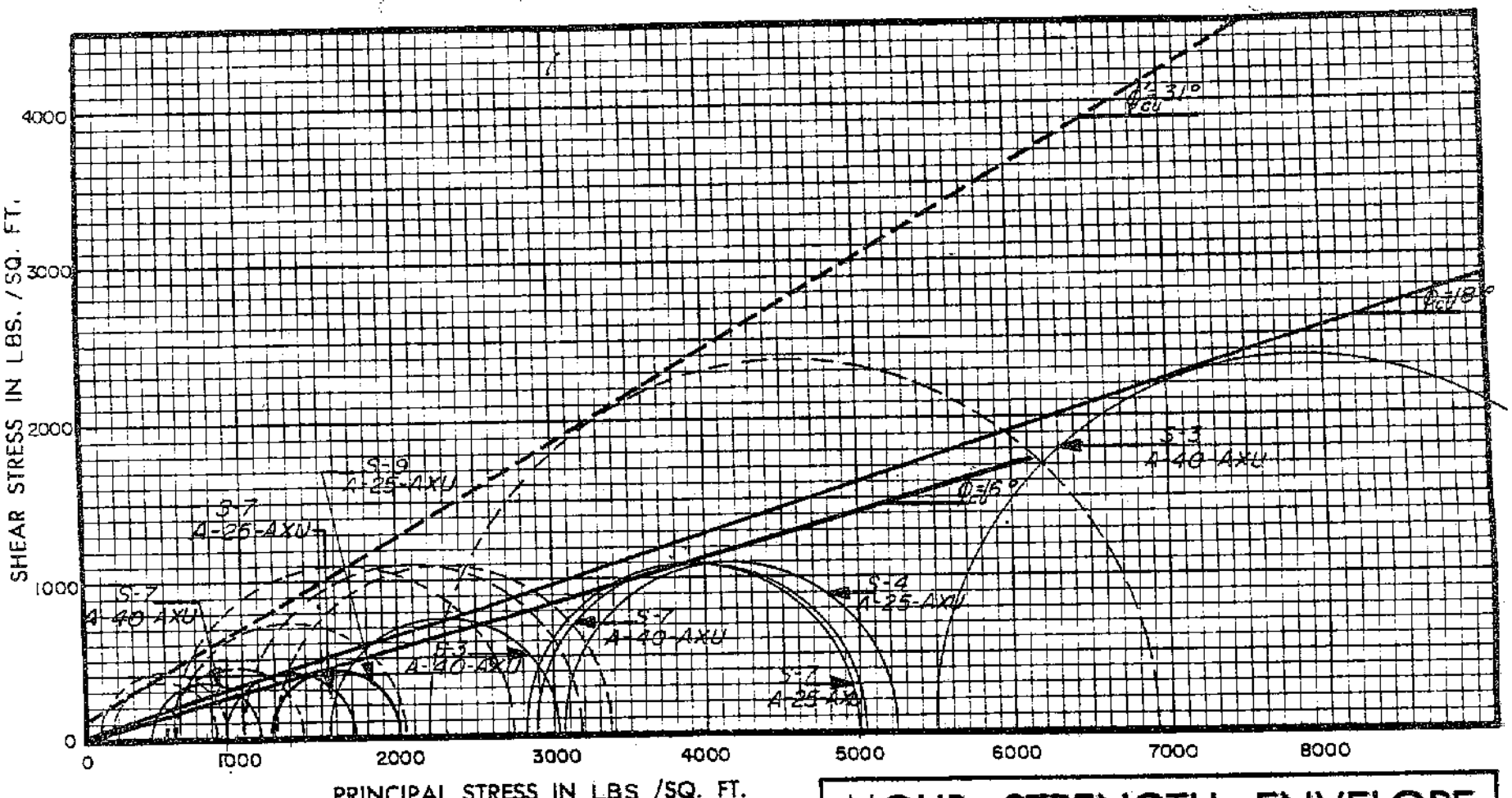
- ■ 1/4 TSF
- ○ 1/2 TSF
- × 3/4 TSF
- ● 1 TSF
- □ 1 1/2 TSF
- + 2 TSF
- ▲ Triaxial Dissipation Test, S-1, UP-15-CU
- △ Consolidation Pressure 1/2 to 1 TSF, S-1, UP-15-CU.
- 2I 3 inch Shelby Tube Sample. No. to Left is Sample No.

PROPOSED DAM - TRINIDAD ARM
GATUN LAKE, PANAMA CANAL ZONE

COEFFICIENT OF CONSOLIDATION VS. DEPTH
DRILL HOLE A-25-AXU
W-62-147
FEBRUARY, 1963

Shannon and Wilson, Inc.
Soil Mechanics and Foundation Engineers

FIG. 5-3



MOHR STRENGTH ENVELOPE

TRIAXIAL COMPRESSION TEST

ORGANIC CLAY

W-62-147 *FEBRUARY, 1963*

FIG. 5-4

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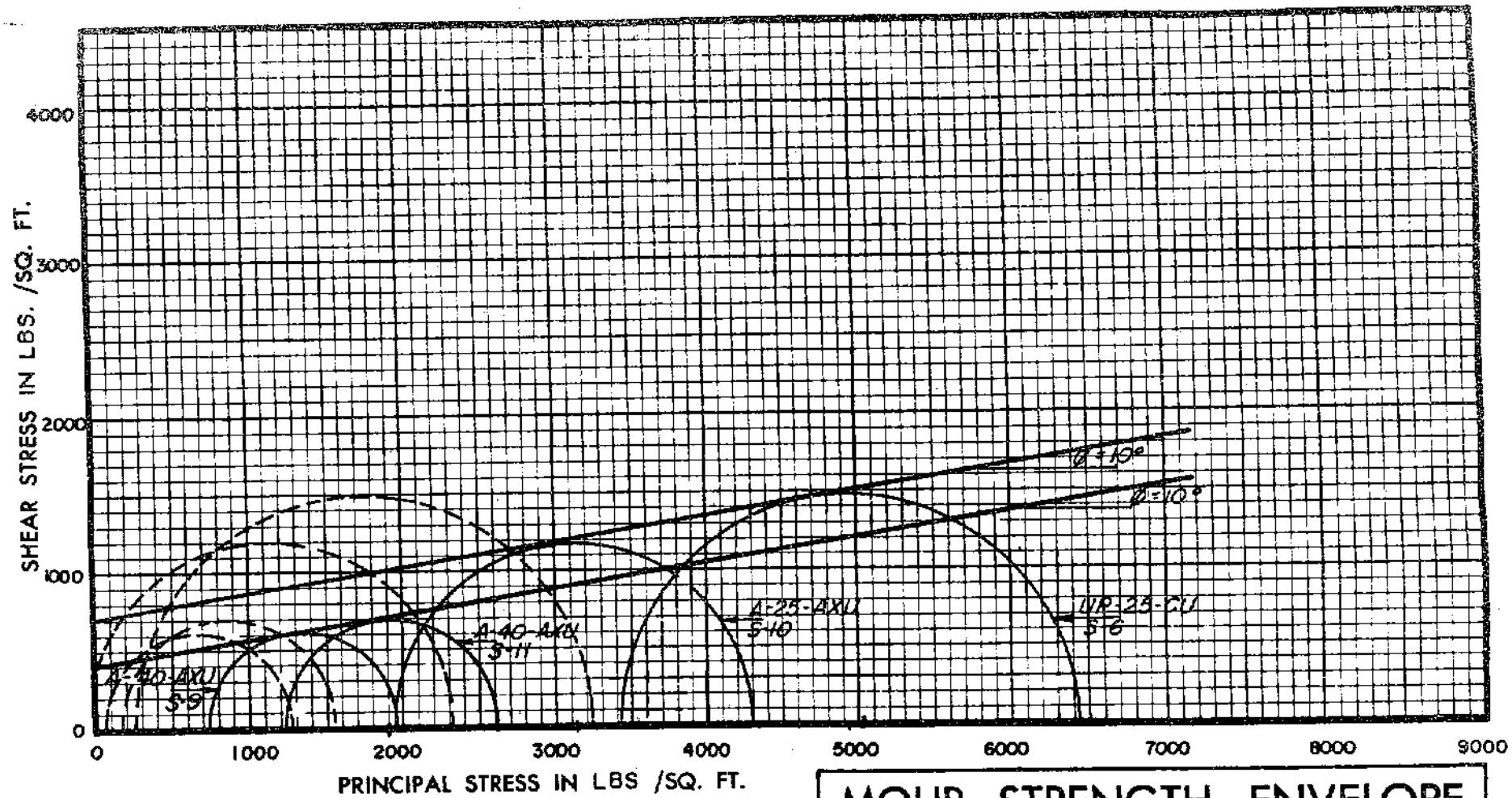
Soil mechanics and foundation engineers
Seattle

TYPE OF TEST

- □ Q
- ■ $\bar{R}$ UNDISTURBED SAMPLES
- □ S
- □ PRESATURATED
- ——— TOTAL STRESSES
- — — — EFFECTIVE STRESSES



MOHR STRENGTH ENVELOPE

TRIAXIAL COMPRESSION TEST

FIBROUS PEAT

W-62-147 *FEBRUARY, 1963*

FIG. 5-5

Shannon and Wilson
Soil mechanics and foundation engineers
Seattle

TYPE OF TEST

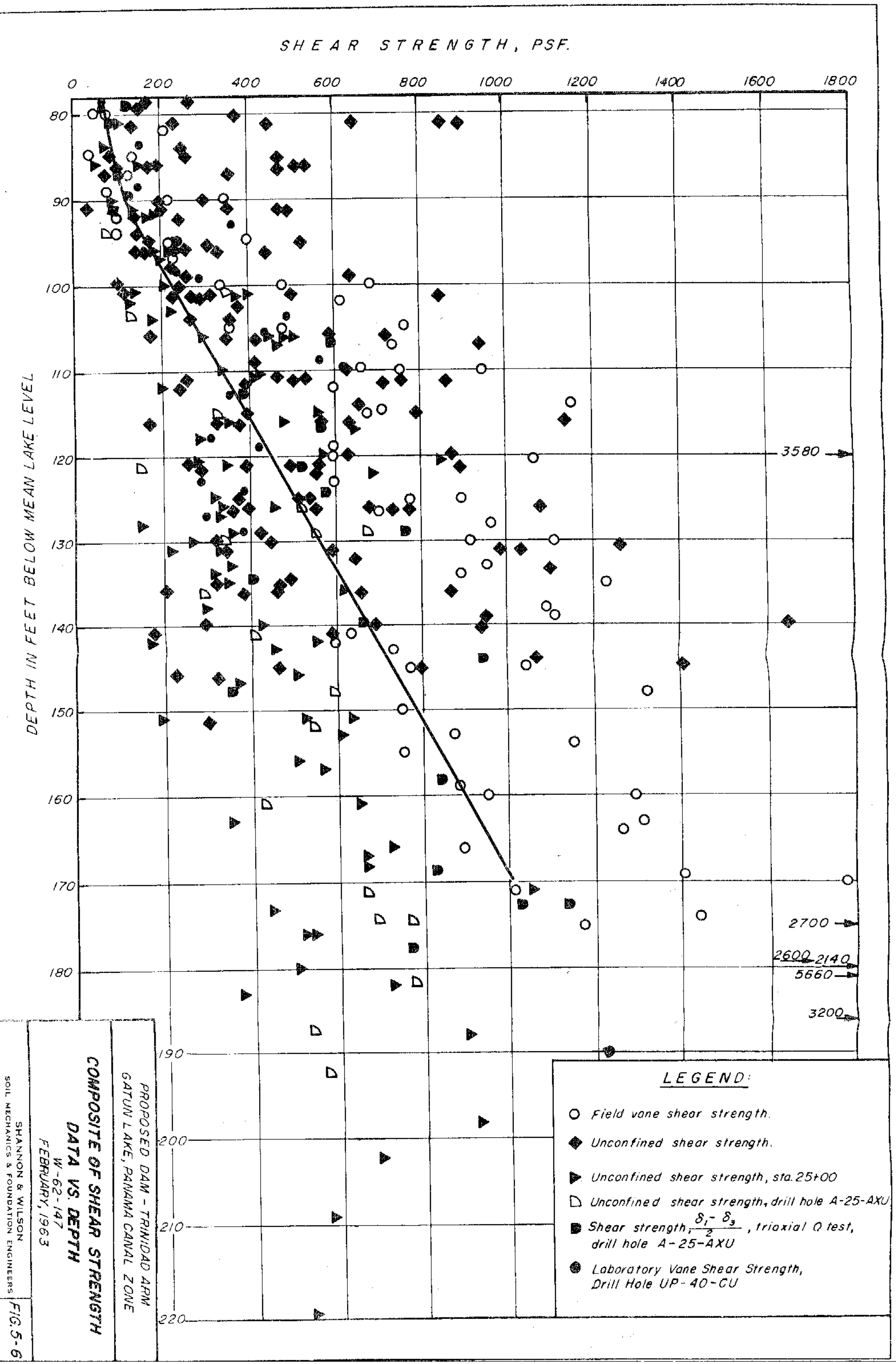
- □ Q
- ■ $\bar{R}$ UNDISTURBED SAMPLE
- □ S
- □ PRESATURATED
- ——— TOTAL STRESSES
- — — — EFFECTIVE STRESSES

field vane shear tests. The sensitivity of the foundation materials ranges from 2 to 9 with a median value of about 5. A value of 5 indicates that severe remolding of the foundation materials, as might be produced by "uncontrolled" barge dumping operations, could result in approximately an 80 percent reduction of foundation shear strength.

5-12. IN-SITU SHEAR STRENGTH.

The "in-situ" shear strength of foundation materials was obtained from evaluation of data from both field and laboratory testing programs. Values of shear strength determined from field vane shear tests and compression tests are plotted on Figure 5-6. The plotted points are widely scattered, but show a general trend of increasing shear strength with depth. The scatter of points is a result of differences in material properties and test conditions. The factors influencing the test data and the reasoning employed in construction of the representative shear strength versus depth line shown on the figure are discussed in the following paragraphs.

Sample disturbance normally produces laboratory shear strengths which are lower than "actual" values. Test specimens are always somewhat disturbed regardless of refinements in sampling procedures and equipment and care in specimen preparation. Disturbance results partly from expansion of the soil structure as in-situ confining pressures are removed during the sampling operation. The soft, normally consolidated clays and peats were particularly susceptible to expansion during sampling since sample pore water pressures were reduced from an equivalent head of water greater than eighty feet to below atmospheric. Additional sample disturbance occurs in the laboratory as the sample is extruded from the Shelby tube and handled during specimen preparation. Unconfined shear strength data reflect the influence of sample disturbance more than do the data from other laboratory strength tests. This influence is revealed by comparison of unconfined shear strength and field vane shear strength data at similar locations (see Figure 5-7). The data indicate that values of shear strength from field vane shear tests are generally 1.5 to 2 times the values of shear strength obtained from unconfined compression tests. Laboratory tests which minimize sample disturbance resulting from handling or



PROPOSED DAM - TRINIDAD ARM
GATUN LAKE, PANAMA CANAL ZONE

COMPOSITE OF SHEAR STRENGTH DATA VS. DEPTH

W-62-147
FEBRUARY, 1963

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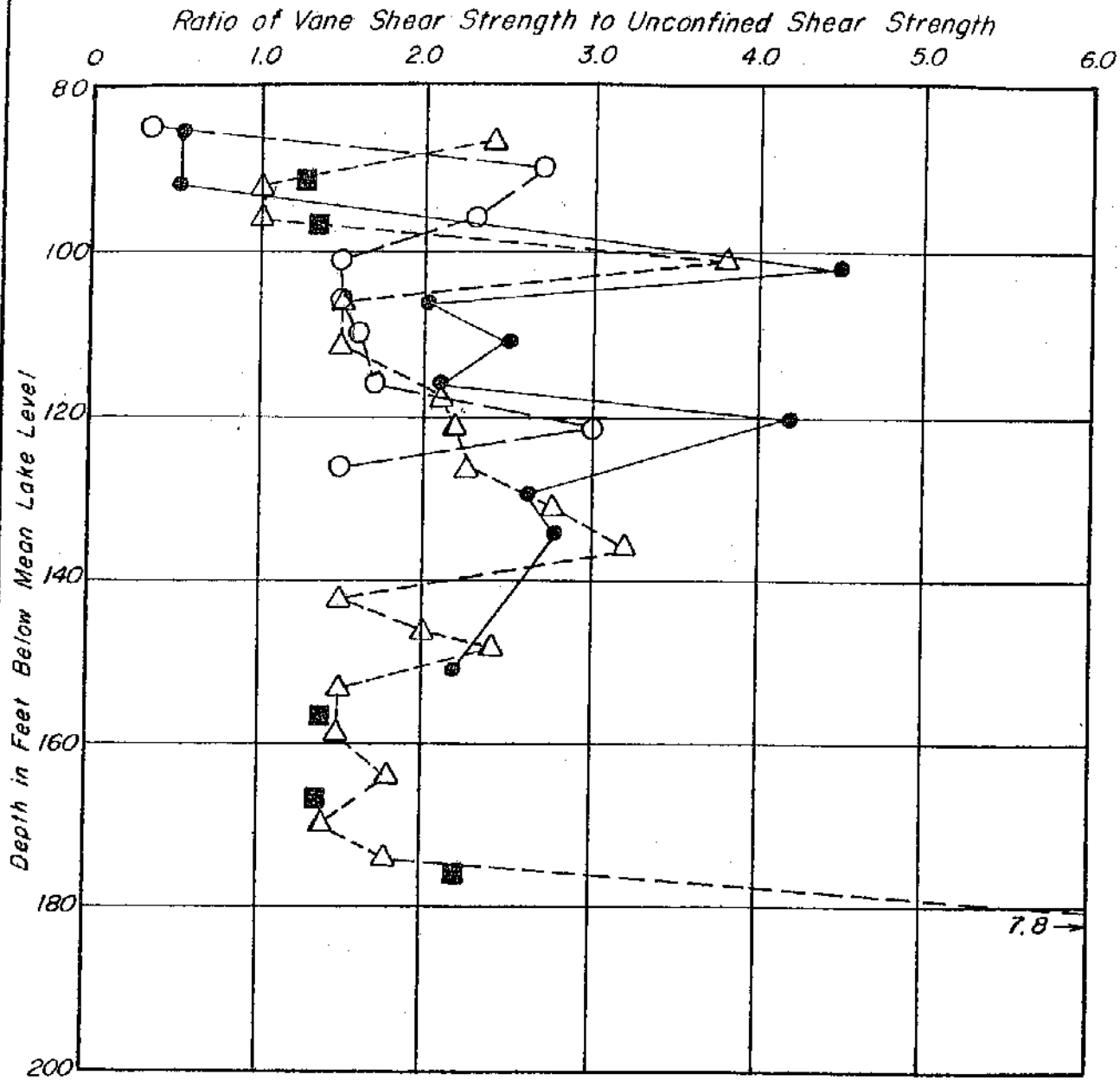
FIG. 5-6



LEGEND:

- ● UP-25-BC
- ○ DN-25-DC
- △ DN-25-CC
- ■ DN-25-BC

PROPOSED DAM - TRINIDAD ARM
GATUN LAKE, PANAMA CANAL ZONE

RATIO OF FIELD VANE SHEAR STRENGTH TO UNCONFINED SHEAR STRENGTH VS. DEPTH

W-62-147 FEBRUARY, 1963

SHANNON & WILSON
SOIL MECHANICS & FOUNDATION ENGINEERS

FIG. 5-7

which attempt to restore in-situ stress conditions to the sample prior to application of the deviator stress, produce the most reliable values of "in-situ" shear strength.

The field vane shear test is most valuable for determining "in-situ" shear strength of soft, normally consolidated soils which are likely to be disturbed by sampling operations. Drill barge movements and the existence of pieces of wood and zones of peat in the foundation materials, however, affected the vane shear test data at the Trinidad site. The vane assembly at shallow depths was more sensitive to barge movements than at deeper depths. The foundation materials near the mudline, therefore, were more likely disturbed and such disturbance probably resulted in low values of vane shear strength. The influence of disturbance on field vane test data at shallow depths is indicated by the ratio of field vane shear strength and unconfined shear strength resulting in values of 1 close to the mudline (see Figure 5-7). The foundation materials to about depth 50 feet below the mudline generally contain chunks of wood and zones or seams of fibrous peat. These materials affected rotation of the vane and resulted in high values of shear strength. Below depth 50 feet, the amount of organic material decreased and the results of the vane test are not believed to have been significantly affected by pieces of foreign matter.

Based upon consideration of the factors affecting shear strength data, it was concluded that the undrained (Q) triaxial tests and the field vane shear tests, particularly below depth 50 feet (reference mudline) provided the most reliable values of "in-situ" shear strength.

Since the undrained (Q) triaxial tests and most of the field vane shear tests were conducted at station 25+00, it was necessary to determine whether these data were representative of the weaker foundation materials. Index properties as well as values of unconfined shear strength of various samples taken at station 25+00 were compared with the index properties and unconfined shear strength data at other locations. This comparison revealed that values of unconfined shear strength at station 25+00 locations were among the lowest values of strength found (see Figure 5-6). In addition, a series of laboratory vane shear tests was conducted on samples from drill hole UP-40-CU to investigate the low values of unconfined shear strength previously obtained

on samples from drill hole UP-40-CC, particularly at shallow depths. These values of vane shear strength, allowing for factors which affect the test data, were considered comparable to values of shear strength obtained at similar depths from Q tests in drill hole A-25-AXU.

In summary, analysis of all strength data indicates that values of shear strength determined along station 25+00 were representative of the "weaker" foundation materials throughout the site.

Based upon the foregoing evaluation of strength data, a "representative" shear strength versus depth line was constructed as shown in Figures 5-2 and 5-6. The slope of this line as defined by the Cu/p[1] ratio is equal to 0.46. From other investigations in similar foundation materials, the Cu/p ratio has been evaluated to be in the order of 0.39[2] or about 15 percent lower (see Figure 5-2). The "representative" shear strength with depth line constructed for this site is, therefore, in general agreement with the findings of other investigations.

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1 Cu/p where Cu = undrained shear strength and
p = vertical effective overburden pressure.

2 Bishop, A. W. and Henkel, D. J., "The Measurement of Soil Properties in the Triaxial Test," Edward Arnold Ltd., London, 1957, p. 98.

SECTION VI

SETTLEMENT ANALYSES

6-1. GENERAL.

The series of consolidation tests performed in the Seattle laboratory indicates that the clay and peat commonly encountered at the site are highly compressible, extremely impervious and, in general, normally consolidated. Due to the impervious nature and the great thickness of the compressible strata, the process of consolidation under the weight of the proposed embankment will be extremely slow. In localized areas, the soft compressible stratum was found to be mantled by several feet of stiff desiccated clay which is considered less pervious and compressible than the underlying soft material. The presence of this stiff cap will retard the process of consolidation and decrease total settlement in these areas.

Differential settlement will develop due to variations in foundation soil and drainage conditions as well as differences in loading conditions. The settlement near the toe of the dam, for example, will be appreciably less than that beneath the crest of the dam.

The settlement analyses presented in this section are based on the results of field explorations and laboratory testing, as well as theoretical considerations and assumptions. It should be emphasized that the results thus obtained are estimates of the foundation settlements to be anticipated and, as a result, substantial deviations will probably develop between the predicted and actual settlements. As field settlement data become available, the "time versus settlement" relationships obtained from the theoretical analyses should be revised so that future settlements can be more accurately predicted.

6-2. MAJOR FACTORS INFLUENCING SETTLEMENT ANALYSES.

The following paragraphs discuss the major factors considered in estimating foundation settlements.

6-3. FOUNDATION CONDITIONS.

Thicknesses of compressible strata were interpreted from the results of field explorations. Based on the boring logs at drill holes A-25-AX and A-40-AX, thicknesses of 120 and 65 feet respectively, were chosen for the analyses. These are considered to be typical of the thickness of compressible strata existing in the area between station 15+00 and 40+00, except in those areas where stiff, desiccated clay was encountered which capped the soft, compressible stratum.

6-4. LOADING.

The saturated unit weight of the underwater portion of the fill was assumed to be 120 pounds per cubic foot. The weight effective in producing settlements (submerged unit weight) is, therefore, 57.5 pounds per cubic foot. For the fill above lake level, a wet unit weight of 120 pounds per cubic foot was used.

The fill placement schedule was assumed to be linear with time as follows. The fill will be dumped to elevation +75 during the first three years. During the fourth year the fill will remain unchanged at elevation +75. In the fifth year the fill will be placed to the crest of the dam and foundation loading will be completed. The loading in the analyses does not include the additional fill placed to compensate for settlement which takes place during construction. The effect of this additional fill on the settlement, however, will be nominal.

6-5. STRESSES.

In the central portion of the Dam, the vertical stresses in the foundation material calculated using the elastic theory were found to be practically constant with depth to bedrock, the stresses at depth being equal to the pressure applied at the mudline. The stresses at the toe area, however, will

be greater than the applied pressure and will increase with depth. Due to the flexible nature of the fill structure, some irregular stress concentrations are likely to develop, but only uniform stress distributions were assumed in the analyses.

6-6. LABORATORY DATA.

A typical Stress versus Percent Settlement relationship was chosen from the results of the consolidation tests and was employed on the basis that the compressible foundation materials are normally consolidated. The values of coefficient of consolidation, C_v, were used in establishing the relationships between time and increase in intergranular pressure at various depths.

6-7. THEORETICAL ASPECTS.

The settlement analyses are based on theory of consolidation and, therefore, deal only with the ultimate settlement which occurs with dissipation of excess porewater pressure.

The average degree of consolidation of the foundation material at any given time in the construction schedule was determined from curves developed by R. L. Schiffman[1]. These curves extend the theory of consolidation to include variations in rate of loading which occur during construction and variations in the coefficient of permeability which occur during the process of consolidation.

The effect on settlement of secondary consolidation and lateral creep of the soft compressible stratum is considered to be relatively minor in the time span considered and hence is not included in the analyses.

6-8. PRACTICAL ASPECTS.

It is expected that dumping the first several feet of fill will cause some disturbance of the foundation material at the mudline and that some of the fill material will penetrate into the soft foundation. There is, however, no reliable

---

1 Schiffman, R. L., "Consolidation of Soil Under Time-Dependent Loading and Varying Permeability," Proceedings HRB Vol. 37, 1958.

way of evaluating the extent and effect of such disturbance on the consolidation characteristics of the materials. The analyses presented herein assume that there will be no disturbance when the initial load is applied. We strongly recommend that construction materials and procedures designed to minimize disturbance be specified.

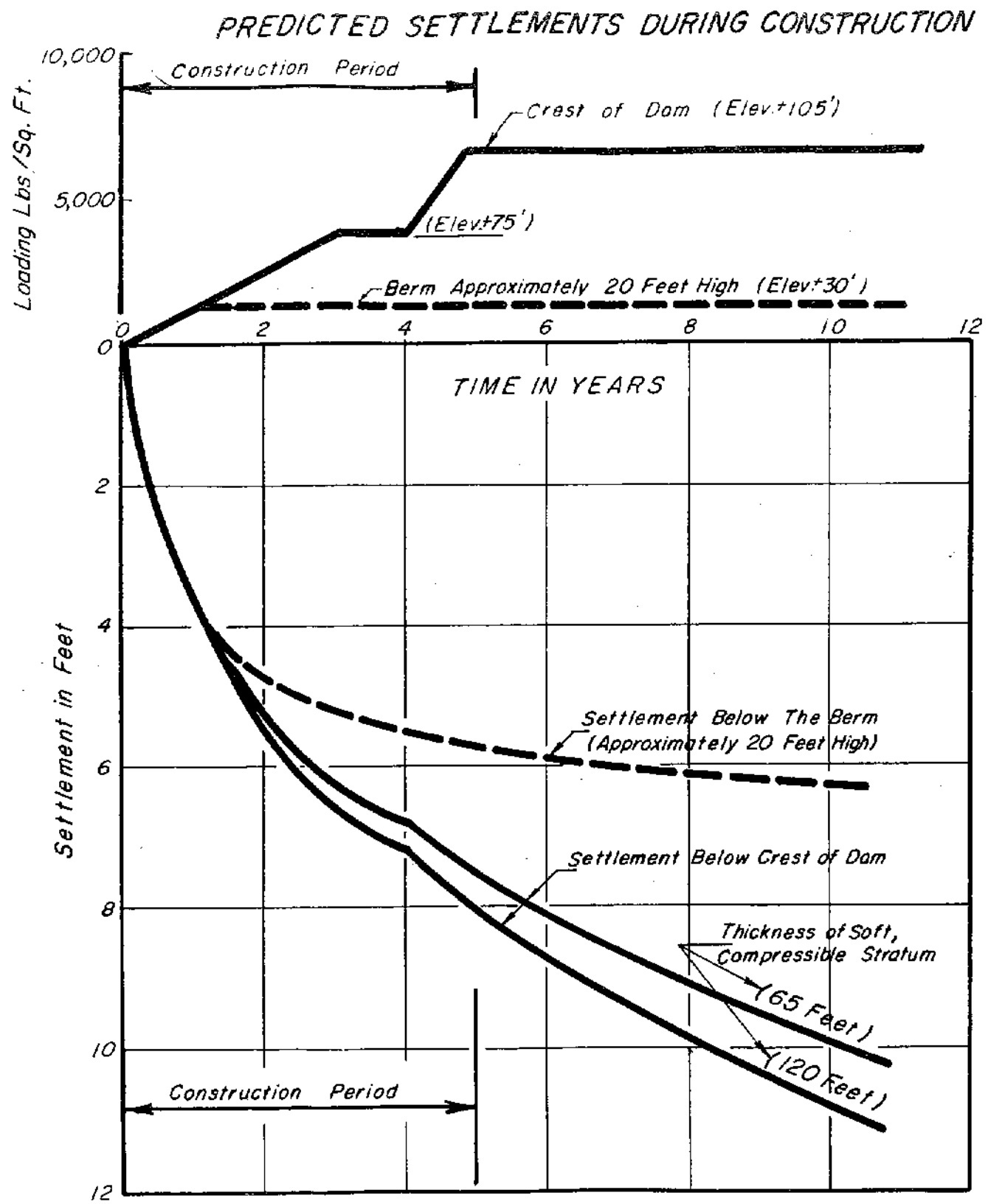
6-9. SETTLEMENT DURING CONSTRUCTION.

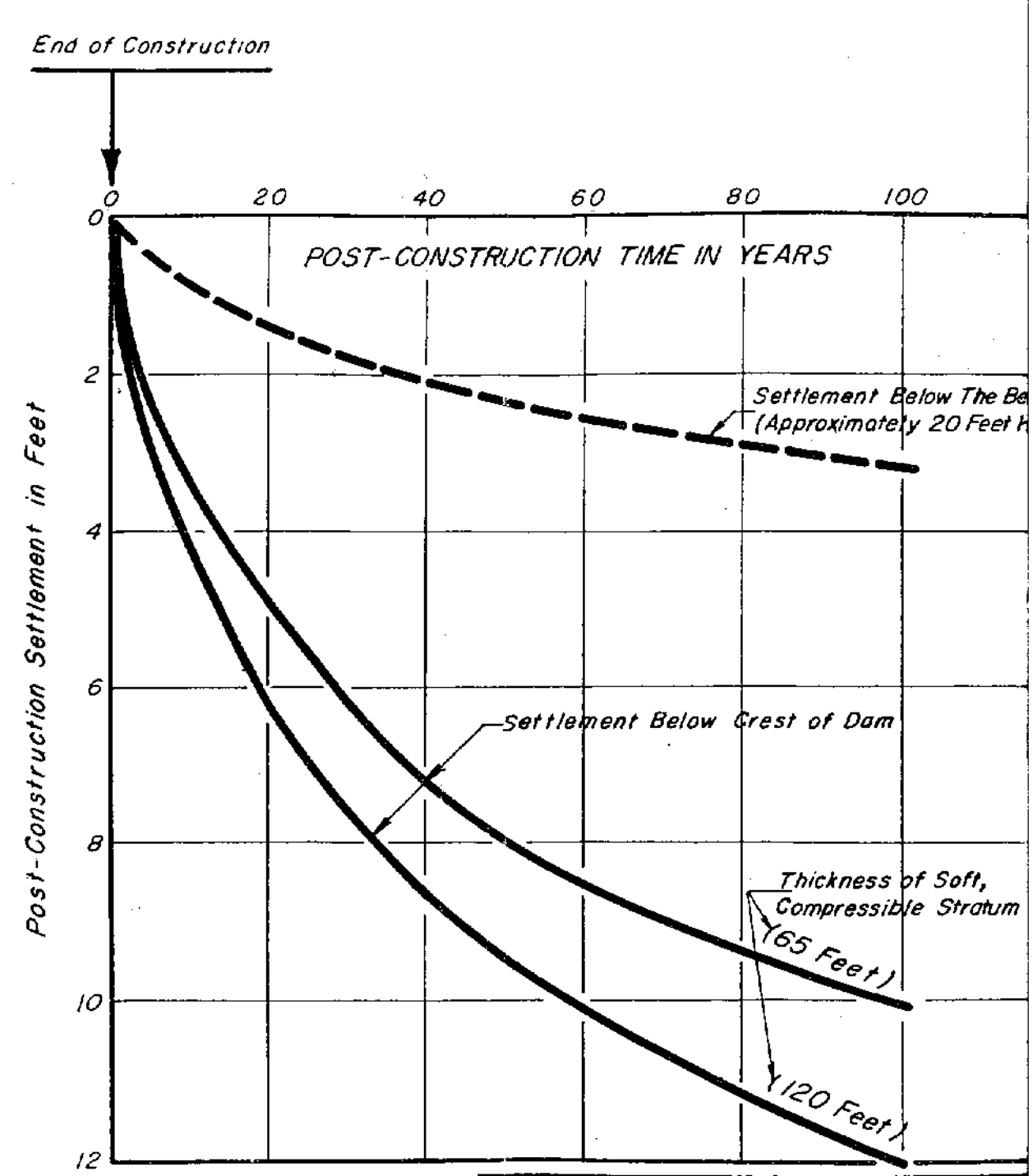
The "Time versus Settlement" relationships calculated on the foregoing bases are illustrated in Figure 6-1 together with the assumed time-loading schedule. Settlements were analysed for two different soil profiles: One with the compressible stratum 120 feet thick and the other with the compressible stratum 65 feet thick. In addition, two different loading conditions were considered: One below the crest of the dam and the other under a berm at elevation +30. The time versus settlement relationships beneath the dam crest are indicated by solid lines and those beneath the elevation +30 berm by dotted lines. The difference in the settlements at the berm for the two assumed thicknesses of compressible strata is minor and, therefore, is not differentiated in Figure 6-1.

The analyses indicate that the central portion of the dam will settle as much as 8 feet before the fill structure is completed. The settlements of the dam outside station 15+00 and station 40+00 will gradually decrease to almost zero as the compressible stratum becomes thinner toward the abutments. In view of the probably flexible nature of the fill structure and the gradual change in the magnitude of the settlements, the dam embankment, in our opinion, will not be adversely affected by differential settlements occurring during construction.

6-10. SETTLEMENT AFTER CONSTRUCTION.

Large post-construction settlements will continue at a decreasing rate over a long period of time. This trend is illustrated in Figure 6-1. As time proceeds, variations in the existing foundation and loading conditions will result in substantial differential settlements.





PROPOSED DAM - TRINIDAD ARM
GATUN LAKE, PANAMA CANAL ZONE

PREDICTED FOUNDATION SETTLEMENTS

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The settlement analyses indicate that between the Axis of the dam and the berm at elevation +30, a horizontal distance of approximately 350 feet, differential foundation settlements will be in the following order of magnitude:

at the end of construction	1 to 3 feet
5 years after construction	3 to 5 feet
20 years after construction	5 to 7 feet
50 years after construction	7 to 10 feet

In the longitudinal section, the anticipated differential settlements will be greater where the compressible stratum thins toward the Guacha Island abutment than those anticipated near the Mainland abutment. The total foundation settlement at station 40+00 is represented approximately by the curve in Figure 6-1 for the compressible stratum 65 feet thick, whereas the settlement at station 48+00 will be practically zero. The difference in settlement over this distance of approximately 800 feet may reach somewhere in the order of 8 feet by the end of construction and as much as 15 feet in 50 years after the dam is constructed. In the areas near station 40+00 and station 35+00, where stiff desiccated clays mantle the thick compressible materials, greater differential settlements may develop. It is believed, however, that the mode and magnitude of these differential settlements will not impair the performance of the Dam.

Settlements will require placement of additional rolled fill material at periodic intervals to maintain the design crest elevation of the dam. The amount of fill material required and the time intervals for placement can be estimated from Figure 6-1. Additional settlement caused by placement of this fill material at periodic intervals will be negligible.

It will be recognized that, to provide for placing additional fill at the crest to compensate for settlement, provision will have to be made in the design for an initial crest width which is wider than the nominal design width. To provide additional width economically, it would be permissible to steepen the embankment slopes for the upper ten feet of the dam to perhaps a slope of 2 Horiz. to 1 Vert.

SECTION VII

SHEAR STRENGTH INCREASE WITH TIME

7-1. GENERAL.

Consolidation of the foundation materials under dam embankment loads will result in an increase in foundation shear strength. Since foundation shear strengths are low, any increase in strength resulting from the consolidation process should be considered in the design of the dam section. The gain in foundation shear strength resulting from the consolidation process, therefore, was determined in this investigation for various periods in the construction schedule.

7-2. THEORETICAL CONSIDERATIONS.

Shear strength gain is a function of the intergranular pressure which gradually increases as consolidation proceeds. A study was made to determine the relationships between time and increase in intergranular pressure by use of curves developed by R. L. Schiffman[1].

In the study, various drainage conditions and drainage layer thicknesses were assumed. It was found, however, that the depth affected by foundation consolidation in the initial five year construction period did not significantly vary from about depth 20 feet. In all calculations an effective drainage zone was assumed to exist between the embankment fill and the foundation.

Based on available information for fill materials and the shear strength data of the foundation material, two sets of assumptions were made: (1) Fill saturated unit weight equal to 100 pounds per cubic foot, foundation strength parameters, $c = 0$ and $\emptyset = 19^{\circ}$, and (2) Fill saturated unit weight equal to 120

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[1] Refer to paragraph 6-7.

pounds per cubic foot, foundation strength parameters, $c = 0$ and $\emptyset = 17^{\circ}$. The values of fill unit weight used are tentative pending further fill characteristic studies.

7-3. INCREASE-IN-SHEAR-STRENGTH DETERMINATION AND RESULTS.

The increase in shear strength of the foundation material with time was evaluated in the following manner. The increase in effective stress occurring in the foundation due to consolidation under embankment loads was determined at a particular depth and at a given time. The sum of the increase in effective stress and the effective overburden pressure was considered an effective consolidation pressure, σ_c. The shear strength of the foundation material consolidated under an effective pressure, σ_c, was determined from the Mohr Strength Envelope plot on Figure 5-4.

The results of the evaluation of shear strength gain with time are shown by curves on Figures 7-1 through 7-3. Only curves derived from the second set of assumptions are presented. The variation in results between the two sets of assumptions will not significantly influence the stability analyses.

The curves of foundation shear strength for use in the stability analyses are presented for certain times in the construction schedule and for particular portions of the dam section. The curves in Figure 7-1 for example, present foundation shear strength existing at the end of the first year of construction with the assumption that the height of fill would have reached approximately the elevation shown. The shear strength profiles on Figures 7-1, 7-2, and 7-3 are applicable under various portions of the proposed dam section as noted on each figure. Values of shear strength for use in the stability analyses can be selected from the curves for each portion of the dam or interpolated between portions as noted on the figures.

On Figure 7-3 are shown the shear strength profiles at the end of both the 3rd and 4th year of construction. These profiles are based on the assumption that the fill will be at about elevation +75 at the end of both periods. For evaluation of the dam topped to elevation +105, the shear strength profile given for the end of the fourth year should be used.

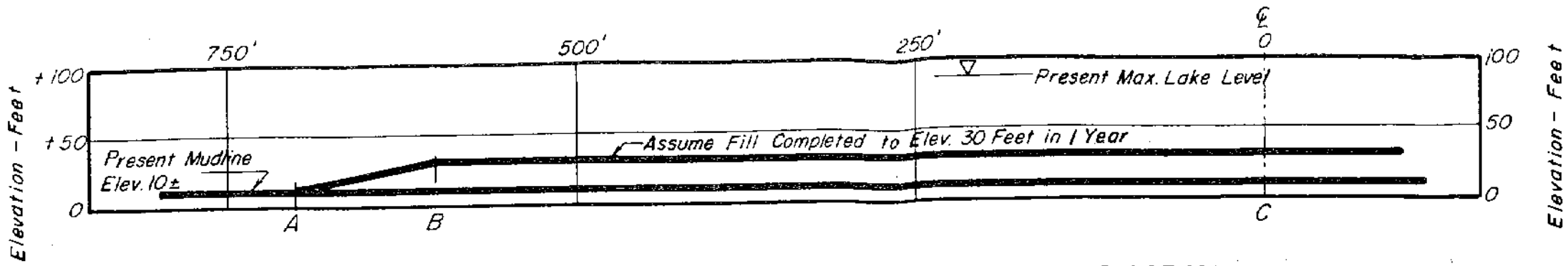


DIAGRAM I

(Existing Shear Strength Vs. Depth Relationship)

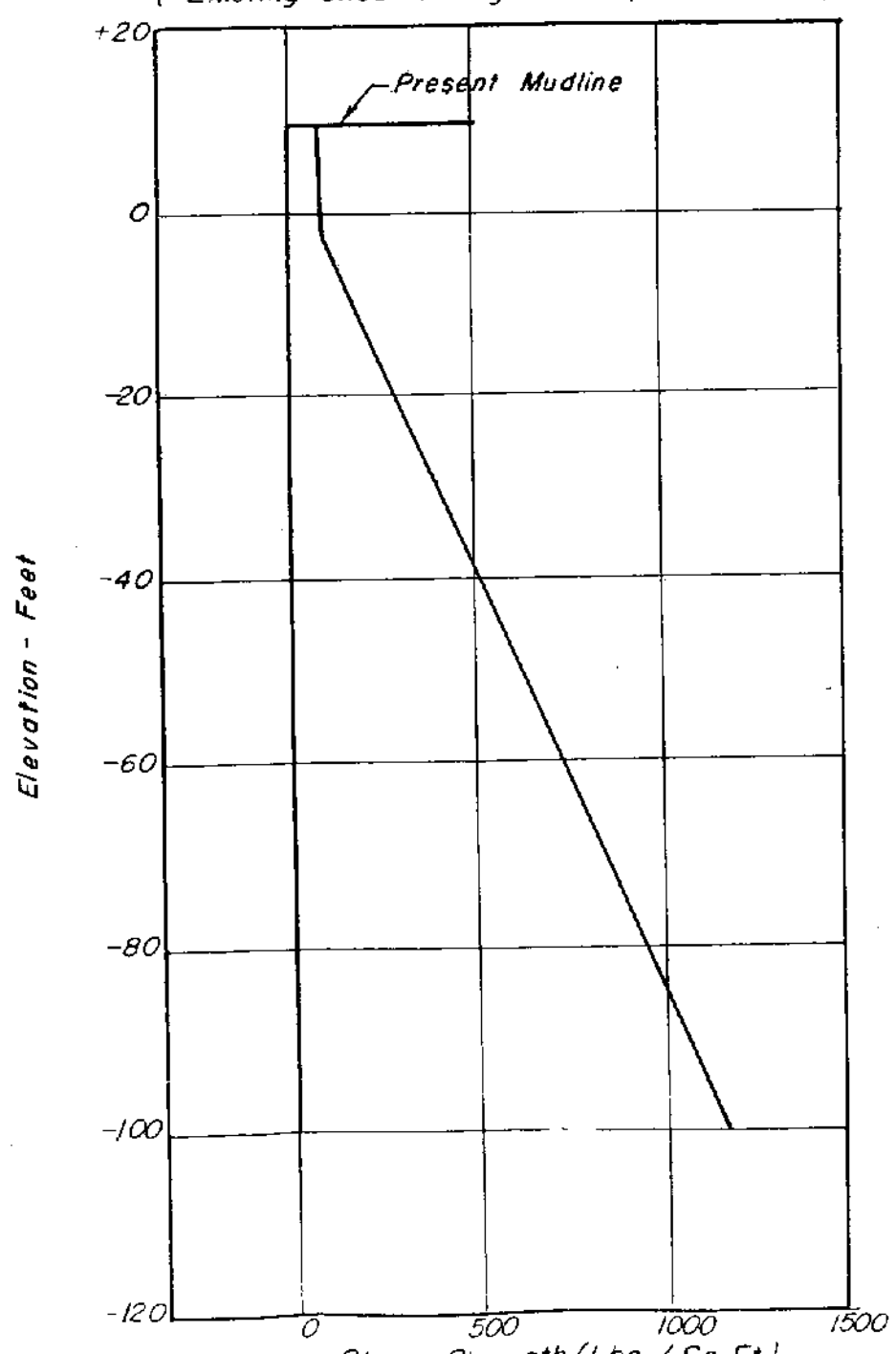
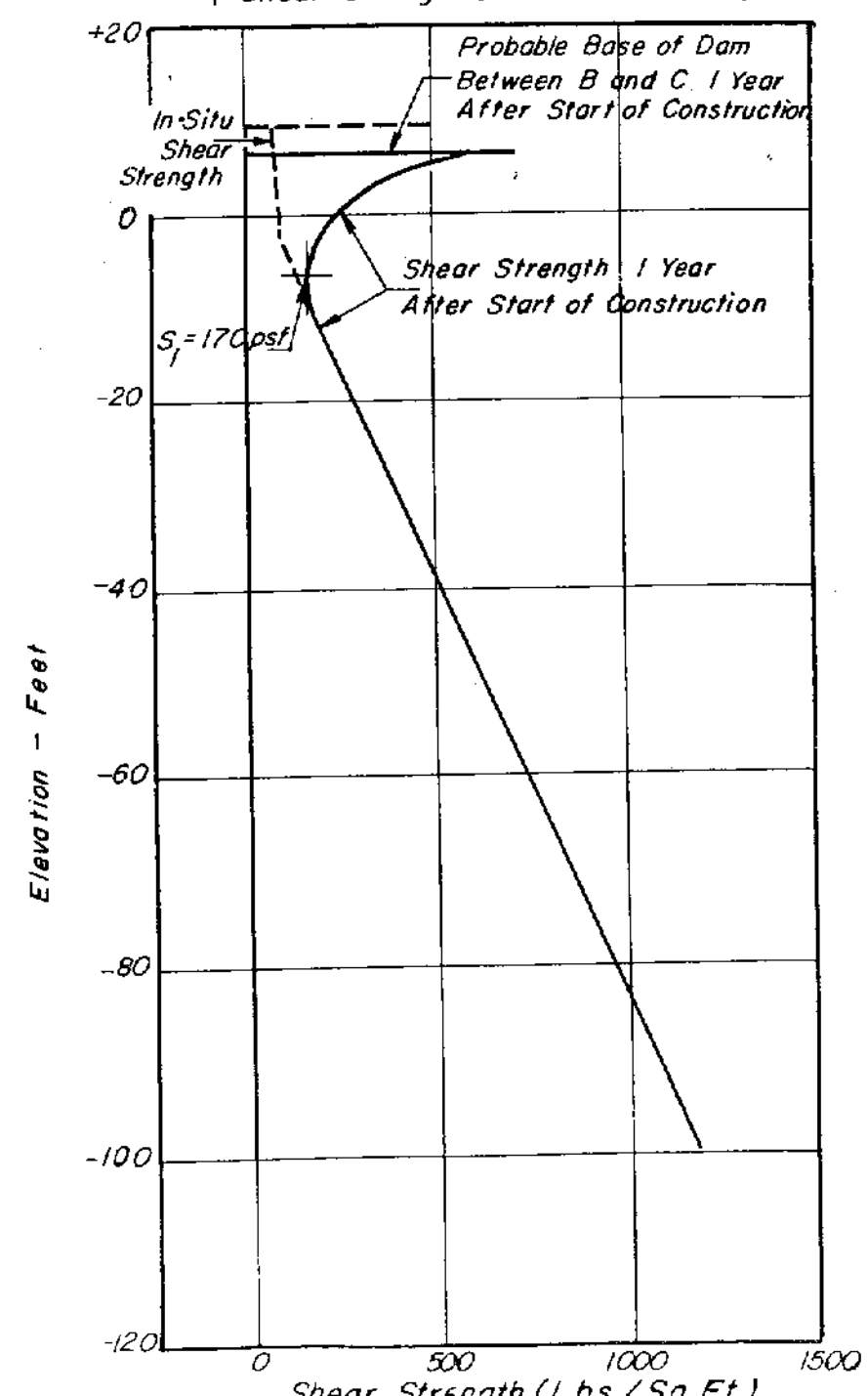


DIAGRAM II

(Shear Strength Between B and C)



NOTES:

1. Between A and B, Use Shear Strength Interpolated from Diagrams I and II
2. Between B and C, Use Diagram II
3. S_f = Calculated Minimum Shear Strength 1 Year After Start of Construction.

