Vepco

VIRGINIA ELECTRIC AND POWER COMPANY, RICHMOND, VIRGINIA 23209

May 27, 1971

(12)

U.S. Army Engineer District, Norfolk
Corps of Engineers
Foot of Front Street
Norfolk, Virginia 23210

Attention: Mr. C. S. Anderson
Geology and Foundation Section

Gentlemen:

FOUNDATION INVESTIGATION-CRANEY ISLAND

Enclosed is a copy of a foundation investigation from our Churchland Substation to our Sewells Point Substation prepared for us by Dames & Moore. Plate 1 of this report shows the location of the boring logs.

If we may be of further service, please do not hesitate to call me.

Very truly yours,

T. E. Rodgers, Jr.
Structural Engineer

Frank Elliot

Report
Foundation Investigation

Proposed 230-KV Transmission Line
Churchland Substation to Sewells Point Substation

Eastern Virginia
Virginia Electric & Power Company

NY-3
EO-2
AT-1

4718-036-10

TABLE OF CONTENTS

	PAGE
INTRODUCTION	1
GENERAL	1
DESIGN CONSIDERATIONS	2
PURPOSE	4
SCOPE OF INVESTIGATION	4
SITE CONDITIONS	5
SURFACE FEATURES	5
SUBSURFACE FEATURES	6
GROUND WATER	7
DISCUSSION AND RECOMMENDATIONS	7
GENERAL	7
PILE FOUNDATIONS	9
Steel H-Piles	9
Cast-in-Place Concrete Piles	14
Lateral Capacity	18
CORROSION PROTECTION	21
FIELD CONTROL	22
SITE ACCESSIBILITY	22
LIST OF PLATES	23
APPENDIX	

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December 7, 1970

Virginia Electric & Power Company
P. O. Box 1194
Richmond, Virginia 23219

Attention: Mr. T. E. Rodgers, Jr.

Gentlemen:

We submit herewith ten copies of our "Report, Foundation Investigation, Proposed 230-KV Transmission Line, Churchland Substation to Sewells Points Substation, Eastern Virginia, for the Virginia Electric & Power Company."

This report presents the results of the investigation conducted for the foundations of the pole structures for the transmission line. The line will extend northward from the Churchland Substation, then eastward across the mouth of the Elizabeth River, then northward to the Sewells Point Substation.

In order to expedite foundation planning and design, preliminary design criteria developed during our investigation were transmitted to Mr. T. E. Rodgers, Jr. of the Virginia Electric & Power Company as they became available.

Yours very truly,

DAMES & MOORE

Robert M. Perry

Robert M. Perry

RMP-CTM ed

Report

Foundation Investigation

Proposed 230-KV Transmission Line

Churchland Substation to Sewells Point Substation

Eastern Virginia

Virginia Electric & Power Company

INTRODUCTION

GENERAL

This report presents the results of a foundation investigation conducted for a proposed 230-KV Transmission Line which will be constructed in eastern Virginia for the Virginia Electric & Power Company. The transmission line route will extend a total of approximately seven miles from the Churchland Substation north to the Sewells Point Substation.

The line will extend from the Churchland Substation north and then east a distance of approximately three and one-half miles to a terminal enclosure at Craney Island. From Craney Island the line will extend northeast approximately one and one-half miles across the mouth of the Elizabeth River to Tanner Point. The river crossing portion of the line was not within the scope of this investigation. From Tanner Point, the line will extend approximately one and one-half miles north to a point approximately one-half mile from the Sewells Point Substation. At this point, the transmission line will extend along an existing right-of-way which was the subject of a previous investigation. The results of that investigation were submitted to the Virginia Electric & Power Company in our report* dated April 29, 1965.

A foundation investigation was conducted previously for the Churchland Substation, the results of which were presented to the Virginia Electric and Power Company in our report** dated June 10, 1968.

The route of the portion of the transmission line that was investigated for this report is shown on the Map of Area, Plate 1. The locations of the test borings along the proposed route of the transmission line are also shown. Borings H-10 through H-15 which are shown on the Plot Plan were drilled along the originally proposed route of the transmission line that was subsequently revised.

DESIGN CONSIDERATIONS

It is understood that the 230-KV transmission line considered herein will be supported on a series of single steel pole structures. It is estimated that the spans between the poles will generally be on the order of 500 to 1,000 feet.

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*"Report, Foundation Investigation, Proposed Transmission Line, Thole Street to Sewells Point Substation, Segment II, Station 94+15 to Station 254+75, Norfolk, Virginia, Virginia Electric & Power Company"

**"Report, Foundation Investigation, Proposed 230-KV Transmission Line, Portsmouth Power Station to Chuckatuck Substation, Eastern Virginia, Virginia Electric & Power Company"

The base of each pole will be subjected to a downward vertical load, a horizontal load, and an overturning moment which will be transmitted directly to the foundation. The estimated base moments for the poles range from 2,500 foot-kips to 8,000 foot-kips. Estimated horizontal loads range from 30 to 100 kips.

It is understood that the estimated loads described above represent the total of all design loads including dead load, wind load and conductor pull. Generally, for tangent structures, the anticipated live load created by hurricane wind represents the major portion of the total estimated load. For angle structures, the maximum live load considered in design would result from an accumulation of one-half inch of radial ice plus four pounds per square inch of wind pressure on the conductor and 6.4 pounds per square inch of wind pressure on the pole structure.

The estimated pole reactions indicated above include certain overload factors which are considered appropriate in the design of the pole structure. We understand that, in addition to these overload factors, a safety factor of 1.25 is incorporated to allow for possible variation in the behavior of the foundation soils from that anticipated in design. We believe that the combination of overload factors and the safety factor for foundation design will provide a reasonable ultimate factor of safety against the possibility of failure of a pole foundation under maximum loading conditions.

PURPOSE

The purpose of this investigation was as follows:

1) to explore the subsurface soil and ground water conditions along the route of the proposed transmission line;

2) to recommend suitable types of foundations for support of the transmission line poles;

3) to develop criteria for use in designing these foundations;

4) to discuss the anticipated performance of these foundations with respect to settlement and lateral deflections; and

5) to discuss possible construction problems which may be encountered during foundation installation.

SCOPE OF INVESTIGATION

The foundation investigation for the proposed transmission line consisted of a comprehensive program which included field explorations, laboratory testing and engineering analyses.

The subsurface soil and ground water conditions underlying the proposed route were explored by drilling a total of 21 test borings to depths ranging from 41 to 96 feet below the ground surface. In addition, eight borings were drilled along the portion of the route which was later revised. The locations of these borings are shown on Plate 1.

The field explorations were conducted under the technical direction and supervision of Dames & Moore Soil Mechanics Engineers. Undisturbed soil samples, suitable for laboratory testing, were extracted from each boring utilizing the Dames & Moore soil sampler. These soil samples were transported to our New York office where they were further examined and subjected to appropriate laboratory tests in order to evaluate the physical characteristics of the soils.

The results of the field explorations and laboratory tests, which provide the bases for our engineering analyses and recommendations, are presented in the Appendix to this report.

SITE CONDITIONS

SURFACE FEATURES

From the Churchland Substation north approximately one mile (Borings H-16 to H-19), the line traverses a moderately wooded low-lying area containing numerous small streams and swampy areas. The line then crosses the Craney Island Naval Reservation and passes through an area containing underground storage tanks (Borings H-19A to H-21).

The line then proceeds east across approximately one-half mile of area which is a dredged spoils dump for the U. S. Army Corps of Engineers. This area (Borings H-22 and H-23) has been filled to approximately Elevation +5 or lower with soft silty and clayey soils which afford very little support for men and equipment. Future plans call for filling the area to Elevation +18 with dredged spoils. The filling will be done subsequent to the construction of the proposed transmission line poles.

After crossing the spoils area, the line passes adjacent to the Craney Island Naval Reservation and extends to the terminal enclosure on the west bank of the mouth of the Elizabeth River (Borings H-24, H-25 and H-7 through H-9). The line in this area traverses what appears to be a dike constructed of a mixture of sand and clay soils.

After crossing the Elizabeth River, the line emerges at the terminal enclosure at Tanner Point. From the terminal enclosure at Tanner Point, the line heads north through the Hampton Roads Army Terminal. The southernmost one-half mile of the line (Borings H-6, H-4) passes through a lowlying area covered with high weeds. The line emerges from this area onto relatively flat level ground.