PROPOSED DAM - TRINIDAD ARM
GATUN LAKE, PANAMA CANAL ZONE

DATA FOR STABILITY ANALYSES

Depth Vs. Shear Strength

1 YEAR

After Start of Construction

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FIG. 7

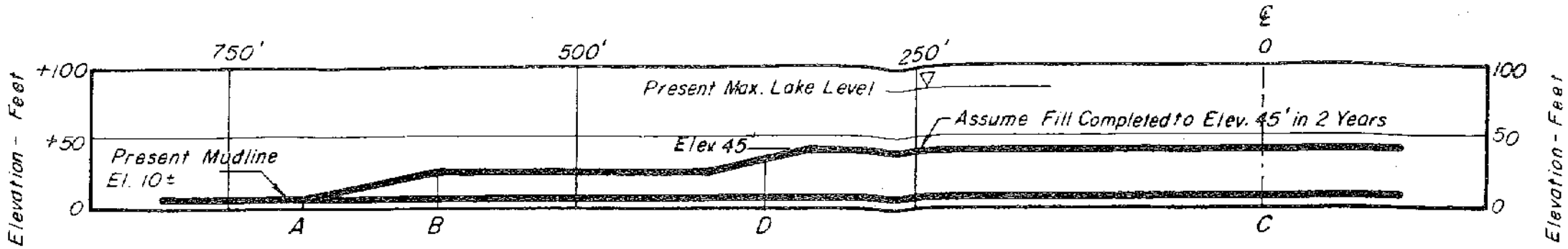


DIAGRAM III
(Shear Strength Between B and D)

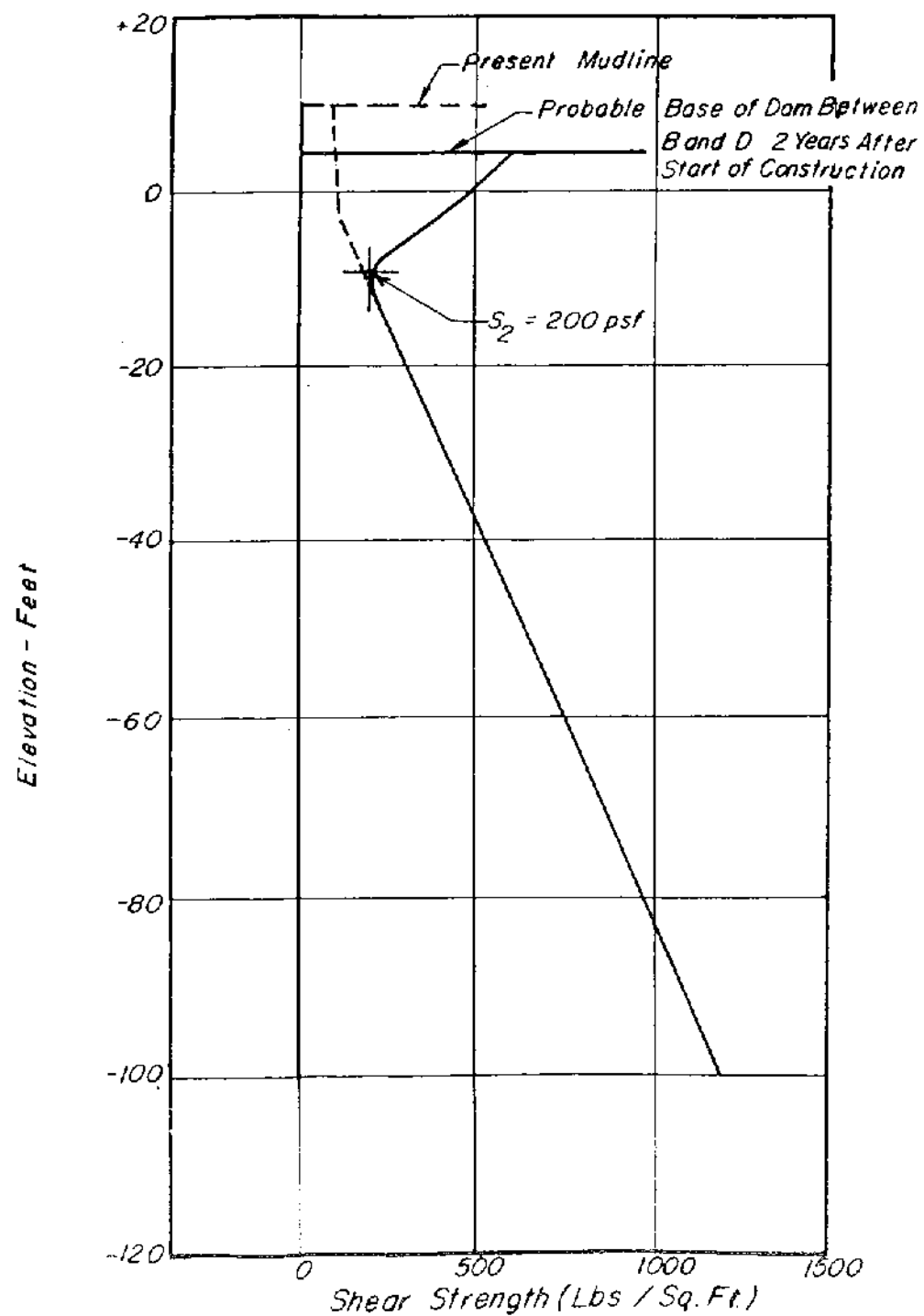
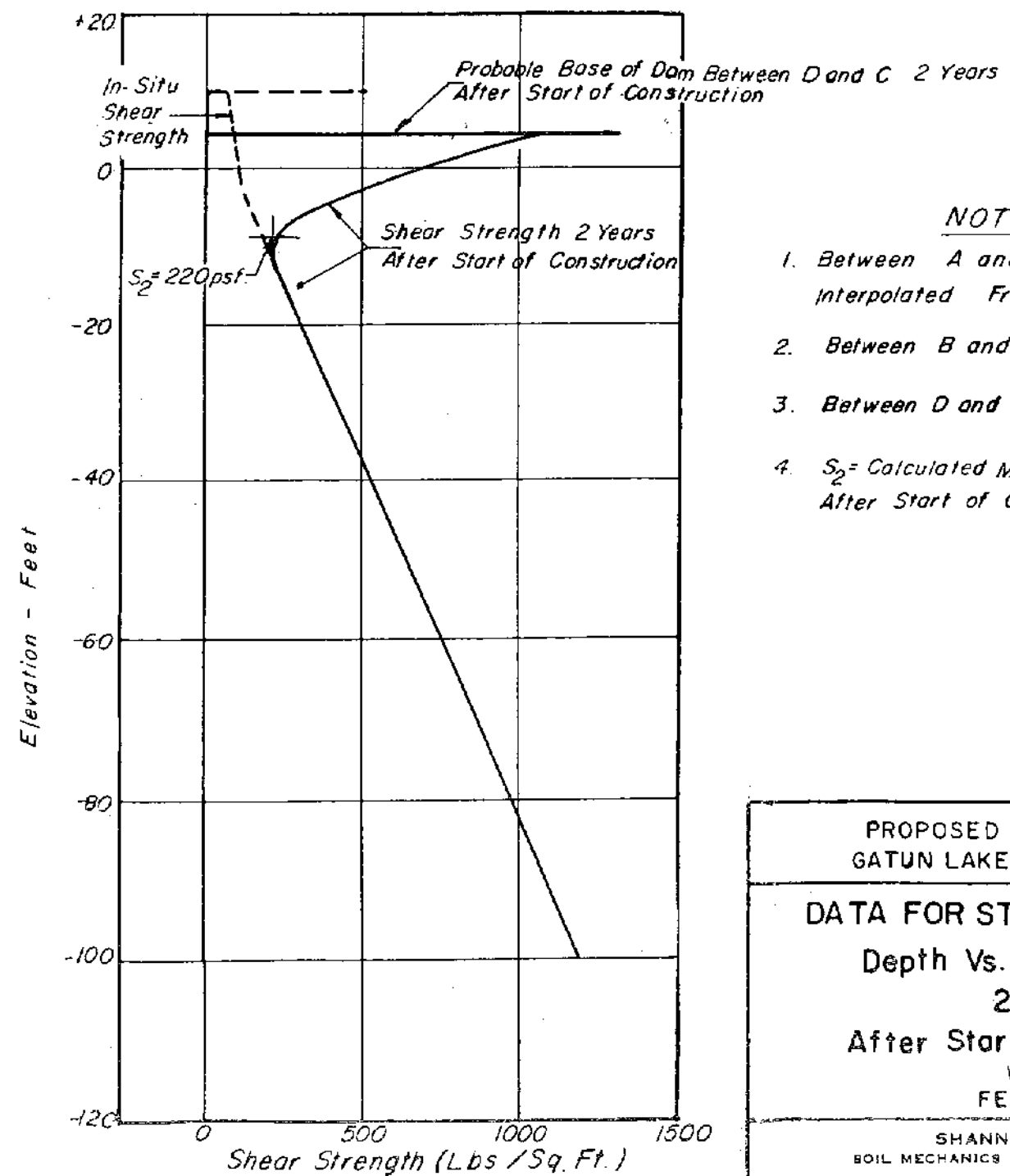


DIAGRAM IV
(Shear Strength Between D and C)



NOTES

1. Between A and B, Use Shear Strength Interpolated From Diagrams I (Fig. 7-1) and III
2. Between B and D, Use Diagram III
3. Between D and C, Use Diagram IV
4. S_2 = Calculated Minimum Shear Strength 2 Years After Start of Construction.

PROPOSED DAM - TRINIDAD ARM
GATUN LAKE, PANAMA CANAL ZONE

DATA FOR STABILITY ANALYSES
Depth Vs. Shear Strength
2 YEARS
After Start of Construction
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FIG. 7-2

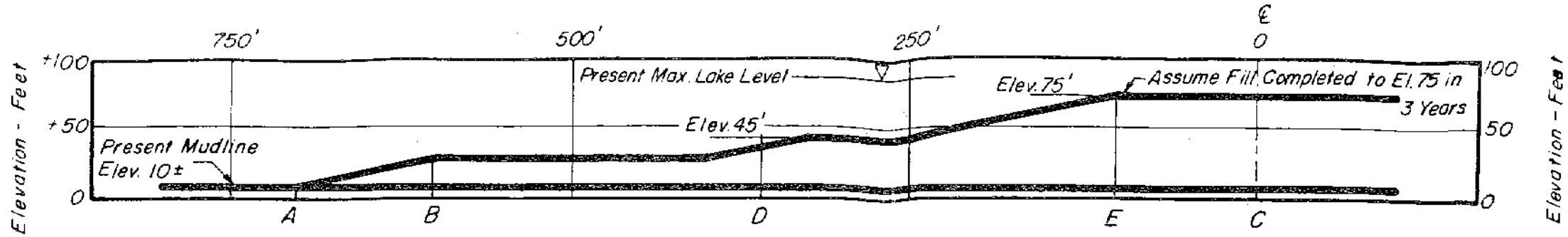


DIAGRAM V
(Shear Strength Between B and D)

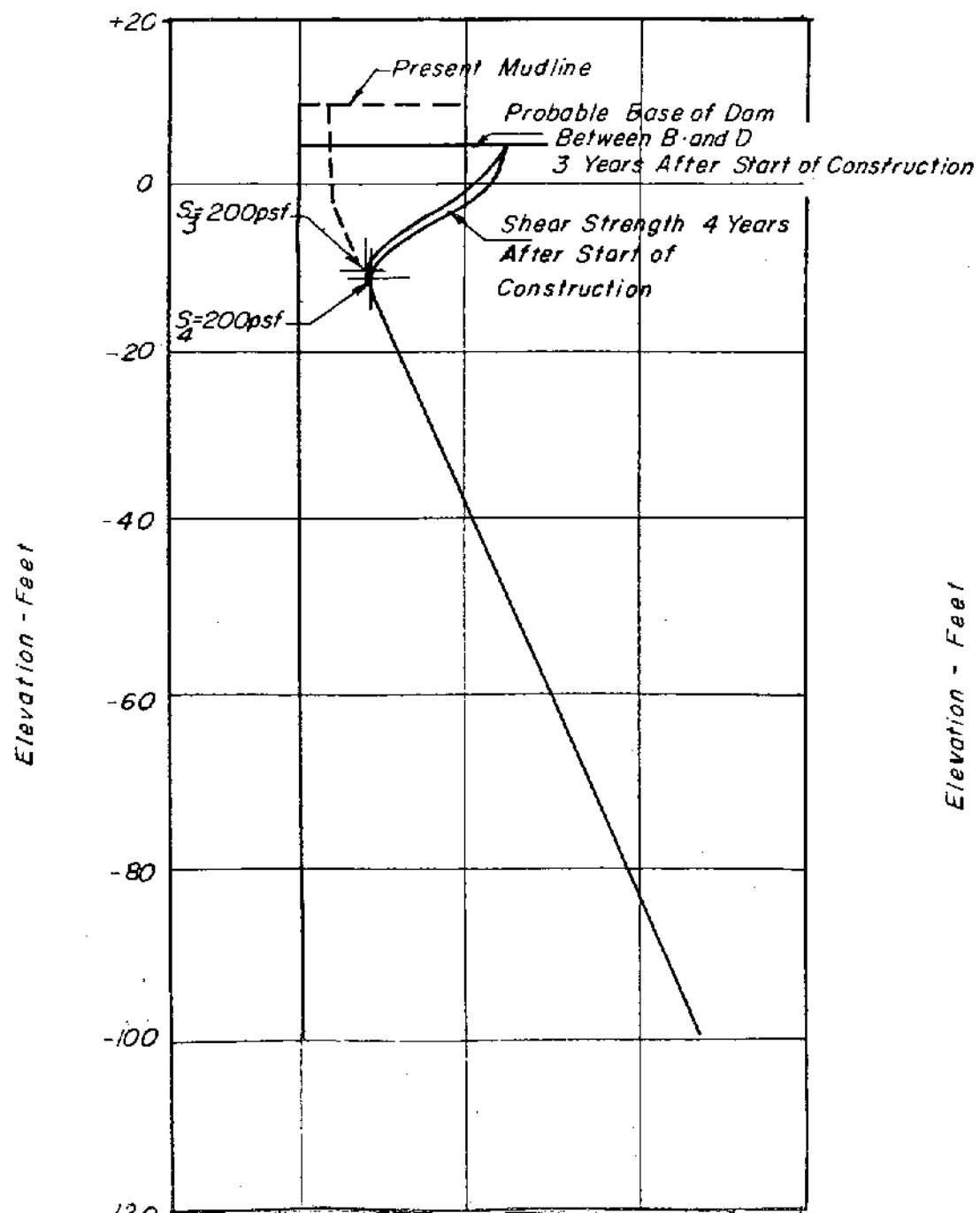
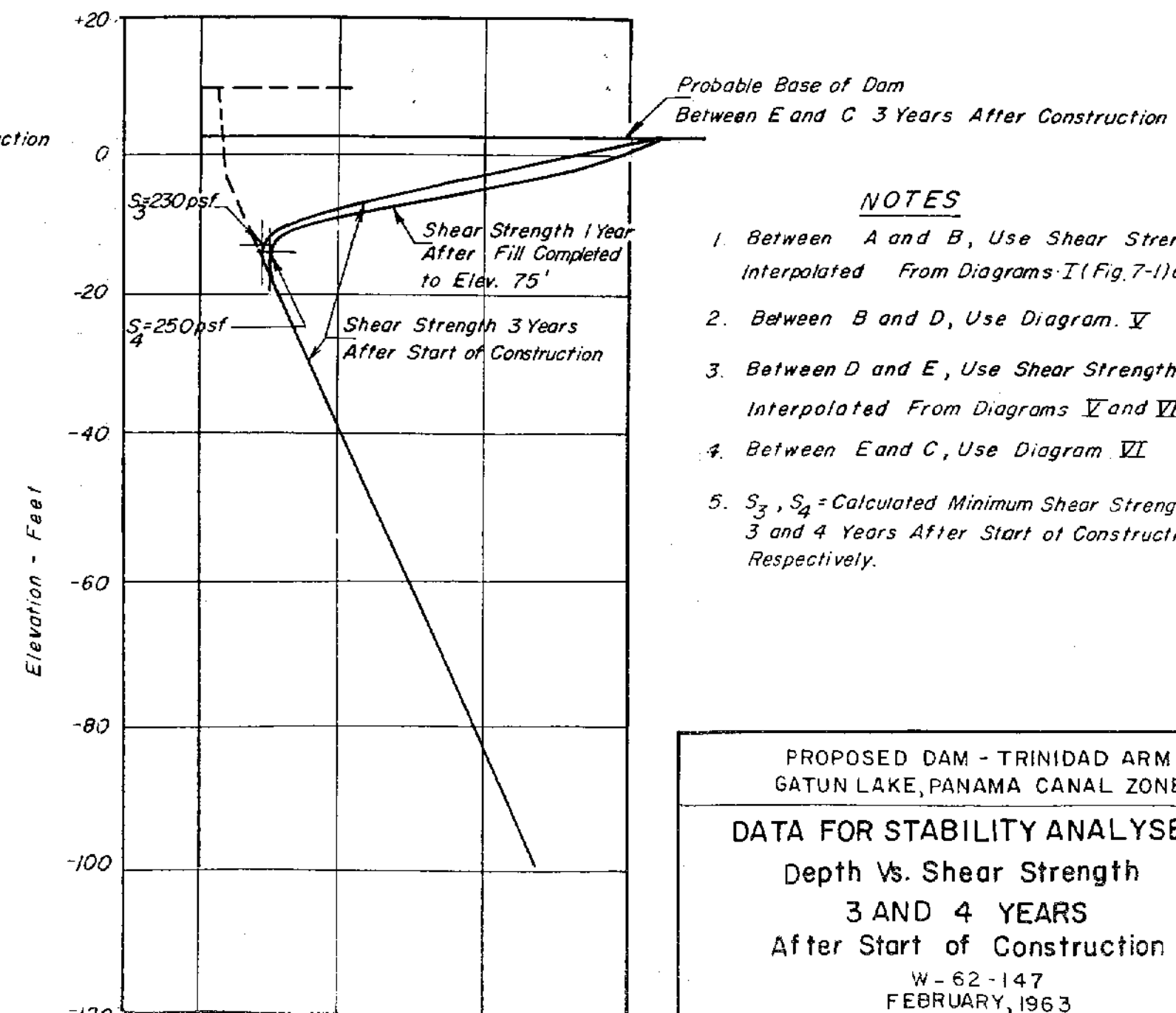


DIAGRAM VI
(Shear Strength Between E and C)



NOTES

1. Between A and B, Use Shear Strength Interpolated From Diagrams I (Fig. 7-1) and V
2. Between B and D, Use Diagram. V
3. Between D and E, Use Shear Strength Interpolated From Diagrams V and VI
4. Between E and C, Use Diagram VI
5. S_3, S_4 = Calculated Minimum Shear Strength 3 and 4 Years After Start of Construction, Respectively.

PROPOSED DAM - TRINIDAD ARM
GATUN LAKE, PANAMA CANAL ZONE

DATA FOR STABILITY ANALYSES
Depth Vs. Shear Strength
3 AND 4 YEARS
After Start of Construction

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The shear strength curves are based upon theoretical concepts incorporating laboratory test results as well as assumptions relating to drainage layer thicknesses and drainage conditions. These curves, therefore, may be at variance with actual field conditions, however any deviation will be, in our opinion, on the side of safety. Field measurements to discern variations from assumed conditions in the strength gain with time study are recommended and are discussed in Section VIII.

Values of shear strength determined in this study do not reflect any consideration of loss in shear strength resulting from material disturbance. The curves are not applicable, therefore, if disturbance or remolding of the foundation materials occurs from barge dumping operations or stability failures. It is considered of utmost importance to control both materials and procedures used in the initial blanketing phases of embankment construction to reduce, as much as practical, loss in shear strength resulting from foundation disturbance.

SECTION VIII

RECOMMENDATIONS

8-1. GENERAL.

The factor of safety, as determined from stability analyses, is dependent upon the properties of both the fill and foundation materials. Preliminary stability analyses have utilized assumed values of saturated unit weight and angle of shearing resistance, Ø, of the fill materials. To assure satisfactory values of factor of safety of the proposed dam section, and in recognition of the significant influence of the fill properties on the factor of safety, it is recommended that the properties of the fill be investigated more completely by field test programs. It would appear from the proposed construction schedule that field tests conducted during the first two years of construction would be practicable, useful and economical. In addition, since stability analyses have utilized values of foundation shear strength estimated from both office and laboratory studies, it is advisable to install field measuring devices and gages to confirm some of the assumptions and evaluations employed in these studies.

8-2. INSTALLATION OF MEASURING DEVICES.

Field measuring devices should be installed to measure pore water pressures in the foundation materials and both vertical and horizontal displacements of the dam section occurring during and after construction.

8-3. PORE WATER PRESSURE MEASUREMENTS.

It is recommended that pore water pressure measuring devices be installed in the foundation to confirm rate of consolidation assumptions used in both the shear strength gain with time study and the settlement study. Higher rates of consolidation occurring in the field will result in a higher gain of foundation

shear strength with time than presently assumed in our stability studies. A higher gain in shear strength with time would increase the factor of safety determined from stability studies of the dam section. If significant savings in embankment quantities would result, some design modification in the dam section could be made during construction. Lower than assumed rates of consolidation occurring in the field would lower the factor of safety of the final design section, since foundation strength build-up would occur at a slower rate than assumed in the stability studies. If this occurs, a slower rate of embankment construction, for instance, could be adopted in potentially unstable areas.

Pore pressure devices should be embedded within the top 20 feet of the foundation at select locations on the Axis and near the toe portion of the Dam. It is estimated that seven devices installed along station 25+00 and seven devices along station 40+00 would provide adequate pore water pressure data. Three of each group of seven devices should be placed on the Axis and the remaining should be placed near both the upstream and downstream toes. The available types of gages, which would properly function under existing and proposed site conditions, as well as an estimate of equipment and installation costs are presently under study.

8-4. HORIZONTAL AND VERTICAL DISPLACEMENT MEASUREMENTS.

It is recommended that gages be installed to measure vertical and horizontal displacements occurring in the dam section during and after construction.

8-5. Horizontal Displacements. Gages installed to reflect horizontal movements would be useful indicators of areas of potential instability. Each gage could consist of a 4 foot x 4 foot concrete plate or equivalent, a cable, with one end attached to the plate, strung over a pulley supported by a buoy, and a counter-weight attached to the free end of the cable. The plate should be placed on the foundation material just outside and adjacent to the toe, either before or immediately after construction of the first layer. This arrangement would permit dumping of fill material without buoy interference. The initial location of the buoy would be accurately determined by appropriate survey

methods. During subsequent construction, buoy locations would be determined at periodic intervals and compared with initial locations. Unusually large horizontal movements would reflect the development of large shearing strains in the foundation, identifying possible imminent embankment failure. This area could then be more closely inspected by underwater reconnaissance. Detection of potentially unstable areas might permit adoption of corrective measures which would prevent foundation failure.

It is estimated that a total of 22 horizontal displacement gages placed at the upstream and downstream toe sections of the Dam would permit measurement of horizontal movement, particularly within the central portion of the dam. These gages, as presently envisioned, would be placed between stations 15+00 and 40+00 at 250 foot intervals.

8-6. Vertical Displacements. To measure settlements occurring at the Axis of the proposed Dam, it is proposed that settlement monuments or plates be installed in the initial layers of select blanket material. These settlement monuments could consist of 5 foot square steel plates embedded in the fill with 3-inch pipe or similar welded to the plate. Pipe sections of pre-measured length could be added by divers to extend the pipe up through the fill as the fill is placed. Settlement measurements at these monuments could ultimately be made with an engineers level. Settlement data obtained in this manner would assist in estimating additional quantities of rolled fill material required in future years to maintain crest elevation and the estimated time intervals for such maintenance.

About ten gages placed at 500 foot intervals on the Axis would permit measurement of dam settlements occurring along the Axis. In addition to these, some of the horizontal displacement gages located near the upstream and downstream toes might be arranged to permit measurement of settlements occurring near the toe of the Dam.

8-7. STUDY OF IN-PLACE FILL CHARACTERISTICS.

It is recommended that the physical properties of the in-place embankment fill materials be studied during the first two years of construction. This study would enable a comparison to be made between the in-place values and the presently assumed values of saturated unit weight and angle of shearing resistance, Ø. The stability of the proposed dam section could then be reviewed and the design modified, if necessary, in the early stages of construction without placing any foreseeable burden on the contractor.

The characteristics of the embankment material, such as saturated unit weight and angle of shearing resistance, Ø, will be dependent upon the in-place structure of the embankment resulting from barge dumping fill material in water. The embankment structure formed by this process may be both loose and layered.

The structure and saturated unit weight of the embankment material could be determined as follows. A large steel box (10' x 10' x 10') would be placed in the initial layers of select fill. After dumping sufficient material to fill the box, the box would be raised to the lake surface. Volume and weight measurements would then be made to determine the saturated unit weight of the fill material. Possible layering of the fill sample would be determined by visual examination. In addition, characteristics of the fill sample, such as grading, angularity, and hardness would also be determined. The knowledge of these characteristics would be helpful in evaluating the angle of shearing resistance, Ø, of the fill material. The above procedure could also be repeated in the initial layers of "random" fill material.

8-8. PROJECT PERSONNEL.

This report is the result of the combined efforts of the following personnel of Shannon and Wilson, Inc., under the direct supervision of T. M. Leps and L. K. Bestwick. The report was written by L. R. Squier, T. L. Olmsted, and T. Akagi. Illustrations were prepared for publication by J. J. Zimcik. The Seattle Laboratory testing, supervised by G. Yamane and T. E. Kirkland, was performed by H. W. McCutcheon, H. G. Aggeler Jr., and R. C. Beltran. All field operations were supervised by D. H. Henkel. Field exploration inspection was performed by T. Akagi and T. L. Olmsted. The Field Laboratory was operated by P. Thordarson.

Respectfully submitted,
SHANNON AND WILSON, INC.

THOMAS M. LEPS
STATE OF WASHINGTON
REGISTERED
PROFESSIONAL ENGINEER

by T. M. Leps
T. M. Leps

LEROY K. BESTWICK
STATE OF WASHINGTON
REGISTERED
PROFESSIONAL ENGINEER

by L. K. Bestwick
L. K. Bestwick

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APPENDIX A

LABORATORY TESTING - PROCEDURES AND TEST RESULTS

APPENDIX A

LABORATORY TESTING - PROCEDURES AND TEST RESULTS

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APPENDIX A

LABORATORY TESTING - PROCEDURES AND TEST RESULTS

A-1. GENERAL.

Laboratory tests on foundation clay and peat materials were performed both in Seattle, Washington and in a field laboratory established in Gatun, Panama Canal Zone.

The field laboratory was established in order to perform more economically the bulk of routine classification and unconfined compression tests. This laboratory was staffed with qualified Shannon and Wilson personnel assisted by personnel provided by the Panama Canal Company.

Undisturbed samples were shipped to Seattle for triaxial and consolidation tests and other specialized tests not within the capabilities of the field laboratory because of equipment limitations. Routine classification tests and unconfined compression tests were also performed in Seattle to supplement field laboratory data.

A-2. SEATTLE LABORATORY.

In the Seattle laboratory, selected undisturbed samples were visually classified and portions were tested for water content, Atterberg Limits, and unit weight. Specific gravity tests and hydrometer analyses were performed on typical samples of clay material. Undisturbed and remolded strengths of clay and peat materials were obtained from triaxial tests, unconfined compression tests and laboratory vane shear tests. Consolidation characteristics of the clay and peat materials were investigated using the standard consolidation test procedure. Triaxial dissipation tests were performed to confirm coefficient of consolidation (C_v) values obtained from consolidation tests.

A-3. CLASSIFICATION TESTS.

The following paragraphs discuss the techniques and procedures used in performing the various classification tests.

A-4. Natural Water Content. The natural water content is defined as the weight of water in a soil expressed as a percentage of the dry soil weight. Natural water contents of the foundation materials were determined at selected intervals for all classified undisturbed samples. In addition, water contents of the various strength and consolidation test specimens were also determined. The oven temperature for drying of clay samples was maintained close to 105°C, but for drying of peat samples the temperature was reduced to about 80°C to avoid oxidation of organic matter. The natural water content determinations are summarized in Table A-1.

A-5. Atterberg Limit Tests. Standard Atterberg Limit Tests were performed on representative samples of clay and clayey peat materials in accordance with ASTM Designation D 423-54T and ASTM Designation D 424-54T except as noted below. In general, one Atterberg Limit Test was conducted on representative material from each classified Shelby tube sample. The liquid limit was determined using the "one-point method" in accordance with the following formula developed by the Corps of Engineers:[1]

$$W = W_n \left(\frac{N}{25}\right)^{0.121}$$

Where W_n = the water content of the soil which closes in N blows in the standard liquid limit device.

In all cases two separate water content determinations were made at identical blow counts. The plastic limit was also determined as the average water content of two separate test threads. Sample preparation for the tests was limited

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1 "Simplification of Liquid Limit Tests," WES Technical Memorandum No. 3-286, Report No. 1, Waterways Experiment Station, Vicksburg, Miss., June, 1949.

TABLE A-1 SUMMARY OF SEATTLE LABORATORY TEST DATA

Boring No.	Sample No.	Depth, feet	Wet Density pcf	Natural Water Content, percent	Atterberg Limits LL	PL	PI	Other Tests	Sample Classification
A-2S-AXU				358					Very soft, gray-brown, leafy and fibrous PEAT with wood fragments, hairlike roots and very sparse clots of gray clayey silt (1/8 in. diameter).
				405				*V=212 V_R=27 S_t=7.92	
	1	77.3-79.8	60.5	574				$\bar{Q}_s$=119	
				349	337	165	172	C	
				790					
			64.5					$\bar{R}_s$ = 215	Brown fibrous to finely divided PEAT with some leaves and wood fragments.
			66.0	264				$\bar{Q}_s$ = 93	
	2	80.6-83.1		250.3				V=242 V_R=54 S_t = 4.47	
				215	219	84	135	C S.G. = 1.57	
								S.G. = 2.64	Soft, gray-brown organic CLAY with scattered rusty yellow-brown spots and sparse light gray concretions 1/16 in.-1/4 in. diameter.
	3	84.8-87.3		72.2				V=128 V_R=50 S_t = 2.58	
			94.2	73.2				$\bar{Q}_s$ = 113	
				91.0	89	44	45	C	Soft, gray-brown, organic CLAY, mottled with yellow-brown streaks. (Gray fine sand layers to 1/2 in. at top of sample).
	4	87.3-89.8		83.4				V=158 V_R=30 S_t = 5.27	
			93.3	82.1				$\bar{Q}_s$ = 127	
				114.7					
			80.2	124.8				U_s = 50	Very soft, dark gray, organic CLAY with organic pieces scattered throughout.
	5	89.9-92.4		128.0				V=102 V_R=44 S_t = 2.3	
				114.9	123	53	70		
				85.6					
			83.2	141				$\bar{Q}_s$ = 228	Soft, gray to gray-brown, organic CLAY, mottled with yellow and black carbonized organic materials.
			85.3	143				$\bar{R}_s$ = 423	
				136	142	64	78	C	
				146				V=152 V_R=28 S_t=5.74	Soft, dark brown, fibrous PEAT with wood fragments.
	7	94.9-97.4	68.6	306				V = 352	
				183					Soft, light tan to gray-brown, mottled organic CLAY with abundant organic material. Layered with thin partings of fibrous, leafy to woody peat. Organic clay is bleached in 1/8 in.-3/4 in. zones above peat partings.
			74.8	167				U_s = 132	
	9	98.8-102.3		151	174	78	96	C	
				177				V_1=358 V_{R1}=42 S_{t2}=8.53	
			102	184				$\bar{R}_s$ = 398	
				192					
				193				V_2=334 V_{R2}=66 S_{t2}=5.1	
				202					Interbedded soft organic CLAY and firm fibrous PEAT.
	10	103.9-105.4	64.7	271				$\bar{R}_s$ = 1230	
								*See Sheet 9 for Legend.	

TABLE A-1 SUMMARY OF SEATTLE LABORATORY TEST DATA

Boring No.	Sample No.	Depth, feet	Wet Density pcf	Natural Water Content, percent	Atterberg Limits LL	PL	PI	Other Tests	Sample Classification
A-2S-AXU			70.0	202				$\bar{Q}_s$=594	Dark brown clayey PEAT. Soft, light gray, organic CLAY with trace of organic material.
	11	105.4-107.9	65.3	288				U_s =66	Dark brown PEAT with trace of thin (1/8 in.) clay layers. Peat is horizontally stratified with leafy material.
				214	273	114	159	C	
				260					
				282				V=522 V_R=152 S_t = 3.45	
				117.2				V_1=312 V_{R1}=92 S_t=3.4	Soft, brown, organic CLAY with organic matter scattered throughout. Some pieces of wood evident.
				130.4					
			85.6	117.0					
				99.9	119	51	68	C	
	13	110.4-112.9		101.3				V_2=356 V_{R2}=84 S_t=4.2	
			82.5	121.5	145	60	85	U_s = 326	
				126.2					
			80.5	142				$\bar{Q}_s$ = 383	
				141					Fibrous PEAT.
				126				V_1 =488 V_{R1}=102 S_{t1}=4.77	
				126					
			79.8	151				$\bar{Q}_s$=563	
	15	115.6-118.1		128				V_2=458 V_{R2}=110 S_{t2}=4.26	Soft, brown, organic silty CLAY with partings (1/32 in.) of dark organic material.
				140	155	61	94	C	
			79.2	124				U_s=148	
				132					
			80.9	131				$\bar{Q}_s$ = 522	Fibrous PEAT
				99.9				V=430 V_R=86 S_t=4.94	Soft dark brown-gray, mottled, organic silty CLAY with sparse organic material.
	17	120.6-123.1		99.3				C S.G.=2.56	
				99.0	123	57	66		
			81.6	98.4				U_s=519	Soft, brown, organic CLAY with streaks of tan organic clay. Layers compressed organic material and wood.
				135					
				107.0				V_1=312 V_{R1}=90 S_{t1}=3.5	Soft to medium, dark gray organic silty CLAY with yellow pockets of silty clay.
								$\bar{Q}_1$=Void	
			84.6	123.4					
			83.8	125.8					
	18	123.1-125.6	90.9	106.1				$\bar{Q}_{s2}$(Med.)=584	
			89.3	107.5				$\bar{Q}_{s3}$(Slow)=572 Q_{4s}(Fast)=605	
				102.5				V_2=435 V_{R2}=100 S_{t2}=4.26	
			91.0	103.1					
			89.1	101.8				U_{1s}=670 U_{2s}=550	
				112.1				C	
			85.2	192				U_s=337	Mottled tan-gray organic silty CLAY with irregular voids to 1/8 in. diameter.
				106				V=568 V_R=98 S_t=5.8	
	19	126.5-129.1		110	162	69	93	C	
				154					
			85.2	109.8				$\bar{Q}_s$=765 (partially consolidated)	
				107					
				106					Mottled olive tan to olive, organic silty CLAY, with very sparse organic material (stippled pattern).
				101					
			85.8	57				V_1=366 V_{R1}=75 S_t=4.59	
								U_s=294 (Low-Fracture)	
				90	103	43	60	C	
	21	131.8-134.3		94.7				V_2=408 V_{R2}=78 S_t=5.24	
			87.4	99.1				$\bar{Q}_s$=409	
				99					

TABLE A-I SUMMARY OF SEATTLE LABORATORY TEST DATA

Boring No	Sample No.	Depth, feet	Wet Density pcf	Natural Water Content, percent	Atterberg Limits LL	PL	PI	Other Tests	Sample Classification*
R-25-AXU	23	132.0-139.5	95.6	76.7 74.9 67.6 73.0 69.6 70.8	85	40	45	V_1=482 V_{R1}=62 S_{t1}=7.77 U_s=403 V_2=468 V_{R2}=50 S_{t2}=9.34 C $\bar{Q}_s$=666	Mottled olive-tan to olive organic silty CLAY with occasional light yellow concretions, thin (less than 1/32 in.) silt partings with leaf impressions. Some organic fragments and rust color spots.
	25	142.5-145.0	92.7 91.0	62.1 81.0 80.7 79.0 82.2 83.5 80.4 83.2 79.0	99	44	55	C V=564 V_R=116 S_t=4.91 $\bar{Q}_s$=948 U_s=594	Medium, gray to light gray, organic CLAY with yellow very dense silt layers and layers and partings of fine sand. Some peat and other organic materials. Some brown organic CLAY
	27	147.5-149.5	90.0 92.1	89.5 85.0 82.4 85.8 84.7				$\bar{Q}_s$=385 V=386 V_R=86 S_t=5.86 U_s=544	Gray brown silty organic CLAY containing scattered yellow siltstone nodules (max. 3/4 in.) and scattered organic fragments. Some yellow streaks and pockets similar to nodules but not hard.
	33	157.6-160.1	93.1 93.7	81.0 82.0 80.9 79.4	116	45	70	U_s=425 $\bar{Q}_s$=848 V=822 V_R=126 S_t=6.54 C	Gray-brown silty organic CLAY with stratified 1/8-1/4 in. thick yellow, soft to locally hard "concretionary" streaks and layers. Thin gray to yellow-gray silt streaks, pockets, and partings.
	34	157.1-159.6	94.9 96.3	83.4 90.1 70.7 70.9 70.9	88	52	36	V=922 V_R=182 S_t=5.05 U_s=664 $\bar{Q}_s$=840 C	Gray-brown silty organic CLAY with very fine sand partings and thin stiff peat laminations. Gray-brown silty organic CLAY with occasional silt to very fine sand streaks and partings, and occasional yellow streaks of soft silty clay. Sparse organic streaks.
	35	170.5-173.1	95.0 95.5	81.8 81.9 77.1 82.6 81.6 79.8 79.3 79.1 134.6	111	51	60	U_{s1}=785 U_{2s}=681 C V=652 V_R=196 S_t=3.51 $\bar{Q}_{s1}$=1155 $\bar{Q}_{s2}$=1035	Medium, dark brown, organic CLAY with organic matter scattered throughout; slight odor, occasional thin (1/2 to 1/4 in.) layer of yellow silt and siltstone and trace of very thin seams or pockets of fine sand.
	51	177.0-179.5	94.2 93.5	83.1 76.1 72.9 79.4	112	47	65	C $\bar{Q}_s$=782 V=870 V_R=152 S_t=5.73 U_s=776	Gray-brown, slightly organic silty CLAY with thin very fine sand-silt partings and occasional yellow alterations. Occasional thin (1/32 in.) peat parting. Yellow portion is very sensitive clay.
	53	183.0-185.5	93.7	75.3 76.6 72.6				U_s=537	Brown-gray silty CLAY with very fine sandy silt. Some organic matter and siltstone nodules.

TABLE A-I SUMMARY OF SEATTLE LABORATORY TEST DATA

Boring No	Sample No.	Depth, feet	Wet Density pcf	Natural Water Content, percent	Atterberg Limits LL	PL	PI	Other Tests	Sample Classification
A-25-AXU	55	188.0-191.5	93.4 95.7	73.2 70.2 75.6 75.4 74.2				U=577 V=862 V_R=178 S_t=4.84 $\bar{Q}_s$=1240	Gray-brown silty CLAY with very fine sandy partings and pockets. Occasional pockets and thin partings of organic material. Yellow alterations, generally soft or slightly harder than sample, occasionally concretionary and hard.
	59	201.0-203.5		60.1 59.2 99.0 56.2					Gray silty fine SAND with thin organic clay partings. Some thin peat layers (1/8 to 1/4 in.). Trace of shells. Occasional yellow silty clay layers.
	60	204.0-206.5	95.5	63.8 63.9 68.8				$\bar{Q}_s$=2050	Medium, layered brown-gray fine SAND and gray organic silty CLAY to organic clay.
	63	213.0-215.5	82.8	111.2 112.0 114.7	178	108	70	C $\bar{Q}_s$=2180	Stiff, dark brown, irregularly layered, woody PEAT with gray silty clay.
	67	223.0-225.5		49.6 62.2					Gray-brown silty fine to medium SAND.
	68	227.2-229.7							Medium gray and brown, organic clayey silty medium to fine SAND.
	69	230.2-232.7	92.5	41.3 60.5 38.1				$\bar{Q}_s$=779	Dense gray-brown silty fine to medium SAND with some organic material.
UP-25-CU	S-6	106.0-108.5	65.7	219 178				$\bar{Q}_s$ = 1490	Very soft gray organic CLAY with organic inclusions. Firm black fibrous PEAT. Same as top.

TABLE A-I SUMMARY OF SEATTLE LABORATORY TEST DATA

Boring No.	Sample No.	Depth, feet	Wet Density pcf	Natural Water Content, percent	Atterberg Limits LL	PL	PI	Other Tests	Sample Classification
DF-25-CU	S-13	106.0-108.5 (5" Shelby)		140 321 194					Very soft gray organic CLAY with organic inclusions. Firm black fibrous PEAT. Same as top.
DN-25-CU	S-2	95.0-97.5 (5" Shelby)	78.4	161 152 184 219 210				$\bar{Q}_{s_1}$=206 S_R = 2.60 $\bar{Q}_{s_2}$=202 S_R = 1.30 $\bar{Q}_{s_3}$=119 S_R = 0.47 $\bar{Q}_{s_4}$=107 S_R = 0.03	Very soft black organic peaty CLAY.
	S-3	79.0-81.5		574 650 580				Photo - See fig. 3-4	Very soft brown loose decayed fibrous PEAT.
	S-5	85.0-87.5		94.5 379 244				Photo - See fig. 3-4	Very soft blue-gray CLAY. Very soft gray peaty organic CLAY. Very soft dark gray organic CLAY interbedded with PEAT.
	S-7	91.0-93.5		137 155 143				Photo - See fig. 3-4	Very soft brown and brown-gray organic CLAY with peat inclusions.
	S-9	97.0-99.5		173 184 185				Photo - See fig. 3-4	Very soft gray organic CLAY interbedded with brown fibrous PEAT.
	S-11	103.0-105.5		306 374 410				Photo - See fig. 3-5	Very soft dark gray organic CLAY interbedded with dark brown peat and wood.
	S-13	109.0-111.5		236 237 309				Photo - See fig. 3-5	Very soft dark gray organic CLAY laminated with brown compact PEAT.

TABLE A-I SUMMARY OF SEATTLE LABORATORY TEST DATA

Boring No.	Sample No.	Depth, feet	Wet Density pcf	Natural Water Content, percent	Atterberg Limits LL	PL	PI	Other Tests	Sample Classification
DN-25-CU	S-15	118.0-120.5		108 110 112				Photo - See fig. 3-5	Very soft gray-brown organic CLAY with occasional silt pockets.
	S-17	124.0-126.5		102 89.0 101				Photo - See fig. 3-5	Very soft brown and dark gray organic CLAY with scattered siltstone nodules.
A-40-AXU	S-II	87.5-90.0 (5" Shelby)		66.7 68.9 66.8 68.5 80.7 84.1					Very soft very decayed PEAT and light blue-gray CLAY. Very soft light blue-gray slightly organic CLAY.
	S-3	83.0-85.5	95.8 95.7	66.3 63.6 65.0 70.5 63.1				$\bar{R}_{s_1}$ = 2380 (σ_c = 5480) $\bar{R}_{s_2}$ = 745 (σ_c = 1550)	Very soft gray-brown organic silty CLAY stratified with thin layers of organic material.
	S-7	93.0-95.5	91.8 97.5	97.0 89.2 88.0 88.3 83.5				$\bar{R}_{s_1}$ = 1085 (σ_c 2860) $\bar{R}_{s_2}$ = 343 (σ_c = 430)	Very soft blue gray organic CLAY with organic pockets and occasional layers of clayey peat to 1 inch thick.
	S-9	98.0-100.5	73.9	215 148 130 149 131				$\bar{R}_s$ = 505	Soft black PEAT interbedded with soft organic CLAY. Soft dark to light gray organic CLAY with layers of black organic material.
	S-11	103.0-105.5	68.3	231 263 384				$\bar{R}_s$ = 690	Black slightly clayey fibrous PEAT.
	S-13	106.0-110.5		346 337 424					Compact black and brown PEAT with large wood fragment 0.5 feet thick.

TABLE A-I SUMMARY OF SEATTLE LABORATORY TEST DATA

Boring No.	Sample No.	Depth, feet	Wet Density pcf	Natural Water Content, percent	Atterberg Limits LL	PL	PI	Other Tests	Sample Classification
UP-40-CU	S-1	78.5-80.8		186 277 344				$V_1 = 77$ $V_{R_1} = 15$ $S_{t_1} = 4.8$ $V_2 = 62$ $V_{R_2} = 16$ $S_{t_2} = 3.9$	Very soft decayed PEAT with pockets of very soft gray organic clay.
	S-1	85.0-87.5 (S = Shelby)	77.2 78.5 75.4 75.4	317 264 198 204 212 208 235				Q_{s_1} = Void $Q_{s_2} = 195$ $S_R = 0.62$ $Q_{s_3} = 200$ $S_R = 2.69$ $Q_{s_4} = 182$ $S_R = 0.29$ $Q_{s_5} = 173$ $S_R = 0.12$	Very soft black to dark brown PEAT. Very soft brown-gray organic CLAY with pockets of clayey peat.
	S-3	83.5-85.8		225 178 170 395				$V = 159$ $V_R = 27$ $S_t = 5.9$	Very soft black-brown decayed PEAT with layers of very soft gray organic peaty clay. Very soft gray organic CLAY with layers of black peat.
	S-5	88.5-90.8		130 112 108 113				$V = 152$ $V_R = 51$ $S_t = 3.0$	Soft gray-mottled-yellow organic CLAY with pockets and layers of black peat.
	S-7	93.5-95.8		169 182 406 318				$V = 367$ $V_R = 52$ $S_t = 7.0$	Soft gray organic CLAY with pockets of black peat. Soft black PEAT with some peaty CLAY. Soft black PEAT.
	S-9	98.5-100.8		219 154 430 588 125				$V_1 = 242$ $V_{R_1} = 29$ $S_{t_1} = 8.3$ $V_2 = 294$ $V_{R_2} = 44$ $S_{t_2} = 6.7$	Very soft brown organic CLAY with pockets of organic material Brown decayed fibrous PEAT. Very soft brown organic CLAY with pockets of organic material.
	S-11	103.5-105.8		304 384 368 384 455				$V_2 = 488$ $V_{R_2} = 59$ $S_{t_2} = 8.3$ $V_1 = 440$ $V_{R_1} = 41$ $S_{t_1} = 10.7$	Firm brown fibrous PEAT with occasional layers of organic clay.
	S-13	108.5-110.8		581 650 522 558 545				$V_1 = 566$ $V_{R_1} = 64$ $S_{t_1} = 8.9$ $V_2 = 628$ $V_{R_2} = 68$ $S_{t_2} = 9.2$	Firm black decayed fibrous PEAT.

TABLE A-I SUMMARY OF SEATTLE LABORATORY TEST DATA

Boring No.	Sample No.	Depth, feet	Wet Density pcf	Natural Water Content, percent	Atterberg Limits LL	PL	PI	Other Tests	Sample Classification
UP-40-CU	S-14	112.5-114.8		361 590 503 486				$V = 360$ $V_R = 134$ $S_t = 2.7$	Firm dark brown decayed fibrous PEAT.
	S-15	117.5-119.8		150 141 161				$V_2 = 314$ $V_{R_2} = 62$ $S_t = 5.1$ $V_1 = 424$ $V_{R_1} = 54$ $S_t = 7.9$	Soft gray organic CLAY with pockets of peat.
	S-16	122.5-124.8		145 144 199 190 181				$V_1 = 286$ $V_{R_1} = 46$ $S_{t_1} = 6.2$ $V_2 = 388$ $V_{R_2} = 50$ $S_{t_2} = 7.8$	Very soft gray-brown to dark gray organic CLAY with pockets of peat.
	S-17	127.5-129.8		142 140 142 152 144 145				$V_2 = 300$ $V_{R_2} = 56$ $S_{t_2} = 5.4$ $V_1 = 388$ $V_{R_1} = 58$ $S_{t_1} = 6.7$	Very soft brown and gray organic peaty CLAY with roots, leaves, twigs, and peat pockets.
DN-40-CU	S-9	122.0-124.3		64.4 53.5 53.0					Medium light brown SAND Soft dark gray organic CLAY with leaves and stems. Soft blue-gray CLAY with siltstone nodules and occasional organics.
UP-15-CU	S-1	90.0-92.5	84.9 91.5	106 104 95.0 96.0 62.0 95.5 81.0	125	37	88	C Triaxial Dissipation Test (See Table A-4) Special Atterberg Limit Tests (See Table A-4) Triaxial Dissipation Test (See Table A-4)	Very soft purple-gray organic CLAY.
	S-11	101.0-103.3		230					Interbedded PEAT and peaty CLAY.
UP-50-CU	S-5	87.0-89.3		Sand 57.1 Clay 70.3					Interbedded brown and gray silty fine SAND and gray organic CLAY with occasional black organic inclusions.

TABLE A-I SUMMARY OF SEATTLE LABORATORY TEST DATA

Boring No.	Sample No.	Depth, feet	Wet Density pcf	Natural Water Content, percent	Atterberg Limits LL	PL	PI

LEGEND

C	Consolidation Test
LL	Liquid Limit
PL	Plastic Limit
PI	Plasticity Index
$\bar{Q}_s$	Shear Strength, psf, determined from undrained triaxial test. Bar indicates test conducted with pore pressure measurements.
$\bar{R}_s$	Shear Strength, psf, determined from consolidated undrained triaxial test. Bar indicates test conducted with pore pressure measurements.
S.G.	Specific Gravity
S_R	Strain Rate, percent strain per minute.
S_t	Sensitivity (V/V_R)
U_s	Shear Strength, psf, determined from unconfined compression test.
V	Undisturbed Laboratory Vane Shear Strength, psf.
V_R	Remolded Laboratory Vane Shear Strength, psf.
σ_c	Consolidation Pressure

NOTE

Refer to Table A-5 for detailed summary of triaxial test results.

to removing visible pieces of leaves, roots and twigs. Oven temperatures were regulated according to the types of materials tested, as noted previously in the natural water content section. The liquid limit and plastic limit values determined in the Atterberg Limit Tests are presented in Table A-1.

A special series of Atterberg Limit Tests were conducted on clay material from samples S-5, drill hole A-40-AXU and S-1, drill hole UP-15-CU to examine in qualitative terms the amount of organic material contained in typical clay samples. Both samples were from about depth 90 feet below mean lake level. In this series, the liquid limit and plastic limit of the clay material in the natural state, after air-drying, and after oven-drying were determined. The presence of organic material in a soil sample can be generally inferred from this type of test series as oven-drying produces irreversible changes in the organic constituents that significantly lower the liquid limit. Sample preparation included removal of visible chunks and pockets of organic matter prior to testing. The results of this test series are shown in Table A-2. The test results indicate that organic matter is present in S-5, but is absent in S-1.

TABLE A-2. SUMMARY OF SPECIAL ATTERBERG LIMIT TESTS

Sample S-5 DH A-40-AXU			
Material Condition	Liquid Limit	Plastic Limit	Plasticity Index
From Natural State	130	52	78
After Air-Drying	126	52	74
After Oven-Drying	104	45	59
Sample S-1 DH UP-15-CU			
Material Condition	Liquid Limit	Plastic Limit	Plasticity Index
From Natural State	117	37	80
After Air-Drying	127	47	80
After Oven-Drying	113	37	76

A-6. Unit Weight Determination. The unit weight of the undisturbed samples of peat and clay was determined by measuring the diameter and height of an extruded sample and weighing the specimen on a gram scale. Unit weights were determined on all unconfined and triaxial test specimens. The unit weight determinations are summarized in Table A-1.

A-7. Specific Gravity Determination. Three specific gravity tests were performed on representative samples of the foundation material in accordance with ASTM designation D 854-52. Entrapped air was removed from the material by subjecting the pycnometer containing the soil to a partial vacuum while the pycnometer was immersed in warm water. The results of this test are presented in Table A-1.

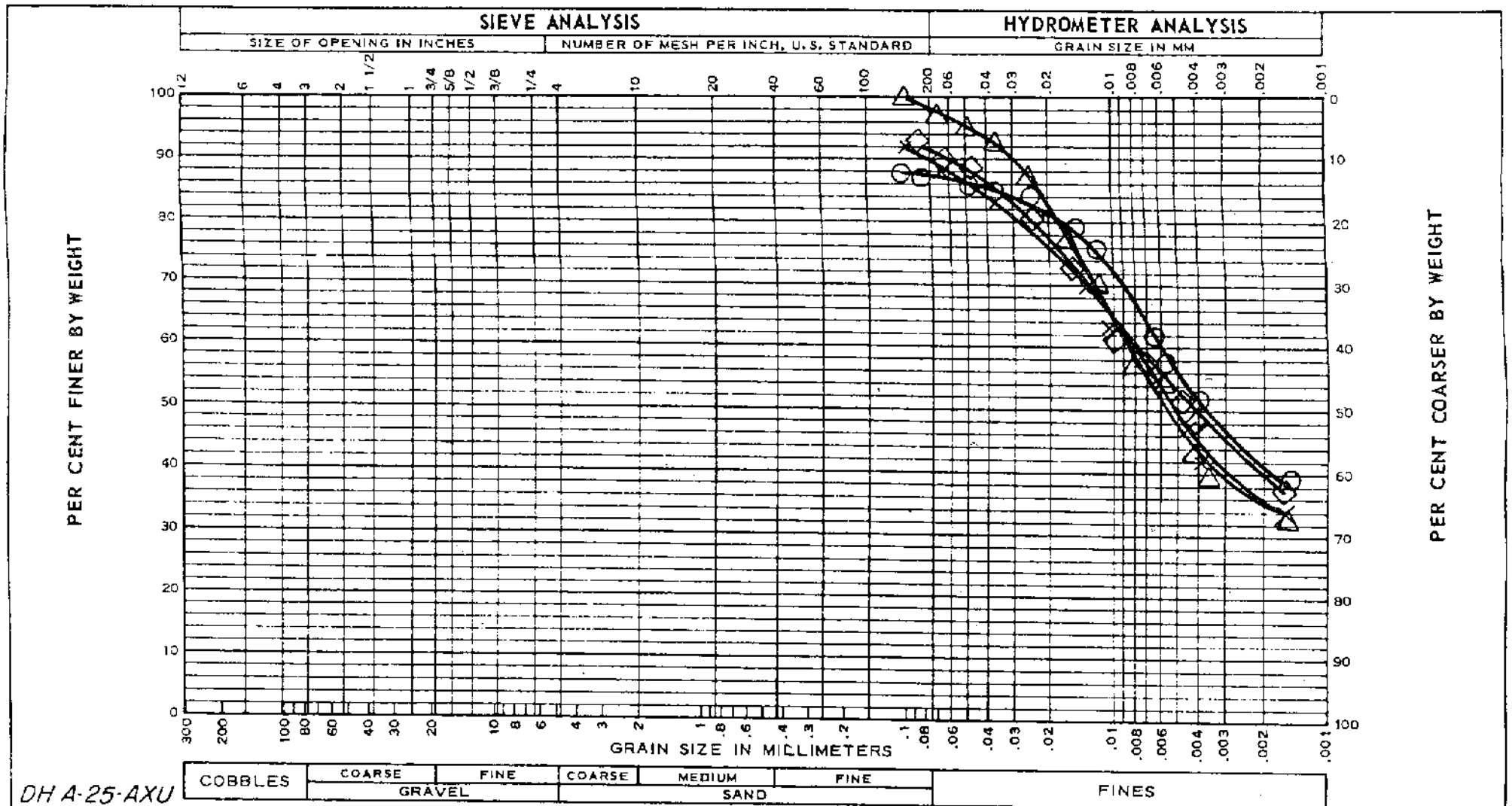
A-8. Hydrometer Analysis. The distribution of particle sizes in typical samples of foundation material were determined in the hydrometer analysis using, in general, the procedure outlined in ASTM designation D 422-54T. The dispersing agent used was sodium silicate. The grain size curves for the selected samples are shown in Figure A-1.

A-9. UNCONFINED COMPRESSION TESTS.

Unconfined compression tests were conducted on undisturbed specimens from drill hole A-25-AXU utilizing a portable, hand-operated, strain-controlled apparatus supplied by Soiltest Incorporated. Specimen height to diameter ratios were generally 2:1 with sample heights predominantly 6 inches or 3.5 inches. Water contents were determined before and after each test. The applied rate of strain for most tests varied from 1 to 2 percent per minute. The strain at failure for most of the specimens ranged from 4 to 7 percent. The results of the unconfined compression tests are summarized on Table A-1.

A-10. LABORATORY VANE SHEAR TESTS.

Laboratory vane shear tests were conducted on samples from drill holes A-25-AXU and UP-40-CU. The latter series of tests were undertaken to further investigate the "low" values of shear strength obtained from field laboratory unconfined compression tests on samples from drill hole UP-40-CC.



DH A-25-AXU

SAMPLE NO.	DEPTH -FT.	CLASSIFICATION	NAT. W. C.	LL	PL	PI	Specific Gravity
△ S-3	84.8-87.3	Gray-brown organic silty CLAY	72%				2.64
○ S-17	120.6-123.1	Dark gray-brown organic silty CLAY	99%	123	57	66	2.56
× S-25	142.5-145.0	Gray organic silty CLAY	80%	99	44	55	2.7-*
◇ S-35	170.6-173.1	Dark brown organic silty CLAY	96%	111	51	60	2.7-*

*Assumed Values

CLASSIFICATION TEST

GRAIN SIZE DISTRIBUTION CURVES

FEBRUARY, 1963

W-62-147

Sherman Wilson

SOIL MECHANICS AND FOUNDATION ENGINEERS

SEATTLE

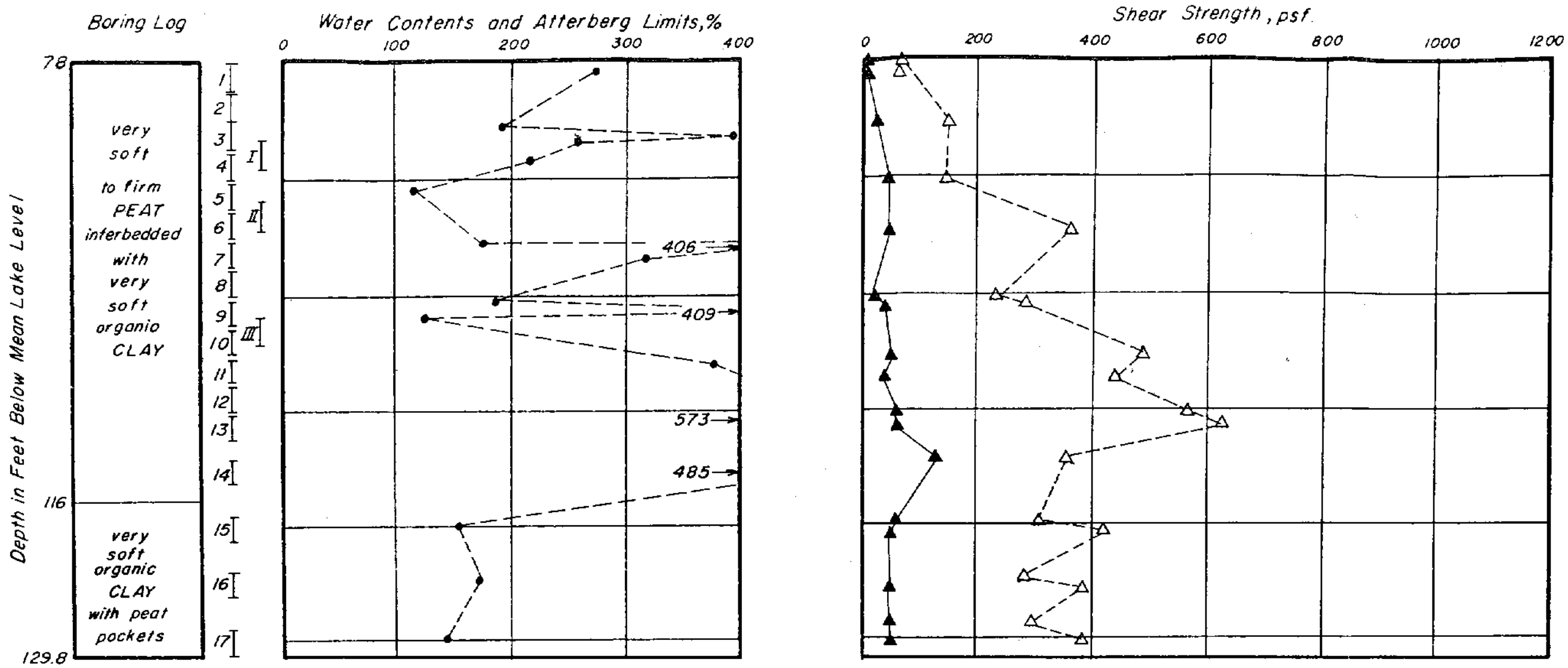
FIG. A-1

The vane shear tests were performed on undisturbed and remolded samples of clay and peat materials. Shear strengths of the fibrous peat materials determined from the tests were generally higher than those determined from the Q triaxial tests. This was due, essentially, to the "interlocking" nature of the peat fibers. Shear strengths determined from the laboratory vane shear tests on clay were generally in agreement with shear strengths determined from the Q triaxial tests, and were higher than the shear strength values determined from the unconfined compression tests. The results of the vane tests on the undisturbed and remolded peat and clay materials are summarized in Table A-1 and shown on Figure A-2.

The laboratory vane used for the tests was a four-bladed model (blade height 1 inch, diameter 0.68 inch, thickness 0.02 inch) which was positioned horizontally and rotated in a ball-bearing guide. The specimen was held in a tight fitting clamp to prevent rotation as torque was applied. A cord attached to a suspended container and counterweight rode over a pulley connected to the vane shaft. Torque was applied by adding water to the container. As failure was approached, the rate of application of torque was reduced and was stopped as soon as slight movement of the vane was observed. Torque was then determined by measuring the volume of water added to the container. Applied torque reached a well defined peak value, particularly in the undisturbed clays. After the undisturbed shear strength torque was determined, the material was thoroughly remolded, and the torque required to again rotate the vane was immediately measured. The undisturbed and remolded values of shear strength were used to obtain the sensitivity of the material; the sensitivity being the ratio of the undisturbed to remolded values of shear strength.

The time required to complete a test on undisturbed material varied according to the relative consistency of the material. The time for completion of a test, therefore, varied from about 2 minutes near the mudline to about 7 to 10 minutes at and below a depth of 50 feet below the mudline. Time to failure for remolded specimens was about 2 minutes.

A program of study has been initiated to investigate the possible gain in shear strength of the foundation clays with time after remolding. Past experience has shown that some normally consolidated clays will "set-up" or gain



LEGEND:

△ Undisturbed Laboratory Vane Shear Strength, psf

▲ Remolded Laboratory Vane Shear Strength, psf

● Natural Water Content

2 3" O.D. Shelby Tube Sample. No. to Left is Sample Number

II 5" O.D. Shelby Tube Sample. No. to Left is Sample Number

PROPOSED DAM – TRINIDAD ARM
GATUN LAKE, PANAMA CANAL ZONE

BORING LOG AND LABORATORY TEST RESULTS, BORING UP-40-CU

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strength with time after having been disturbed or remolded. This phenomenon is known as the thixotropic effect.

To investigate the thixotropic effect in the foundation clays, a series of undisturbed vane shear tests was conducted on clay material contained in tube sample S-II from drill hole A-40-AXU. After the undisturbed shear strengths were determined and the initial conditions established, the material was thoroughly mixed. The remolded shear strength of this material was then determined. Specimens were prepared in aluminum containers and sealed to prevent loss of moisture. At periodic intervals, successive specimens will be opened, one for each interval, and the shear strength of the material will be determined utilizing the vane apparatus. The values of shear strength obtained from this approach will indicate the degree of shear strength gain, if any, with time. The data obtained for the relatively short intervals will be extended to greater intervals of time by extrapolation.

This study will be helpful in evaluating the shear strength of the foundation materials at various times after complete remolding or disturbance caused by, for example, foundation failures or barge-dumping operations.

A-11. TRIAXIAL COMPRESSION TESTS.

A series of undrained (Q) triaxial tests was performed on undisturbed samples of clay and peat to determine the "in-situ" shear strength of the foundation material.

In addition, consolidated-undrained (R) tests were conducted on undisturbed samples of clay and peat to determine the shear strength parameters, cohesion, c, and angle of shearing resistance, Ø, for these materials. Undrained (Q) triaxial tests were also performed on undisturbed samples of clay to investigate the effect of rate of shear on values of shear strength. A detailed summary of triaxial test data relative to test conditions and test results is presented in Table A-3.

A-12. Triaxial Specimens. All triaxial specimens were taken from samples extruded from either 3-inch or 5-inch O.D. Shelby tubes. The specimens were cylindrical with diameters of either 2.90 inches or 1.4 inches. The 1.4-inch

TABLE A-3 SUMMARY OF TRIAXIAL TEST DATA

Sample No.	Depth, Ft.	Classification	Type of Test	H, in.	σ_3, psf	σ_c, psf	B	u_o, psf	u_f, psf	E_f, %	s_R, %/min.	$(\sigma_1 - \sigma_3)_f$, psf	$\left(\frac{\sigma_1 - \sigma_3}{2}\right)_f$, psf
DRILL HOLE A-25-AXU													
S-1	77.3-79.8	PEAT	$\bar{Q}$	6.00	4970	-	0.98	4850	4980	9.7	0.74	245	122
S-2	80.6-83.1	Organic CLAY	$\bar{Q}$	5.88	5250	-	0.98	5160	5250	8.0	0.89	192	96
S-3	84.8-87.3	Organic CLAY	$\bar{Q}$	6.81	5800	-	0.98	5750	5760	10.8	0.79	232	116
S-4	87.3-89.8	Organic CLAY	$\bar{Q}$	6.63	6000	-	0.98	5900	5980	8.8	0.38	261	130
"	"	Organic CLAY	$\bar{R}$	6.16	7100	1050	-	6050	6510	10.3	0.61	2175	1090
S-7	94.9-97.4	Organic CLAY	$\bar{Q}$	*3.50	6480	-	0.96	6200	6390	4.8	0.21	467	233
"	"	Organic CLAY	$\bar{R}$	*3.50	880	880	-	0	252	3.7	0.93	867	433
"	"	Organic CLAY	$\bar{R}$	6.13	8900	2850	-	6050	7900	7.5	0.68	2140	1070
S-9	99.8-102.3	Organic CLAY	$\bar{R}$	5.25	7100	-	-	6200	7000	7.2	0.40	817	408
S-10	102.9-105.4	PEAT	$\bar{R}$	7.00	8560	1920	-	6640	8600	12.9	0.05	2500	1250
S-11	105.4-107.9	PEAT	$\bar{Q}$	6.19	7000	-	0.93	6450	7050	5.0	0.36	1230	615
S-13	110.4-112.9	Organic CLAY	$\bar{Q}$	5.50	9960	-	0.94	9360	10060	4.0	0.19	785	393
S-15	115.6-118.1	Organic CLAY	$\bar{Q}$	6.19	8250	-	0.94	7740	8250	5.8	0.36	1150	575
S-17	120.6-123.1	Organic CLAY	$\bar{Q}$	5.44	8720	-	0.94	8180	8600	2.9	0.59	1070	535
S-18	123.1-125.6	Organic CLAY	$\bar{Q}$	*3.50	8800	-	0.94	8300	8600	4.8	0.30	1170	585
"	"	Organic CLAY	$\bar{Q}$	*3.81	8800	-	0.92	8200	8750	5.8	2.58	1240	620
S-19	125.6-128.1	Organic CLAY	$\bar{Q}$	5.88	9250	-	0.86	7920	8650	5.6	0.63	1570	785
S-21	131.8-134.3	Organic CLAY	$\bar{Q}$	6.31	9650	-	0.97	9370	9550	4.6	0.58	840	420
S-23	137.0-139.5	Organic CLAY	$\bar{Q}$	6.13	10100	-	0.94	9480	9870	5.3	0.59	1365	682
S-25	142.5-145.0	Organic CLAY w/sand and silt partings	$\bar{Q}$	*3.50	10650	-	0.90	9600	10400	2.9	0.95	1942	971
S-27	147.5-149.5	Same as S-25	$\bar{Q}$	6.63	10950	-	0.96	10500	10800	10.7	0.60	790	395
S-31	157.6-160.1	Organic CLAY	$\bar{Q}$	5.63	11900	-	0.92	10900	11600	3.3	0.66	1740	870
S-34	167.1-169.6	Same as S-25	$\bar{Q}$	6.19	12700	-	0.95	12150	12400	5.2	0.65	1720	860
S-35	170.6-173.1	Same as S-25	$\bar{Q}$	*3.25	13100	-	0.89	11700	12650	5.4	1.07	2370	1185
"	"	Same as S-25	$\bar{Q}$	*3.50	15100	-	0.92	14000	14900	6.0	1.01	2130	1065
S-51	177.0-179.5	Same as S-25	$\bar{Q}$	6.50	13500	-	0.94	12750	13150	3.5	0.53	1600	800

TABLE A-3 SUMMARY OF TRIAXIAL TEST DATA

Sample No.	Depth, Ft.	Classification	Type of Test	H, in.	σ_3, psf	σ_c, psf	B	u_o, psf	u_f, psf	E_f, %	s_R, %/min.	$(\sigma_1 - \sigma_3)_f$, psf	$\left(\frac{\sigma_1 - \sigma_3}{2}\right)_f$, psf
DRILL HOLE A-25-AXU Continued													
S-55	189.0-191.5	Silty slightly org. CLAY	$\bar{Q}$	6.47	14400	-	0.91	13200	13900	2.5	0.59	2540	1270
S-60	204.0-205.5	Silty CLAY and fine SAND	$\bar{Q}$	7.69	16120	-	0.91	13300	15200	3.6	0.51	4200	2100
S-63	213.0-215.5	Silty CLAY & PEAT	$\bar{Q}$	4.75	16600	-	0.87	14500	17650	3.8	0.75	4460	2230
S-65	227.2-229.7	Org. clayey silty SAND	$\bar{Q}$	7.81	18650	-	0.93	14300	17000	2.3	0.21	4250	2125
S-69	230.2-232.7	Silty fine Sand w. org.	$\bar{Q}$	6.63	18200	-	0.98	17800	17600	6.0	0.50	1600	800
DRILL HOLE A-40-AXU													
S-3	83.0-85.5	Organic CLAY	$\bar{R}$	7.00	5480	5480	-	0	3300	12.1	0.11	4750	2350
"	"	Organic CLAY	$\bar{R}$	7.00	6910	1550	-	5360	6350	13.5	0.05	1490	745
S-7	93.0-95.5	Organic CLAY	$\bar{R}$	7.00	432	432	-	0	244	3.2	0.05	686	343
"	"	Organic CLAY	$\bar{R}$	7.00	2880	2880	-	0	1660	10.6	0.05	2170	1085
S-9	98.0-100.5	PEAT	$\bar{R}$	6.33	720	720	-	0	662	7.5	0.05	1260	630
S-11	103.0-105.5	PEAT	$\bar{R}$	7.00	7600	1210	-	6390	7450	5.9	0.03	1438	719
DRILL HOLE UP-40-CU													
S-1	85.0-87.5	Organic CLAY w/peat	$\bar{Q}$	*3.50	5750	-	0.92	5150	5450	5.3	2.67	400	200
"	"	"	$\bar{Q}$	*3.50	5750	-	0.93	5300	5650	7.4	0.62	390	195
"	"	"	$\bar{Q}$	*3.50	5750	-	0.94	5400	5500	8.3	0.29	363	182
"	"	"	$\bar{Q}$	*3.50	5750	-	0.95	5450	5510	8.2	0.12	345	173
DRILL HOLE DN-25-CU													
S-2	95.0-97.5	Org. peaty CLAY	$\bar{Q}$	*3.50	6350	-	0.99	6150	6380	7.8	2.50	412	206
"	"	"	$\bar{Q}$	*3.50	6390	-	0.97	6210	6350	5.2	1.30	404	202
"	"	"	$\bar{Q}$	*3.50	6390	-	0.97	6200	6310	12.1	0.47	238	119
"	"	"	$\bar{Q}$	*3.50	6390	-	0.97	6200	6150	8.2	0.03	213	107
DRILL HOLE UP-25-CU													
S-5	106.0-108.5	PEAT	$\bar{R}$	6.00	10220	3430	-	6790	10060	7.9	0.04	3100	1550

LEGEND

$\bar{Q}$ - Undrained triaxial test. Bar indicates test with pore water pressure measurements.

$\bar{R}$ - Consolidated-Undrained triaxial test. Bar indicates test with pore water pressure measurement.

H (*) - Height of specimen. Diameter 2.90 inches. Asterisk (*) indicates specimen 1.40 inches in diameter.

σ_3 - Confining pressure.

σ_c - Consolidation pressure.

B - u_o / σ_3

u_o - Pore water pressure under confining pressure, σ_3, prior to application of deviator stress.

u_f - Pore water pressure at failure.

E_f - Percent strain at failure.

s_R - Rate of shear, percent strain per minute.

$(\sigma_1 - \sigma_3)_f$ - Deviator stress at failure

$\left(\frac{\sigma_1 - \sigma_3}{2}\right)_f$ - Shear Strength

diameter specimens were trimmed from larger diameter samples and ranged in height from 3.25 to 3.81 inches. The 2.90-inch diameter specimens were untrimmed and ranged in height from 5.25 to 7.81 inches.

A-13. Undrained (Q) Tests. Undrained (Q) tests were performed on 34 samples taken from various depths in drill hole A-25-AXU. An applied cell pressure equal to the total overburden pressure existing at sample depth was maintained throughout each test. Pore pressure readings were taken, but they may not reflect the pore pressures developed in the zone of shear because of the relatively high rate of strain (0.2 to 1.0 percent per minute) employed in the tests.

Parameter B, which is the ratio of the pore pressure in the sample after stabilization under the confining pressure to the cell pressure, is given for each test in Table A-3. The values of parameter B indicate that the degree of saturation of the samples was close to 100 percent.

The shear strength value obtained from the Q test is equal to one-half the deviator stress at failure. The resultant values of shear strength are summarized in Table A-3.

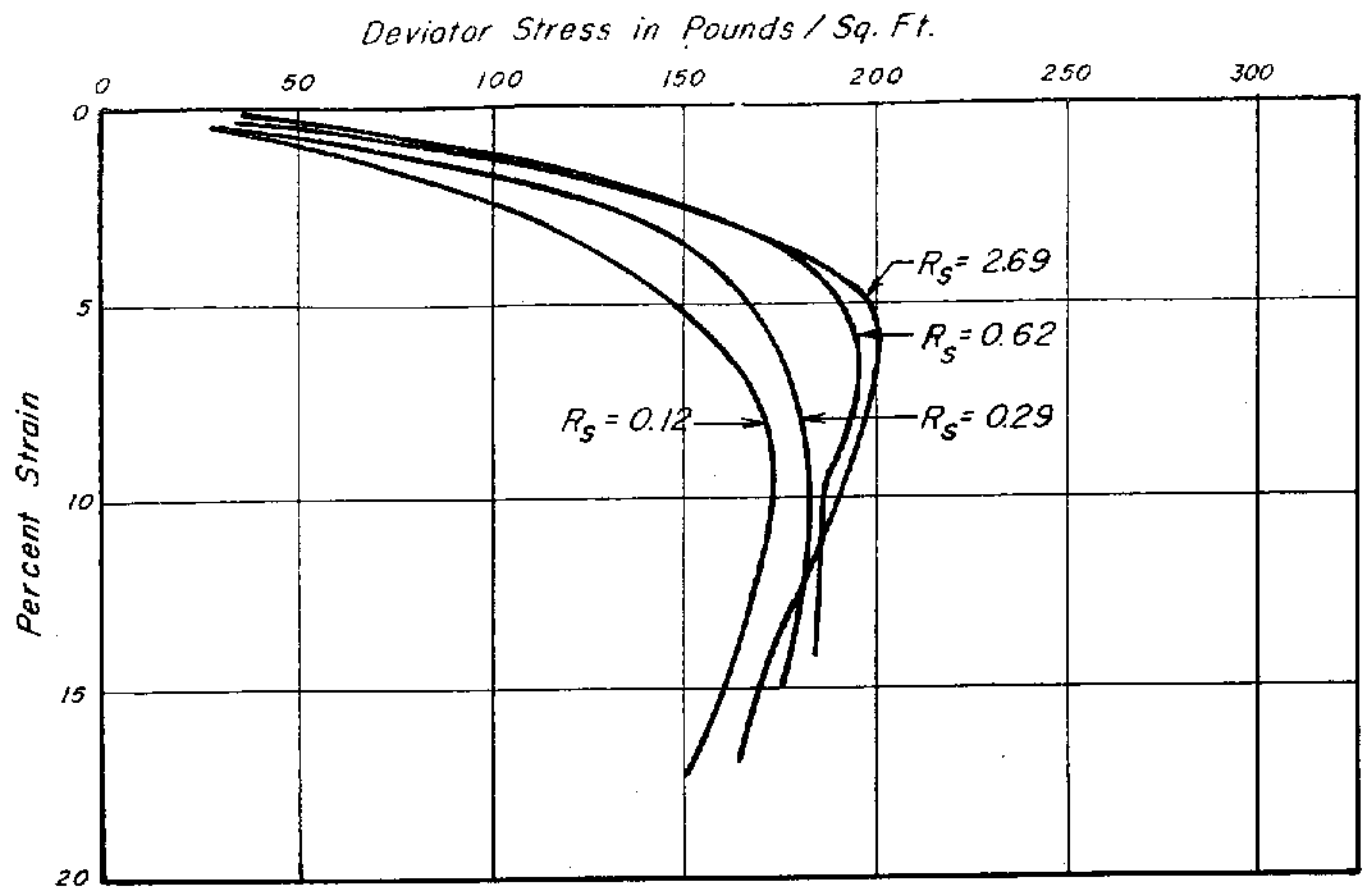
A-14. Rate of Shear Study. The rate of shear, or rate of axial strain, is a significant factor influencing the compressive strength of many soils. The shear strength of normally loaded, sensitive clays is generally a function of the rate of shear employed in the test, increasing as the rate of shear increases. To determine the effect of rate of shear on the shear strength of the soft organic clays, two series of undrained (Q) tests were conducted. Each series was composed of four specimens which were failed at various rates of shear. One test series was performed on four specimens from sample S-1, drill hole UP-40-CU. The other series was performed on four specimens from sample S-2, drill hole DN-25-CU. Specimens were 1.4 inch in diameter and 3.5 inches high.

Each specimen of a particular test series was confined, without drainage, under a pressure equal to the total overburden pressure existing at sample depth. Specimen conditions existing prior to application of the deviator stress, therefore, were nearly identical for each test series. Specimens for both series of tests were failed under rates of shear varying from 0.03 to 2.7 percent strain

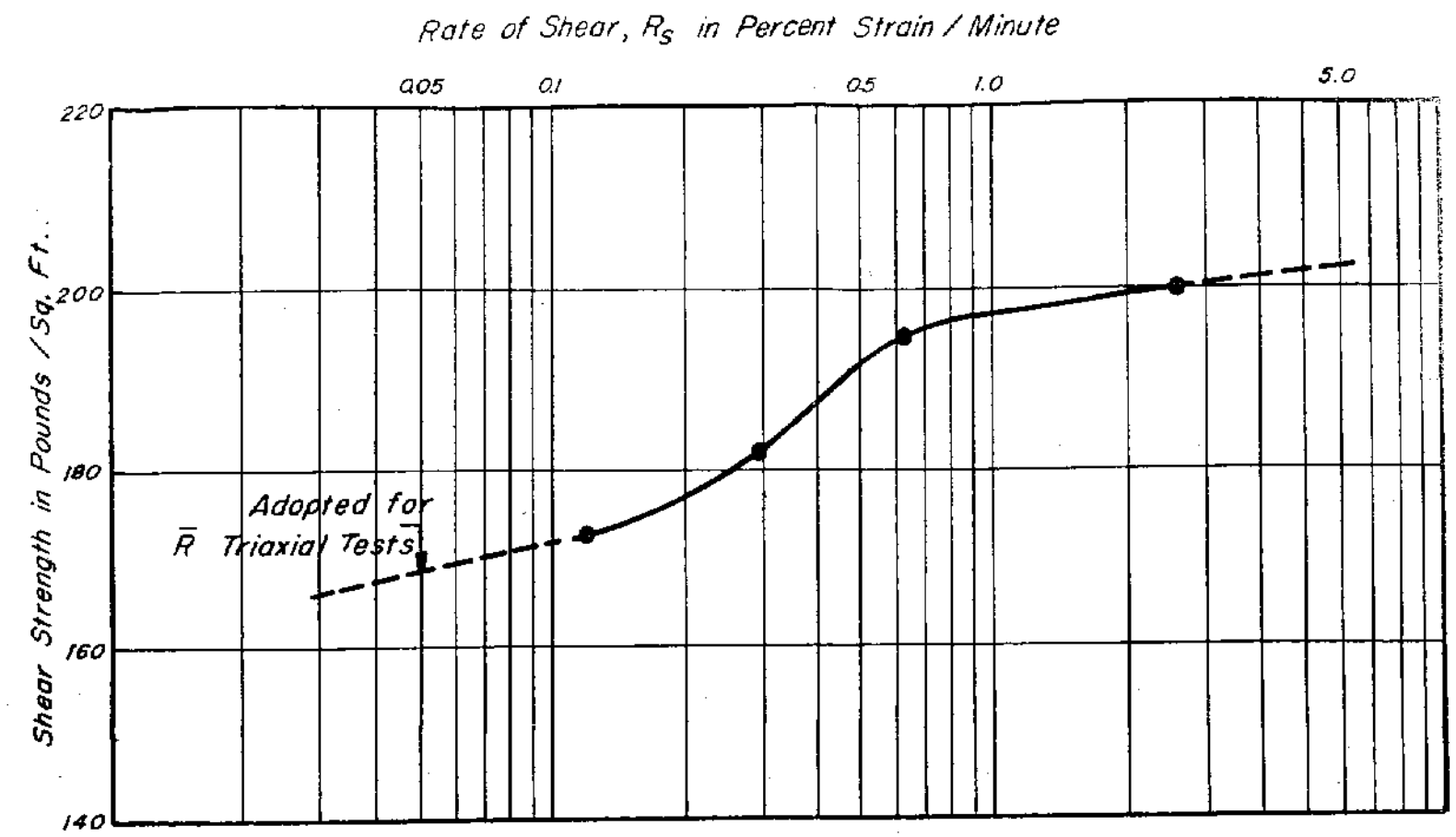
per minute. Test conditions and results are summarized in Table A-3. A graphic presentation of the results of the test series conducted on specimens from sample S-I, drill hole UP-40-CU, is shown on Figure A-3.

The data from this study indicate that a reduction in strain rate produces a lower value of shear strength within the strain rates investigated. The strain at which failure occurs also increases as the rate of shear or strain decreases. In the test series on specimens from sample S-I, drill hole UP-40-CU, for example, the shear strength obtained at a rate of shear of 0.12 percent per minute was about 13.5 percent lower than the shear strength obtained at a strain rate of 2.69 percent per minute. The rate of loading of the foundation material in the field will be very slow, assuming a uniform rate of load application resulting from fill placed from elevation +10 feet to elevation +75 feet in a three-year construction period. The shear strength parameters, cohesion c, and angle of shearing resistance Ø, used in studies relating to the gain in shear strength of the foundation materials with consolidation must be determined, therefore, from laboratory tests simulating as closely as practical the slow rate of field loading. Inspection of the shear strength versus rate of shear curve, Figure A-3, shows that a rate of shear in the order of 0.05 percent strain per minute would provide data representative of slow rate of loading conditions. This rate of shear would also be sufficiently slow to permit measurement of pore pressures in the sample during shear. Consequently, this strain rate was adopted for the remaining consolidated undrained tests to be conducted on the foundation clay and peat materials. No correction for rate of shear was applied to the values of shear strength obtained from either the Q tests or the unconfined compression tests which were generally conducted at higher rates of loading. It was assumed that higher values of shear strength obtained in these tests, resulting from high rates of shear would be essentially compensated for by the fact that sample disturbance in any strength test on sensitive clays, tends to result in lower values of shear strength than those actually existing in the field.

A-15. Consolidated Undrained (R) Tests. Eight consolidated-undrained (R) tests were conducted on clay specimens and four consolidated-undrained tests



STRESS VS. STRAIN CURVES



RATE OF SHEAR VS. SHEAR STRENGTH

NOTES

1. All Tests Undrained (Q) Triaxial Tests
2. All Specimens from Sample S-I, UP-40-CU
   Depth 85.0'-87.5'

PROPOSED DAM - TRINIDAD ARM
GATUN LAKE, PANAMA CANAL ZONE

THE EFFECT OF RATE OF SHEAR
ON SHEAR STRENGTH
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FEBRUARY, 1963

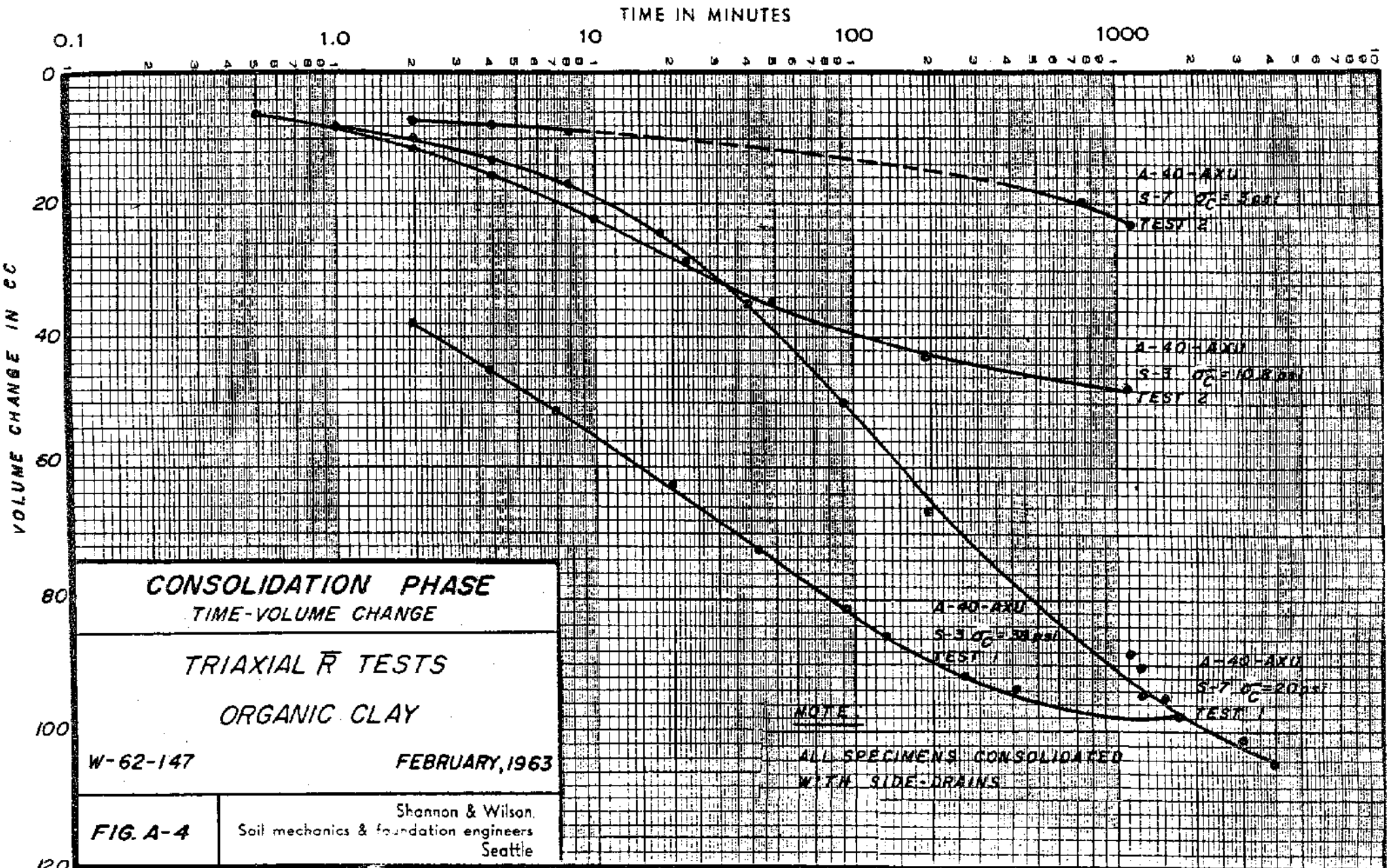
Chagnon and Wilson Inc.

were conducted on peat samples to determine the shear strength parameters of these materials. Four of the R tests were conducted on clay specimens from drill hole A-25-AXU as part of the initial laboratory testing program. These samples were not provided with side drains and were tested at a rate of shear in the order of 0.4 to 0.9 percent strain per minute. Four other tests were conducted on clay samples from drill hole A-40-AXU. These samples were provided with side drains consisting of from four to six strips of blotter paper. The strain rate adopted for testing the latter group of specimens was determined from evaluation of data from the rate of shear study and, generally, was in the order of 0.05 percent strain per minute.

Two procedures were adopted to consolidate the specimens prior to application of the deviator stress. In one method, the specimen was subjected to a cell pressure and, after stabilization of conditions in the sample, drainage and subsequent consolidation of the specimen were permitted. The effective consolidation pressure, σ_c, was then equal to the cell pressure. In the alternate method in-situ conditions were simulated by subjecting the speciment to a cell pressure equal to the total overburden pressure existing at sample depth and allowing conditions in the sample to stabilize. The cell pressure was then increased and consolidation of the sample permitted with a back pressure applied to the pore water equal to the approximate hydrostatic head existing at sample depth. Consolidation was continued until the pore water pressure in the sample dissipated and became equal to the back pressure. The effective consolidation pressure in this approach, therefore, was equal to the difference between the cell pressure and the applied back pressure. Use of the back pressure method causes any air trapped between the rubber membrane and the sample or in the pores of the sample to dissolve. Time-versus-volume change curves representing the consolidation phase in the R tests are shown for both clay and peat specimens on Figures A-4 and A-5.

The results of the consolidated-undrained tests are represented graphically by the stress-strain curves shown of Figures A-6 through A-17.