The topography along the entire route of the transmission line is quite flat with surface elevations ranging from approximately 0 feet to +20 feet. The variations in surface elevation are gradual except where the line crosses eroded stream channels or small local swampy areas, or where the line passes through areas filled to higher grades.

SUBSURFACE FEATURES

The northern portion of the line and much of the southern portion will be established on filled areas (or areas to be filled). The fill soils cannot be readily identified as some have been in place for many years and are similar to the natural soils in the area.

Beneath the fill soils are found variable thicknesses of Pleistocene and Recent deposits of sand, silt and clay. The upper sands are generally moderately compact. The upper silts and clays are soft to medium.

The deeper soils are Miocene deposits of marine origin. The deeper silts and clays are generally stiff to very stiff. The deeper sands are generally moderately compact to compact and contain varying amounts of shell fragments.

More detailed descriptions of the soil conditions encountered in the test borings are shown on the Log of Borings presented in the Appendix to this report.

GROUND WATER

Water level observations made in the test borings during our field investigation indicate that the ground water table is close to the ground surface along the route of the proposed transmission line. This is reasonable because of the low ground surface elevation and the proximity of the site to the River. We believe that during periods of appreciable precipitation or during abnormally high tides, the ground water may rise to and above the ground surface.

DISCUSSION AND RECOMMENDATIONS

GENERAL

It is planned to support the proposed transmission line on single steel poles. In order to resist the design overturning forces, the pole foundations have to be installed at appreciable depths. The combination of granular soils or soft cohesive soils and high water table along most of the route would require expensive dewatering operations to enable the installation of spread foundations. The varying nature of the fill in many areas would also require special consideration if spread foundations were utilized. Consequently, we have not performed detailed analyses for spread foundations, grillages, or other types of foundations constructed in open excavations. Based upon the soil conditions along the transmission line route, it is our opinion that friction pile foundations would be the most suitable type of support for the pole structures.

Large diameter concrete cylinder piles are generally an economical type of foundation for pole structures because only one pile is required for each pole. However, the relatively poor near-surface soils will not offer sufficient lateral resistance to allow the use of this type of foundation. In addition, the highly variable nature of the fill soils that exist in many areas make it difficult to predict conditions other than at the exact locations where the borings were drilled. Consequently, a group of batter piles with a pile cap will be required over most of the line.

Design criteria presented herein consist of ultimate axial capacities for steel H-piles and cast-in-place concrete piles. For areas where the upper soils are competent, lateral capacities are also presented for these foundation types and for steel H-piles and cast-in-place concrete piles with concrete upper sections 36 inches and 54 inches in diameter to a depth of 10 feet.

Axial capacities presented herein are ultimate capacities (safety factor equal to 1) which represent the load at which failure of the foundation would be expected to occur. We believe that the combination of the design overload factors and the foundation safety factor would provide a reasonable factor of safety against the possibility of failure of a pole foundation under maximum loading conditions.

PILE FOUNDATIONS

Steel H-Piles: Subsurface conditions along the route of the proposed transmission line are variable. For the purpose of presenting design criteria for steel H-piles, we have divided the route into a number of sections according to the engineering properties of the subsurface materials. Our recommended ultimate downward and upward axial capacities for steel H-piles for the different sections of the transmission line are presented in Table I on the following page.

The values presented in this table are ultimate axial capacities (factor of safety equal to 1) and should be used in design only with appropriate overload and safety factors. The design criteria presented in Table I are for nominal 12-inch H-pile sections. Axial capacities for steel H-pile sections other than 12 inches can be computed by multiplying the values presented in Table I by the following factors:

NOMINAL PILE SIZE (inches)	CAPACITY FACTOR
8-BP	0.67
10-BP	0.83
14-BP	1.17

No reduction in the capacity of an individual pile will be required for group action provided that the minimum center-to-center pile spacing is three times the nominal pile size.