BORING	SAMPLE	DEPTH	HEIGHT	DIAMETER	CLASSIFICATION
A-40-AXU	S=3	83'-85.5'	7.0 in.	2 7/8 in. (TEST 1)	Very soft organic silty CLAY
			7.0 in.	2 7/8 in. (TEST 2)	
	S-7	93'-95.5'	7.0 in.	2 7/8 in. (TEST 1)	Very soft organic CLAY
			7.0 in.	2 7/8 in. (TEST 2)	



CONSOLIDATION PHASE

TIME-VOLUME CHANGE

TRIAXIAL $\bar{R}$ TESTS

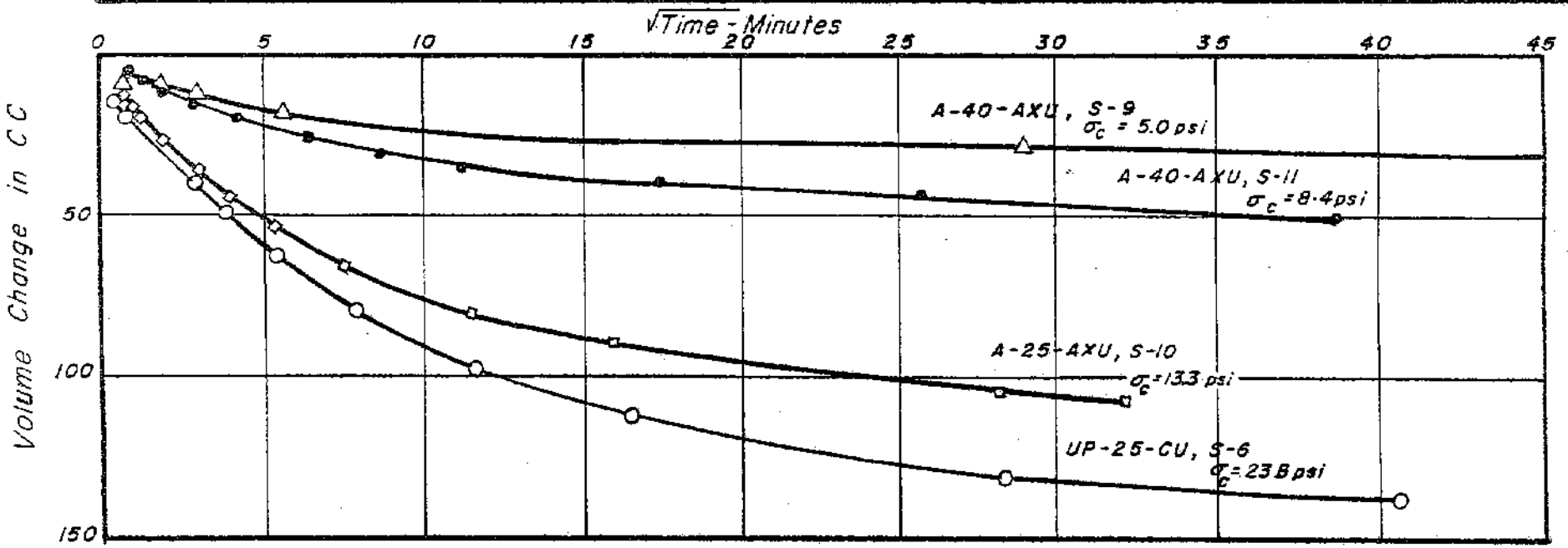
ORGANIC CLAY

W-62-147 FEBRUARY, 1963

FIG. A-4

Shannon & Wilson,
Soil mechanics & foundation engineers
Seattle

Boring	Sample	Depth	Height	Diameter	Classification:
A-40 AXU	S-9	98.0-100.5	6.1"	2.9"	Slightly clayey fibrous PEAT
	S-11	103.0-105.0	7.0"	2.9"	Slightly clayey fibrous PEAT
A-25-AXU	S-10	102.9-105.4	7.0"	2.9"	Slightly clayey fibrous PEAT
UP-25 CU	S-6	106.0-108.5	6.0"	2.9"	Slightly clayey fibrous PEAT



CONSOLIDATION PHASE

TIME-VOLUME CHANGE

TRIAXIAL $\bar{R}$ TESTS

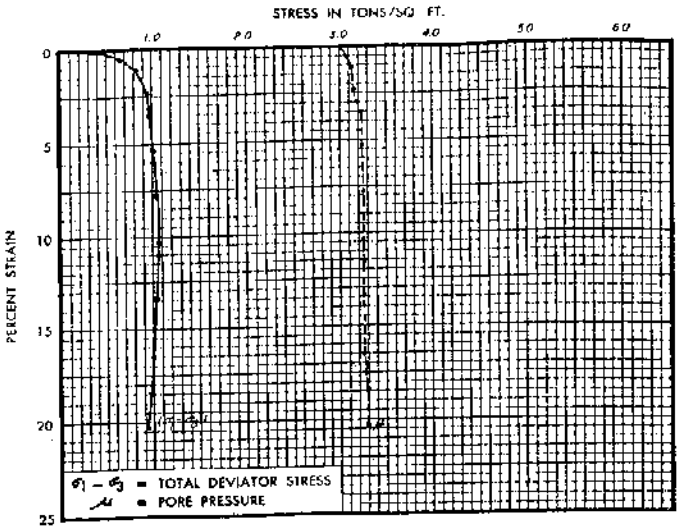
PEAT

W-62-147 FEBRUARY, 1963

FIG. A-5 Shannon and Wilson, Inc. Soil Mechanics and Foundation Engineers

Note

All Samples Consolidated with Side Drains.



TYPE OF TEST

☐ Q ■ $\bar{R}$ ☐ S ☐ PRESATURATED

REMARKS: RATE OF SHEAR, R_s = 0.61% / MIN.

σ_3 = CHAMBER PRESSURE = 3.53 TONS/SQ. FT.

SPECIMEN

CLASSIFICATION: SOFT DARK GRAY ORGANIC CLAY

☐ COMPACTED ■ UNDISTURBED

COMPACTIVE EFFORT:

HEIGHT = 6.16 inches

DIAMETER = 2.9 inches

WET UNIT WT. 92.5 pcf.

WATER CONTENT:

BEFORE TEST = — %

AFTER TEST = 76.8 %

AFTER SATURATION = ____ %

FAILURE SKETCH

TRIAXIAL COMPRESSION TEST

STRESS vs. STRAIN

BORING A-25-AXU

SAMPLE S-4

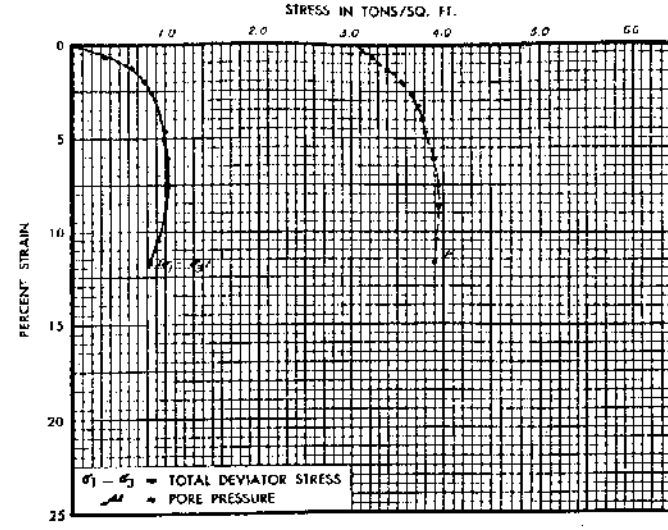
W-62-147 FEBRUARY, 1963

FIG. A-6

Shannon and Wilson

Soil mechanics and foundation engineers

Seattle



TYPE OF TEST

☐ Q ■ $\bar{R}$ ☐ S ☐ PRESATURATED

REMARKS: RATE OF SHEAR, R_s = 0.68% / MIN.

σ_3 = CHAMBER PRESSURE = 4.45 TONS/SQ. FT.

SPECIMEN

CLASSIFICATION: SOFT, DARK GRAY AND BROWN ORGANIC CLAY

☐ COMPACTED ■ UNDISTURBED

COMPACTIVE EFFORT:

HEIGHT = 8.13 inches

DIAMETER = 2.9 inches

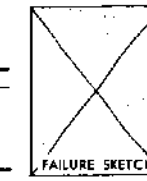
WET UNIT WT. 71.0 pcf.

WATER CONTENT:

BEFORE TEST = 200.0 %

AFTER TEST = 147.0 %

AFTER SATURATION = ____ %



FAILURE SKETCH

TRIAXIAL COMPRESSION TEST

STRESS vs. STRAIN

BORING A-25-AXU

SAMPLE S-7

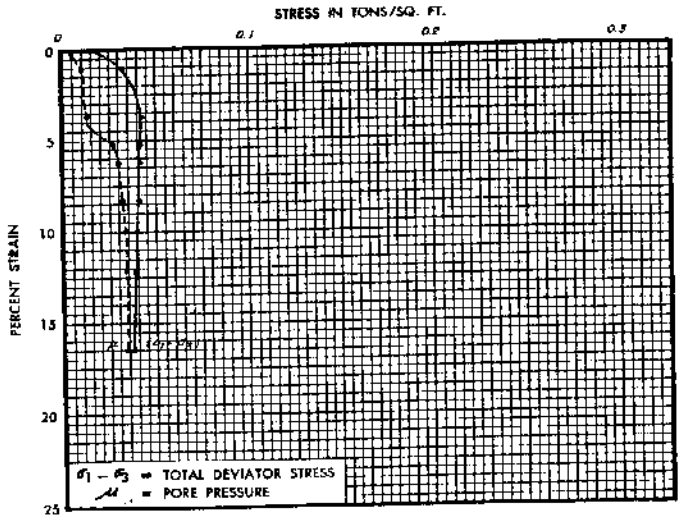
W-62-147 FEBRUARY, 1963

FIG. A-7

Shannon and Wilson

Soil mechanics and foundation engineers

Seattle



TYPE OF TEST

☐ Q ■ $\bar{R}$ ☐ S ☐ PRESATURATED

REMARKS: RATE OF SHEAR, R_s = 0.93% / MIN.

σ_3 = CHAMBER PRESSURE = 0.43 TONS/SQ. FT.

SPECIMEN

CLASSIFICATION: SOFT, DARK GRAY AND BROWN ORGANIC CLAY

☐ COMPACTED ■ UNDISTURBED

COMPACTIVE EFFORT:

HEIGHT = 3.50 inches

DIAMETER = 1.4 inches

WET UNIT WT. 85.5 pcf.

WATER CONTENT:

BEFORE TEST = 142.1 %

AFTER TEST = 124.2 %

AFTER SATURATION = ____ %

FAILURE SKETCH

TRIAXIAL COMPRESSION TEST

STRESS vs. STRAIN

BORING A-25-AXU

SAMPLE S-7

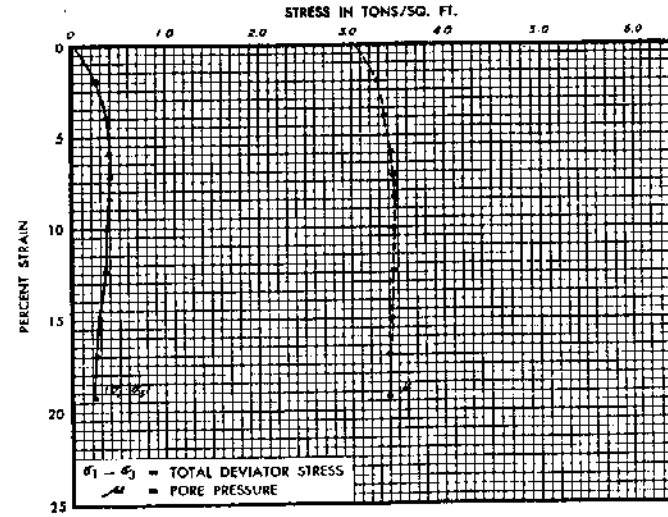
W-62-147 FEBRUARY, 1963

FIG. A-8

Shannon and Wilson

Soil mechanics and foundation engineers

Seattle



TYPE OF TEST

☐ Q ■ $\bar{R}$ ☐ S ☐ PRESATURATED

REMARKS: RATE OF SHEAR, R_s = 0.40% / MIN.

σ_3 = CHAMBER PRESSURE = 3.55 TONS/SQ. FT.

SPECIMEN

CLASSIFICATION: VERY SOFT, BROWN ORGANIC CLAY

☐ COMPACTED ■ UNDISTURBED

COMPACTIVE EFFORT:

HEIGHT = 5.75 inches

DIAMETER = 2.9 inches

WET UNIT WT. 102.0 pcf.

WATER CONTENT:

BEFORE TEST = — %

AFTER TEST = 184.0 %

AFTER SATURATION = ____ %

FAILURE SKETCH

TRIAXIAL COMPRESSION TEST

STRESS vs. STRAIN

BORING A-25-AXU

SAMPLE S-9

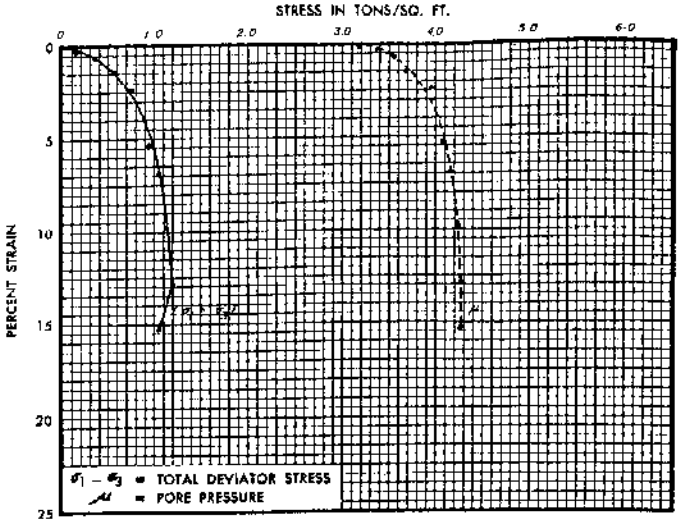
W-62-147 FEBRUARY, 1963

FIG. A-9

Shannon and Wilson

Soil mechanics and foundation engineers

Seattle



TYPE OF TEST

☐ Q ■ $\bar{R}$ ☐ S ☐ PRESATURATED

REMARKS: RATE OF STRAIN = 0.05% / MIN.
BACK PRESSURE = 3.38 TONS/SQ. FT.

σ_3 = CHAMBER PRESSURE = 4.25 TONS/SQ. FT.

SPECIMEN

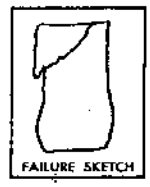
CLASSIFICATION: FIRM SLIGHTLY CLAYEY FIBROUS PEAT

☐ COMPACTED ■ UNDISTURBED

COMPACTIVE EFFORT:

HEIGHT = 7.0 inches
DIAMETER = 2.9 inches
WET UNIT WT. 64.7 pcf.

WATER CONTENT:
BEFORE TEST = 271 %
AFTER TEST = 280 %
AFTER SATURATION = %


FAILURE SKETCH

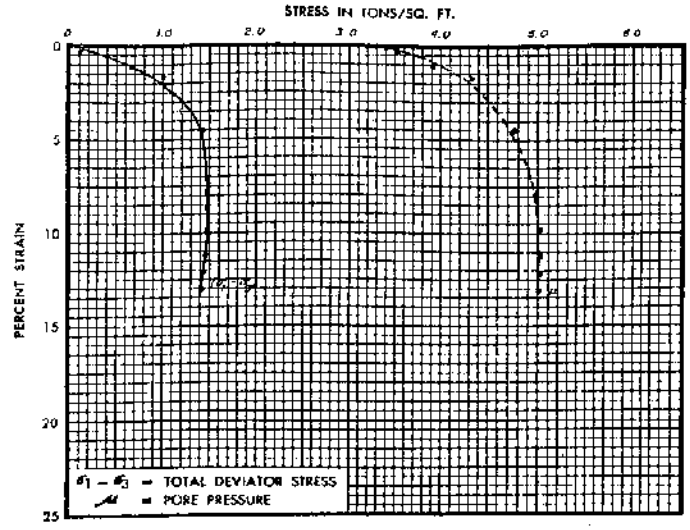
TRIAXIAL COMPRESSION TEST
STRESS vs. STRAIN

BORING A-25-AXU
SAMPLE S-10

W-62-147 FEBRUARY, 1963

FIG. A-10

Shannon and Wilson
Soil mechanics and foundation engineers
Seattle



TYPE OF TEST

☐ Q ■ $\bar{R}$ ☐ S ☐ PRESATURATED

REMARKS: RATE OF STRAIN = 0.042% / MIN.
BACK PRESSURE = 3.37 TONS/SQ. FT.

σ_3 = CHAMBER PRESSURE = 5.07 TONS/SQ. FT.

SPECIMEN

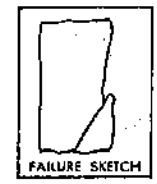
CLASSIFICATION: FIRM SLIGHTLY CLAYEY FIBROUS PEAT

☐ COMPACTED ■ UNDISTURBED

COMPACTIVE EFFORT:

HEIGHT = 6.0 inches
DIAMETER = 2.9 inches
WET UNIT WT. 65.7 pcf.

WATER CONTENT:
BEFORE TEST = 178 %
AFTER TEST = 131 %
AFTER SATURATION = %


FAILURE SKETCH

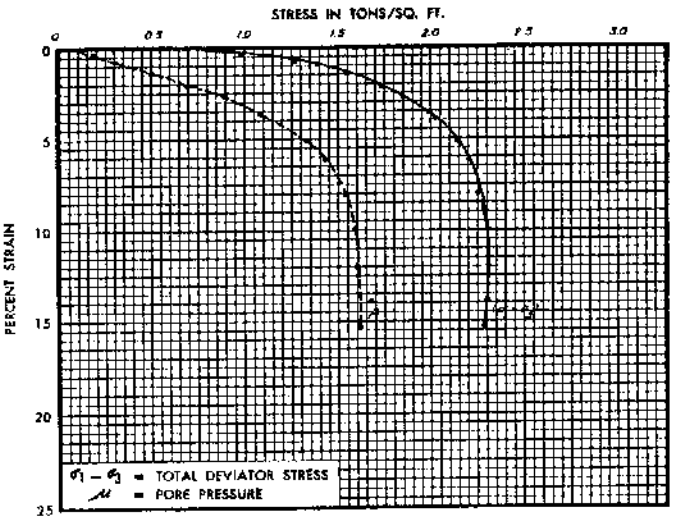
TRIAXIAL COMPRESSION TEST
STRESS vs. STRAIN

BORING UP-25-CU
SAMPLE S-6

W-62-147 FEBRUARY, 1963

FIG. A-11

Shannon and Wilson
Soil mechanics and foundation engineers
Seattle



TYPE OF TEST

☐ Q ■ $\bar{R}$ ☐ S ☐ PRESATURATED

REMARKS: RATE OF SHEAR, R_2 = 0.11% / MIN.

σ_3 = CHAMBER PRESSURE = 2.68 TONS/SQ. FT.

SPECIMEN

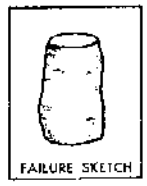
CLASSIFICATION: VERY SOFT, GRAY AND BROWN SILTY ORGANIC CLAY

☐ COMPACTED ■ UNDISTURBED

COMPACTIVE EFFORT:

HEIGHT = 7.0 inches
DIAMETER = 2.9 inches
WET UNIT WT. 98.8 pcf.

WATER CONTENT:
BEFORE TEST = 63.6 %
AFTER TEST = 49.9 %
AFTER SATURATION = %


FAILURE SKETCH

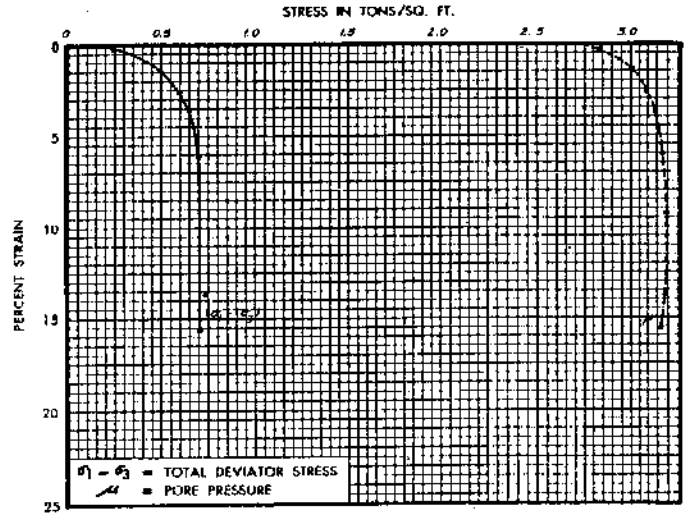
TRIAXIAL COMPRESSION TEST
STRESS vs. STRAIN

BORING A-40-AXU
SAMPLE S-3

W-62-147 FEBRUARY, 1963

FIG. A-12

Shannon and Wilson
Soil mechanics and foundation engineers
Seattle



TYPE OF TEST

☐ Q ■ $\bar{R}$ ☐ S ☐ PRESATURATED

REMARKS: RATE OF SHEAR, R_2 = 0.05%/MIN.

σ_3 = CHAMBER PRESSURE = 3.38 TONS/SQ. FT.

SPECIMEN

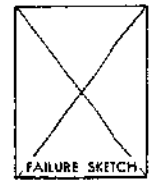
CLASSIFICATION: VERY SOFT, GRAY AND BROWN SILTY ORGANIC CLAY

☐ COMPACTED ■ UNDISTURBED

COMPACTIVE EFFORT:

HEIGHT = 7.0 inches
DIAMETER = 2.9 inches
WET UNIT WT 98.7 pcf.

WATER CONTENT:
BEFORE TEST = 70.5 %
AFTER TEST = 58.0 %
AFTER SATURATION = %


FAILURE SKETCH

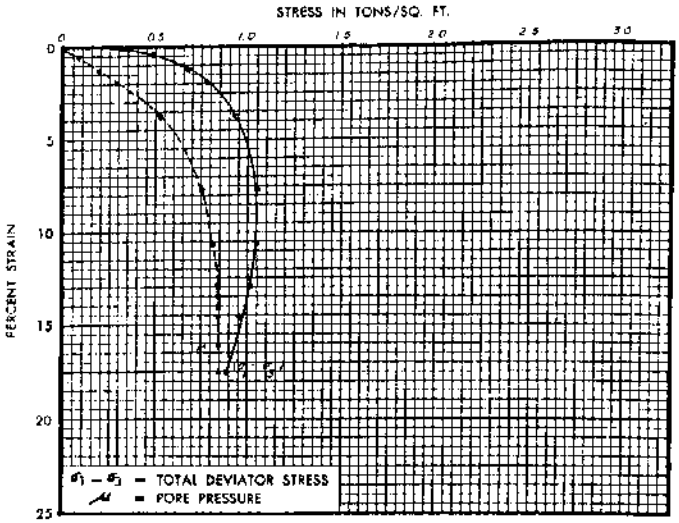
TRIAXIAL COMPRESSION TEST
STRESS vs. STRAIN

BORING A-40-AXU
SAMPLE S-3

W-62-147 FEBRUARY, 1963

FIG. A-13

Shannon and Wilson
Soil mechanics and foundation engineers
Seattle



TYPE OF TEST

☐ Q ■ $\bar{R}$ ☐ S ☐ PRESATURATED

REMARKS: RATE OF SHEAR, R_s = 0.05% / MIN.

σ_3 = CHAMBER PRESSURE = 1.41 TONS/SQ. FT.

FAILURE SKETCH

SPECIMEN

CLASSIFICATION: VERY SOFT, GRAY ORGANIC CLAY

☐ COMPACTED ■ UNDISTURBED

COMPACTIVE EFFORT:

HEIGHT = 7.0 inches

DIAMETER = 2.9 inches

WET UNIT WT. 88.2 pcf.

WATER CONTENT:

BEFORE TEST = 91.8 %

AFTER TEST = 72.5 %

AFTER SATURATION = %

TRIAXIAL COMPRESSION TEST

STRESS vs. STRAIN

BORING A 40-AXU

SAMPLE S-7

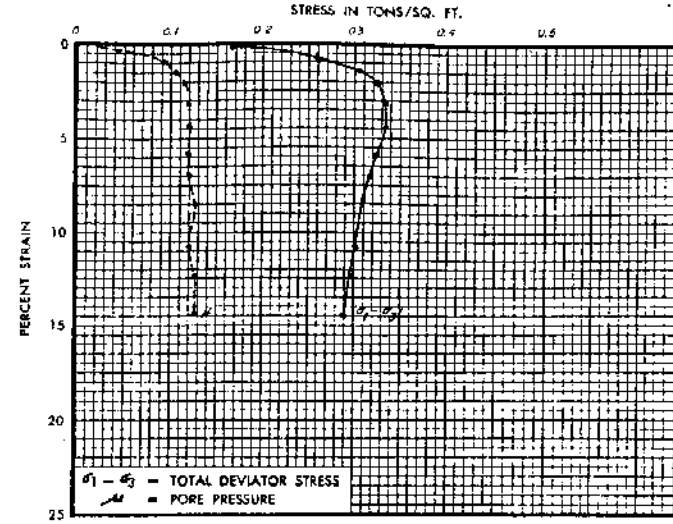
W-62-147 FEBRUARY, 1963

FIG. A-14

Shannon and Wilson

Soil mechanics and foundation engineers

Seattle

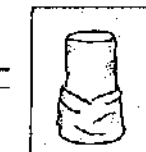


TYPE OF TEST

☐ Q ■ $\bar{R}$ ☐ S ☐ PRESATURATED

REMARKS: RATE OF SHEAR, R_s = 0.05% / MIN.

σ_3 = CHAMBER PRESSURE = 0.21 TONS/SQ. FT.


FAILURE SKETCH

SPECIMEN

CLASSIFICATION: VERY SOFT, GRAY, ORGANIC CLAY

☐ COMPACTED ■ UNDISTURBED

COMPACTIVE EFFORT:

HEIGHT = 7.00 inches

DIAMETER = 2.9 inches

WET UNIT WT. 88.3 pcf.

WATER CONTENT:

BEFORE TEST = 97.6 %

AFTER TEST = 92.9 %

AFTER SATURATION = %

TRIAXIAL COMPRESSION TEST

STRESS vs. STRAIN

BORING A-40-AXU

SAMPLE S-7

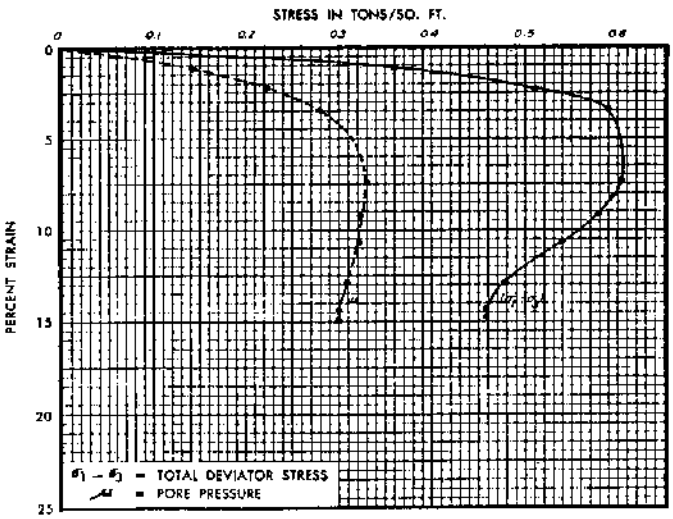
W-62-147 FEBRUARY, 1963

FIG. A-15

Shannon and Wilson

Soil mechanics and foundation engineers

Seattle



TYPE OF TEST

☐ Q ■ $\bar{R}$ ☐ S ☐ PRESATURATED

REMARKS: RATE OF STRAIN = 0.048% / MIN.

σ_3 = CHAMBER PRESSURE = 0.35 TONS/SQ. FT.

FAILURE SKETCH

SPECIMEN

CLASSIFICATION: SOFT, SLIGHTLY CLAYEY FIBROUS PEAT

☐ COMPACTED ■ UNDISTURBED

COMPACTIVE EFFORT:

HEIGHT = 6.125 inches

DIAMETER = 2.9 inches

WET UNIT WT. 73.5 pcf

WATER CONTENT:

BEFORE TEST = 215.0 %

AFTER TEST = 181.5 %

AFTER SATURATION = %

TRIAXIAL COMPRESSION TEST

STRESS vs. STRAIN

BORING A-40-AXU

SAMPLE S-9

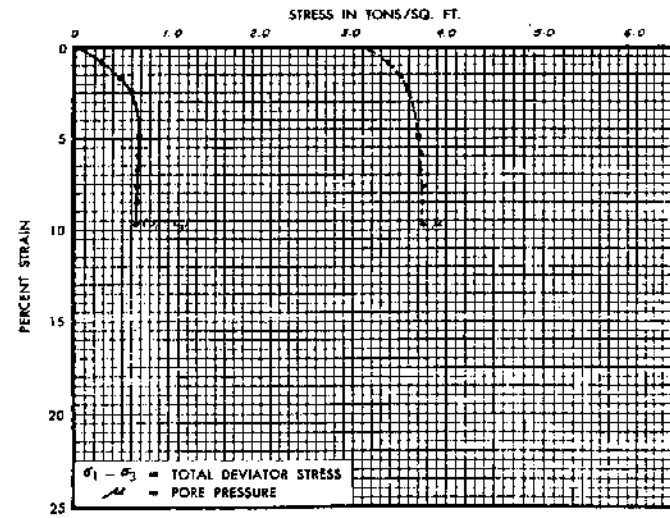
W-62-147 FEBRUARY, 1963

FIG. A-16

Shannon and Wilson

Soil mechanics and foundation engineers

Seattle

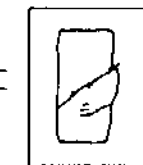


TYPE OF TEST

☐ Q ■ $\bar{R}$ ☐ S ☐ PRESATURATED

REMARKS: RATE OF STRAIN = 0.031% / MIN.
BACK PRESSURE = 3.25 TONS / SQ. FT.

σ_3 = CHAMBER PRESSURE = 3.8 TONS/SQ. FT.


FAILURE SKETCH

SPECIMEN

CLASSIFICATION: SOFT SLIGHTLY CLAYEY FIBROUS PEAT

☐ COMPACTED ■ UNDISTURBED

COMPACTIVE EFFORT:

HEIGHT = 7.0 inches

DIAMETER = 2.9 inches

WET UNIT WT. 88.3 pcf.

WATER CONTENT:

BEFORE TEST = 263 %

AFTER TEST = 241 %

AFTER SATURATION = %

TRIAXIAL COMPRESSION TEST

STRESS vs. STRAIN

BORING A-40-AXU

SAMPLE S-11

W-62-147 FEBRUARY, 1963

FIG. A-17

Shannon and Wilson

Soil mechanics and foundation engineers

Seattle

A-16. CONSOLIDATION TESTS.

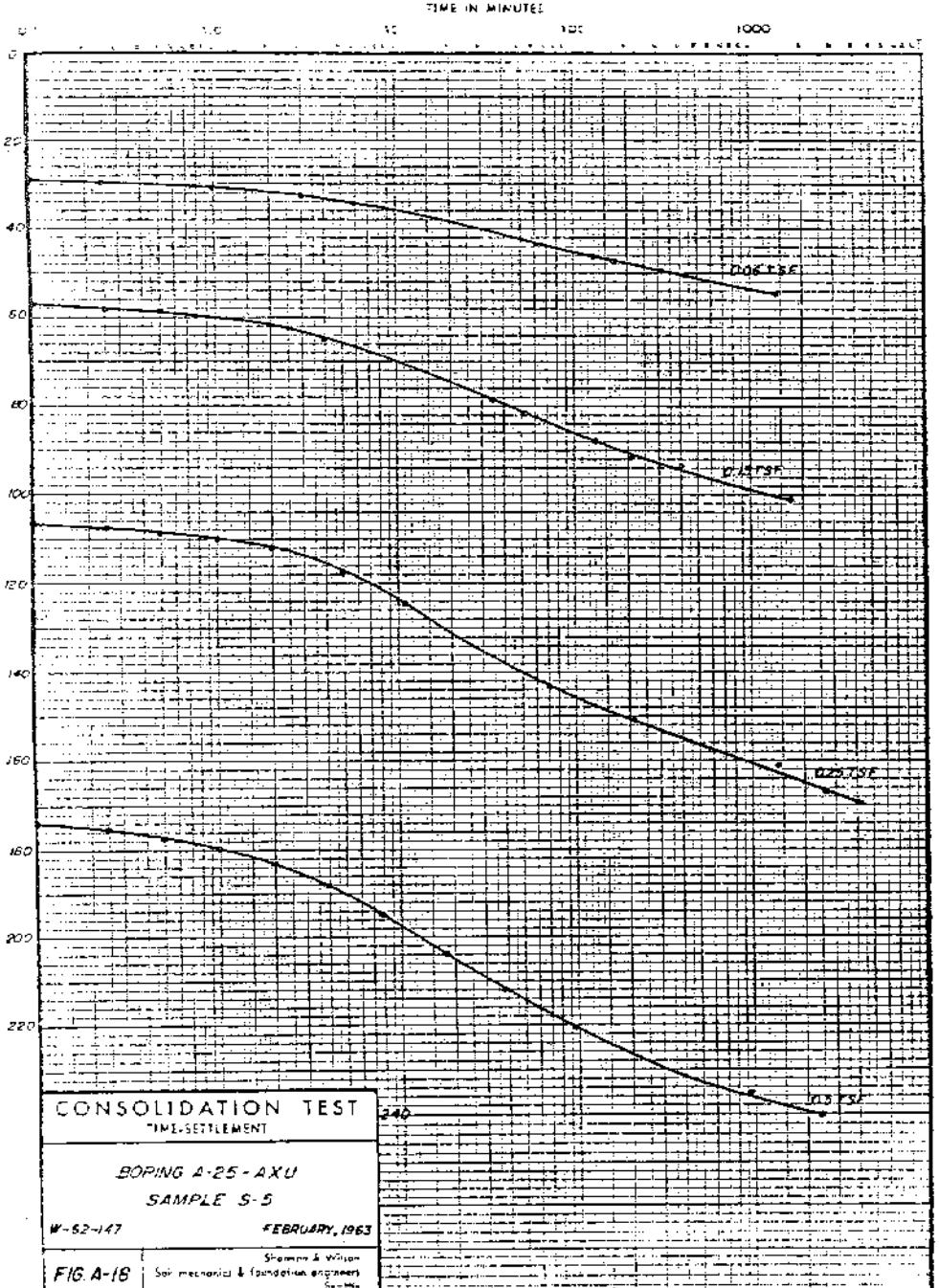
Consolidation properties of the clay and peat materials encountered in drill hole A-25-AXU were investigated by means of 19 consolidation tests performed on samples from various depths in the boring. One consolidation test was also conducted on a specimen from sample S-1, drill hole UP-15-CU for comparison with the consolidation test results obtained from drill hole A-25-AXU and as part of the Triaxial Dissipation Studies outlined in paragraph A-17. The coefficient of consolidation, C_v, which is a measure of the rate of consolidation or settlement of the material, was calculated from the time-dial reading curves for imposed loads of 0.25 to 2 tons per square foot. The calculated values of C_v, at various depths in A-25-AXU and for sample S-1, drill hole UP-15-CU, are presented on Figure 5-3. It will be noted in this figure that the value of C_v obtained from the test on sample S-1, drill hole UP-15-CU, agrees favorably with values of C_v calculated from the series of tests on samples from A-25-AXU. Typical time-versus-dial reading curves for various loadings are shown in Figures A-18 through A-20. The stress versus percent strain curves obtained in the consolidation tests are presented on Figures A-21 and A-22.

The consolidation tests were performed in floating ring consolidometers with drainage from the top and the base of the specimen and with loads applied in the conventional manner. Specimens 2.75 or 2.5 inches in diameter by one-half inch high were employed in the tests on the clay and peat materials. Due to the fibrous nature of the peat and the sensitivity of the clay, some sample disturbance undoubtedly resulted from trimming and handling of the test specimens. Extreme care, however, was employed to insure that sample disturbance was minimized.

A-17. TRIAXIAL DISSIPATION TESTS.

This test series was undertaken to compare values of the coefficient of consolidation, C_v, calculated from the triaxial dissipation tests with those obtained from consolidation tests. It was conceivable, in our opinion, that the value of C_v calculated from consolidation tests might be too low because of gas being developed in the pore space of the sample, reducing sample

BORING	SAMPLE	DEPTH	HEIGHT	DIAMETER	CLASSIFICATION
A-25-AXU	S-5	89.9'-92.4'	1.225 cm	2.75 in.	Very soft dark gray organic CLAY



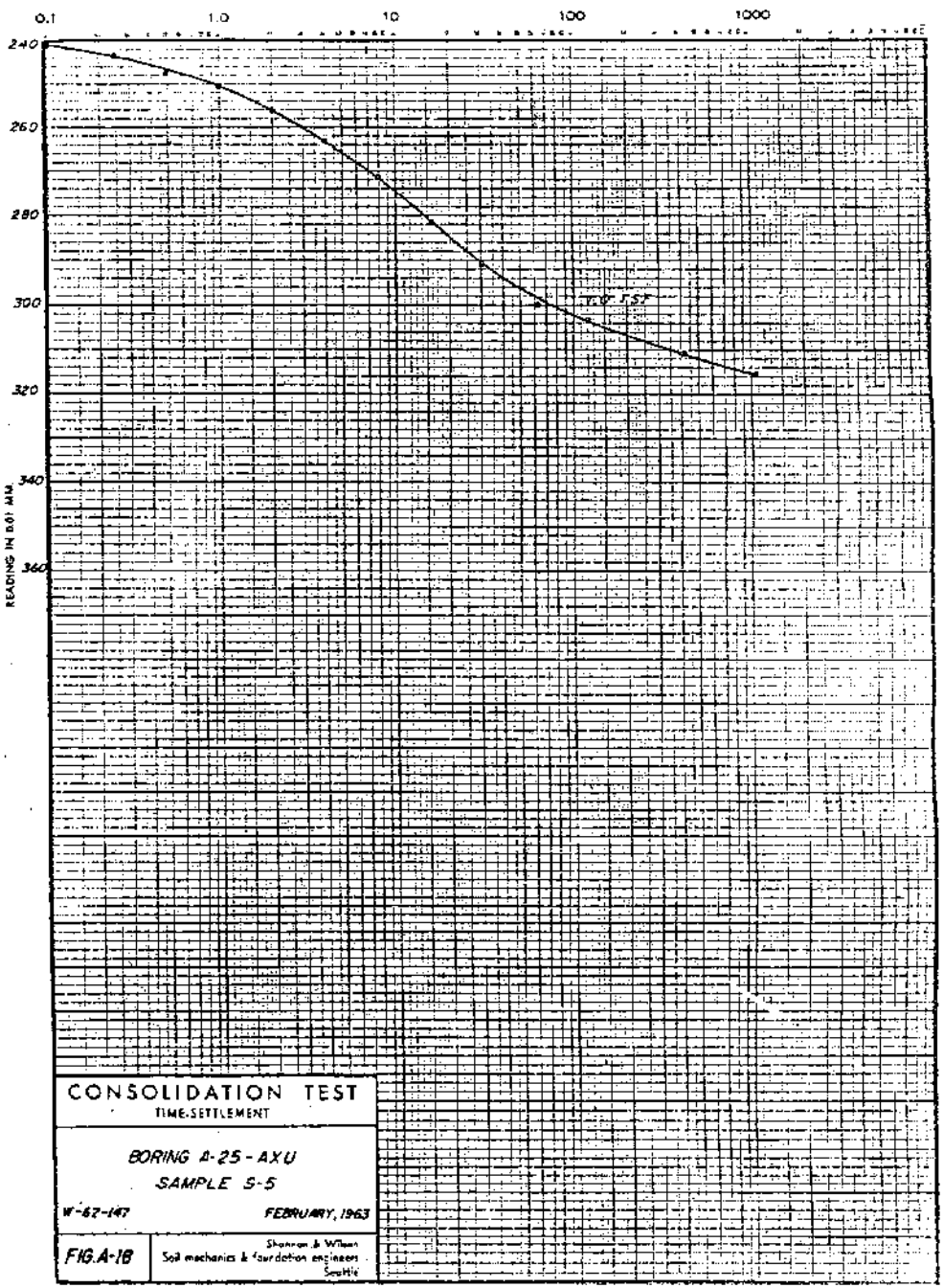
CONSOLIDATION TEST
TIME-SETTLEMENT

BORING A-25-AXU
SAMPLE S-5

W-62-147 FEBRUARY, 1963

FIG. A-18 | Shannon & Wilson Soil mechanics & foundation engineers Seattle

BORING	SAMPLE	DEPTH	HEIGHT	DIAMETER	CLASSIFICATION
A-25-AXU	S-5	89.9'-92.4'	1.225 cm	2.75 in.	Very soft dark gray organic CLAY



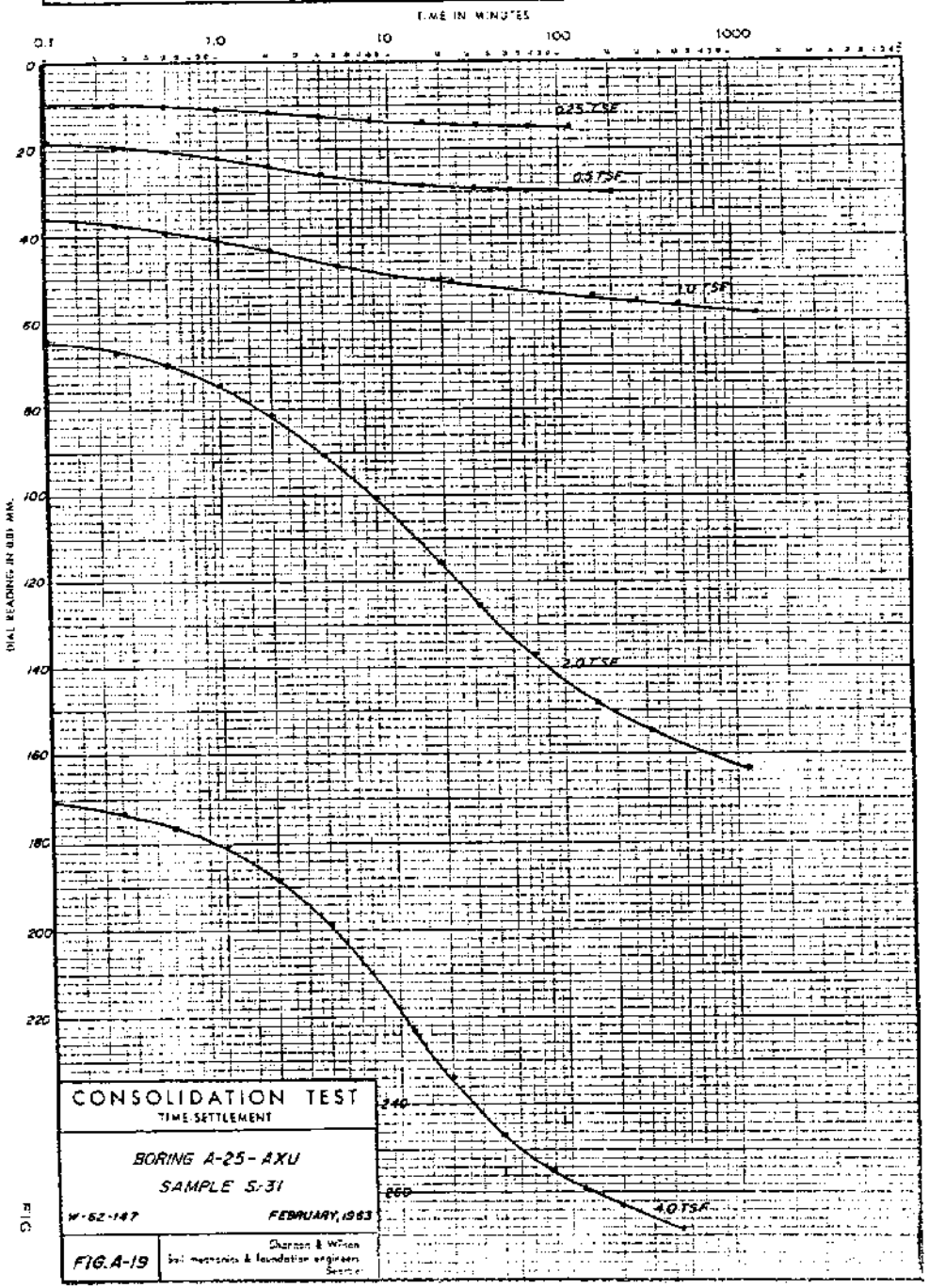
CONSOLIDATION TEST
TIME-SETTLEMENT

BORING A-25-AXU
SAMPLE S-5

W-62-147 FEBRUARY, 1963

FIG. A-18 | Shannon & Wilson Soil mechanics & foundation engineers Seattle

BORING	SAMPLE	DEPTH	HEIGHT	DIAMETER	CLASSIFICATION
A-25-AXU	S-31	157.6'-160.1'	1.167cm	2.75 in.	Medium gray-brown organic silty CLAY



CONSOLIDATION TEST
TIME-SETTLEMENT

BORING A-25-AXU
SAMPLE S-31

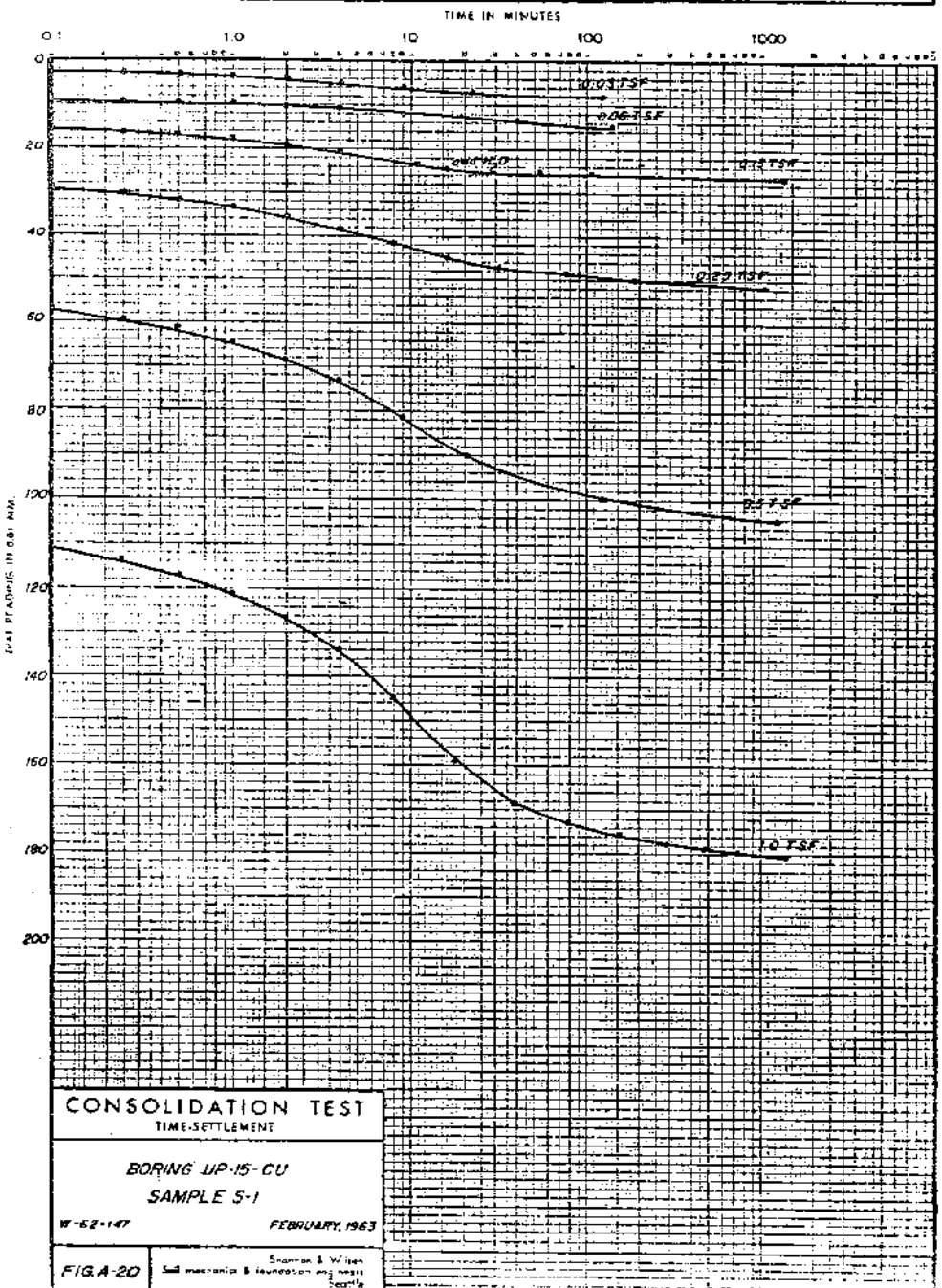
W-62-147 FEBRUARY, 1963

FIG. A-19

Sherman & Wilson
Soil mechanics & foundation engineers
Seattle

FIG.