TABLE I

PILE FOUNDATION DESIGN DATA
(12-inch steel H-piles)

LOCATION	DEPTH OF PENETRATION	ULTIMATE AXIAL CAPACITY (F.S.=1) (kips) UPLIFT	DOWNWARD
Churchland Substation to Station 18+28 (Boring 27A from previous report dated June 10, 1968)	30	not recommended	
	40	not recommended	
	50	80	86
	60	120	128
	70	170	180
Station 18+28 to Station 99+67 (Borings H-16 through H-21)	30*	27	29
	40*	50	54
	50*	80	86
	60	110	118
	70	145	155
	80	187	200
Station 106+20** (Boring H-22)	90	45	45
	100	120	120
	110	200	200
Station 112+90** (Boring H-23)	80	40	40
	90	110	110
	100	190	190

*These depths of penetration are not recommended from Station 67+15 to Station 93+74 (Boring 19A).

**Refer to the discussion in the subsequent paragraphs for special considerations that will be required at Borings H-22 and H-23. At these locations, the indicated depth of penetration is below the present existing grade of +5 feet and not the final planned grade of +18 feet.

TABLE I (continued)

LOCATION	DEPTH OF PENETRATION	ULTIMATE AXIAL CAPACITY (F.S.=1) (kips) UPLIFT	DOWNWARD
Station 119+03 to	30	not recommended	
Station 159+70	40	not recommended	
(Borings H-24 and H-25)	50	not recommended	
	60	not recommended	
	70	not recommended	
	72	145	155
	80	187	200
Station 159+70 to	30	not recommended	
Station 176+98	40	not recommended	
(Borings H-9 and H-8)	50	80	83
	60	115	123
	70	170	180
Station 176+98 to	30	not recommended	
Station 194+68	40	not recommended	
(Boring H-7)	50	not recommended	
	60	not recommended	
	70	125	135
	80	180	195
Station 219+20 to Station	30	not recommended	
240+00 (Borings H-6 and	40	not recommended	
H-5)	50	not recommended	
	60*	75	78
	70*	115	118
	80*	165	168
Station 240+00 to Station	30	not recommended	
252+44 (Borings H-4 and	40	not recommended	
H-3)	50	not recommended	
	60*	110	114
	70*	155	159
	80*	205	210
Station 252+44 to existing	30	not recommended	
Sewells Point-Taussig	40	not recommended	
Tranmission Line (Borings	50	50	56
H-2 through H-1A)	60	80	88
	70	125	135
	80	180	192

*Estimated pile settlement is about one to two inches.

We believe that pile foundations installed to depths indicated in Table I will perform satisfactorily with settlements within tolerable limits. In areas where the settlements appear marginal, we have noted the estimated magnitude of the settlement. In some areas, we have recommended minimum penetration depths for steel H-piles. At these locations, we believe that pile foundations installed to a depth shallower than the minimum recommended depth could experience excessive settlement.

Conditions at Borings H-22 and H-23 require special precautions and design considerations. We understand that this area will be covered by 13 feet of fill dredged from nearby areas. This fill will experience appreciable settlement due to the consolidation of the underlying soft clay which will result in large downdrag (axial) loads on all piles and bending loads on batter piles. We estimate that the axial downdrag loads will be as follows:

NOMINAL PILE SIZE (inches)	DOWNDRAG LOAD IN KIPS Boring H-22	Boring H-23
8	77	70
10	96	88
12	115	105
14	135	123

The axial capacities for 12-inch steel H-piles shown in Table I for Borings H-22 and H-23 have been reduced to take into account the anticipated downdrag load. However, when determining stresses in the piles, the anticipated downdrag loads tabulated above must be considered and added to the structural loads. Axial capacities of piles in this area at H-22 and H-23 will differ before and after fill placement, due to downdrag and increased overburden pressures. The capacities shown in Table I are for the most critical condition.

At the locations of Borings H-22 and H-23, foundations also will have to be designed to support the weight of the fill to be placed directly over the pile cap. We recommend that it be assumed that the fill will weigh 120 pounds per cubic foot. Consequently, if 13 feet of fill is placed, the foundations must be designed to support an additional 1,560 pounds per square foot of pile cap area.