BORING	SAMPLE	DEPTH	HEIGHT	DIAMETER	CLASSIFICATION
UP-15-CU	S-1	90.0'-92.5'	1.18 cm	2.75 in.	Very soft purple-gray organic CLAY



CONSOLIDATION TEST
TIME-SETTLEMENT

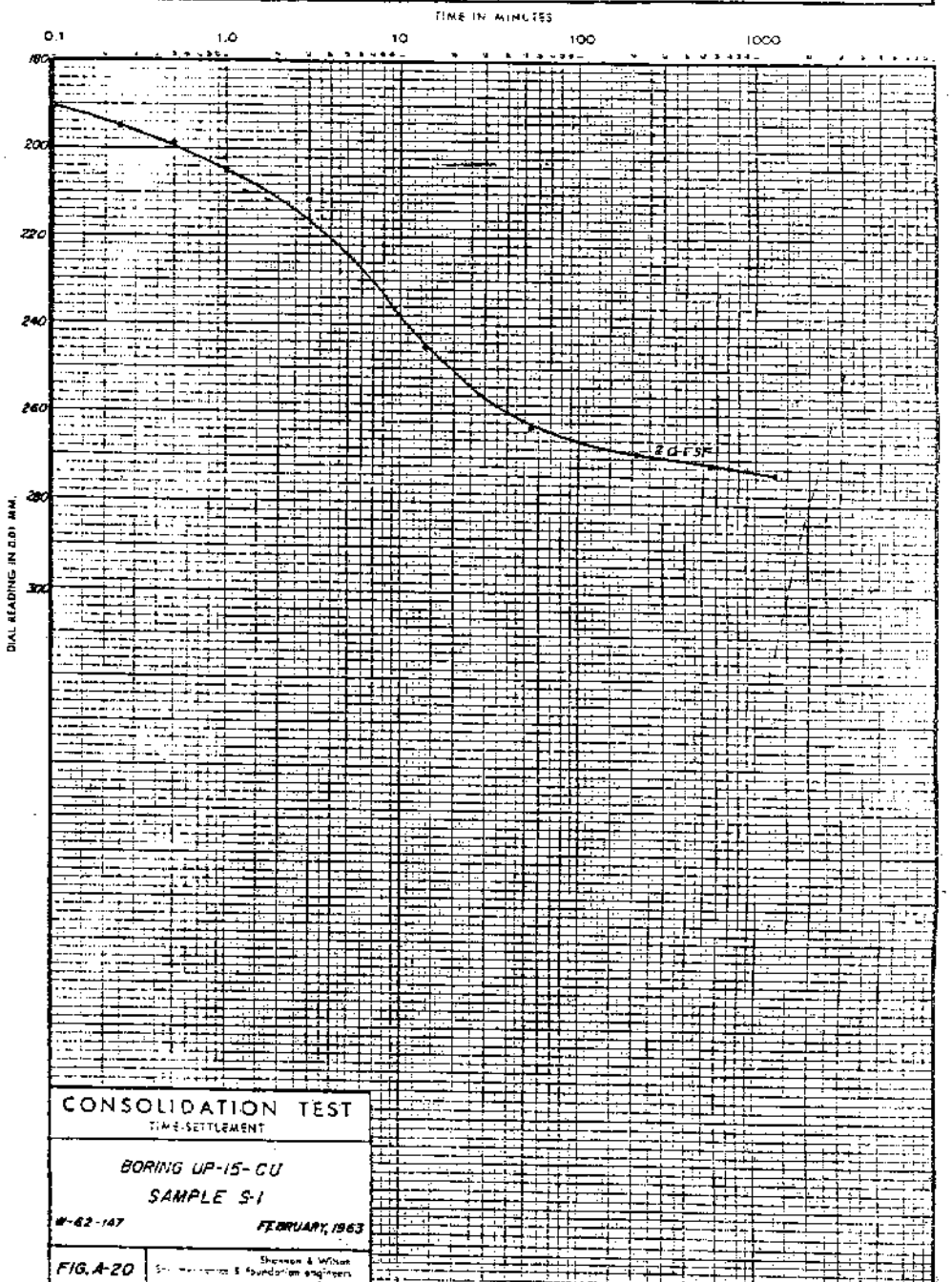
BORING UP-15-CU
SAMPLE S-1

W-62-147 FEBRUARY, 1963

FIG. A-20

Shannon & Wilson
Soil mechanics & foundation engineers
Seattle

BORING	SAMPLE	DEPTH	HEIGHT	DIAMETER	CLASSIFICATION
UP-15-CU	S-1	90.0'-92.5'	1.18 cm	2.75 in.	Very soft purple-gray organic CLAY



CONSOLIDATION TEST
TIME-SETTLEMENT

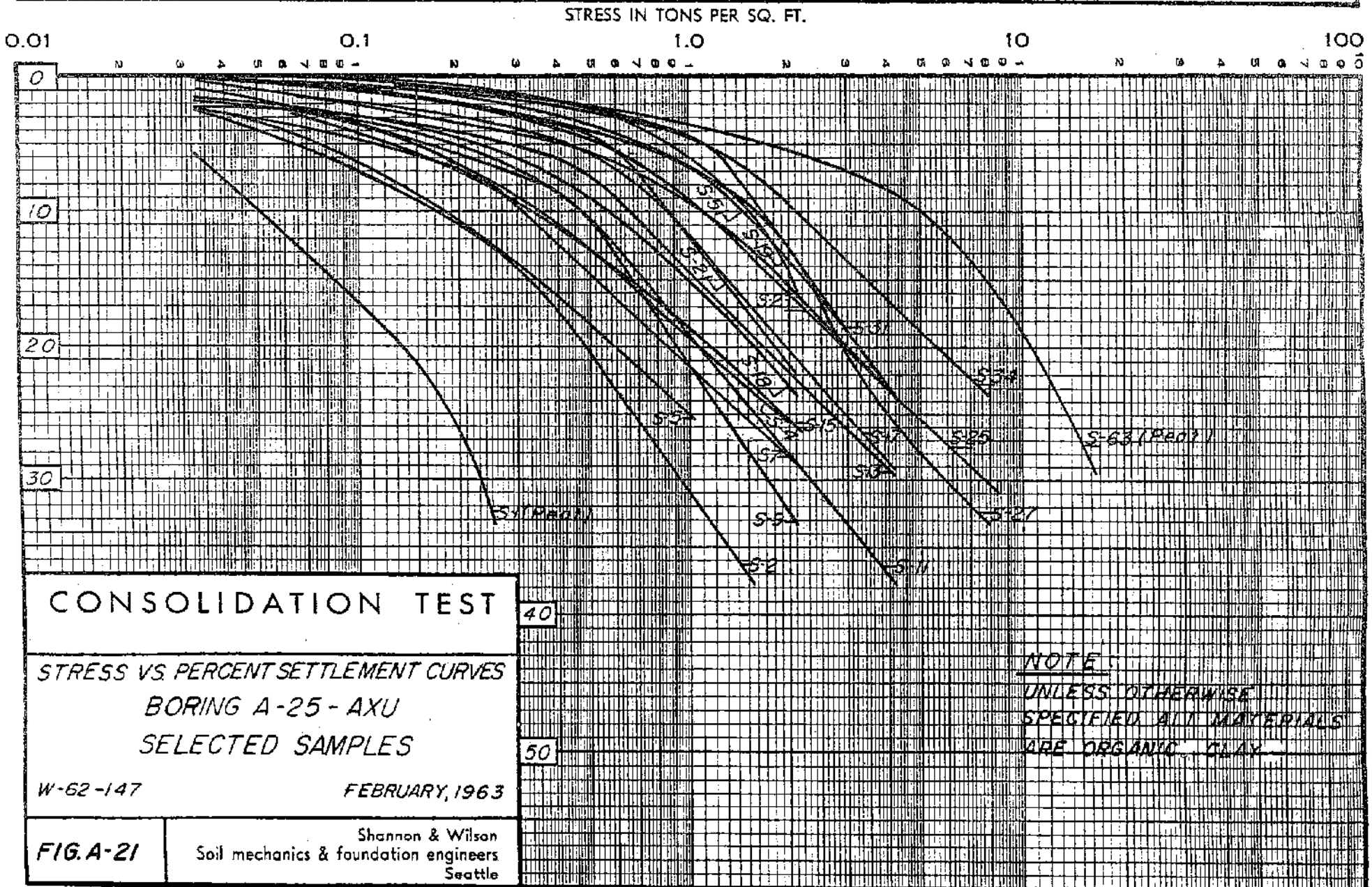
BORING UP-15-CU
SAMPLE S-1

W-62-147 FEBRUARY, 1963

FIG. A-20

Shannon & Wilson
Soil mechanics & foundation engineers
Seattle

BORING	SAMPLE	DEPTH	W. C. BEFORE	W. C. AFTER	HEIGHT	DIAMETER	CLASSIFICATION
A-25-AXU	AS NOTED						



CONSOLIDATION TEST

STRESS VS. PERCENT SETTLEMENT CURVES
BORING A-25-AXU
SELECTED SAMPLES

W-62-147 FEBRUARY, 1963

FIG. A-21

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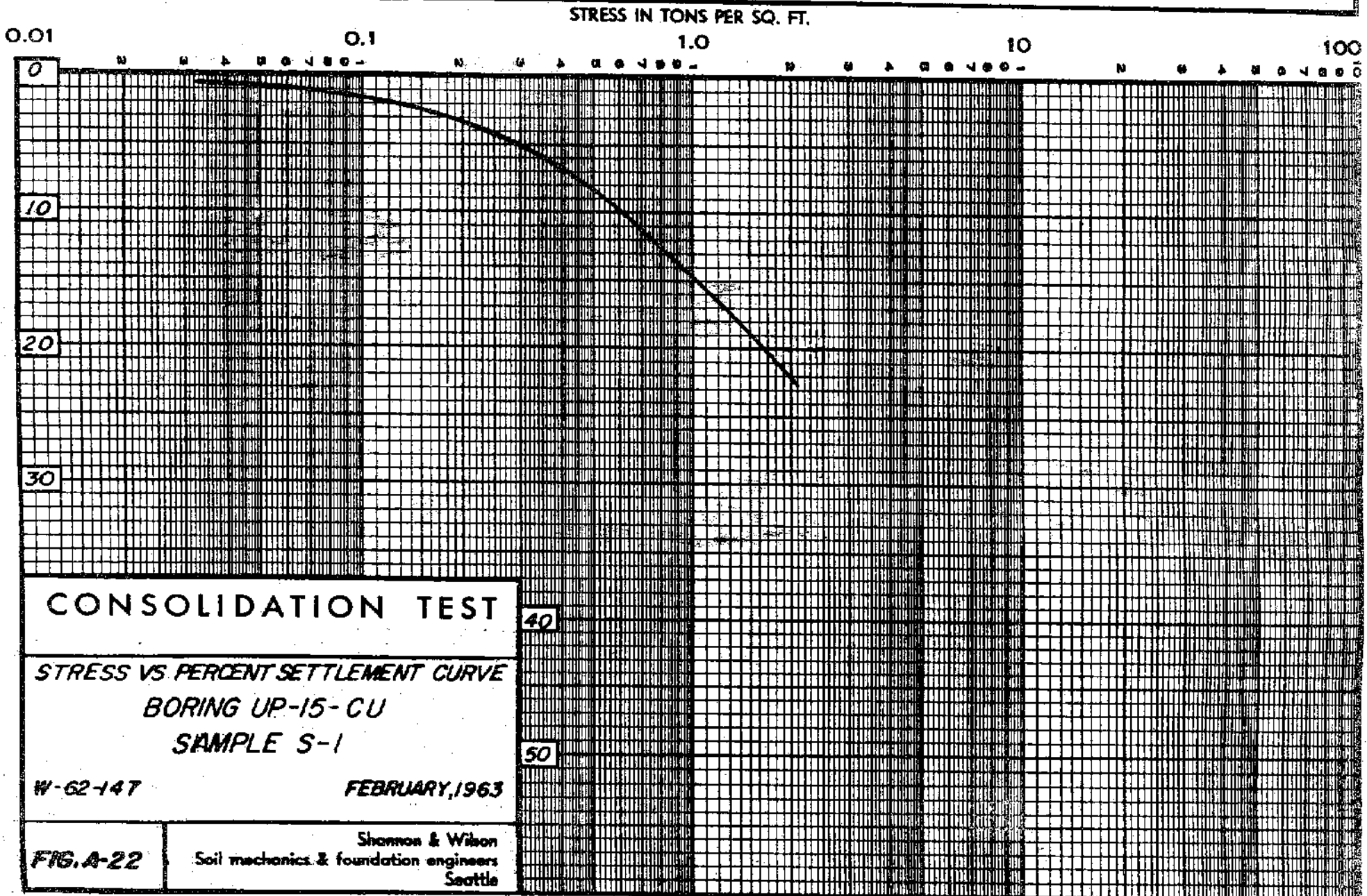
BORING	SAMPLE	DEPTH	W.C. BEFORE	W.C. AFTER	HEIGHT	DIAMETER	CLASSIFICATION
UP-15-CU	S-1	90'-92.5'			1.18cm	2.75in.	Very soft purple-gray organic CLAY



CONSOLIDATION TEST

STRESS VS PERCENT SETTLEMENT CURVE

BORING UP-15-CU

SAMPLE S-1

W-62-147

FEBRUARY, 1963

FIG. A-22

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permeability. Sample expansion and possible inclusion of gas in the pore space could occur as the in-situ stresses on the sample were removed by the sampling operation. It was believed worthwhile, therefore, to investigate in the triaxial test apparatus, the values of C_v which would be obtained if the sample was returned to simulated in-situ conditions and then permitted to drain under an increment of load obtained by increasing the cell pressure. Any gas in the pores of the specimen would in this manner be absorbed in the pore water under the applied pressures.

Clay specimens from sample S-1, drill hole UP-15-CU were prepared; one specimen was prepared for a consolidation test and two specimens were prepared for the triaxial dissipation tests. The specimen heights for the dissipation tests were 3.5 inches and 1 inch, with both specimens 1.4 inch in diameter. Specimens were prepared without side-drains and placed in the triaxial apparatus. The specimens were then subjected to a confining pressure, without sample drainage, equal to the approximate in-situ total pressure at sample depth. The pore water pressure in the sample was determined after conditions stabilized. The cell pressure was then increased by a pre-determined amount, $\Delta\sigma_3$, and the sample allowed to drain. Sample volume change and pore pressure measurements were made from the top and the bottom of the sample, respectively. The pore pressures were determined using a differential gage which indicated the difference in pressure between the pore pressure in the sample and a constantly maintained back-pressure. The back pressure was equal in all cases to the pore pressure existing in the sample prior to the increase in cell pressure, $\Delta\sigma_3$. Values of C_v were determined from a plot of volume change, ΔV, versus square root of time, t.

In order to compare values of C_v obtained from the above dissipation tests with those obtained from consolidation tests, one consolidation test was conducted.

The results of this test series are summarized in Table A-4.

The values of the coefficient of consolidation, C_v, calculated from the triaxial dissipation data are somewhat greater than those obtained from the consolidation test data. The variation in values of C_v, however, are not significantly great to warrant further study. Consequently, it is reasonable to

assume that the values of C_v calculated from consolidation test data are of the right order of magnitude for use in rate of consolidation studies.

TABLE A-4: COMPARISON OF COEFFICIENT OF CONSOLIDATION, C_v, FROM TRIAXIAL DISSIPATION TESTS AND CONSOLIDATION TEST DATA

Sample	Description	Specimen Height, inches	Coefficient of Consolidation, C_v, cm^2/min	
			Triaxial Dissipation Test	Consolidation Test
S-1, UP-15-CU, 90.0-92.5	Very soft purple-gray CLAY	1	0.026[1]	
		3.5	0.026[2]	
		0.5		0.012[3]
				0.009[4]

NOTES

1. Consolidation pressure, σ_c = 1600 psf.
2. Consolidation pressure, σ_c = 720 psf.
3. Load increment, Δp = 1000 psf.
4. Load increment, Δp = 2000 psf.

A-18. FIELD LABORATORY.

The field laboratory established in Gatun, Panama Canal Zone, was responsible for the visual classification of all samples obtained from the classification borings, for the performance of classification tests on these samples, and for determining the unconfined compressive strength of representative samples. The results of field laboratory testing are summarized in Table A-5 and presented on Figures A-23 through A-42.

A-19. VISUAL CLASSIFICATION.

Each Shelby tube sample was carefully extruded, using a mechanical, hand-operated sample extruder. Total recovery and sample quality were

recorded. A representative zone of sufficient length and consistency to permit testing was selected from the sample and trimmed for an unconfined compression test. The remaining sample was then visually classified in detail. A representative portion of the sample was selected for Atterberg Limit determinations and stored in a sealed container. Portions of the sample were selected for water content determinations, placed in tarred containers and immediately weighed.

A-20. CLASSIFICATION TESTS.

A-21. Natural Water Contents. Water contents of selected portions of each classification boring sample were determined using conventional techniques. All samples were dried in an oven operating at a temperature of 105 degrees Centigrade.

A-22. Atterberg Limit Tests. Liquid limit determinations were made in the field in accordance with the procedures given in paragraph A-5, with the exception that, due to lack of oven space, only one water content determination was made for each test. Accuracy was assured, however, by requiring that the same blow count be obtained three times before a moisture content sample was taken. Plastic limits were determined in accordance with the procedures given in paragraph A-5.

A-23. Unit Weight Determinations. Unit weights were determined for all unconfined test specimens using the same method used in the Seattle Laboratory (refer to paragraph A-6).

A-24. UNCONFINED COMPRESSION TESTS.

Unconfined compression tests were performed in a strain-controlled apparatus using the same techniques outlined in paragraph A-9.

TABLE A-5 SUMMARY OF FIELD LABORATORY TEST DATA

Boring No.	Sample No.	Depth, feet	Standard Penetration Resistance, blows/ft.	Natural Water Content, percent	Atterberg Limits LL	PL	PI	Shear Strength psf (1/2 qu)	Wet Density pcf	Sample Classification
A-5-AXC	S-1	49.0-51.5	Hydraulic Push	347.0 72.1	125	47	78			Top Nearly liquid dark brown slightly clayey decayed PEAT. Bottom Very soft dark blue and brown organic silty CLAY.
	S-2	52.8-55.3	"	62.4				360	101.0	Stiff to hard reddish brown with light blue streaks organic silty CLAY with occasional soft siltstone gravel to 1-1/2 in. dia.
	S-3	56.5-59.0	"	80.6	128	49	79	502	95.4	Medium stiff to very stiff red with light blue streaks organic silty CLAY with soft siltstone fragments to 1-1/4 in. dia.
	S-4	61.5-64.0	"	72.6	130	56	74	180	95.5	Very stiff light brown to red silty CLAY with soft siltstone fragments to 1-1/4 in. dia.
A-10-AXC	S-1	77.5-80.0	"							Top Nearly liquid dark gray to black silty clayey decayed PEAT.
				90.3 61.4	120	50	70	154	90.5	Middle Very soft light blue organic sensitive silty CLAY. Bottom Soft to medium organic silty CLAY.
	S-2	83.2-85.7	"	54.0	72	33	39	245	101.2	Same as bottom of S-1 with small pockets of fine to medium sand.
	S-3	88.5-91.0	"	54.7	57	36	21	339	99.2	Soft to medium dark gray organic very fine sandy silty CLAY with occasional pockets of peat.
	S-4	93.5-96.0	"	195.3 267.0 361.0				522	73.6	Top Medium dark gray-brown to brown slightly sensitive peaty silty CLAY. Middle Medium dark brown clayey fibrous PEAT. Bottom Well compacted dark brown fibrous PEAT.
	S-5	98.5-101.0	"	269.5				634	68.1	Top Same as bottom of S-4. Bottom Soft dark gray & green organic sensitive fine sandy silty CLAY with some leaves & grass roots.
	S-6	103.5-106.0	"	127.8 51.2						Top Soft dark blue organic silty CLAY with some siltstone fragments to 1 in. dia. and leaves & grass roots. Bottom Moderately hard light to dark blue weathered SILTSTONE.
	S-7	107.0-109.5	"	81.7						Same as bottom of S-5 with leaves and grass roots throughout.
	S-8	112.3-114.8	"	78.9						Same as bottom of S-5 except no organic material.
A-15-AXC	S-1	77.8-80.3	Hydraulic Push	501						Very soft dark brown fibrous PEAT.
	S-2	84.0-86.5	"	77.8	123	41	82	255	90.6	Soft light blue, dark brown & purple organic sensitive silty CLAY.
	S-3	88.4-90.9	"	79.4	95	37	58	293	97.0	Top Soft light gray & purple to dark gray organic sensitive silty CLAY with some fine lenses of leaves and peat. Bottom Same as top except without leaves and peat.
	S-4	94.4-96.6	"	63.3	70	33	37	168	96.0	Very soft dark gray to light blue organic sensitive silty CLAY with fine lenses of very fine sandy silty clay and with occasional sandstone fragments to 1/2 in. dia.

TABLE A-5 SUMMARY OF FIELD LABORATORY TEST DATA

Boring No.	Sample No.	Depth, feet	Standard Penetration Resistance, blows/ft.	Natural Water Content, percent	Atterberg Limits LL	PL	PI	Shear Strength psf (1/2 qu)	Wet Density pcf	Sample Classification
A-15-AXC	S-5	99.5-102.1	Hydraulic Push	62.7 587	66	35	31	245	95.1	Top Same as S-4. Bottom Well compacted dark brown fibrous PEAT.
	S-6	104.5-107.1	"	306				720	73.1	Moderately well compacted dark brown silty clayey fibrous PEAT.
	S-7	109.8-112.3	"	215				531	78.5	Well compacted dark brown and gray clayey fibrous PEAT with peaty silty CLAY layers 1/8 in. thick.
	S-8	114.8-117.3	"	267 191				791	59.6	Top Well compacted dark brown slightly silty fibrous PEAT. Bottom Medium dark brown to black peaty silty CLAY.
	S-9	118.5-121.0	"	115	134	45	89	632	86.7	Medium dark gray to brown sensitive peaty silty CLAY.
	S-10	123.5-126.0	"	113	140	53	87	535	84.2	Same as S-9.
	S-11	128.7-131.2	"	116	142	55	87	443	85.0	Same as S-9 except medium and slightly sensitive.
	S-12	133.7-136.2	"	121	182	52	130	481	90.6	Same as S-11.
	S-13	138.9-141.4	"	33.0	91	20	71	950	119.0	Stiff blue, brown, yellow and red mottled slightly sandy silty CLAY with occasional leaves. Slickensides in blue clay.
	S-14	143.0-145.5	"	31.9 29.9				1070	120.0	Top Stiff multicolored silty CLAY with dense very fine SAND layers and lenses 1/8 in. thick. Bottom Dense red with blue, yellow, and pink clayey fine to coarse SAND with some gravel to 3/4 in. dia.
	S-15	150.5-153.0	"	70.5						Hard multicolored weathered highly fractured SILTSTONE.
A-20-AXC	S-1	79.0-81.5	"	442						Nearly liquid dark brown decayed PEAT
	S-2	84.0-86.5	"	82.3	60	35	25	84	92.9	Very soft light blue to dark gray organic sensitive silty CLAY.
	S-3	89.0-91.5	"	633	53	35	18			Same as S-2 with small pockets of peat.
	S-4	93.1-95.6	"	69.3	83	36	47	92	95.1	Very soft light blue organic silty CLAY.
	S-5	98.3-100.8	"	70.4	77	40	34	252	93.0	Soft dark gray organic sensitive silty CLAY with leaves and occasional sandstone nodules to 1 in. dia.
	S-6	103.3-105.8	"	134 130 272	160	66	94	356	74.0	Top Soft dark gray to black organic sensitive silty CLAY. Middle Soft dark gray to brown sensitive peaty silty CLAY. Bottom Well compacted dark brown clayey fibrous PEAT.
	S-7	108.0-110.5	"	258				420	59.0	Same as bottom of S-6
	S-8	113.0-115.5	"	252				560	72.4	Same as bottom of S-6.
	S-9	118.2-120.7	"	140 105	127	51	76	876	73.1	Top Stiff light gray peaty CLAY and clayey PEAT, layers 1/4 in. thick. Bottom Medium dark gray organic sensitive silty CLAY.

TABLE A-5 SUMMARY OF FIELD LABORATORY TEST DATA

Boring No.	Sample No.	Depth, feet	Standard Penetration Resistance, blows/ft.	Natural Water Content, percent	Atterberg Limits LL	PL	PI	Shear Strength psf (1/2 qu)	Wet Density pcf	Sample Classification
A-20-AXC	S-10	123.2-125.7	Hydraulic Push	104	124	57	67	523	84.3	Soft to medium dark gray and light gray organic silty CLAY.
	S-11	128.5-131.0	"	89.8	115	45	70	325	88.0	Same as S-10
	S-12	133.5-136.0	"	69.0	68	42	26	322	94.2	Same as S-10 with siltstone gravel to 1-1/2 in. dia.
	S-13	138.5-141.1	"	82.7	96	45	51	695	91.4	Same as S-12 with small fragments of peat.
	S-14	143.6-146.1	"	84.0	101	43	58	462	93.5	Same as S-12.
	S-15	148.6-151.3	"	100	103	42	61			Medium to stiff dark gray organic sensitive very fine sandy silty CLAY and stiff dark brown clayey PEAT layers 1/8 in. thick.
	S-16	153.8-156.3	"	71.8	91	42	49			Medium dark gray organic sensitive silty CLAY with soft siltstone nodules to 1/2 in. dia. Bottom 1/2 in. of sample is very fine sandy PEAT.
	S-17	158.5-161.0	"	Clay=92.1 Peat=197	155	48	57		85.3	Medium to stiff dark gray organic silty CLAY with layers 1/4 in. thick of well compacted fibrous PEAT.
	S-18	163.5-166.0	"	128 79.0	115	41	74		91.5	Top Same as S-17. Bottom Medium dark grey organic silty CLAY with occasional pockets of blue weathered siltstone.
	S-19	168.7-171.2	"	76.1	116	43	73			Same as S-18 except medium to stiff with occasional shells and sandstone nodules to 1-1/2 in. dia.
	S-20	173.7-176.2	"	88.7	117	48	69			Medium stiff dark grey peaty silty CLAY.
	S-21	179.0-181.5	"	83.5	122	46	76			Same as S-20
	S-22	184.0-186.5	"	83.6	129	54	75			Same as S-20
	S-23	189.2-191.7	"	78.6	111	46	65			Same as S-20
	S-24	194.2-196.7	"	78.7	140	57	83			Top Same as S-20 with fragments of light blue weathered siltstone. Bottom Light blue weathered SILTSTONE.
	S-25	199.0-201.5	"	48.6						Blue weathered SILTSTONE.
A-30-AXC	S-1	78.7-81.2	"	379						Very soft dark brown fibrous PEAT.
	S-2	83.3-85.8	"	555 142.6						Top Same as S-1 except slightly clayey. Bottom Very soft light blue organic silty CLAY.
	S-3	88.0-90.5	"	54.1						Top Very soft dark gray silty fine SAND with large amounts of organic material and occasional hard siltstone nodules to 1 in. dia.
				79.6	77	39	38			Bottom Very soft dark gray organic fine sandy silty CLAY.
	S-4	93.0-95.5	"	130	175	45	130	158	82.4	Very soft dark grey to light blue organic sensitive silty CLAY with occasional black organic spots.

TABLE A-5 SUMMARY OF FIELD LABORATORY TEST DATA

Boring No.	Sample No.	Depth, feet	Standard Penetration Resistance, blows/ft.	Natural Water Content, percent	Atterberg Limits LL	PL	PI	Shear Strength psf (1/2 qu)	Wet Density pcf	Sample Classification
A-30-AXC	S-5	98.2-100.7	Hydraulic Push	126 103	97	68	29	118	78.4	Top Same as S-4 Bottom Very soft dark gray peaty silty CLAY.
	S-6	103.0-105.5	"	184	225	71	154	273	75.6	Layers of medium stiff clayey fibrous PEAT and soft peaty silty CLAY 1/2 in. thick.
	S-7	108.2-110.7	"	281				622	67.6	Moderately well compacted dark brown clayey fibrous PEAT.
	S-8	113.4-115.9	"	121	74	56	18	402	75.4	Soft dark gray sensitive peaty CLAY.
	S-9	118.1-120.6	"	265	O					Soft dark brown fibrous PEAT.
	S-10	123.1-125.6	"	155	180	74	106	360	74.4	Soft dark gray peaty silty CLAY.
	S-11	128.3-130.8	"	166	165	52	113	438	82.5	Same as S-10
	S-12	133.3-135.8	"	112	135	56	79	473	85.3	Same as S-10
	S-13	138.5-141.0	"	105	128	47	81	301	87.3	Same as S-10
	S-14	143.5-146.0	"	104						Same as S-10
	S-15	148.7-151.2	"	No Recovery						
	S-16	151.7-154.2	"	110	130	53	77			Same as S-10
	S-17	156.7-159.2	"	111						Soft to medium sensitive peaty silty CLAY with occasional broken shells.
	S-18	161.9-164.4	"	97.6	132	55	77			Same as S-17
	S-19	165.9-169.4	"	101						Medium stiff dark gray peaty silty CLAY.
	S-20	172.1-174.6	"	85.6	118	47	71			Same as S-19.
	S-21	177.1-179.6	"	87.7						Same as S-19 with layers of soft siltstone nodules.
	S-22	182.3-184.8	"	85.2	125	45	80			Same as S-19 with 1/2 in. layers of light gray silty clay.
	S-23	187.3-189.8	"	76.2						Same as S-22
	S-24	192.5-195.0	"	74.1						Same as S-22
	S-25	197.5-200.0	"	74.8	107	44	63			Same as S-22
	S-26	202.7-205.2	"	75.5						Same as S-22
	S-27	207.6-210.1	"	47.2	62	36	26			Very stiff light gray clayey SILT with some very fine sand and occasional soft siltstone nodules.
	S-28	212.3-214.8	"	42.8						Stiff light to dark blue, purple and green clayey SILT.
	S-29	216.0-217.5	32	48.3						Soft weathered rust-colored SILTSTONE.

TABLE A-5 SUMMARY OF FIELD LABORATORY TEST DATA

Boring No.	Sample No.	Depth, feet	Standard Penetration Resistance, blows/ft.	Natural Water Content, percent	Atterberg Limits LL	PL	PI	Shear Strength psf (1/2 qu)	Wet Density pcf	Sample Classification
A-35-AXC	S-1	77.0-79.5	Hydraulic Push	229 89.3	128	41	87			Top Nearly liquid dark gray decayed PEAT. Bottom Very soft light blue silty CLAY.
	S-2	80.0-82.5	"	68.2	137	45	92	378	96.9	Medium stiff light blue and dark brown silty CLAY.
	S-3	85.0-87.5	"	59.9 62.1	88	33	55	360	101.9	Top Same as S-2 except medium stiff to stiff. Bottom Soft light blue and dark gray organic sensitive silty CLAY with numerous grass roots.
	S-4	90.0-92.5	"	50.2	67	35	32	200	97.6	Same as bottom of S-3
	S-5	95.0-97.5	"	55.9	66	30	36	331	104	Soft light blue to purple organic sensitive silty CLAY with occasional small pockets of black organic material.
	S-6	100.0-102.5	"	70.9	102	56	46	311	98.4	Soft dark brown to light blue organic sensitive silty CLAY with occasional siltstone nodules to 1-1/2 in. dia. and thin organic lenses 1/8 in. thick 4 in. apart.
	S-7	105.0-107.5	"	69.6	80	22	58	476	94.1	Soft to medium light to dark gray organic sensitive silty CLAY with occasional very fine sandy silty clay lenses 1/16 in. thick.
	S-8	110.0-112.5	"	45.9						Dark gray silty fairly clean SAND with two layers of soft to medium dark gray very fine sandy silty clay 0.2 feet thick and one layer of dark brown and gray sandy peat 0.2 feet thick.
	S-9	115.0-117.5	"	Peat = 99.4 Sand = 50.9						Loose dark gray silty fairly clean SAND with a 0.8 foot layer of soft dark gray very fine sandy silty clayey peat
	S-10	121.5-124.0	"	32.4 107	160	49	111	599	85.5	Top Loose dark gray silty fine to medium SAND (well graded). Bottom Medium dark gray organic sensitive silty CLAY.
	S-11	125.0-127.5	"	107	170	56	114	790	86.2	Medium to stiff dark brown organic silty CLAY with thin layers of black organic material.
	S-12	130.0-132.5	"	96.1	137	61	76	1040	87.5	Same as S-11 except stiff.
	S-13	135.0-137.5	"	104.5	110	50	60	462	88.2	Medium stiff dark gray peaty silty CLAY.
	S-14	140.0-142.5	"	118	91	57	34			Stiff to hard dark brown peaty silty CLAY.
	S-15	145.0-147.5	"	83.9	123	43	80			Stiff dark gray peaty silty CLAY.
	S-16	150.0-152.5	"	87.1	125	54	71			Same as S-15 with occasional shells.
	S-17	155.0-157.5	"	81.9	127	43	84			Medium to stiff dark grey peaty silty CLAY.
	S-18	160.0-162.5	"	83.4	61	46	15			Same as S-17 with occasional shells.
	S-19	165.0-167.5	"	78.5	82	43	39			Same as S-17 except medium.
	S-20	170.0-172.5	"	81.2	116	39	77			Stiff dark gray peaty silty CLAY.
	S-21	175.0-177.5	"	86.8	120	41	79			Same as S-20 with one 2 in. pocket of light blue medium stiff peaty silty clay.
	S-22	180.0-182.5	"	42.5	120	43	77			Stiff to very stiff light blue and brown silty CLAY with occasional small pockets of silty clayey fine sand. Clay shows slickensided surfaces.

TABLE A-5 SUMMARY OF FIELD LABORATORY TEST DATA

Boring No.	Sample No.	Depth, feet	Standard Penetration Resistance, blows/ft.	Natural Water Content, percent	Atterberg Limits LL	PL	PI	Shear Strength psf (1/2 qu)	Wet Density pcf	Sample Classification
A-35-AXC	S-23	185.0-187.5	Hydraulic Push	44.8	75	31	44			Stiff to hard blue to dark gray very fine sandy silty CLAY with occasional hard siltstone fragments 1/2 in. dia.
A-40-AXC	S-1	77.0-79.5	No Recovery							
	S-2	80.0-82.5	Hydraulic Push	94.0	121	47	74	138	92.0	Very soft to soft light blue to dark green organic sensitive silty CLAY.
	S-3	85.0-87.5	"	64.0	69	33	36	173	97.4	Same as S-2 except light blue-green with some grass roots.
	S-4	90.0-92.5	"	85.0	96	37	59	145	88.0	Very soft light blue organic sensitive silty CLAY with a layer of siltstone nodules to 1/4 in. dia.
	S-5	95.0-97.5	"	100	122	39	83	170	84.4	Same as S-4 with layers and pockets of soft silty clayey peat.
	S-6	100.0-102.5	"	167 138				290	79.8	Top Soft dark gray and brown organic sensitive silty CLAY with layers of medium stiff silty clayey PEAT 1/4 in. thick. Bottom Soft dark gray organic sensitive silty CLAY with grass roots and leaves.
	S-7	105.0-107.5	"	200 353				734	66.9	Top Soft to medium dark gray and brown layers and lenses of organic sensitive silty CLAY and fibrous PEAT. Bottom Fairly firm dark brown fibrous PEAT.
	S-8	110.0-112.5	"	454				868	64.3	Well compacted dark brown fibrous PEAT and WOOD.
	S-9	115.0-117.5	"	136	140	40	100	378	85.5	Soft dark gray to brown organic sensitive silty CLAY with frequent layers of fibrous peat and wood.
	S-10	120.0-122.5	"	98.1	154	42	112	393	92.0	Same as S-9 except with occasional layers of peat.
	S-11	125.0-127.5	"	104	138	43	95	553	87.3	Same as S-10 except medium with occasional shells.
	S-12	130.0-132.5	"	120	160	51	109	648	85.4	Top Fairly firm black-brown clayey PEAT. Bottom Medium dark gray to brown organic sensitive silty CLAY with occasional lenses of leaves, peat and wood.
	S-13	135.0-137.5	"	121	148	51	97	667	85.0	Same as bottom of S-12
	S-14	140.0-142.5	"	85.3	124	44	80	594	91.7	Medium dark gray to brown organic sensitive silty CLAY with numerous small pockets of peat and leaves.
	S-15	144.0-146.5	"	46.9	73	36	37	793	101.6	Stiff to very stiff dark blue to brown very fine sandy silty CLAY with a 0.2 ft. layer of soft silty clay.
	S-16	147.0-156.8	NX core	26.7						Hard brown-yellow very fine sandy SILTSTONE.
A-45-AXC	S-1	76.0-78.2	Hydraulic Push	66.4	89	41	48			Soft light blue to dark brown organic sensitive silty CLAY with numerous grass roots.
	S-2	80.0-82.5	"	63.0	96	41	55	238	94.6	Soft dark red-brown and pink organic sensitive silty CLAY.
	S-3	85.0-87.5	"	66.4				529	96.8	Soft to medium dark brown to light blue sensitive silty CLAY (Note: One foot of water trapped between top and bottom of sample.)

TABLE A-5 SUMMARY OF FIELD LABORATORY TEST DATA

Boring No	Sample No.	Depth, feet	Standard Penetration Resistance, blows/ft.	Natural Water Content, percent	Atterberg Limits LL	PL	PI	Shear Strength psf (1/2 qu)	Wet Density pcf	Sample Classification
A-45-AXC	S-4	90.0-92.5	Hydraulic Push	264 100	140	51	89	475	59.4	Top Medium stiff dark brown clayey PEAT. Bottom Soft dark gray organic silty CLAY.
	S-5	95.0-97.5	"	442 143				229	83.4	Top Fairly compact dark brown fibrous PEAT. Bottom Medium dark gray peaty silty CLAY with occasional wood fragments.
	S-6	100.0-102.5	"	81.9	94	48	46	276	79.7	Soft light blue and brown organic sensitive fine sandy silty CLAY with numerous grass roots.
	S-7	105.0-107.5	"	70.7				354	90.5	Stiff light blue and orange slightly silty CLAY.
	S-8	110.0-112.5	"	65.7				513	97.3	Medium dense black, orange and green clayey silty medium to coarse SAND with well-rounded weathered cobbles to 2-1/2 in. dia. (weathered conglomerate)
A-50-AXC	S-1	8.5-10.5	"	71.5 31.4						Top Watery dark brown leaves and wood. Bottom Very stiff dark gray coarse sandy silty CLAY with weathered well-rounded gravel to 1 in. dia. (weathered conglomerate).
	S-2	12.3-13.3	"	41.2	86	39	47			Same as bottom of S-1.
	S-3	20.0-24.7	"	44.8	100	36	64			Very stiff orange-brown with red and purple laminations fine to coarse sandy silty CLAY with well-rounded cobbles to 3 in. dia. (weathered conglomerate).
	S-4	26.5-30.4	"	50.3	113	39	74	2340	106.1	Very stiff light gray with red laminations fine to medium sandy silty CLAY (weathered sandstone).
	S-5	47.5-48.5	"	30.7				1340	101.8	Dense yellow-orange clayey fine to coarse SAND with well-rounded gravel to 1 in. dia. (weathered conglomerate).
A-55-AXC	S-1	43.0-45.5	Hydraulic Push	41.7 42.3	78	31	47	594	111.0	Top Stiff to hard dark gray to brown silty CLAY with numerous organic fragments. Bottom Hard red silty CLAY with weathered well-rounded cobbles to 2-1/2 in. dia. (weathered conglomerate).
	S-2	45.5-48.0	"	47.5	84	47	37	1412	108.5	Very stiff red, blue and purple slightly sandy silty CLAY (weathered sandstone).
	S-3	50.9-53.4	"	52.9	126	55	71			Very stiff orange, yellow and red fine to coarse sandy silty CLAY with occasional sub-rounded cobbles to 1-1/2 in. dia.
	S-4	52.0-52.0	"	43.3	91	33	58			Very stiff light gray, purple and orange fine to coarse sandy CLAY with occasional cobbles to 3 in. dia. (sedimentary tuff).
DN-25-DC	S-1	75.0-81.5	Hydraulic Push	540 130				104	85.0	Top Very soft dark brown fibrous PEAT Bottom Very soft light gray to light brown very organic CLAY with occasional pieces of organic material.
	S-2	85.0-87.5	"	66.1	80	34	46			Very soft gray organic CLAY with occasional organic fragments.
	S-3	90.0-92.5	"	104	134	34	100	137	84.4	Very soft light to dark gray and brown organic sensitive silty CLAY with pockets of peat and wood fragments.
	S-4	95.0-97.5	"	146				184	80.5	Same as S-3

TABLE A-5 SUMMARY OF FIELD LABORATORY TEST DATA

Boring No	Sample No.	Depth, feet	Standard Penetration Resistance, blows/ft.	Natural Water Content, percent	Atterberg Limits LL	PL	PI	Shear Strength psf (1/2 qu)	Wet Density pcf	Sample Classification
DN-25-DC	S-5	100.0-102.5	Hydraulic Push	159				371	74.5	Alternating layers 1/8 in. thick of soft dark gray to brown organic silty CLAY and firm fibrous PEAT.
	S-6	105.0-107.5	"	356				508	66.5	Very stiff well-compacted dark brown fibrous PEAT.
	S-7	110.0-112.5	"	101 293	123	47	76	424	81.5	Top Soft dark gray to brown organic silty CLAY. Bottom Stiff dark brown fibrous PEAT and soft organic silty CLAY layers 1/2 to 1-1/2 in. thick.
	S-8	115.0-117.5	"	Peat=198 Clay=103	124	48	76	472	81.5	Same as bottom of S-7
	S-9	120.0-122.5	"	79.6	95	36	59	346	95.7	Soft dark gray to brown organic sensitive silty CLAY with numerous broken shells and occasional pieces of wood.
	S-10	125.0-127.5	"	89.6	116	39	77	454	83.5	Soft dark gray organic sensitive silty CLAY with thin lenses of clayey peat and soft siltstone nodules to 1 in. dia.
	S-11	130.0-132.5	"	90.7	119	35	84	215	91.0	Soft dark gray organic sensitive silty CLAY with numerous shells and siltstone nodules to 3/4 in. dia.
	S-12	135.0-137.5	"	88.2	112	36	76	619	94.5	Soft to medium dark gray organic silty CLAY with fibrous peat lenses.
	S-13	140.0-142.5	"	73.3				168	114	Soft light gray to blue organic sensitive fine to coarse sandy silty CLAY with occasional gravel to 3/4 in. dia.
	S-14	145.0-147.5	"	57.5	105	39	66			Top Same as S-13 except with cobbles to 2 in. dia. Bottom Medium red and blue coarse to fine sandy silty gravelly (to 1-1/2 in. dia.) CLAY (weathered conglomerate).
	S-15	150.0-152.5	"	47.0	173	42	31	197	109.5	Medium to stiff blue, yellow and gray coarse to fine sandy silty CLAY with some well-rounded gravel to 1/2 in. dia. (weathered conglomerate).
DN-25-CC	S-1	80.0-82.5	Hydraulic Push	565 182						Top Soft brown fibrous PEAT with occasional wood fragments. Bottom Very soft gray peaty silty CLAY.
	S-2	85.0-87.5	"	212 108	138	41	97	50	86.6	Top Very soft dark gray to brown clayey PEAT. Bottom Very soft light gray organic silty CLAY.
	S-3	90.0-92.5	"	135	150	42	108	85	78.1	Very soft light to dark gray and brown organic sensitive silty CLAY with lenses and pockets of organic material and occasional siltstone nodules to 3/4 in. dia.
	S-4	95.0-97.5	"	264 134				231	69.9	Top Soft to medium dark brown silty clayey PEAT Bottom Same as top except with layers of soft gray-brown organic silty clay.
	S-5	100.0-102.5	"	277				137	68.4	Firm gray and brown fibrous PEAT with layers of soft to medium silty clayey peat.
	S-6	105.0-107.5	"	285				474	59.9	Same as S-5
	S-7	110.0-112.5	"	208	250	79	171	416	70.4	Same as S-5 (Limits performed on clayey peat).
	S-8	115.0-117.5	"	172 123	130	43	87	284	81.9	Top Same as S-5 Bottom Soft dark gray organic sensitive silty CLAY with occasional layers of leaves and peat to 1/2 in. thick.