It is believed that, in several areas, steel H-piles may encounter high driving resistance in the compact sand and shell strata which are in some cases partially cemented. Past experience with pile installations in areas with similar subsurface conditions indicates that some difficulty in driving piles along this route should be anticipated. Since the piles must develop resistance to uplift as well as downward forces, they will require relatively high frictional capacity in addition to end bearing. The piles therefore should be driven to the depths indicated in Table I and should not be governed by driving resistance criteria. Jetting of the piles should not be permitted. Because of the possibility of high driving resistance, we recommend that a higher energy pile hammer be utilized than normally would be required for the capacities which we have indicated. The experience gained from previously installed pile foundations for other transmission lines in the area indicates that a hammer with a high frequency (90 to 110 blows per minute) facilitates driving of the piles.

Cast-in-Place Concrete Piles: Our recommended ultimate downward and uplift capacities for cast-in-place concrete piles for the different sections of the transmission line route are presented in Table II on the following page.

The values presented in Table II are ultimate axial capacities (factor of safety equal to 1) and should be used in design only with appropriate overload and safety factors. Design data presented are for 18-inch diameter cast-in-place concrete piles. Axial capacities for cast-in-place concrete piles other than 18 inches in diameter can be computed by multiplying the values presented in Table II by the following factors:

PILE DIAMETER (inches)	CAPACITY FACTOR
16	0.89
20	1.11
24	1.33

No reduction in the capacity of an individual pile will be required for group action provided that the minimum center-to-center pile spacing is three times the pile diameter.

TABLE II

PILE FOUNDATION DESIGN DATA
(18-inch cast-in-place concrete piles)

LOCATION	DEPTH OF PENETRATION (feet)	ULTIMATE AXIAL CAPACITY (F.S.=1) (kips) UPLIFT	ULTIMATE AXIAL CAPACITY (F.S.=1) (kips) DOWNWARD
Churchland Substation to Station 18+28 (Boring 27A) from previous report dated 6/10/68.	30	not recommended	
	40	not recommended	
	50	180	186
	60	270	278
Station 18+28 to Station 99+67 (Borings H-16 through H-21)	30	62	64
	40	122	126
	50	200	206
	60	285	293
except Station 67+15 to Station 93+74 (Boring H-19A)	30	not recommended	
	40	not recommended	
	50	not recommended	
	60	175	183
	70	275	285
Station 99+67 to Station 119+03 (Borings H-22 and H-23)		Concrete piles not recommended in this area because of downdrag loads	
Station 119+03 to Station 140+00 (Boring H-24)	30	not recommended	
	40	not recommended	
	50	not recommended	
	60	not recommended	
	72	192	194
	80	290	300
Station 140+00 to Station 159+70 (Boring H-25)	30	not recommended	
	40	not recommended	
	50	not recommended	
	60	175	183
	70	275	285
Station 159+70 to Station 176+98 (Borings H-9 and H-8)	30	not recommended	
	40	not recommended	
	50	107	110
	60	163	171
	70	242	252

TABLE II (continued)

LOCATION	DEPTH OF PENETRATION (feet)	ULTIMATE AXIAL CAPACITY (F.S.=1) (kips) UPLIFT	DOWNWARD
Station 176+98 to	30	not recommended	
Station 194+68	40	not recommended	
(Boring H-7)	50	not recommended	
	60	not recommended	
	70	197	207
	80	292	305
Station 219+20 to	30	not recommended	
Station 240+00	40	not recommended	
(Borings H-6 and H-5)	50	not recommended	
	60*	157	160
	70*	191	194
	80*	236	239
Station 240+00 to	30	not recommended	
Station 278+18	40	not recommended	
(Borings H-4 through	50	not recommended	
H-1)	60*	180	183
	70*	270	274
Station 278+18 to	30	not recommended	
Existing Sewells Point-	40	112	116
Taussig Transmission	50	169	175
Line (Boring H-1A)	60	253	261

*Estimated pile settlement is about one to two inches for Borings H-6 through H-3.

We believe that pile foundations installed to the depths indicated in Table II will perform satisfactorily with settlements within tolerable limits. In areas where the settlements appear marginal, we have indicated the estimated magnitude of the settlement. In some areas, we have recommended minimum penetration depths for cast-in-place concrete piles. At these locations, we believe that pile foundations installed to a depth shallower than the minimum recommended depth could experience excessive settlement.

We do not recommend that concrete piles be used in the area of Borings H-22 and H-23 because they would be subject to very high bending forces due to the combined lateral and downdrag loads.

The criteria presented for cast-in-place concrete piles apply to piles installed by any one of several procedures. We understand that the method of installation employed by Girdler Foundation & Exploration Company is being considered for this project. In this method, a rotary drilling rig is used to drill a hole of the same diameter as the finished pile to the required depth. An expansive clay slurry is used to prevent the walls of the hole from caving. A prefabricated cage of reinforcing steel is set into the predrilled hole. Concrete is then placed by the tremie method which displaces the stabilizing slurry from the predrilled hole. This procedure has several advantages in that no casing or dewatering is required. However, some degree of risk is involved since the possibility exists that the concrete may become mixed with either the drilling mud or soils which have sloughed from the sides of the hole. Girdler Foundation & Exploration Company has demonstrated the necessary competence and experience to satisfactorily install these foundations. Other local contractors also may be capable of performing this method of installation. However, we recommend that the installation be observed by experienced engineering personnel.

<u>Lateral Capacity:</u> The lateral capacity of a pile foundation is largely influenced by the soil conditions in the upper soil strata. Over most of the route of the proposed transmission line, the upper soils are composed of either soft clay, soft silt, or fill of a variable nature. These soils will not provide adequate lateral support. In these areas which extend from Borings H-1 through Boring H-22, we recommend that batter piles be utilized to resist the lateral loads. The lateral resistance of a batter pile can be considered to be the horizontal component of the axial downward or uplift capacities presented in Tables I and II.

Because of the large areal subsidence anticipated in the area of Borings H-22 and H-23, batter piles at these locations will be subjected to high bending forces. To minimize bending of these piles, we recommend that the piles be installed on a batter of 1 horizontal to 12 vertical or steeper. Piles installed at a 1 horizontal to 12 vertical batter should be designed for bending loads (perpendicular to the pile axis) varying uniformly from 125 pounds per square foot of pile flange area at the top of the pile to 350 pounds per square foot of pile flange area at the elevation of the bottom of the soft soils (Elevation -63 in Boring H-22 and Elevation -53 in Boring H-23).

Batter piles at the locations of Borings H-22 and H-23 should be sufficiently embedded in the pile cap to develop a fixed end condition. The lower end of the piles may be assumed to be fixed at penetrations of five feet into the deep sand strata. At Boring H-22, they may be considered fixed at Elevation -68 and at Boring H-23 they may be considered fixed at Elevation -58 (penetrations of 73 and 63 feet below the existing ground surface, respectively).

Lateral capacities for the section of the route where batter piles are not necessary (Churchland Substation to Boring H-21) are shown in Table III. The indicated lateral capacities are the recommended design loads for a deflection at the ground surface of approximately one-half inch or less. The lateral loads which will result in an actual shear failure of the foundation soils are considerably greater than the capacities indicated in Table III, which are based on deflection criteria.

The indicated capacities are based on the assumption that the head of the pile is free to rotate. If the pile is fixed at the ground surface, preventing rotation, we believe that the lateral capacity of the pile can be increased by a factor of 50 percent.

The lateral capacities indicated in Table III are for 12-inch steel H-piles and 18-inch and 24-inch diameter cast-in-place concrete piles and for these piles with 36-inch diameter and 54-inch diameter cast-in-place concrete upper sections to a depth of 10 feet. The lateral capacities indicated for piles with 36-inch diameter and 54-inch diameter built-up upper sections can be increased by a factor of 1.25 if the upper section is constructed with a square cross section having a side dimension equal to the indicated diameter.

TABLE III

PILE FOUNDATION DESIGN DATA

LATERAL CAPACITY*
(kips)

Section of Route	12-Inch Steel-H Pile	18-inch Diameter Concrete Pile	12-inch Steel-H pile or 18-inch Diameter Concrete pile with a 36-inch diameter top, 10 feet deep	12-inch Steel-H pile or 18-inch Diameter Concrete pile with a 54-inch diameter top 10 feet deep	24-Inch Diameter Concrete Pile	24-inch Diameter Concrete pile with a 36-inch diameter top, 10 feet deep	24-inch Diameter Concrete pile with a 54-inch diameter top, 10 feet deep
Churchland Substation to Station 99+67** (Borings 27A [of June 10, 1968 report] to H-21)	6	8	12	15	10	13	16

*Capacity for a deflection of one-half inch or less at the ground surface.