TABLE A-5 SUMMARY OF FIELD LABORATORY TEST DATA

Boring No.	Sample No.	Depth, feet	Standard Penetration Resistance, blows/ft.	Natural Water Content, percent	Atterberg Limits LL	PL	PI	Shear Strength psf (1/2 qu)	Wet Density pcf	Sample Classification
DN-25-CC	S-9	120.0-122.5	Hydraulic Push	98.4	125	43	82	270	83.3	Same as S-8 except peat layers to 1/16 in. thick and with occasional yellow silt lenses.
	S-10	125.0-127.5	"	99.2	120	50	70	341	86.5	Soft dark gray to brown organic very sensitive silty CLAY with layers of very fine silty sand to 1/16 in. thick and occasional siltstone nodules to 3/4 in. dia.
	S-11	130.0-132.5	"	77.9	90	39	51	338	82.7	Soft dark gray organic very sensitive silty CLAY with occasional siltstone nodules to 1 in. dia.
	S-12	135.0-137.5	"	75.6 80.8	85	39	46	332	96.1	Top Same as S-11 Bottom Alternating layers of soft gray-brown peaty CLAY, firm fibrous PEAT, and fine gray SAND about 1/2 inch thick.
	S-13	140.0-142.5	"	90.7 83.0	94	39	55	553	92.0	Top Same as bottom of S-12 Bottom Soft to medium gray organic silty CLAY with occasional layers of peat 1/8 in. thick and siltstone nodules to 2 in. dia.
	S-14	145.0-147.5	"	88.3	97	40	57	509	91.9	Medium dark gray organic sensitive slightly silty CLAY with thin lenses of fine silty sand spaced about every three inches and with occasional shells.
	S-15	150.0-152.5	"	99.1	115	42	73	645	90.7	Same as S-14 with alternating layers of fibrous peat 1/4 in. thick and fine silty sand 1/8 in. thick spaced about every 1-1/2 inches.
	S-16	155.0-157.5	"	78.8	92	39	53	511	93.6	Medium dark gray organic sensitive silty CLAY with layers of clayey fibrous peat 1/8 in. thick every 2 inches. Occasional hard siltstone nodules to 1 in. dia.
	S-17	160.0-162.5	"	91.1	109	39	70	650	93.9	Same as S-16 except with occasional lenses of peat instead of layers.
	S-18	165.0-167.5	"	Clay= 86.1 Peat= 158	111	41	70	730	90.0	Medium dark gray organic sensitive silty CLAY with layers of fibrous PEAT 1/2 in. thick every 1-1/2 inches.
	S-19	170.0-172.5	"	87.9	127	43	84	1058	92.4	Medium dark gray organic sensitive silty CLAY with occasional pockets of peat to one in. dia.
	S-20	175.0-177.5	"	84.5	122	41	81	642	91.7	Same as S-19 with wood fragments.
	S-21	180.0-182.5	"	78.5 49.7	97	38	59	514 729	97.0 107.0	Top Same as S-19. Bottom Medium stiff light blue and gray organic slightly sensitive CLAY with subangular fragments of hard welded tuff to 2 in. dia. at the extreme bottom of sample.
DN-25-BC	S-1	76.2-78.7	"	No Recovery						
	S-2	81.0-83.5	"	261					68.6	Very soft gray-brown silty clayey fibrous PEAT.
	S-3	85.0-88.5	"	78.3	78	35	43		92.4	Very soft dark gray to yellow-gray organic very sensitive silty CLAY with numerous peat fragments.
	S-4	91.0-93.5	"	131	159	64	95	175	85.5	Very soft dark gray to brown organic silty CLAY with spots of black organic material.
	S-5	96.0-98.5	"	221				194	76.1	Medium light gray to dark brown silty clayey fibrous PEAT with occasional wood fragments.
	S-6	101.0-103.5	"	284						Medium-stiff dark brown slightly silty fibrous PEAT with some wood fragments.

TABLE A-5 SUMMARY OF FIELD LABORATORY TEST DATA

Boring No.	Sample No.	Depth, feet	Standard Penetration Resistance, blows/ft	Natural Water Content, percent	Atterberg Limits LL	PL	PI	Shear Strength psf (1/2 qu)	Wet Density pcf	Sample Classification
DN-25-BC	S-7	105.0-108.5	Hydraulic Push	155						Very soft gray organic sensitive silty CLAY and silty clayey PEAT, interbedded-layers 1/2 in. thick.
	S-8	111.0-113.5	"	143	196	55	141			Same as S-7
	S-9	116.0-118.5	"	103	120	52	68			Soft dark brown-gray organic very sensitive silty CLAY with fine sand partings.
	S-10	121.0-123.5	"	115	128	55	73			Same as S-9 with yellow silty clay lenses.
	S-11	125.0-128.5	"	116						Soft brown-gray organic silty CLAY with layers of blue-gray silty fine sand and firm brown fibrous peat 1/2 in. thick.
	S-12	131.0-133.5	"	68.7	105	38	67			Soft brown-gray organic silty CLAY with fine sand and organic partings and hard yellow siltstone nodules to 1 in. dia.
	S-13	136.0-138.5	"	96.5	96	41	55			Same as S-12
	S-14	141.0-143.5	"	78.6	103	41	62			Same as S-12 with more siltstone nodules and with layers of yellow silty clay 1/4 in. thick.
	S-15	146.0-148.5	"	86.4	115	43	72			Same as S-14
	S-16	151.0-153.5	"	83.5	115	41	74			Same as S-14
	S-17	156.0-158.5	"	85.2	106	44	62	550		Same as S-14 except medium
	S-18	161.0-163.5	"	92.1	114	46	68			Medium dark gray organic slightly silty CLAY with fine sand partings and thin layers of firm brown fibrous peat.
	S-19	166.0-168.5	"	81.6	107	43	64	670	93.0	Medium dark to light brown organic silty CLAY with occasional fine sand partings.
	S-20	171.0-173.5	"	75.2	105	41	64			Same as S-19
	S-21	176.0-178.5	"	82.5 45.2	117	44	73	520	95.5	Top Same as S-19 except medium to medium-stiff and with abundant shell fragments at lower end. Bottom Medium stiff dark brown-gray organic silty CLAY with numerous angular fragments (2 in. dia.) of blue-green hard very fine sandy siltstone (weathered).
	S-22	181.7-182.6	"	34.5						Hard blue-green very fine sandy weathered SILTSTONE with red-brown stains throughout.
UP-25-BC	S-1	80.2-82.7	Hydraulic Push	928 377						Top Very soft brown slightly clayey fibrous PEAT. Bottom Soft gray-brown clayey fibrous PEAT.
	S-2	84.7-87.2	"	105	136	41	95	147	59.5	Very soft light gray organic CLAY with brown peat pockets.
	S-3	90.3-92.8	"	77.6	80	34	46	174	85.0	Very soft light gray organic sensitive CLAY with one 1/2 in. thick layer of black clayey fibrous peat.
	S-4	95.0-97.5	"	72.3	97	38	59			Very soft light gray organic CLAY with one 0.3 foot layer of light gray silty clayey very fine sand.

TABLE A-5 SUMMARY OF FIELD LABORATORY TEST DATA

Boring No.	Sample No.	Depth, feet	Standard Penetration Resistance, blows/ft.	Natural Water Content, percent	Atterberg Limits LL	PL	PI	Shear Strength psf (1/2 qu)	Wet Density pcf	Sample Classification
UP-25-BC	S-5	100.0-102.5	Hydraulic Push	80.5						Top Soft brown-gray organic silty CLAY layers 1/10 in. thick interbedded with layers and lenses 1/8 in. thick of light gray silty fine SAND and brown fibrous PEAT.
				77.1	79	39	40	127	94.5	Bottom Very soft brown-gray organic sensitive silty CLAY.
	S-6	105.0-107.5	"	112	124	50	74	296	84.0	Soft to medium light gray-brown organic silty CLAY with lenses of black and brown peat 1/8 in. thick.
	S-7	110.5-112.9	"	Clay= 155 Peat= 171				406	75.4	Layers 1/2 in. thick of soft light gray organic silty CLAY and medium stiff brown to black clayey PEAT.
	S-8	115.5-118.0	"	239				552	72.6	Medium to stiff dark brown fibrous PEAT with thin layers of organic silty clay.
	S-9	119.8-122.3	"	Clay= 101 Peat= 211				844	68.5	Same as S-8
	S-10	127.5-130.0	"	103	122	62	60	362	79.5	Soft gray-brown organic very sensitive silty CLAY with occasional layers of peat 1/16 in. thick.
	S-11	132.3-134.8	"	82.2	88	37	51	318	92.5	Soft gray organic sensitive silty CLAY with occasional siltstone nodules.
	S-12	137.1-139.6	"	No Recovery						
	S-13	141.0-143.5	"	83.0	103	39	64			Soft dark gray organic clayey SILT with very fine sandy silt partings and occasional hard siltstone layers to 1/4 in. thick.
	S-14	145.0-147.5	"	71.5	103	37	66			Soft dark gray organic sensitive silty CLAY with yellow clayey silt layers (1/4 in.), black organic partings and sandy silt partings about every 1/8 inch.
	S-15	150.1-152.6	"	83.6	131	37	94	521	81.5	Same as S-14 except medium with occasional hard yellow siltstone nodules to 1/2 in. dia.
	S-16	155.1-157.6	"	81.4	101	34	67			Same as S-15.
	S-17	160.3-162.8	"	73.2	98	43	55			Same as S-15
	S-18	165.3-167.8	"	75.5	103	44	59			Medium brown-gray organic silty CLAY with numerous brown organic partings and occasional yellow siltstone nodules to 1/2 in. dia.
	S-19	170.5-173.0	"	80.5	105	48	57			Same as S-18 with occasional peat layers 1/8 in. thick and silty fine sand partings.
	S-20	175.5-178.0	"	73.5	108	48	60			Same as S-19
	S-21	181.6-183.1	5	84.2	111	42	69			Same as S-19
	S-22	185.7-187.2	7	No Recovery						
	S-23	187.5-189.5	7	51.9	75	59	16			Medium brown-gray organic fine sandy silty CLAY with brown fibrous peat layers, silty sand lenses and hard yellow siltstone nodules to 1/2 in. dia.
	S-24	191.7-193.2	14	71.6						Loose brown-gray to blue-gray silty fine to medium SAND with occasional wood fragments to in. dia.

TABLE A-5 SUMMARY OF FIELD LABORATORY TEST DATA

Boring No.	Sample No.	Depth, feet	Standard Penetration Resistance, blows/ft.	Natural Water Content, percent	Atterberg Limits LL	PL	PI	Shear Strength psf (1/2qu)	Wet Density pcf	Sample Classification
UP-25-BC	S-25	197.3-199.3	9	56.8						Top Same as S-24 Bottom Medium dark gray-brown organic silty CLAY.
	S-26	205.0-207.0	30	51.2	113	54	59			Very stiff tan mottled blue fine sandy silty CLAY.
	S-27	207.0-208.5	89	36.9						Same as S-26 except with blue-gray hard claystone nodules to 1 in. dia.
	S-28	208.5-216.1	NX Core							Subangular nodules to 2 in. dia of hard blue-gray weathered gray-tan claystone and hard gray welded tuff in a blue-gray silty clay binder (weathered CLAYSTONE).
UP-25-CC	S-1	78.0-80.5	Hydraulic Push	595						Very soft brown silty clayey fibrous PEAT.
	S-2	83.0-85.5	"	147	250	56	194	69	80.5	Top 0.2 ft. Soft brown-gray organic silty clayey PEAT. Bottom Soft light gray-brown very organic silty CLAY becoming peaty with black sand partings at the bottom 0.5 ft. of sample.
	S-3	88.2-90.7	"	115.8	155	48	107			Very soft light gray organic silty CLAY with occasional black organic sandy pockets.
	S-4	93.2-95.7	"	89.4	113	39	74	105	94.5	Very soft light gray organic silty CLAY.
	S-5	98.4-100.9	"	112	122	47	75	208	84.8	Very soft dark gray organic silty CLAY with small pockets of black organic material and one hard yellow siltstone nodule 3 in. diameter.
	S-6	102.9-105.4	"	Peat= 187 Clay= 142				180	80.0	Very soft brown to gray clayey fibrous PEAT with occasional layers of soft sensitive organic silty clay 1/8 in. thick.
	S-7	105.7-108.2	"	171				472	73.0	Soft to medium brown-gray silty clayey fibrous PEAT.
	S-8	111.1-113.6	"	171				205	74.7	Alternating layers 1/8 in. thick of soft to medium light gray organic silty CLAY and medium stiff brown fibrous PEAT.
	S-9	116.3-118.8	"	258				644	71.1	Medium to stiff dark brown silty clayey fibrous PEAT.
	S-10	121.5-124.1	"	Peat= 277 Clay= 198				682	70.9	Same as S-9 with layers of soft silty clay 1/8 in. thick occuring about every 3/4 inch.
	S-11	126.5-129.0	"	119	117	44	73	142	84.2	Soft gray-brown organic sensitive silty CLAY with layers 1/2 to 1/8 in. thick of clayey fibrous peat occuring about every 2 inches.
	S-12	131.7-134.2	"	73.6	86	40	46	359	95.9	Soft light to dark gray organic very sensitive very fine sandy silty CLAY with lenses of very fine sandy silty clayey fibrous peat.
	S-13	170.0-139.5	"	82.0	89	43	46	292	90.0	Soft gray to brown organic sensitive silty CLAY with lenses of gray-blue very fine sandy silty clay 1/5 in. thick.
	S-14	141.7-144.2	"	81.2	91	43	48	454	91.7	Same as S-13 with lenses 1/8 in. thick of fibrous peat.
	S-15	146.5-149.0	"	80.7	115	44	72	367	92.5	Same as S-13.

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UP-25-CC	S-16	151.7-154.2	Hydraulic Push	87.6	110	43	67	603	89.9	Same as S-14 except medium.
	S-17	157.0-159.5	"	79.4	104	36	68	587	92.4	Top Medium dark gray to brown organic sensitive silty CLAY with layers 1/8 in. thick of very fine silty sand about every 1 inch. Bottom Medium dark gray to brown organic sensitive silty CLAY with layers 3/4 in. thick of light brown sandy clay.
	S-18	162.0-164.5	"	Clay=68.0 Peat=136	107	40	67	355	91.9	Alternating layers about 1 inch thick of soft organic very fine sandy silty CLAY and medium stiff fibrous PEAT.
	S-19	167.0-169.5	"	85.9	112	48	64	671	92.1	Medium dark gray-brown organic sensitive silty CLAY with occasional soft sandstone nodules to 1-1/2 in. diameter.
	S-20	172.0-174.5	"	82.4	108	40	68	444	91.0	Medium dark gray organic sensitive silty CLAY with occasional lenses of silty clayey fine sand 1/8 inch thick.
	S-21	177.2-181.0	"	No Recovery						
	S-22	181.5-184.0	"	69.2	87	39	48	377	96.4	Soft to medium dark gray to brown organic sensitive silty CLAY with occasional lenses 1/8 in. thick of silty fine sand.
	S-23	186.5-189.0	"	70.9	98	42	56	905	95.0	Same as S-22 except medium stiff.
	S-24	191.5-194.1	"	93.5	101	46	55			Same as S-22.
	S-25	197.2-199.7	"	Clay=98.0 Peat=93.6	105	42	63	938	92.4	Alternating layers of medium to stiff dark gray organic silty CLAY, firm fibrous PEAT, and silty fine SAND, 1 inch, 1/2 inch and 1/2 inch thick respectively.
	S-26	202.4-204.9	"	Clay=73.0 Peat=111.3	100	40	60	694	92.6	Same as S-25.
	S-27	208.2-210.7	"	95.5				582	90.5	Top Same as S-25.
				58.0	68	38	30			Bottom Light gray silty clayey fine SAND with occasional 1/8-inch layers of fibrous peat with broken shells.
	S-28	213.4-215.9	"	59.7	76	40	36			Medium to stiff dark gray organic fine sandy silty CLAY with occasional layers of light gray hard siltstone and sandstone.
	S-29	219.0-221.5	"	53.4	66	39	27	543	107.5	Same as S-28.
	S-30	224.2-226.7	"	93.5				106	86.5	Loose dark gray organic silty fine to medium SAND with occasional pockets of peat and shells.
	S-31	228.8-230.3	10	50.1						Top Same as S-30.
				75.5	118	44	74			Bottom Medium dark gray fine sandy silty CLAY with numerous small peat pockets.
	S-32	234.0-235.5	Hydraulic Push	42.5					103	Medium dark gray slightly silty fine SAND with small pockets of peat.

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Boring No.	Sample No.	Depth, feet	Standard Penetration Resistance, blows/ft.	Natural Water Content, percent	Atterberg Limits LL	PL	PI	Shear Strength psf (1/2 qu)	Wet Density pcf	Sample Classification
UP-25-CC	S-33	236.8-239.3	Hydraulic Push	42.3					120	Medium dense dark gray to brown silty fine to medium SAND with numerous sandstone nodules to 1/2 in. diameter and shells.
	S-34	240.1-242.5	"	47.3						Top Same as S-33.
				41.8					107	Bottom Fairly clean dark gray to black medium to coarse sandy GRAVEL (1/2 in. diameter).
	S-35	242.7-245.2	"	25.8						Top Dark gray to black fairly clean medium to coarse SAND with some gravel to 3/4 in. diameter.
				40.5					108	Middle Dark gray silty clayey fine SAND.
				35.3						Bottom Same as top.
	S-36	248.7-249.7	"							Same as S-35.
	S-37	251.8-251.9	"	No Recovery						
	S-38	252.5-253.2	130/1 in	No Recovery						
	S-39	253.2-254.1	NX Core							Very hard black brittle BASALT with iron pyrite and calcite filled fractures.
UP-25-DC	S-1	78.5-81.0	Hydraulic Push	727						Very soft dark brown slightly clayey PEAT.
	S-2	85.0-87.5	"	98.9	85	33	53			Very soft light blue to purple organic silty CLAY with numerous black organic pockets and roots.
	S-3	90.2-92.7	"	69.8	60	36	44			Very soft light blue to purple organic silty CLAY with silty very fine sand lenses 1/16 in. thick.
	S-4	95.2-97.7	"	136	175	61	114	238	82.5	Very soft light blue to purple organic silty CLAY with numerous black organic pockets.
	S-5	100.4-102.9	"	149						Top Very soft dark organic sandy silty CLAY.
				157	170	70	100	222	76.5	Bottom Soft dark gray organic peaty silty CLAY.
	S-6	105.4-107.9	"	178	210	68	142	449	75.5	Soft light gray peaty silty CLAY with numerous stiff fibrous peat layers 1/2 in. thick.
	S-7	110.0-112.5	"	256	250	96	154	338	71.9	Top Same as S-6.
				267						Bottom Firm dark brown clayey fibrous PEAT.
	S-8	113.5-116.0	"	147.5	175	59	116	555	77.3	Soft to medium light gray peaty silty CLAY with occasional wood fragments.
	S-9	118.6-121.1	"	327				578	73.0	Firm dark brown slightly clayey fibrous PEAT.
	S-10	123.5-125.0	"							Top Soft dark brown organic sensitive silty CLAY.
				370						Middle Firm dark brown fibrous PEAT.
				109	129	41	88	320	89.5	Bottom Soft dark gray organic sensitive silty CLAY.
	S-11	128.7-131.2	"	97.7	119	47	72	265	95.1	Same as bottom of S-10 with occasional sandstone nodules to 3/4 in. diameter.
	S-12	133.7-136.2	"	73.7	79	35	44	349	104.5	Soft light blue to dark gray organic sensitive very fine sandy silty CLAY with occasional siltstone cobbles to 1-1/2 in. diameter and fine lenses of clayey fine sand.

TABLE A-5 SUMMARY OF FIELD LABORATORY TEST DATA

Boring No.	Sample No.	Depth, feet	Standard Penetration Resistance, blows/ft.	Natural Water Content, percent	Atterberg Limits LL	PL	PI	Shear Strength psf (1/2 qu)	Wet Density pcf	Sample Classification
UP-25-DC	S-13	138.9-141.4	Hydraulic Push	89.1	105	47	58	435	94.8	Soft dark gray organic sensitive silty CLAY with occasional small pockets of organic material and shells.
	S-14	143.8-146.3	"	97.8	106	56	50			Medium dark brown to gray organic silty CLAY.
	S-15	149.0-151.5	"	144 90.9	101	47	54			Top Medium stiff dark brown to black peaty silty CLAY. Bottom Medium stiff dark gray organic sensitive silty CLAY with numerous lenses of dark gray very fine sandy silty clay.
	S-16	154.0-156.5	"	83.7	110	51	59			Same as S-15 except with lenses of peat 1/16 in. thick.
	S-17	159.2-164.0	"							Same as S-15.
	S-18	164.2-166.7	"	86.4	113	50	63			Medium dark gray organic silty CLAY with occasional fine lenses of dark gray silty fine sandy clay and fine lenses of PEAT.
	S-19	169.4-171.9	"	Clay=87.2 Peat=115	118	53	65			Alternating layers to 6 inches thick of medium stiff dark gray organic silty CLAY and firm dark brown fibrous PEAT.
	S-20	174.3-176.8	"	Clay=70.1 Peat=97.7						Top Alternating layers to 1 in. thick of medium dark gray very fine sandy silty CLAY and dark brown well compacted fibrous PEAT. Bottom 0.2 ft Medium dark gray organic sensitive silty CLAY.
	S-21	179.5-182.0	"	83.2	119	48	71			Same as bottom of S-20 with occasional pockets of peat.
	S-22	184.5-187.0	"	83.6	110	45	55			Same as S-21.
	S-23	189.7-192.3	"	93.4	135	56	79			Medium dark brown to black peaty silty CLAY.
	S-24	194.6-197.1	"	87.8	124	55	69			Same as S-23.
	S-25	199.8-204.0	"	83.3						Top Same as S-23. Bottom Medium stiff light blue slightly sandy silty CLAY.
	S-26	204.2-207.0	"	58.8	107	42	65			Same as bottom of S-25.
	S-27	209.8-212.0	"	48.5						Same as S-25 except hard (weathered siltstone).
	S-28	212.0-214.2	37							Hard brown weathered SILTSTONE.
DN-40-DC	S-1	80.0-82.5	"	60.3	102	45	57	544	100	Medium light blue-brown organic sensitive silty CLAY with numerous grass roots.
	S-2	85.0-87.5	"	61.2	73	36	37	471	97.1	Soft blue to brown organic sensitive very fine sandy silty CLAY.
	S-3	90.0-92.5	"	54.7 206	68	30	38	136	85.5	Top Very soft purple sensitive very fine sandy silty CLAY with occasional siltstone nodules to 1-1/2 in. diameter. Bottom Soft dark brown clayey fibrous PEAT.
	S-4	95.0-97.5	"	88.6				320	86.4	Top Soft light gray organic sensitive silty CLAY.

TABLE A-5 SUMMARY OF FIELD LABORATORY TEST DATA

Boring No.	Sample No.	Depth, feet	Standard Penetration Resistance, blows/ft.	Natural Water Content, percent	Atterberg Limits LL	PL	PI	Shear Strength psf (1/2 qu)	Wet Density pcf	Sample Classification
DN-40-DC	S-4	95.0-97.5	Hydraulic Push	131	113	43	70			Bottom Soft dark brown-gray peaty silty CLAY.
	S-5	100.0-102.5	"	126				367	92.1	Same as bottom of S-4.
	S-6	105.0-107.5	"	369 118				471	80.9	Top Well compacted dark brown fibrous PEAT. Bottom Soft dark gray sensitive peaty silty CLAY.
	S-7	110.0-112.5	"	132 257	150	75	75	456	80.1	Top Same as bottom of S-5. Bottom Well compacted dark brown fibrous PEAT.
	S-8	115.0-117.5	"	159 240				1144	74.3	Top Medium stiff dark gray sensitive peaty silty CLAY. Bottom Well compacted dark brown fibrous PEAT.
	S-9	120.0-122.5	"	75.2	117	47	70	569	88.6	Medium light blue to dark gray slightly peaty organic sensitive silty CLAY with numerous shells.
	S-10	125.0-127.5	"	97.0				698	89.7	Same as S-9.
	S-11	130.0-132.5	"	84.2				595	85.4	Same as S-9 except no shells.
	S-12	135.0-137.5	"	129	155	76	79			Medium stiff dark gray sensitive peaty silty CLAY.
	S-13	140.0-142.5	"	101	134	48	86			Same as S-12 except less peaty.
	S-14	145.0-147.5	"	91.0	135	46	89			Same as S-12 except with shells.
	S-15	150.0-152.5	"	55.2	83	36	47			Top Same as S-14. Bottom Medium light gray and blue organic very fine sandy silty CLAY with angular cobbles to 2-1/2 in. diameter.
DN-40-CC	S-1	77.0-79.5	Hydraulic Push	63.9				267	99.2	Soft to medium light blue to reddish brown organic slightly silty CLAY with occasional grass roots.
	S-2	80.0-82.5	"	54.0				897	104.2	Medium to stiff, red, blue, yellow and green mottled organic slightly silty CLAY with grass roots.
	S-3	84.0-86.5	"	54.3				472	103.2	Same as S-2.
	S-4	90.0-92.5	"	60.9				488	101.0	Soft to medium stiff light blue to dark gray organic sensitive silty CLAY with occasional siltstone nodules to 1/4 in. diameter and one fine sand layer 1/8 in. thick.
	S-5	95.0-97.5	"	86.7	110	60	30	440	92.0	Soft dark brown-gray very organic sensitive silty CLAY with small peat pockets and one fine sand layer 1/2 in. thick.
	S-6	100.0-102.5	"	92.1 71.2	94	44	50	377	98.0	Top Same as S-5 except soft to medium and with a 0.4 ft. thick layer of fine sand. Bottom Soft to medium dark gray organic silty CLAY with small pockets and lenses of leaves and peat.

TABLE A-5 SUMMARY OF FIELD LABORATORY TEST DATA

Boring No.	Sample No.	Depth, feet	Standard Penetration Resistance, blows/ft.	Natural Water Content, percent	Atterberg Limits LL	PL	PI	Shear Strength psf (1/2 qu)	Wet Density pcf	Sample Classification
DN-40-CC	S-7	105.0-107.5	Hydraulic Push	Clay=75.5 Sand=21.5 Peat=201				Peat 945	Peat 77.5	Alternating layers of soft to medium dark gray to brown organic sensitive CLAY, grey silty SAND, and firm brown fibrous PEAT.
	S-8	110.0-112.5	"	Clay=89.5 Sand=48.7 Peat=145	122	59	63	391	89.5	Same as S-7
	S-9	113.0-115.5	"	Sand=57.5 Peat=133					Sand 106	Gray silty fine SAND with layers of firm dark brown fibrous PEAT.
	S-10	120.0-122.5	"	94.8	126	40	86	895	87.0	Medium dark brown very organic slightly silty CLAY with small pockets and lenses of peat.
	S-11	125.0-127.5	"	41.4	107	29	78	732 (slicken side)	112.3	Medium to stiff light blue with green and red spots slightly silty CLAY.
	S-12	129.0-131.5	"	43.9	124	31	93	1272 (slicken-side)	112.3	Same as S-11 except very stiff.
	S-13	132.0-134.5	"	47.9	111	29	82	1102 (slicken-side)	110	Same as S-12.
	S-14	138.0-140.5	"	45.4	84	29	55	958 slicken-side)	113.0	Same as S-12.
DN-40-BC	S-1	77.0-79.5	Hydraulic Push	67.1	127	43	84			Soft light blue to brown organic silty CLAY with occasional grass roots.
	S-2	80.0-81.9	"	67.2	125	42	84	854	100.2	Medium dark gray-brown organic sensitive silty CLAY with grass roots.
	S-3	85.0-87.5	"	65.4	124	32	92	519	100.8	Medium dark brown and blue organic sensitive silty CLAY.
	S-4	90.0-92.5	"	67.8	93	32	61	357	98.2	Soft light blue to purple organic silty CLAY with occasional lenses of silty clayey fine sand 1/10 in. thick.
	S-5	95.0-97.5	"	84.0	102	46	57	243	91.1	Very soft dark gray-brown organic sensitive slightly fine sandy silty CLAY with layers of clayey silty fine sand to 3 in. thick.
	S-6	100.0-102.5	"	75.2	88	35	53	506	91.5	Medium gray to brown organic sensitive silty CLAY with occasional siltstone nodules to 3/4 in. diameter and numerous organic pockets 1/4 in. diameter.
	S-7	105.0-107.5	"	73.2	112	42	70	591	91.5	Medium dark gray-brown organic very sensitive silty CLAY with an occasional small organic pocket and with a layer of firm sandy peat 1 in. thick.
	S-8	110.0-112.5	"	89.0	126	49	77	763	89.6	Medium dark gray-brown organic sensitive CLAY with occasional siltstone nodules to 1/2 in. diameter.

TABLE A-5 SUMMARY OF FIELD LABORATORY TEST DATA

Boring No.	Sample No.	Depth, feet	Standard Penetration Resistance, blows/ft.	Natural Water Content, percent	Atterberg Limits LL	PL	PI	Shear Strength psf (1/2 qu)	Wet Density pcf	Sample Classification
DN-40-BC	S-9	115.0-117.5	Hydraulic Push	101				539	90.2	Medium dark gray-brown organic sensitive silty CLAY with numerous peat pockets and occasional shells.
	S-10	120.0-122.5	"	108	146	43	105	456	85.7	Same as S-9 except no shells and with more peat.
	S-11	125.0-127.5	"	96.5				1084	88.2	Medium dark gray organic silty CLAY with numerous leaves and peat fragments.
	S-12	130.0-132.5	"	41.1	96	27	69	920	112.9	Medium to stiff light blue and green silty CLAY with a layer of light gray silty clayey fine sand 1/2 in. thick.
	S-13	135.0-137.3	"	47.7	107	32	75	866	108.5	Medium to stiff red with light blue slightly silty CLAY.
	S-14	140.0-142.5	"	42.3				1650 (Slicken-side)	112.0	Same as S-13 except very stiff.
	S-15	144.0-146.5	"	47.3				1417	110.5	Same as S-14
UP-40-BC	S-1	77.0-79.5	"	708						Nearly liquid dark gray decayed PEAT.
	S-2	80.0-82.5	"	257						Very soft watery dark gray clayey PEAT.
	S-3	85.0-87.5	"	191	120	49	71	98	75.5	Very soft dark brown-gray organic sensitive peaty silty CLAY.
	S-4	90.0-92.5	"	115	137	50	87	95	86.7	Same as S-3 except light blue and purple.
	S-5	95.0-97.5	"	165 341	180	57	123			Top Same as S-3 Bottom Soft dark brown silty clayey PEAT.
	S-6	100.0-102.5	"	402 164				223	70.2	Top Soft dark brown silty clayey PEAT. Bottom Soft dark gray-brown sensitive peaty silty CLAY.
	S-7	105.0-107.5	"	289 352				450	68.6	Top Firm black-brown fibrous PEAT. Middle Same as bottom of S-6. Bottom Dark brown hard wood.
	S-8	110.0-112.5	"	506				714	65.5	Firm dark brown fibrous PEAT.
	S-9	115.0-117.5	"	123	137	46	91	172	82.0	Top 0.1 ft. Same as S-8 Bottom Very soft dark gray organic sensitive silty CLAY with numerous leaves and lenses of peat.
	S-10	120.0-122.5	"	133	152	58	94	290	81.8	Same as bottom of S-9 except soft and with occasional shells.
	S-11	125.0-127.5	"	131	169	52	117	352	82.5	Soft dark gray to brown organic sensitive peaty silty CLAY.
	S-12	130.0-132.5	"	129 121	157	71	86			Top 0.3 ft. Firm dark brown fibrous PEAT with shells. Bottom Same as S-11
	S-13	135.0-137.5	"	108	130	48	82	358	86.1	Soft dark gray organic sensitive silty CLAY with occasional leaves and siltstone nodules.
	S-14	140.0-142.5	"	74.5 48.5	85	35	50			Top Same as S-13 Bottom Soft light blue organic fine sandy silty CLAY with occasional siltstone fragments to 3/4 in. dia.

TABLE A-5 SUMMARY OF FIELD LABORATORY TEST DATA

Boring No	Sample No.	Depth, feet	Standard Penetration Resistance, blows/ft.	Natural Water Content, percent	Atterberg Limits LL	PL	PI	Shear Strength psf (1/2qu)	Wet Density pcf	Sample Classification
UP-40-BC	S-15	145.0-147.5	Hydraulic Push	56.8	92	38	54	326	98.1	Soft light blue, purple, and green very fine sandy silty CLAY with siltstone fragments to 3/4 in. dia. and a wood fragment 1/2 x 4 inches.
	S-16	150.0-152.5	"	43.9	72	26	46	305	109.8	Stiff to very stiff light blue, brown, and green very fine sandy silty CLAY with siltstone fragments to 1/4 in. dia. (weathered siltstone).
UP-40-DC	S-1	78.0-81.5	"	1145						Nearly liquid dark brown silty decayed PEAT.
	S-2	85.0-87.5	"	Peat= 740 Clay= 233	260	77	183	74	72.1	Alternating layers of very soft sensitive peaty CLAY and clayey PEAT 1/2 in. thick.
	S-3	90.0-92.5	"	117	131	46	85	20	79.2	Very soft light blue to purple organic silty CLAY with numerous air pockets.
	S-4	95.0-97.5	"	147 337	145	53	92	145	73.0	Top Very soft dark gray peaty silty CLAY. Bottom Soft dark brown silty clayey fibrous PEAT.
	S-5	100.0-102.5	"	Peat= 319 Clay= 154	190	74	116	118	71.6	Alternating layers 3/4 in. thick of very soft light gray organic peaty silty CLAY and soft clayey PEAT
	S-6	105.0-107.5	"	464				180	68.0	Soft dark brown clayey fibrous PEAT.
	S-7	110.0-112.5	"	683				253	63.1	Firm dark gray fibrous PEAT.
	S-8	115.0-117.5	"	205				322	73.8	Alternating layers and lenses 1/8 to 1/2 in. thick of soft light gray peaty silty CLAY and clayey PEAT.
	S-9	120.0-122.5	"	143	180	34	146	261	76.4	Soft dark gray-brown slightly peaty organic CLAY.
	S-10	125.0-127.5	"	141	150	46	104	398	78.6	Same as S-9
	S-11	130.0-132.5	"	125				345	80.7	Same as S-9
	S-12	135.0-137.5	"	730	88	29	59	210	103	Soft dark gray peaty silty CLAY with fine gravelly sand throughout.
	S-13	140.0-142.5	"	55.5	69	34	35	181	106	Medium stiff light blue, purple, gray, green, and red silty CLAY with gravel to 3/4 in. dia.
	S-14	145.0-147.5	"	50.1				228	105	Same as S-13 (weathered sandstone).
UP-40-DC	S-1	77.0-79.5	"	86.6	142	43	99	166	91.2	Very soft light gray organic sensitive silty CLAY with occasional grass roots. Top 1/2 ft. is peaty.
	S-2	80.0-82.5	"	73.3	121	48	73	445	97.5	Medium to stiff dark brown-gray and light blue organic silty CLAY.
	S-3	85.0-87.5	"	74.2 109	104	38	66	184	92.5	Top Medium light blue organic silty CLAY. Bottom Very soft dark gray organic silty CLAY.
	S-4	90.0-92.5	"	121 87.3	110	39	71	238	94.5	Top Soft dark gray peaty silty CLAY and firm clayey PEAT layers. Bottom Soft light gray organic silty CLAY with occasional grass roots.

TABLE A-5 SUMMARY OF FIELD LABORATORY TEST DATA

Boring No.	Sample No.	Depth, feet	Standard Penetration Resistance, blows/ft.	Natural Water Content, percent	Atterberg Limits LL	PL	PI	Shear Strength psf (1/2qu)	Wet Density pcf	Sample Classification
UP-40-DC	S-5	95.0-97.5	Hydraulic Push	134 463	165	54	131			Top Same as bottom of S-4 Bottom Firm dark brown fibrous PEAT.
	S-6	100.0-102.5	"	314				845	65.0	Same as bottom of S-5
	S-7	105.0-107.5	"	98.6	98	39	59	414	92.9	Medium dark gray organic silty CLAY with occasional gravel to 1/2 in. dia. and one layer of firm fibrous peat 1 in. thick.
	S-8	110.0-112.5	"	46.5 64.2	80	37	43	234	99.5	Top Soft to medium light blue gravelly silty CLAY with occasional siltstone nodules. Bottom Soft dark green sensitive silty CLAY.
	S-9	115.0-117.5	"	47.4	105	24	81	564	110.0	Top Stiff to very stiff light gray and yellow-orange slightly gravelly CLAY. Bottom Stiff to very stiff silty CLAY with numerous well-rounded weathered cobbles to 1 in. dia. (weathered conglomerate).
	S-10	115.0-121.5	NX Core	34.8						Same as bottom of S-9.

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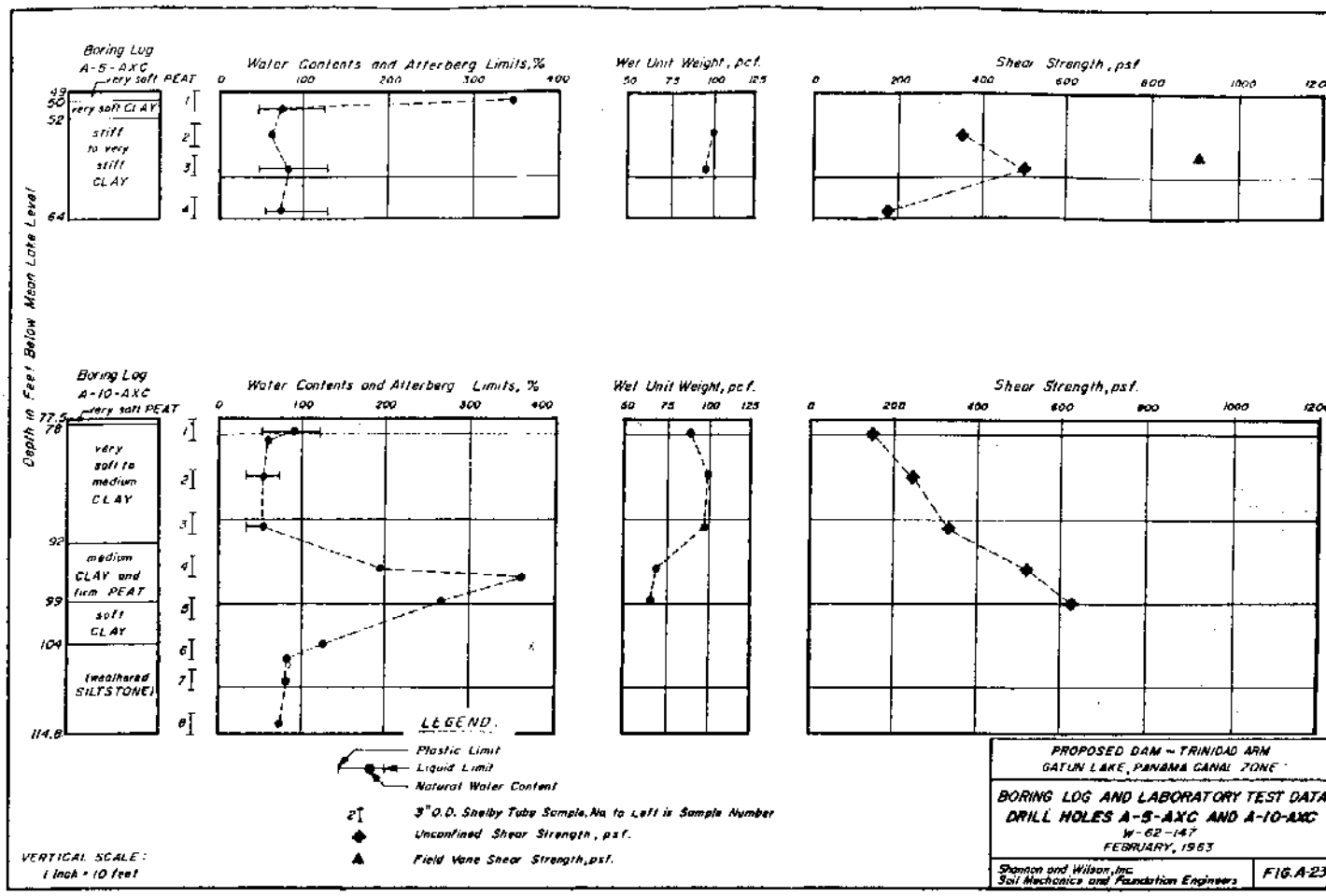
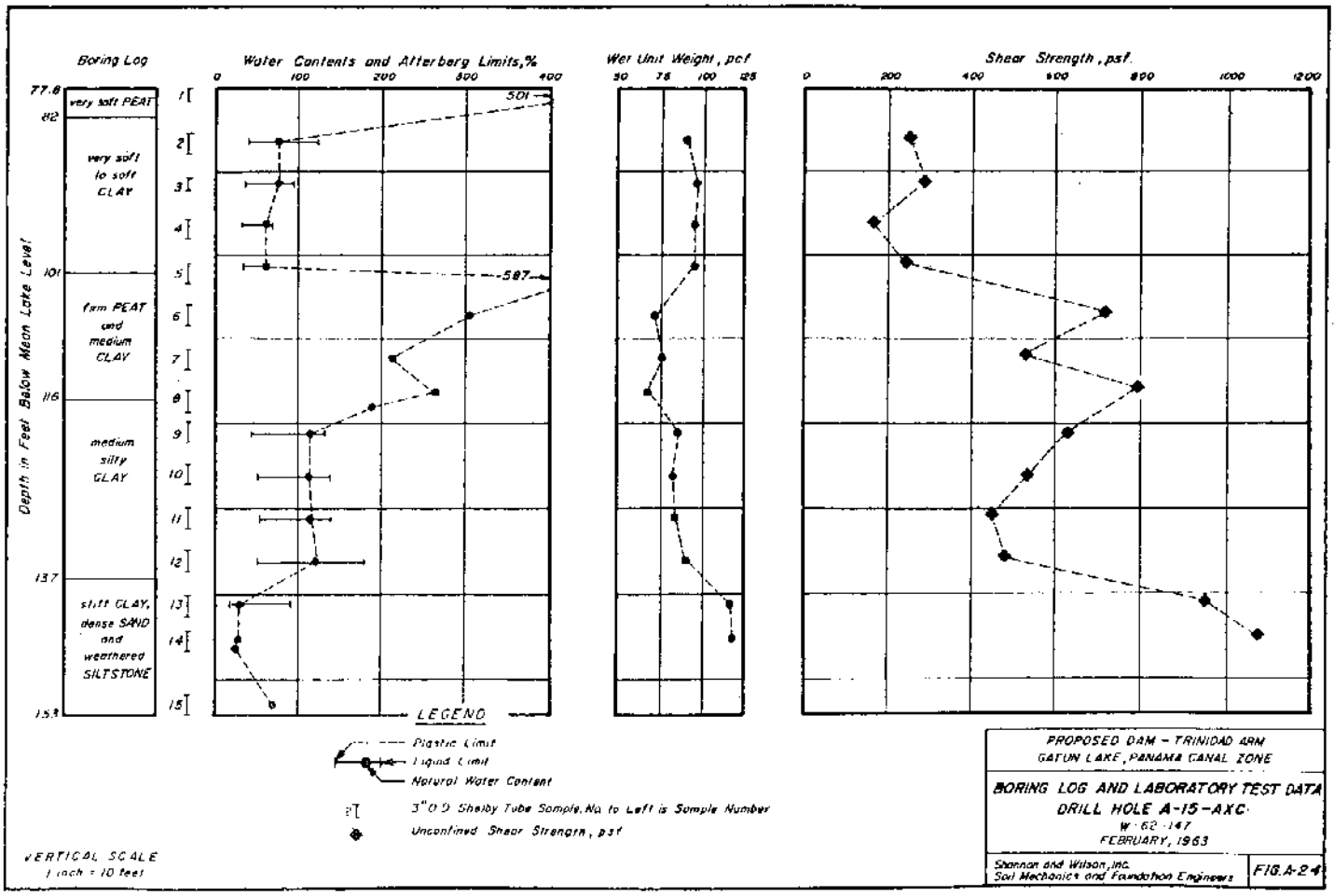

Boring Log
A-5-AXC
very soft PEAT
very soft CLAY
stiff to very stiff CLAY
Water Contents and Atterberg Limits, %
Wet Unit Weight, pcf.
Shear Strength, psf
Boring Log
A-10-AXC
very soft PEAT
very soft to medium CLAY
medium CLAY and firm PEAT
soft CLAY
(weathered SILTSTONE)
Depth in Feet Below Mean Lake Level
LEGEND.
Plastic Limit
Liquid Limit
Natural Water Content
3" O.D. Shelby Tube Sample, No. to Left is Sample Number
Unconfined Shear Strength, psf.
Field Vane Shear Strength, psf.
VERTICAL SCALE:
1 inch = 10 feet
PROPOSED DAM – TRINIDAD ARM
GATUN LAKE, PANAMA CANAL ZONE
BORING LOG AND LABORATORY TEST DATA
DRILL HOLES A-5-AXC AND A-10-AXC
W-62-147
FEBRUARY, 1963
Shannon and Wilson, Inc.
Soil Mechanics and Foundation Engineers
FIG. A-23


Boring Log
Water Contents and Atterberg Limits, %
Wet Unit Weight, pcf
Shear Strength, psf.
very soft PEAT
very soft to soft CLAY
firm PEAT and medium CLAY
medium silty CLAY
stiff CLAY, dense SAND and weathered SILTSTONE
Depth in Feet Below Mean Lake Level
LEGEND
Plastic Limit
Liquid Limit
Natural Water Content
3" O.D. Shelby Tube Sample, No. to Left is Sample Number
Unconfined Shear Strength, psf
VERTICAL SCALE
1 inch = 10 feet
PROPOSED DAM – TRINIDAD ARM
GATUN LAKE, PANAMA CANAL ZONE
BORING LOG AND LABORATORY TEST DATA
DRILL HOLE A-15-AXC
W-62-147
FEBRUARY, 1963
Shannon and Wilson, Inc.
Soil Mechanics and Foundation Engineers
FIG. A-24