**Refer to the text of this report for special procedures to be used if soft surface soils encountered. At the location of Boring H-19A, batter piles are recommended.

At any tower locations where surface peat is encountered, the peat should be removed and replaced with compacted granular fill. Where the surficial peat extends to a depth of more than a few feet or where the magnitude of lateral loading is great, it may be necessary and/or more economical to utilize batter piles to develop the necessary resistance to lateral thrust.

The available passive resistance of the soil adjacent to the pile caps may be considered in evaluating the resistance of the foundation to the lateral loading. The available passive resistance of the soil acting on the side of the pile cap can be computed by assuming that the soil acts as a fluid with a density of 150 pounds per cubic foot. The backfill soils adjacent to the pile caps should be placed in layers approximately 9 inches in thickness and each layer should be compacted to a density of at least 95 percent of the maximum density determined by the AASHO T-180-64 Test Procedure (Modified AASHO) in order to provide the passive resistance indicated above.

CORROSION PROTECTION

Based on our expereince with similar soils we investigated for nearby transmission lines, we believe that some of the soils possess concentrations of soluble sulphates which could adversely affect concrete while other soils have low pH values which could have a corrosive effect on steel.

We recommend that either Type II sulphate resistant Portland cement be used for all concrete foundations and a rich concrete (high cement content) mix be employed or Type V cement be used.

Several methods are available to protect steel piles from the effects of the potentially corrosive soils. These include:

1) protecting the piles with an epoxy coating;

2) using a heavier pile section to allow for possible corrosion; and

3) using corrosion resistant steel pile sections such as Mariner steel developed by U. S. Steel Corporation.

FIELD CONTROL

The foundation design criteria we have developed are based upon subsurface information which is interpolated from widely spaced borings. Consequently, the subsurface conditions at pole locations may vary from the assumed conditions. In view of this, we recommend that control by engineering personnel be exercised during foundation installation in order to correlate conditions between borings and assure that foundations are properly installed.

SITE ACCESSIBILITY

The route of the proposed transmission line is generally clear, with the exception of the moderately heavily wooded areas along the southern portion of the line. Most of the route would be accessible except for the localized marsh areas along the southern portion of the line and the U. S. Army Corps of Engineers spoils area. Because of the wet and soft conditions in these latter areas, surface soils will not withstand heavy traffic loads. Consequently, the mobilization of heavy construction equipment will be quite difficult in these areas and the use of construction mats or fill roadways will be necessary to mobilize equipment to the pole locations.

It has been observed that the condition of the ground surface in many areas deteriorates badly after periods of rainfall. We believe that if heavy rain occurs during construction, access to a number of locations and travel along the right-of-way on the southern portion of the line will be difficult for heavy equipment.

-oOo-

The following Plate and Appendix are attached and complete this Report:

Plate 1	-	Plot Plan (Showing Route of Transmission Line and Boring Locations)
Appendix	-	Field Explorations and Laboratory Tests