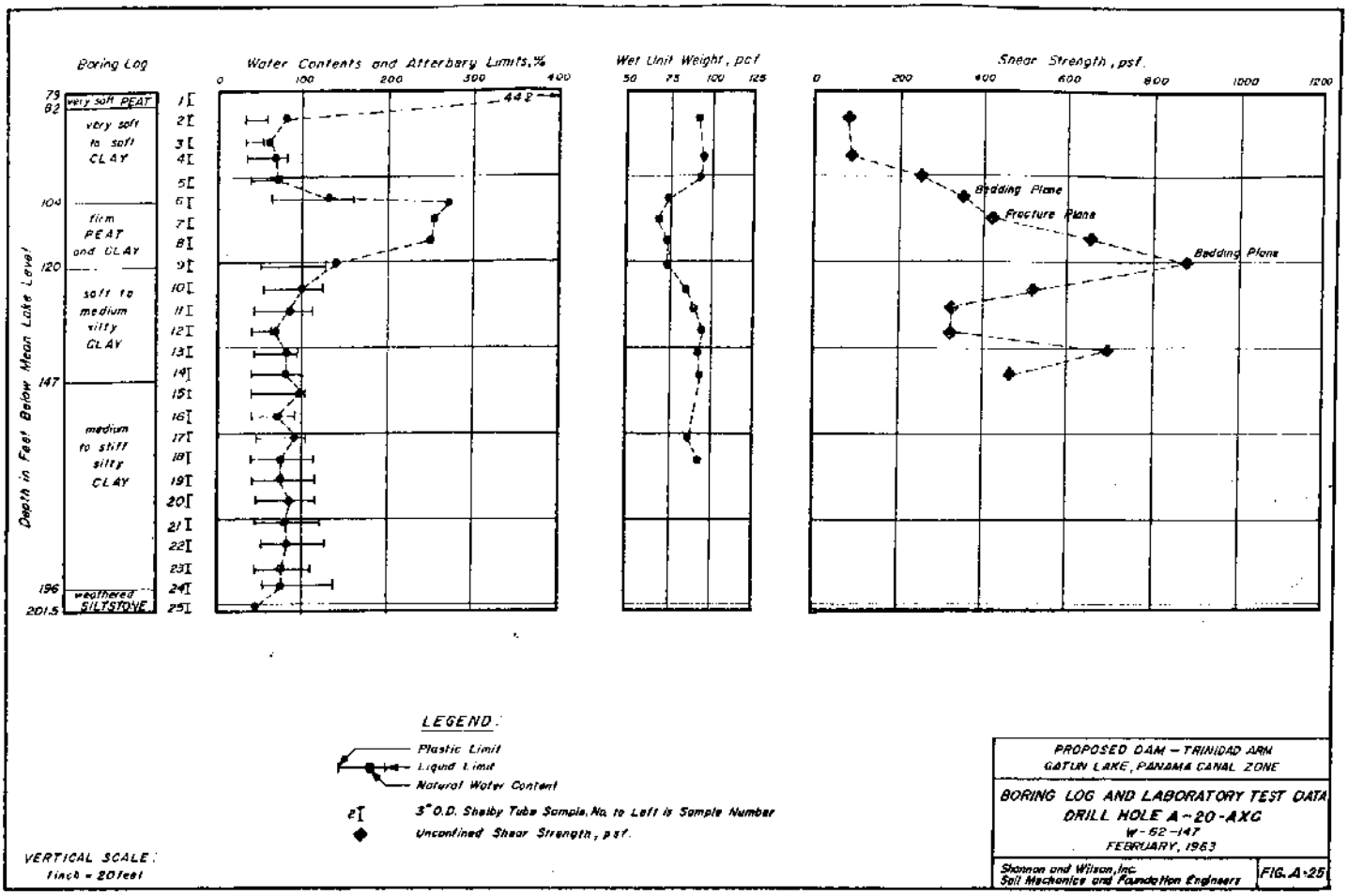

Boring Log
Water Contents and Atterberg Limits, %
Wet Unit Weight, pcf
Shear Strength, psf.
Depth in Feet Below Mean Lake Level
very soft PEAT
very soft to soft CLAY
firm PEAT and CLAY
soft to medium silty CLAY
medium to stiff silty CLAY
weathered SILTSTONE
Bedding Plane
Fracture Plane
Bedding Plane
LEGEND:
Plastic Limit
Liquid Limit
Natural Water Content
3" O.D. Shelby Tube Sample, No. to Left is Sample Number
Unconfined Shear Strength, psf.
VERTICAL SCALE: 1 inch = 20 feet
PROPOSED DAM – TRINIDAD ARM
GATUN LAKE, PANAMA CANAL ZONE
BORING LOG AND LABORATORY TEST DATA
DRILL HOLE A-20-AXC
W-62-147
FEBRUARY, 1963
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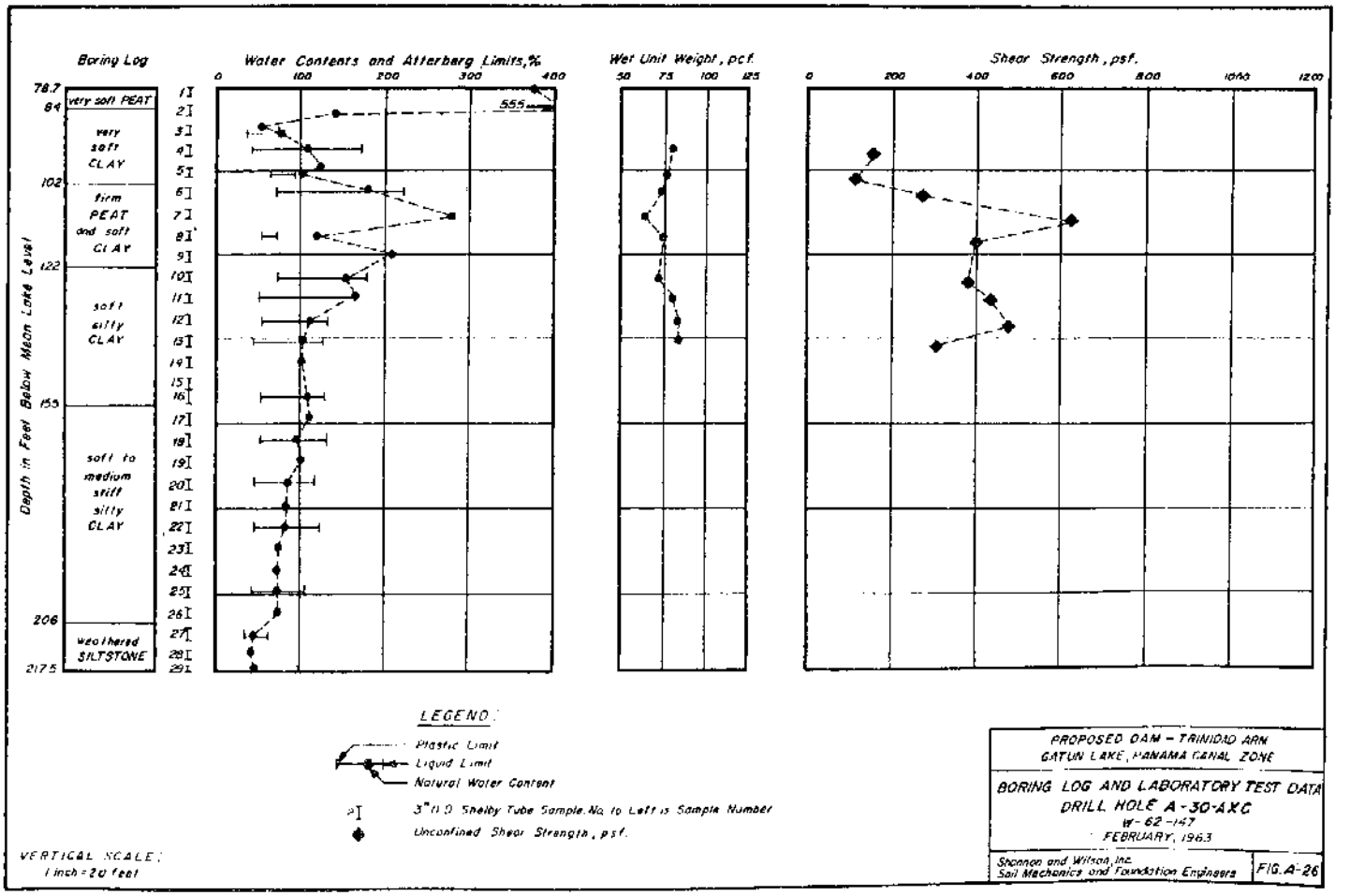
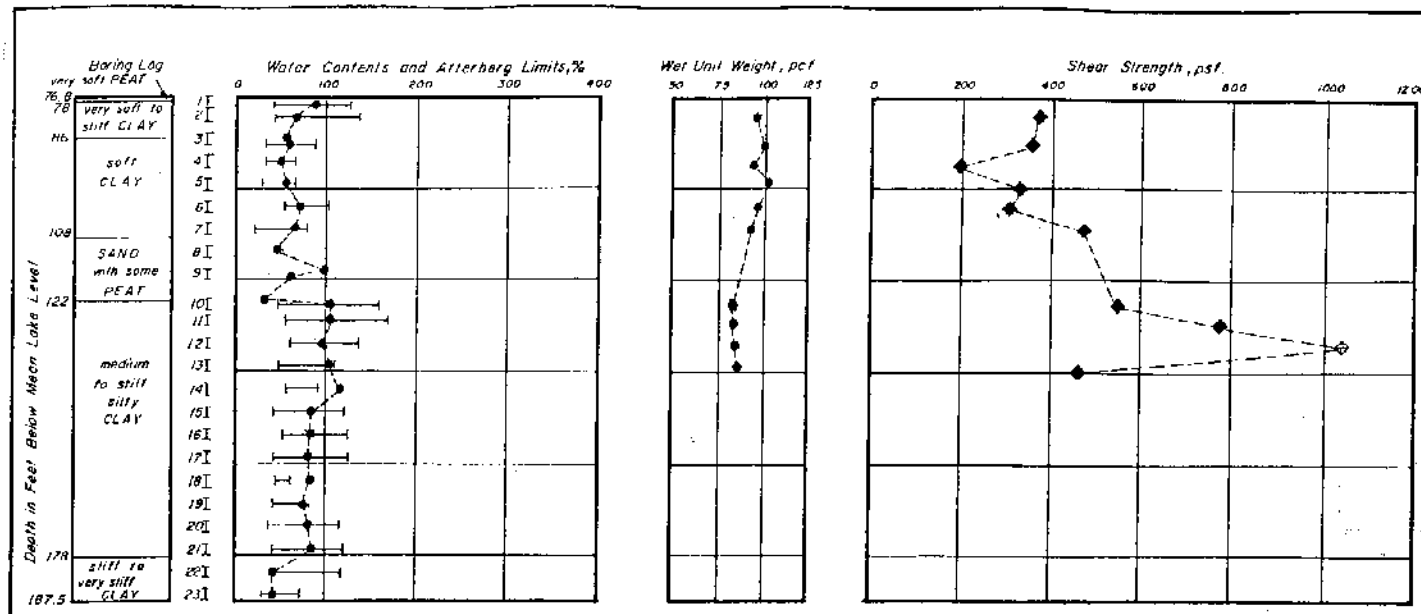
FIG. A-25


Boring Log
Water Contents and Atterberg Limits, %
Wet Unit Weight, pcf.
Shear Strength, psf.
Depth in Feet Below Mean Lake Level
very soft PEAT
very soft CLAY
firm PEAT and soft CLAY
soft silty CLAY
soft to medium stiff silty CLAY
weathered SILTSTONE
LEGEND:
Plastic Limit
Liquid Limit
Natural Water Content
3" O.D. Shelby Tube Sample, No. to Left is Sample Number
Unconfined Shear Strength, psf.
VERTICAL SCALE: 1 inch = 20 feet
PROPOSED DAM – TRINIDAD ARM
GATUN LAKE, PANAMA CANAL ZONE
BORING LOG AND LABORATORY TEST DATA
DRILL HOLE A-30-AXC
W-62-147
FEBRUARY, 1963
Shannon and Wilson, Inc.
Soil Mechanics and Foundation Engineers
FIG. A-26



LEGEND:

- Plastic Limit
- Liquid Limit
- Natural Water Content
- 2 3" O.D. Shelby Tube Sample. No. to Left is Sample Number
- Unconfined Shear Strength, psf.

VERTICAL SCALE: 1 inch = 20 feet

PROPOSED DAM – TRINIDAD ARM
GATUN LAKE, PANAMA CANAL ZONE

BORING LOG AND LABORATORY TEST DATA
DRILL HOLE A-35-AXC
W-62-147
FEBRUARY, 1963

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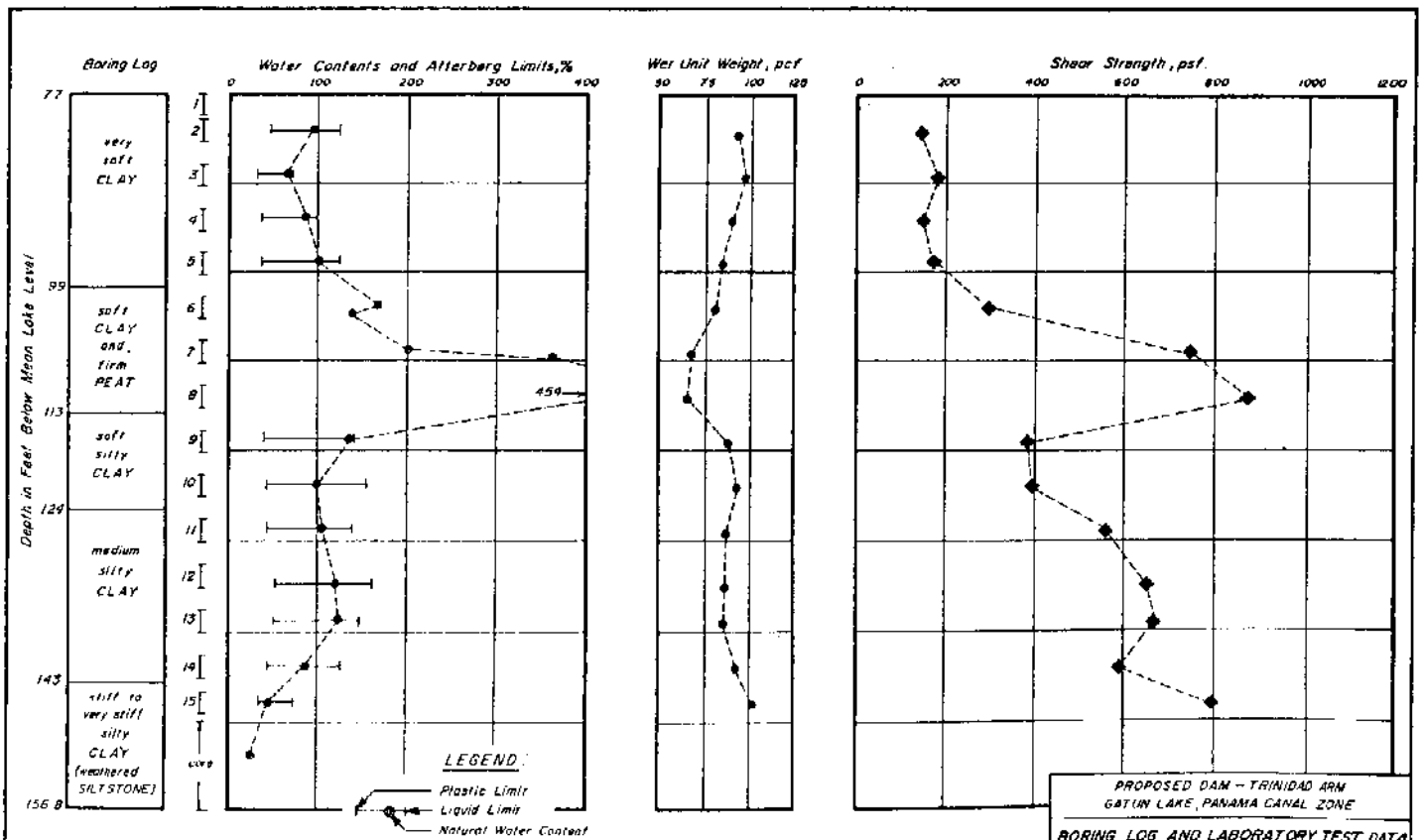
FIG. A-27



LEGEND:

- Plastic Limit
- Liquid Limit
- Natural Water Content
- 2 3" O.D. Shelby Tube Sample. No. to Left is Sample Number
- Unconfined Shear Strength, psf.

VERTICAL SCALE: 1 inch = 10 feet

PROPOSED DAM – TRINIDAD ARM
GATUN LAKE, PANAMA CANAL ZONE

BORING LOG AND LABORATORY TEST DATA
DRILL HOLE A-40-AXC
W-62-147
FEBRUARY, 1963

Shannon and Wilson, Inc.
Soil Mechanics and Foundation Engineers

FIG. A-28

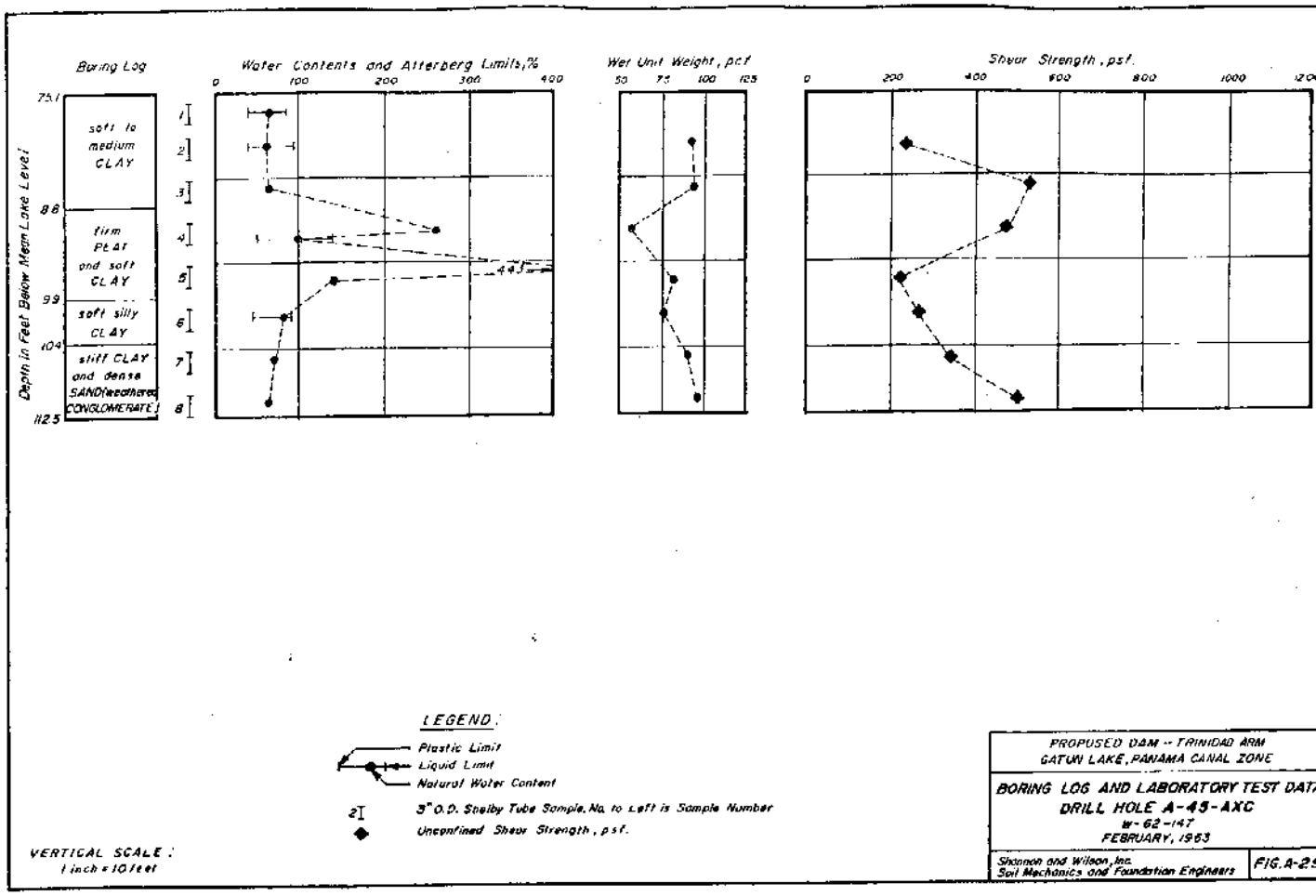

Boring Log
Water Contents and Atterberg Limits, %
Wet Unit Weight, pcf
Shear Strength, psf.
Depth in Feet Below Mean Lake Level
soft to medium CLAY
firm PEAT and soft CLAY
soft silty CLAY
stiff CLAY and dense SAND (weathered CONGLOMERATE)
75.1
88
99
104
112.5
443
LEGEND:
Plastic Limit
Liquid Limit
Natural Water Content
3" O.D. Shelby Tube Sample, No. to Left is Sample Number
Unconfined Shear Strength, psf.
VERTICAL SCALE: 1 inch = 10 feet
PROPOSED DAM - TRINIDAD ARM
GATUN LAKE, PANAMA CANAL ZONE
BORING LOG AND LABORATORY TEST DATA
DRILL HOLE A-45-AXC
W-62-147
FEBRUARY, 1963
Shannon and Wilson, Inc.
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FIG. A-29

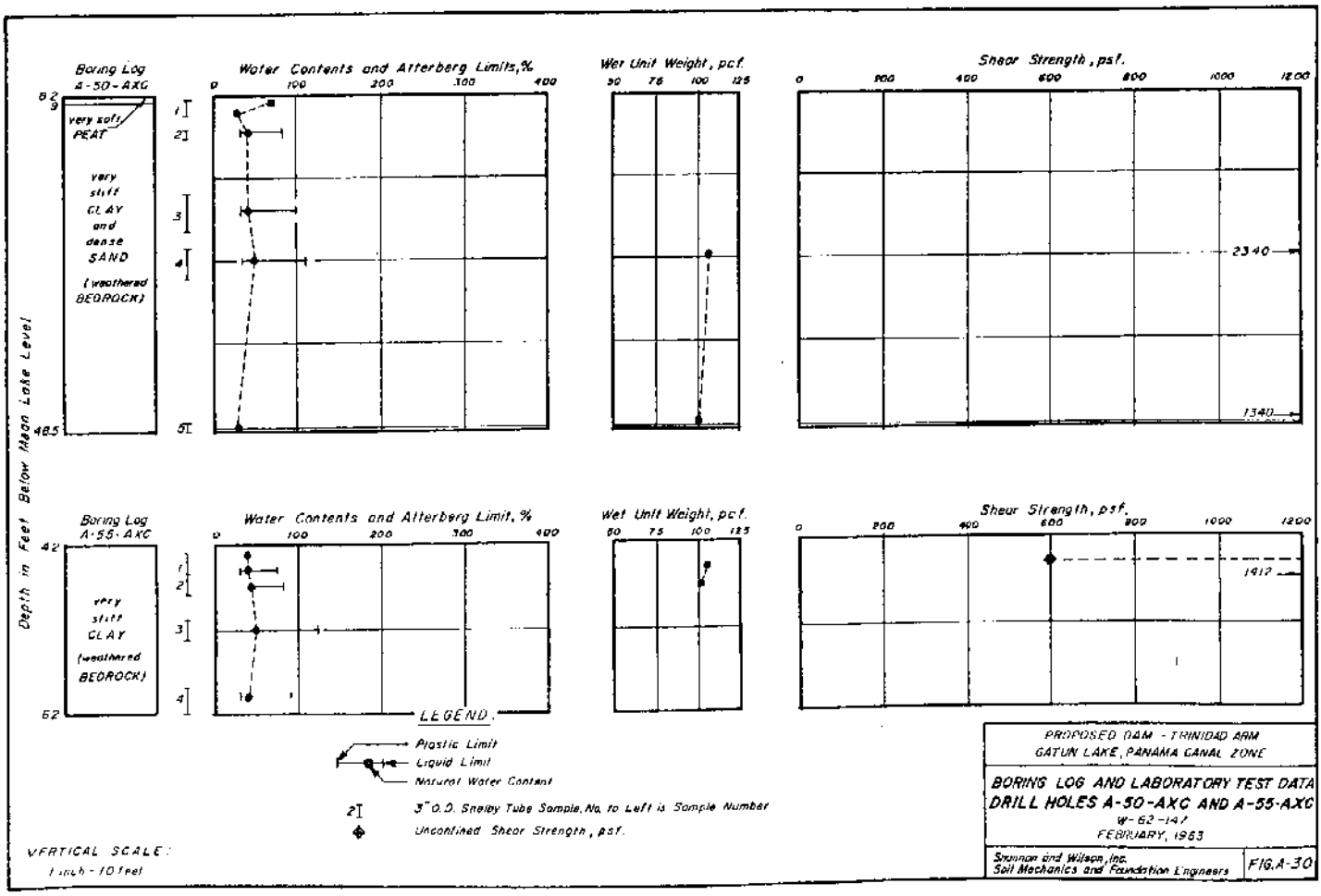
Boring Log A-50-AXC
Water Contents and Atterberg Limits, %
Wet Unit Weight, pcf
Shear Strength, psf.
Depth in Feet Below Mean Lake Level
very soft PEAT
very stiff CLAY and dense SAND (weathered BEDROCK)
82
9
48.5
2340
1340
Boring Log A-55-AXC
Water Contents and Atterberg Limit, %
Wet Unit Weight, pcf.
Shear Strength, psf.
very stiff CLAY (weathered BEDROCK)
42
62
1412
LEGEND:
Plastic Limit
Liquid Limit
Natural Water Content
3" O.D. Shelby Tube Sample, No. to Left is Sample Number
Unconfined Shear Strength, psf.
VERTICAL SCALE: 1 inch = 10 feet
PROPOSED DAM - TRINIDAD ARM
GATUN LAKE, PANAMA CANAL ZONE
BORING LOG AND LABORATORY TEST DATA
DRILL HOLES A-50-AXC AND A-55-AXC
W-62-147
FEBRUARY, 1963
Shannon and Wilson, Inc.
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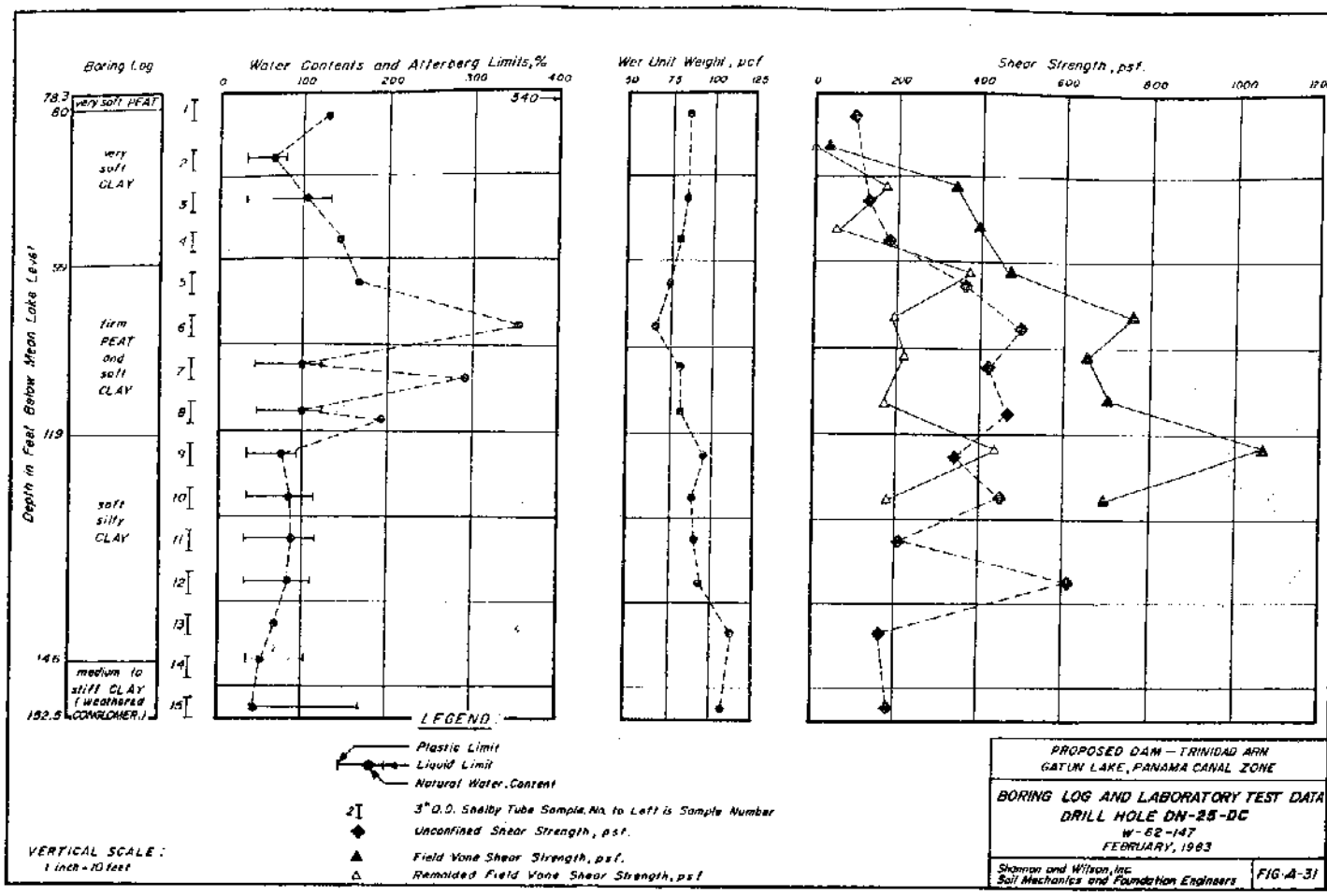
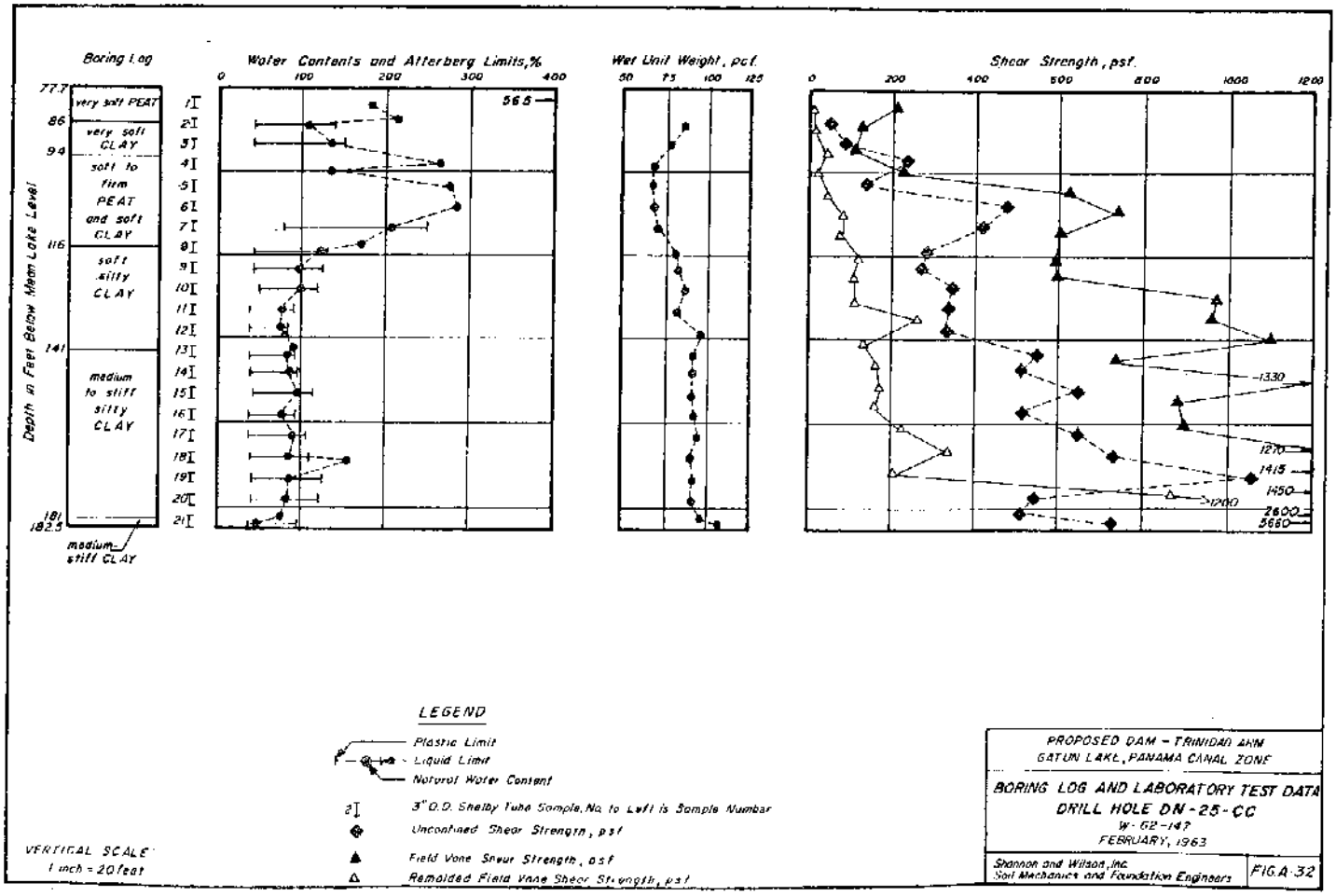
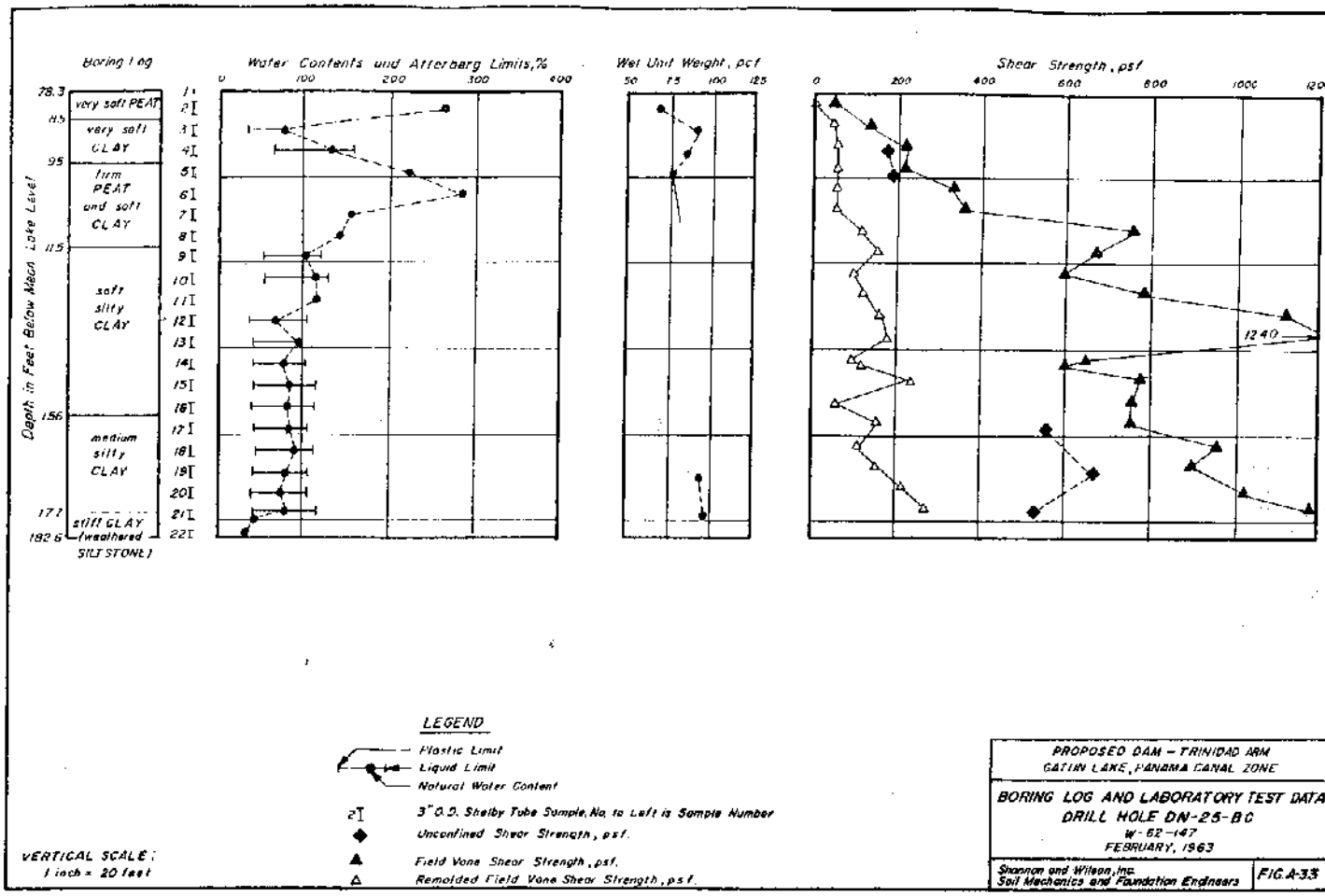
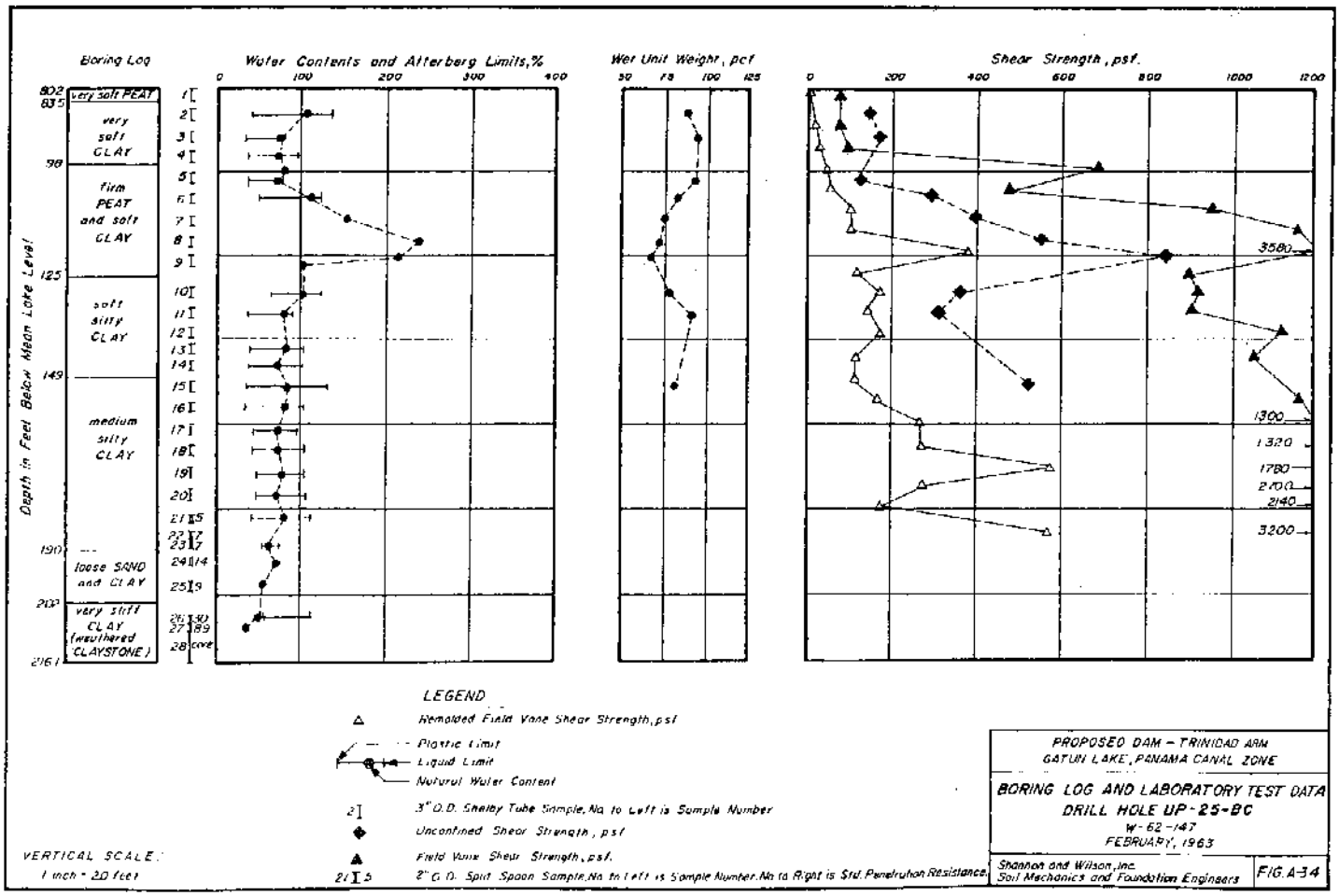
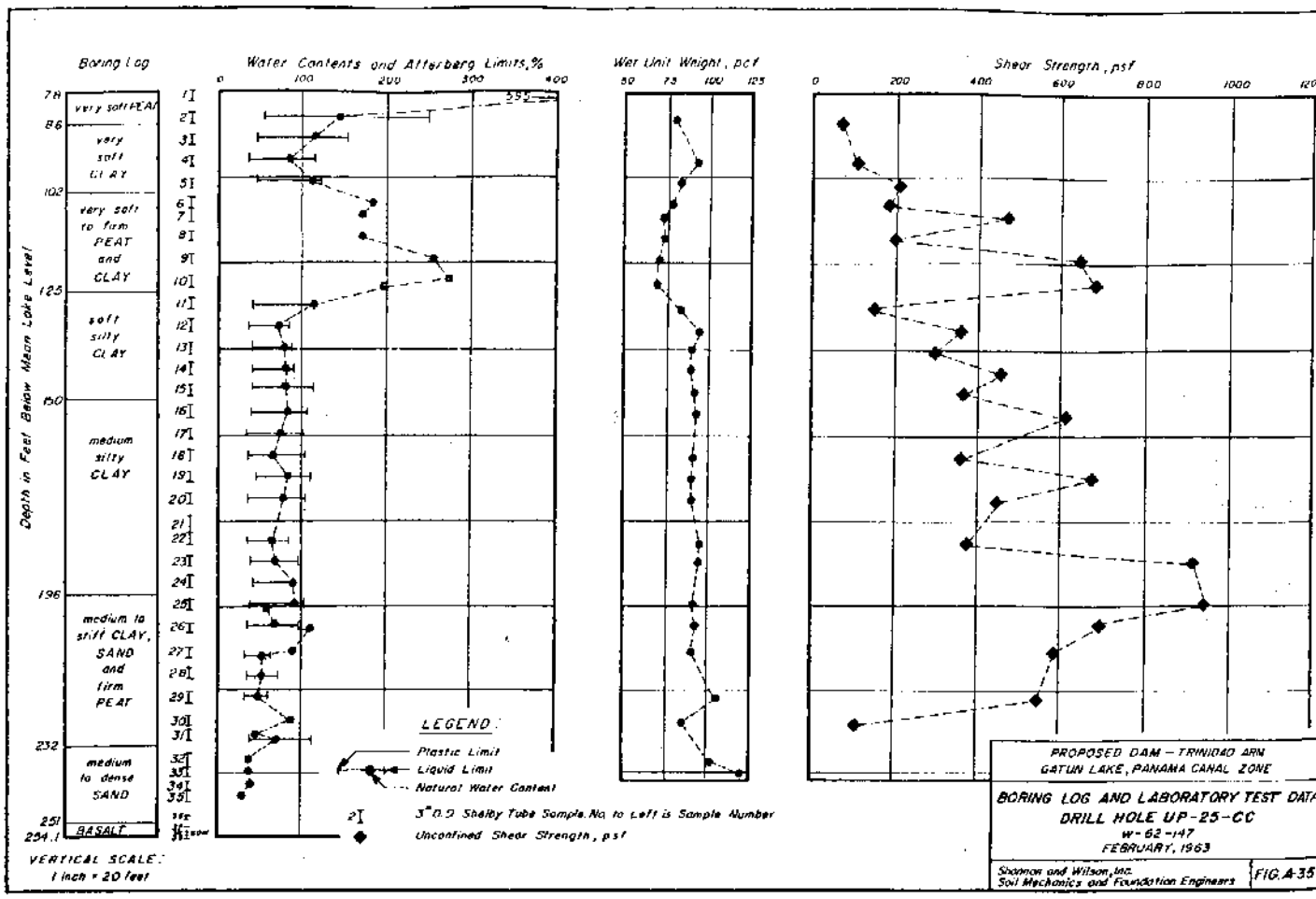
FIG. A-30

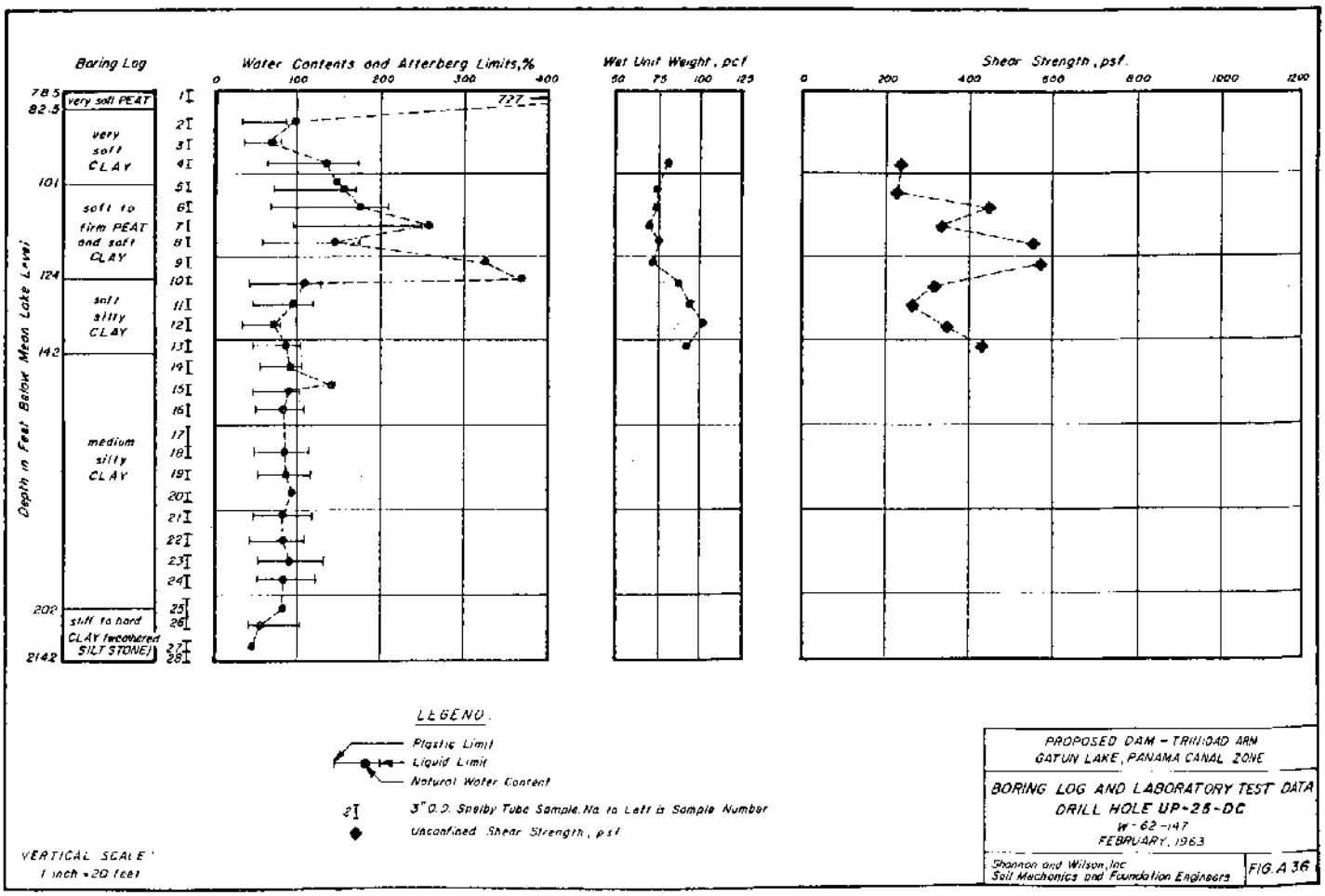

Boring Log
Water Contents and Atterberg Limits, %
Wet Unit Weight, pcf
Shear Strength, psf.
Depth in Feet Below Mean Lake Level
very soft PEAT
very soft CLAY
firm PEAT and soft CLAY
soft silty CLAY
medium to stiff CLAY (weathered CONGLOMER.)
LEGEND
Plastic Limit
Liquid Limit
Natural Water Content
3" O.D. Shelby Tube Sample, No. to Left is Sample Number
Unconfined Shear Strength, psf.
Field Vane Shear Strength, psf.
Remolded Field Vane Shear Strength, psf
VERTICAL SCALE: 1 inch = 10 feet
PROPOSED DAM — TRINIDAD ARM
GATUN LAKE, PANAMA CANAL ZONE
BORING LOG AND LABORATORY TEST DATA
DRILL HOLE DN-25-DC
W-62-147
FEBRUARY, 1963
Shannon and Wilson, Inc.
Soil Mechanics and Foundation Engineers
FIG. A-31

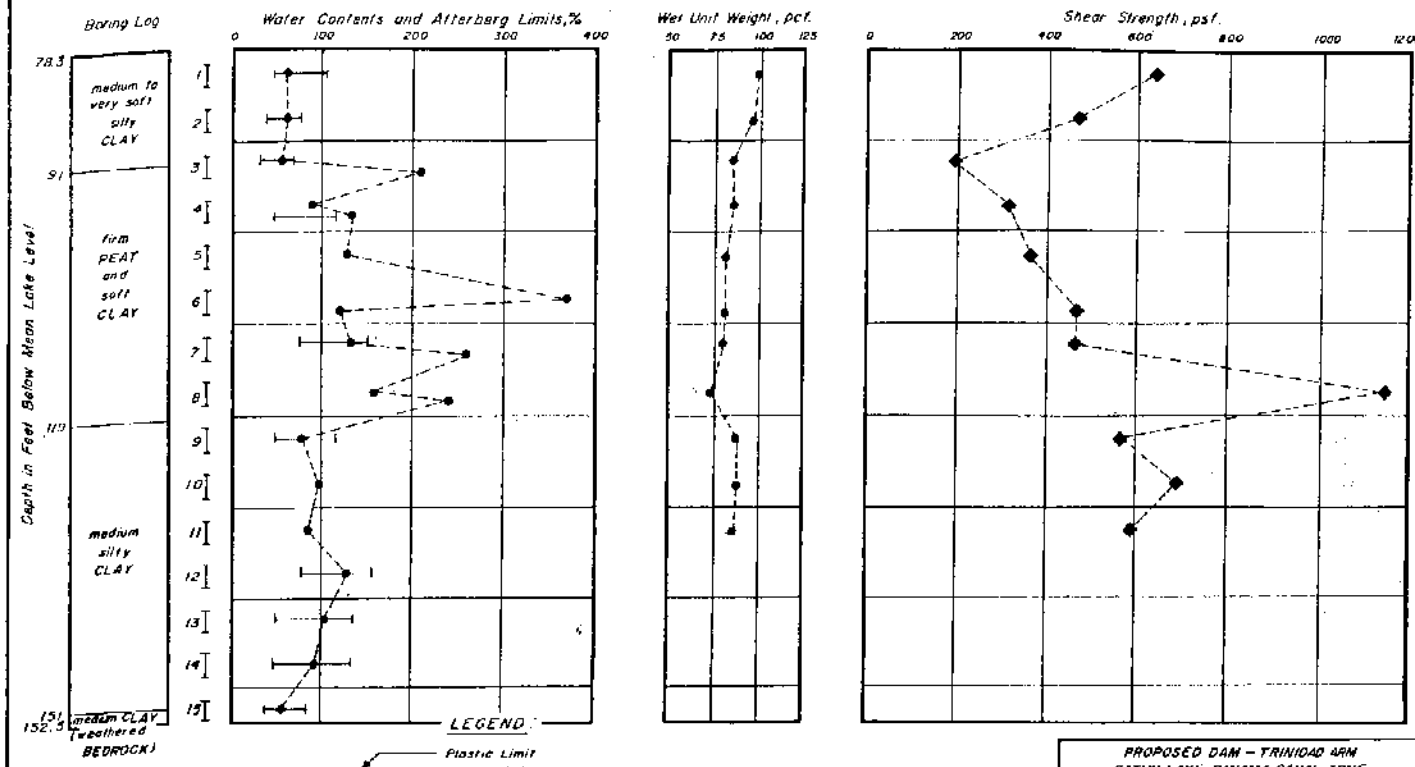

Boring Log
Water Contents and Atterberg Limits, %
Wet Unit Weight, pcf
Shear Strength, psf.
Depth in Feet Below Mean Lake Level
very soft PEAT
very soft CLAY
soft to firm PEAT and soft CLAY
soft silty CLAY
medium to stiff silty CLAY
medium stiff CLAY
LEGEND
Plastic Limit
Liquid Limit
Natural Water Content
3" O.D. Shelby Tube Sample, No. to Left is Sample Number
Unconfined Shear Strength, psf
Field Vane Shear Strength, psf
Remolded Field Vane Shear Strength, psf
VERTICAL SCALE: 1 inch = 20 feet
PROPOSED DAM — TRINIDAD ARM
GATUN LAKE, PANAMA CANAL ZONE
BORING LOG AND LABORATORY TEST DATA
DRILL HOLE DN-25-CC
W-62-147
FEBRUARY, 1963
Shannon and Wilson, Inc.
Soil Mechanics and Foundation Engineers
FIG. A-32


Boring Log
Water Contents and Atterberg Limits,%
Wet Unit Weight, pcf
Shear Strength, psf
Depth in Feet Below Mean Lake Level
very soft PEAT
very soft CLAY
firm PEAT and soft CLAY
soft silty CLAY
medium silty CLAY
stiff CLAY (weathered SILTSTONE)
1240
LEGEND
Plastic Limit
Liquid Limit
Natural Water Content
3" O.D. Shelby Tube Sample, No. to Left is Sample Number
Unconfined Shear Strength, psf.
Field Vane Shear Strength, psf.
Remolded Field Vane Shear Strength, psf.
VERTICAL SCALE: 1 inch = 20 feet
PROPOSED DAM – TRINIDAD ARM
GATUN LAKE, PANAMA CANAL ZONE
BORING LOG AND LABORATORY TEST DATA
DRILL HOLE DN-25-BC
W-62-147
FEBRUARY, 1963
Shannon and Wilson, Inc.
Soil Mechanics and Foundation Engineers
FIG. A-33


Boring Log
Water Contents and Atterberg Limits,%
Wet Unit Weight, pcf
Shear Strength, psf.
Depth in Feet Below Mean Lake Level
very soft PEAT
very soft CLAY
firm PEAT and soft CLAY
soft silty CLAY
medium silty CLAY
loose SAND and CLAY
very stiff CLAY (weathered CLAYSTONE)
3580
1300
1320
1780
2100
2140
3200
LEGEND
Remolded Field Vane Shear Strength, psf
Plastic Limit
Liquid Limit
Natural Water Content
3" O.D. Shelby Tube Sample, No. to Left is Sample Number
Unconfined Shear Strength, psf
Field Vane Shear Strength, psf.
2" O.D. Split Spoon Sample, No. to Left is Sample Number, No. to Right is Std. Penetration Resistance
VERTICAL SCALE: 1 inch = 20 feet
PROPOSED DAM – TRINIDAD ARM
GATUN LAKE, PANAMA CANAL ZONE
BORING LOG AND LABORATORY TEST DATA
DRILL HOLE UP-25-BC
W-62-147
FEBRUARY, 1963
Shannon and Wilson, Inc.
Soil Mechanics and Foundation Engineers
FIG. A-34


Boring Log
Water Contents and Atterberg Limits,%
Wet Unit Weight, pcf
Shear Strength, psf
Depth in Feet Below Mean Lake Level
very soft PEAT
very soft CLAY
very soft to firm PEAT and CLAY
soft silty CLAY
medium silty CLAY
medium to stiff CLAY, SAND and firm PEAT
medium to dense SAND
BASALT
LEGEND:
Plastic Limit
Liquid Limit
Natural Water Content
3" O.D. Shelby Tube Sample. No. to Left is Sample Number
Unconfined Shear Strength, psf
VERTICAL SCALE: 1 inch = 20 feet
PROPOSED DAM – TRINIDAD ARM
GATUN LAKE, PANAMA CANAL ZONE
BORING LOG AND LABORATORY TEST DATA
DRILL HOLE UP-25-CC
W-62-147
FEBRUARY, 1963
Shannon and Wilson, Inc.
Soil Mechanics and Foundation Engineers
FIG. A-35


Boring Log
Water Contents and Atterberg Limits,%
Wet Unit Weight, pcf
Shear Strength, psf
Depth in Feet Below Mean Lake Level
very soft PEAT
very soft CLAY
soft to firm PEAT and soft CLAY
soft silty CLAY
medium silty CLAY
stiff to hard CLAY (weathered SILTSTONE)
LEGEND:
Plastic Limit
Liquid Limit
Natural Water Content
3" O.D. Shelby Tube Sample. No. to Left is Sample Number
Unconfined Shear Strength, psf
VERTICAL SCALE: 1 inch = 20 feet
PROPOSED DAM – TRINIDAD ARM
GATUN LAKE, PANAMA CANAL ZONE
BORING LOG AND LABORATORY TEST DATA
DRILL HOLE UP-26-DC
W-62-147
FEBRUARY, 1963
Shannon and Wilson, Inc.
Soil Mechanics and Foundation Engineers
FIG. A-36



PROPOSED DAM – TRINIDAD ARM
GATUN LAKE, PANAMA CANAL ZONE

BORING LOG AND LABORATORY TEST DATA
DRILL HOLE DN-40-DC
W-62-147
FEBRUARY, 1963

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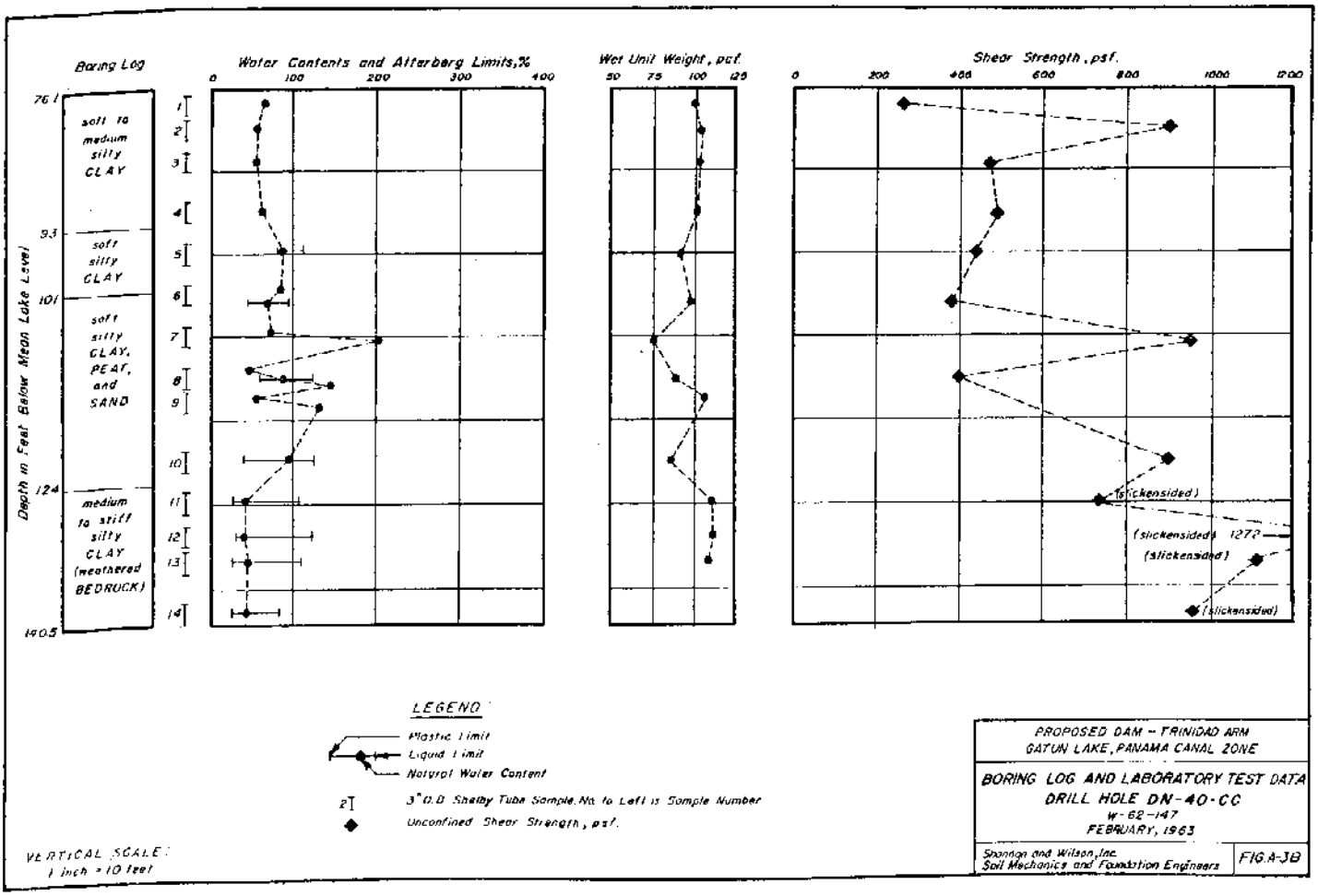
FIG. A-37



PROPOSED DAM – TRINIDAD ARM
GATUN LAKE, PANAMA CANAL ZONE

BORING LOG AND LABORATORY TEST DATA
DRILL HOLE DN-40-CC
W-62-147
FEBRUARY, 1963

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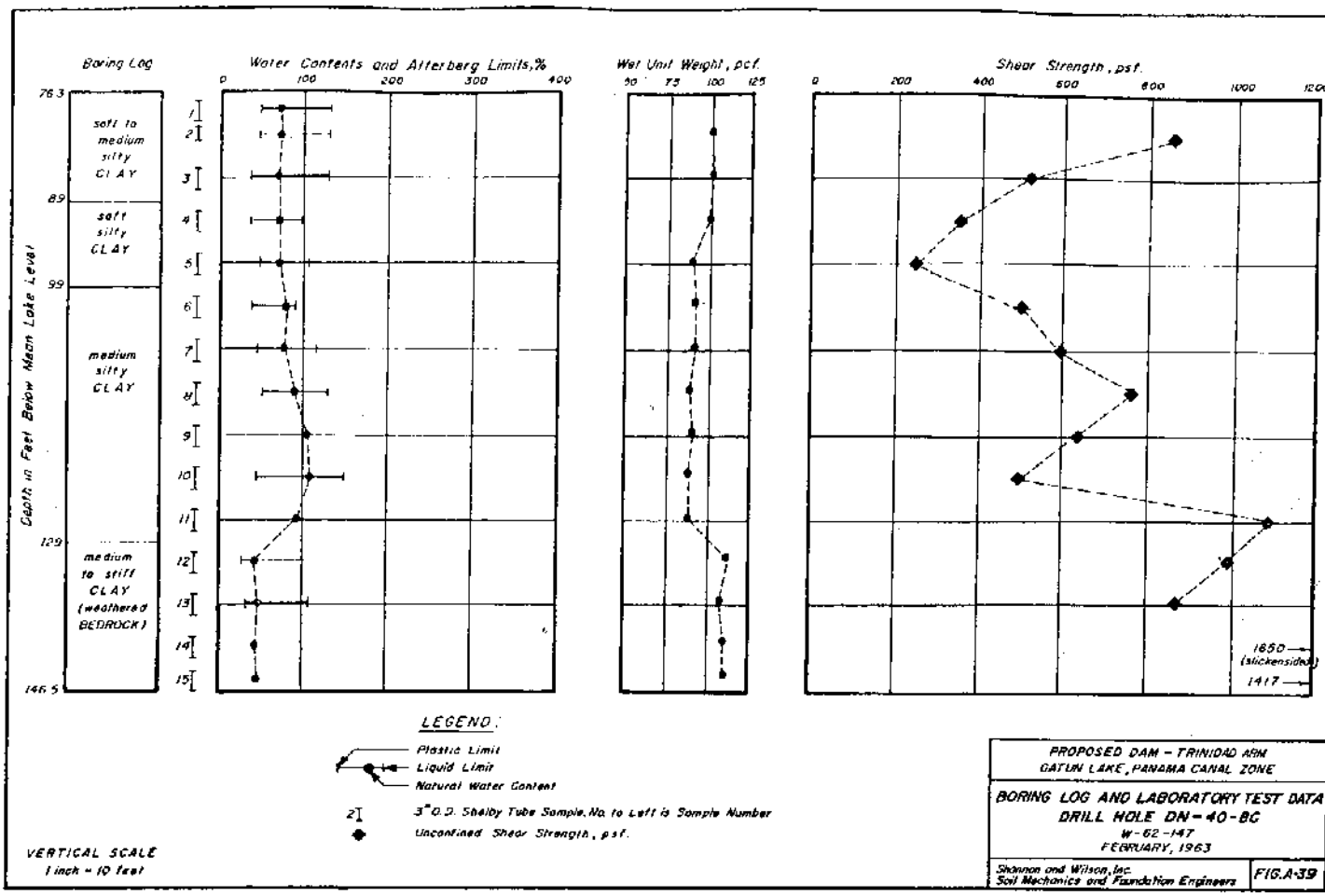
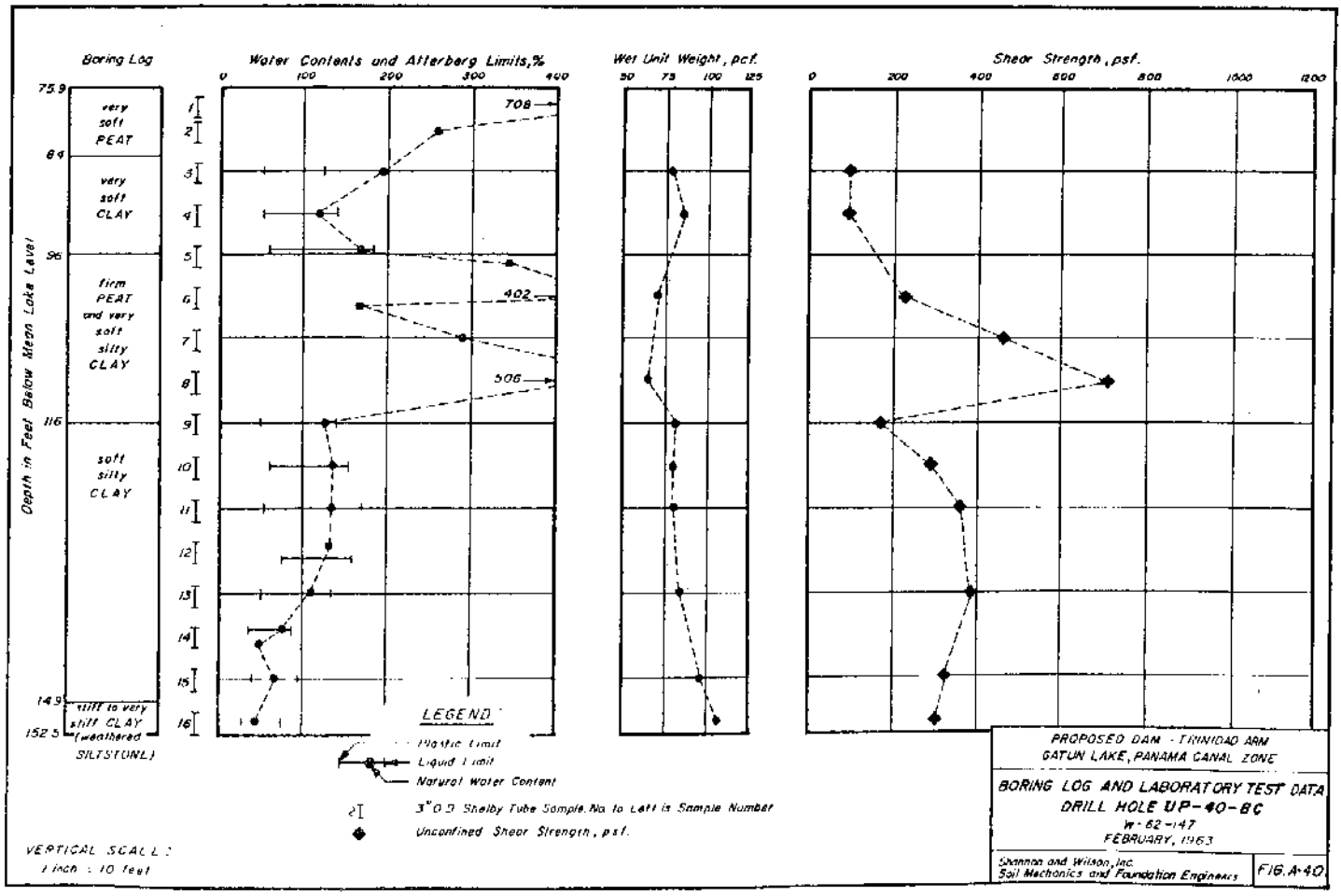
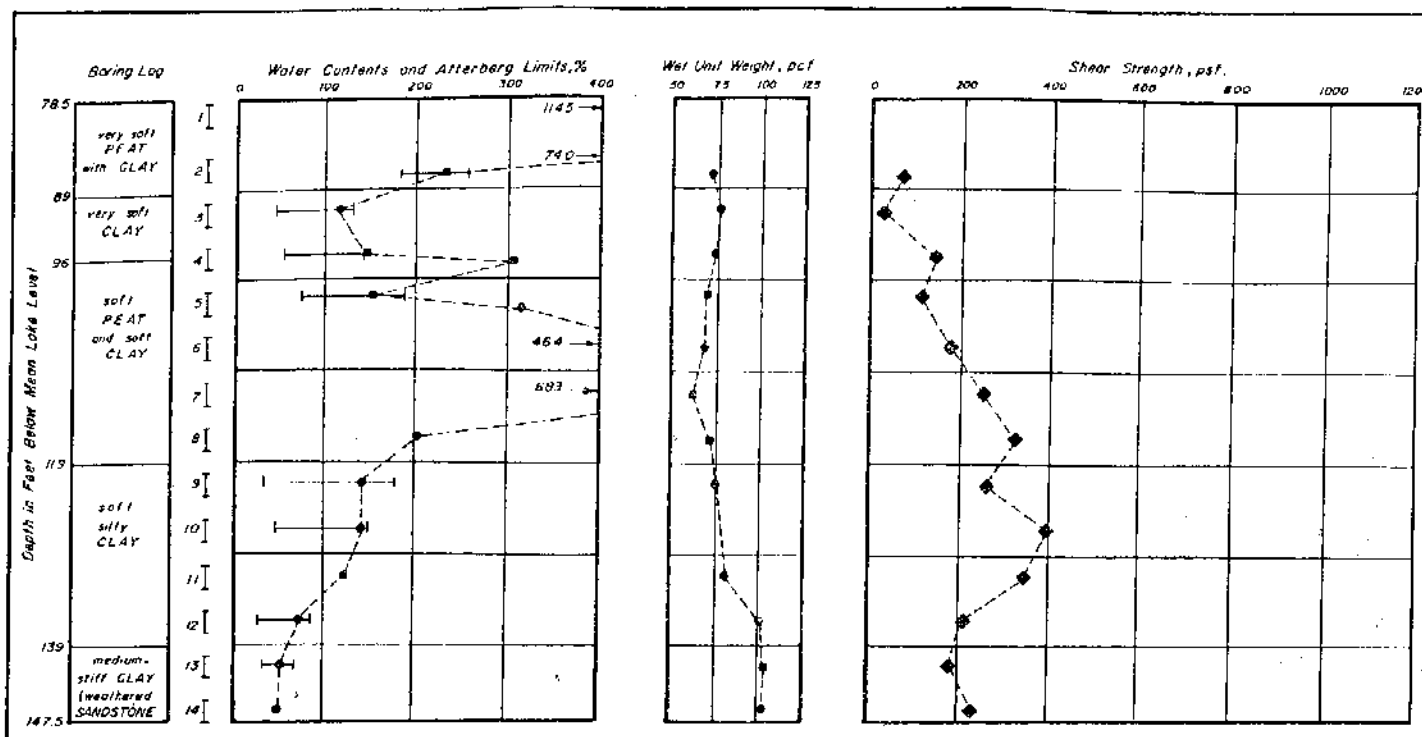
FIG. A-38


Boring Log
Water Contents and Atterberg Limits,%
Wet Unit Weight, pcf.
Shear Strength, psf.
Depth in Feet Below Mean Lake Level
76.3
soft to medium silty CLAY
89
soft silty CLAY
99
medium silty CLAY
129
medium to stiff CLAY (weathered BEDROCK)
146.5
1650 (slickensided)
1417
LEGEND:
Plastic Limit
Liquid Limit
Natural Water Content
3" O.D. Shelby Tube Sample, No. to Left is Sample Number
Unconfined Shear Strength, psf.
VERTICAL SCALE
1 inch = 10 feet
PROPOSED DAM - TRINIDAD ARM
GATUN LAKE, PANAMA CANAL ZONE
BORING LOG AND LABORATORY TEST DATA
DRILL HOLE DN-40-BC
W-62-147
FEBRUARY, 1963
Shannon and Wilson, Inc.
Soil Mechanics and Foundation Engineers
FIG. A-39


Boring Log
Water Contents and Atterberg Limits,%
Wet Unit Weight, pcf.
Shear Strength, psf.
Depth in Feet Below Mean Lake Level
75.9
very soft PEAT
84
very soft CLAY
96
firm PEAT and very soft silty CLAY
116
soft silty CLAY
149
stiff to very stiff CLAY (weathered SILTSTONE)
152.5
708
402
506
LEGEND:
Plastic Limit
Liquid Limit
Natural Water Content
3" O.D. Shelby Tube Sample, No. to Left is Sample Number
Unconfined Shear Strength, psf.
VERTICAL SCALE:
1 inch = 10 feet
PROPOSED DAM - TRINIDAD ARM
GATUN LAKE, PANAMA CANAL ZONE
BORING LOG AND LABORATORY TEST DATA
DRILL HOLE UP-40-BC
W-62-147
FEBRUARY, 1963
Shannon and Wilson, Inc.
Soil Mechanics and Foundation Engineers
FIG. A-40



LEGEND:

Plastic Limit
Liquid Limit
Natural Water Content

2 3" O.D. Shelby Tube Sample, No. to Left is Sample Number

Unconfined Shear Strength, psf

VERTICAL SCALE:
1 inch = 10 feet

PROPOSED DAM – TRINIDAD ARM
GATUN LAKE, PANAMA CANAL ZONE

BORING LOG AND LABORATORY TEST DATA
DRILL HOLE UP-40-CC
W-62-147
FEBRUARY, 1963

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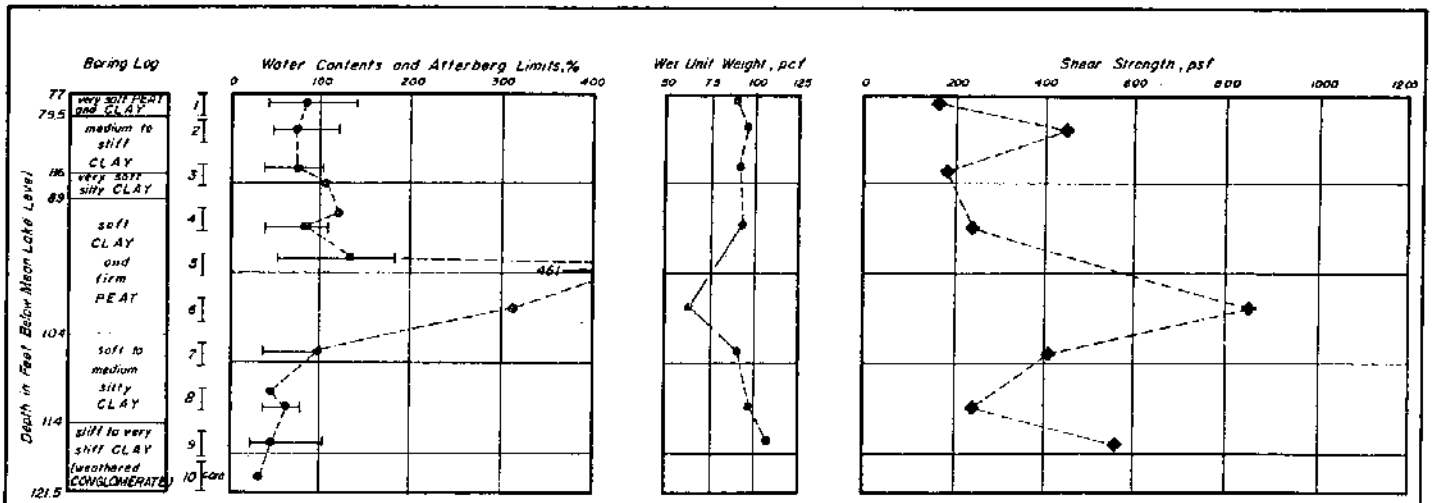
FIG. A-41



LEGEND:

Plastic Limit
Liquid Limit
Natural Water Content

2 3" O.D. Shelby Tube Sample, No. to Left is Sample Number

Unconfined Shear Strength, psf.

VERTICAL SCALE:
1 inch = 10 feet

PROPOSED DAM – TRINIDAD ARM
GATUN LAKE, PANAMA CANAL ZONE

BORING LOG AND LABORATORY TEST DATA
DRILL HOLE UP-40-DC
W-62-147
FEBRUARY, 1963

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FIG. A-42

APPENDIX B

FIELD EXPLORATIONS - EQUIPMENT AND PROCEDURES

APPENDIX B

FIELD EXPLORATIONS - EQUIPMENT AND PROCEDURES

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APPENDIX B

FIELD EXPLORATIONS – EQUIPMENT AND PROCEDURES

B-1. GENERAL.

This appendix is concerned with the details of the field explorations conducted at the dam site between 27 September 1962 and 13 December 1962. The details of the preliminary geophysical study performed during January and February 1962 are contained in Appendix A of the Tudor Engineering Company report titled "Study and Report on Increasing the Water Supply of the Panama Canal," dated June 1962.

B-2. EQUIPMENT AND PERSONNEL.

Three barge-mounted drill rigs were utilized to perform all field vane testing and soil sampling. The drill rigs consisted of two Sullivan model 37 skid rigs, each mounted on a 20-foot by 30-foot wooden-decked steel-pontoon barge, and one Sullivan model 200 truck-mounted drill rig secured to a 20-foot by 40-foot steel barge. These rigs, each with one driller and generally four helpers, were provided by the Core Drilling Unit of the Dredging Division of the Panama Canal Company. All field operations were supervised and inspected by Shannon and Wilson Incorporated personnel with occasional assisting personnel provided by the Engineering Division of the Panama Canal Company.

A 36-foot power launch, operated by one Launch Operator and one Seaman, was utilized to move the barges to new locations as well as to transport the crew between the drill barges and Gatun Landing at Gatun. One two-way radio mounted in the launch and one portable two-way radio provided communication between field operations on Gatun Lake and the field office located on the mainland. An 18-foot outboard speed boat provided by the Panama Canal Company was used for high-speed transportation to and from drilling operations.

B-3. METHOD OF LOCATING EXPLORATIONS.

Marker buoys placed by the Survey Division of the Panama Canal Company provided the initial drill barge locations. Some difficulties were encountered due to movement of the marker buoys by daily channel traffic and by stump-breaking operations, which occasionally required resetting of the buoys. The buoy locations as well as the final barge locations were determined using sextant methods. Angular measurements were taken between fixed points on Trinidad Grande and Trinidad Chiquita Islands, and fixed points on Gaucha and Ormit Islands, or points on the mainland designated Guava and "A". All locations east of station 30+00 used the points on Trinidad Grande and Chiquita Islands, and Points Guava and "A" on the mainland. All locations west of station 30+00 used the points on Trinidad Grande and Chiquita Islands, and on Gaucha and Ormit Islands. Measurements were read to the nearest minute and plotted on an angular grid chart prepared by the Survey Division. The resulting boring locations are considered accurate to within ten feet.

B-4. CONE PENETROMETER BORINGS.

The cone penetrometer equipment consisted of a three-inch diameter, 65-degree steel cone welded to a two-foot section of N-rod, and a drive hammer weighing approximately 140 pounds.

The initial procedure for performing a cone penetrometer test was to lower the drill rods, with cone tip attached until the weight of the drill rod column was supported by the underlying foundation material. Two methods of performing the cone penetrometer tests were then used; either a continuous drive method or a case-and-drive method, both using the 140-pound drive hammer with a 30-inch drop. In the continuous drive method, driving was stopped only to add another section of N-rod or to make equipment repairs or adjustments. The case-and-drive method involved a repeated sequence of driving the cone ten feet and washing casing ten feet. This was continued until a pre-determined maximum number of blows per foot was reached. In both methods the number of blows per foot were recorded. A comparison of these two methods used at

boring A-25-AXP and a typical penetrometer test profile are shown on Figure B-1.

B-5. CLASSIFICATION BORINGS.

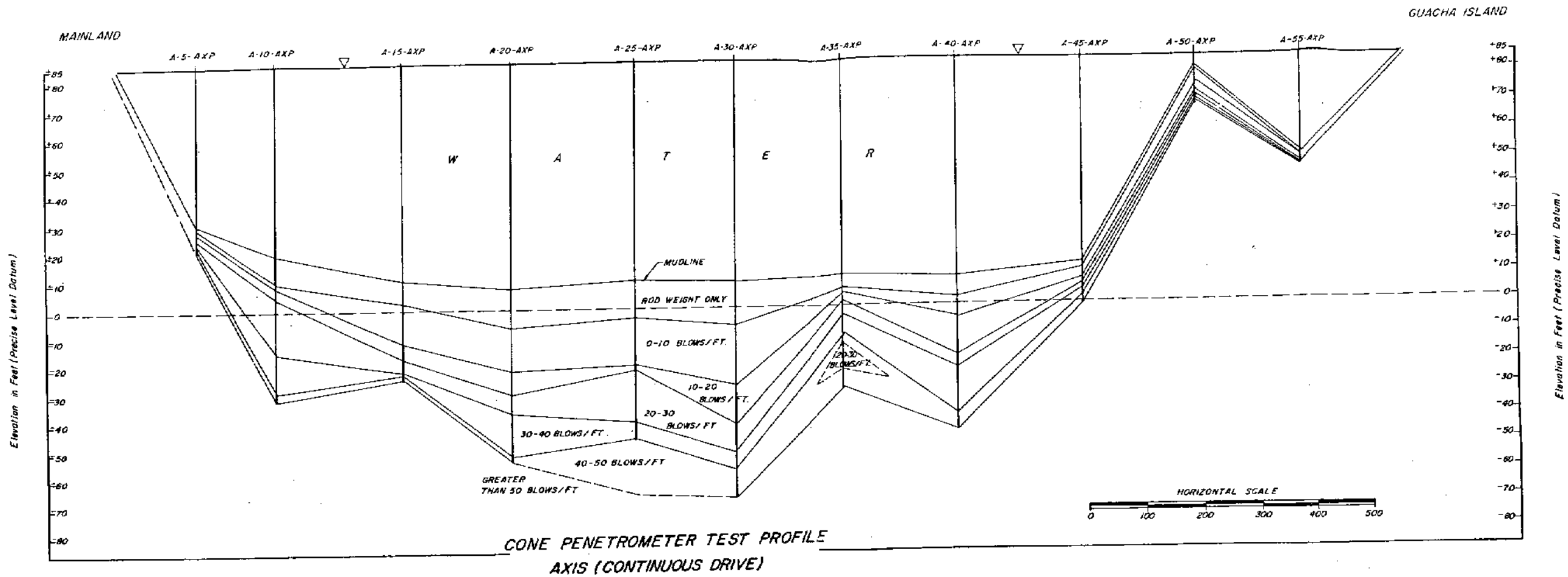
Sampling equipment used in the classification borings consisted of a conventional piston sampler and a hydraulically-operated Osterberg piston sampler, both using three-inch steel Shelby tubing. Later in the program another Osterberg sampler was obtained which replaced the conventional piston sampler. Tubing was protected from corrosion by a thin film of rust preventive oil compound applied by the tubing fabricator.

In each case the boring was advanced by washing four-inch casing to within five feet of the desired sample depth. The sampler was then lowered down the casing and pushed to sample depth. In the case of the Osterberg sampler, water pressure was applied through the drill rods to the sampler, extending the Shelby tube into the undisturbed material. The sampler was then extracted from the boring, the filled Shelby tube removed, sample classified and waxed, and a new Shelby tube installed in the sampler. The conventional piston sampler was used in nearly the same manner, except inner rods were used to hold the piston stationary while the Shelby tube was extended into the undisturbed material by hand pressure applied at the drill rig or by the hydraulic drill head of the drill rig.

Samples were taken approximately every five feet except where local conditions made it necessary to sample at smaller or slightly greater intervals. Where materials were too stiff to be sampled with piston samplers, either a two-inch drive sampler or an M-series NX core barrel was used. Sampling generally was performed to a depth of ten feet below the bottom of the Atlantic Muck materials.

B-6. UNDISTURBED BORINGS.

The sampling equipment used in the undisturbed borings consisted of both three-inch and five-inch Osterberg piston samplers. No conventional piston samplers were used in these borings.



CONE PENETROMETER TEST PROFILE
AXIS (CONTINUOUS DRIVE)

LEGEND

A-10-AXP Boring Number (See Figure 1-1)

○ Continuous Drive Blow Count

△ Blow Count at Mid-Point of Ten-Foot Drive After Washing Casing

PROPOSED DAM-TRINIDAD ARM
GATUN LAKE, PANAMA CANAL ZONE

TYPICAL CONE PENETROMETER TEST PROFILE AND PENETRATION RESISTANCE RESULTS

W-62-147
FEBRUARY, 1963

Shannon and Wilson
Soil mechanics and foundation engineers
Seattle

FIG. B-1

The sampling techniques used in the undisturbed borings were essentially the same as those used in the classification borings, except that the sampling interval was approximately continuous. In cases where both classification and undisturbed borings were made at the same station, the undisturbed boring samples were taken in the intervals not sampled in the classification boring.

The undisturbed borings were advanced to a depth of approximately fifty feet below lake bottom with the exception of the initial undisturbed boring made at Station 25+00 on the Axis, where nearly continuous sampling was performed to bedrock.

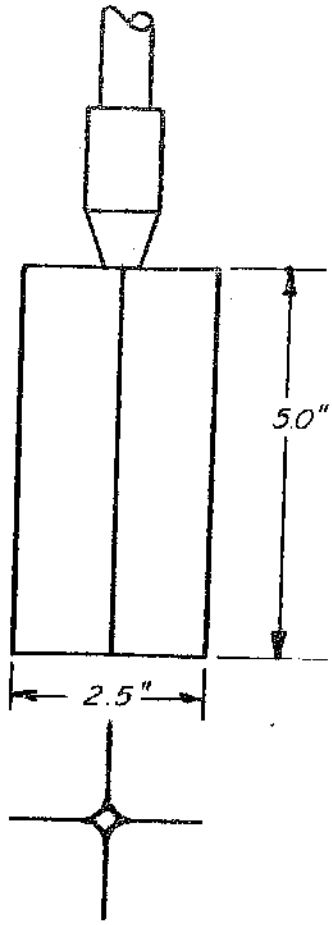
B-7. FIELD VANE SHEAR TESTS.

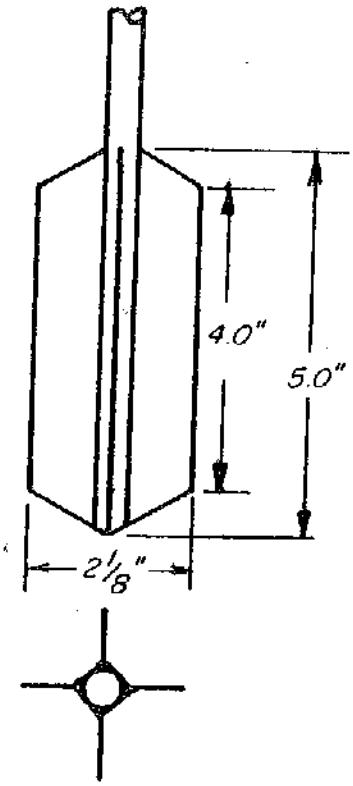
The equipment used in the vane shear tests consisted of a 50 foot-pound capacity torque wrench (readings to nearest 0.5 foot-pound), a 150 foot-pound capacity torque wrench (readings to nearest foot-pound), steel vanes (see Figure B-2), and a series of adapters, bearings, and supporting equipment.

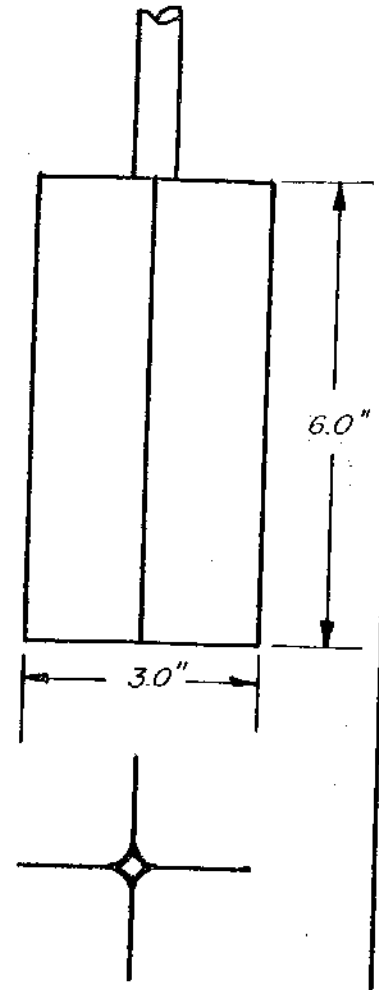
An attempt was made to use a strain controlled device with which a vane rotation of 0.1 degree per minute could be maintained. However, vertical movement of the drill barge caused by wave action tended to disengage the vane rod stem from the torque applicator assembly, and as a result the use of this type of device was discontinued.

The vane shear test borings were advanced by washing casing to within two feet of test depth. The vane attached to the A-rods was lowered through the casing to test depth. Generally, the rod weight was sufficient to penetrate the undisturbed soil; in cases where it was not, additional hand pressure was used to advance the vane. The rods were supported by a collar resting on a bearing in the top of the casing. Alignment was provided by bearings placed every twenty or thirty feet in the rods (see Figure B-2).

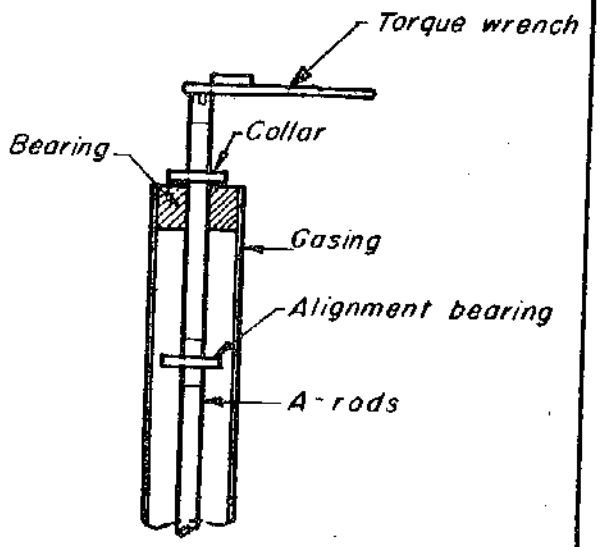
The maximum force required to rotate the vane apparatus was measured by gradually increasing the pressure on the torque wrench until a peak pressure was reached and a distinct decrease in force required to rotate the vane was noted. The vane apparatus was then rotated at a fast rate through three complete revolutions after which another torque measurement was made. The

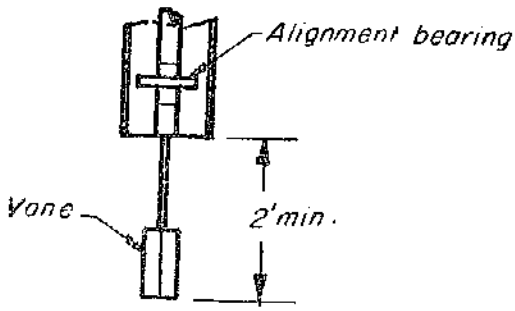






TYPICAL VANE SHEAR TEST SETUP





PROPOSED DAM - TRINIDAD ARM
GATUN LAKE, PANAMA CANAL ZONE

FIELD VANES AND TYPICAL TEST SETUP

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SHANNON & WILSON
SOIL MECHANICS & FOUNDATION ENGINEERS

FIG.B-2

apparatus was then allowed to stand undisturbed for ten minutes. After this period another torque measurement was taken to note any thixotropic effect. The vane assembly was then removed from the boring, and the rods lowered without the vane to the same depth. The maximum force required to overcome frictional resistance within the system was measured. The rods were again removed, the casing washed another five feet, and the entire test procedure repeated.

The number of field vane shear tests was limited due to operational difficulties. A few vanes were lost as apparent metal fatigue resulted in rods being sheared off in the boring. Additional difficulties in the vane shear tests were produced by rough water conditions which moved the barge and caused abnormally high frictional resistance, erratic readings, and soil disturbance.

The results of the field vane shear tests are shown on Figures A-23, A-31, A-32, A-33, and A-34, Appendix A. The plotted values of remolded shear strength were determined immediately after remolding.