Respectfully submitted,

DAMES & MOORE

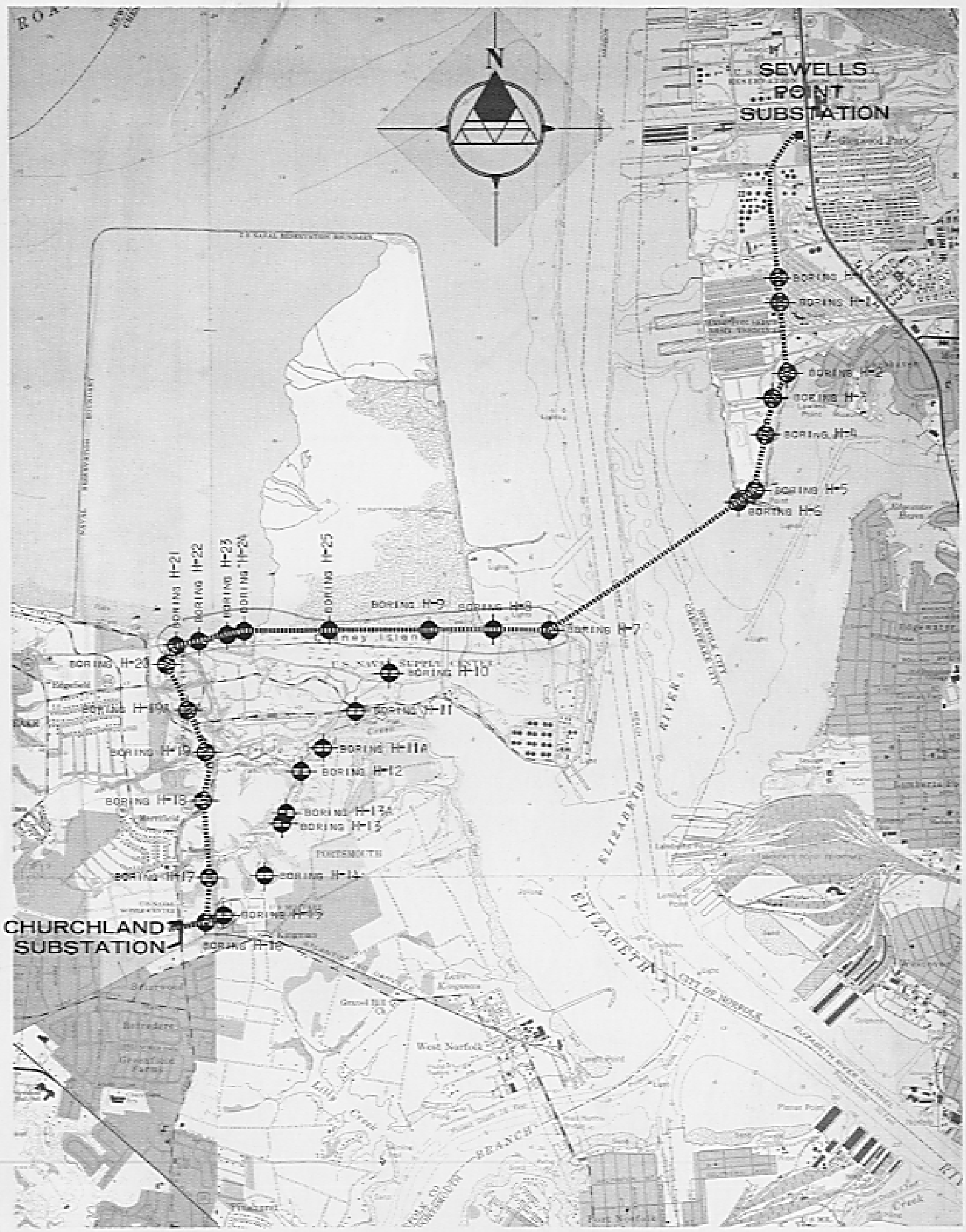
Robert M. Perry

Robert M. Perry

Charles T. Melick

Charles T. Melick

RMP-CTM ed



PLOT PLAN

SHOWING ROUTE OF TRANSMISSION LINE AND BORING LOCATIONS

NOTE:

BORINGS H-10 THROUGH H-15 WERE DRILLED ALONG THE ORIGINAL TRANSMISSION LINE ROUTE WHICH WAS SUBSEQUENTLY REVISED.

REFERENCE:

THIS MAP WAS PREPARED USING PORTIONS OF THE FOLLOWING U.S.G.S QUADRANGLES - NORFOLK NORTH, VA., 1965, BOWERS HILL, VA., 1965, NORFOLK SOUTH, VA., 1955 AND NEWPORT NEWS SOUTH, VA. 1964.

DAMES & MOORE

PLATE I

FILE 4718-026
BY ____ DATE FEB. '69
CHECKED BY ____ DATE ____

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

APPENDIX

FIELD EXPLORATIONS AND LABORATORY TESTS

FIELD EXPLORATIONS

The subsurface conditions underlying the proposed transmission line route were explored by drilling a total of 21 test borings to depths ranging from 41 to 96 feet below the ground surface. In addition, eight borings were drilled along a portion of the route which was later revised. The locations of all borings are shown on the Plot Plan, Plate 1.

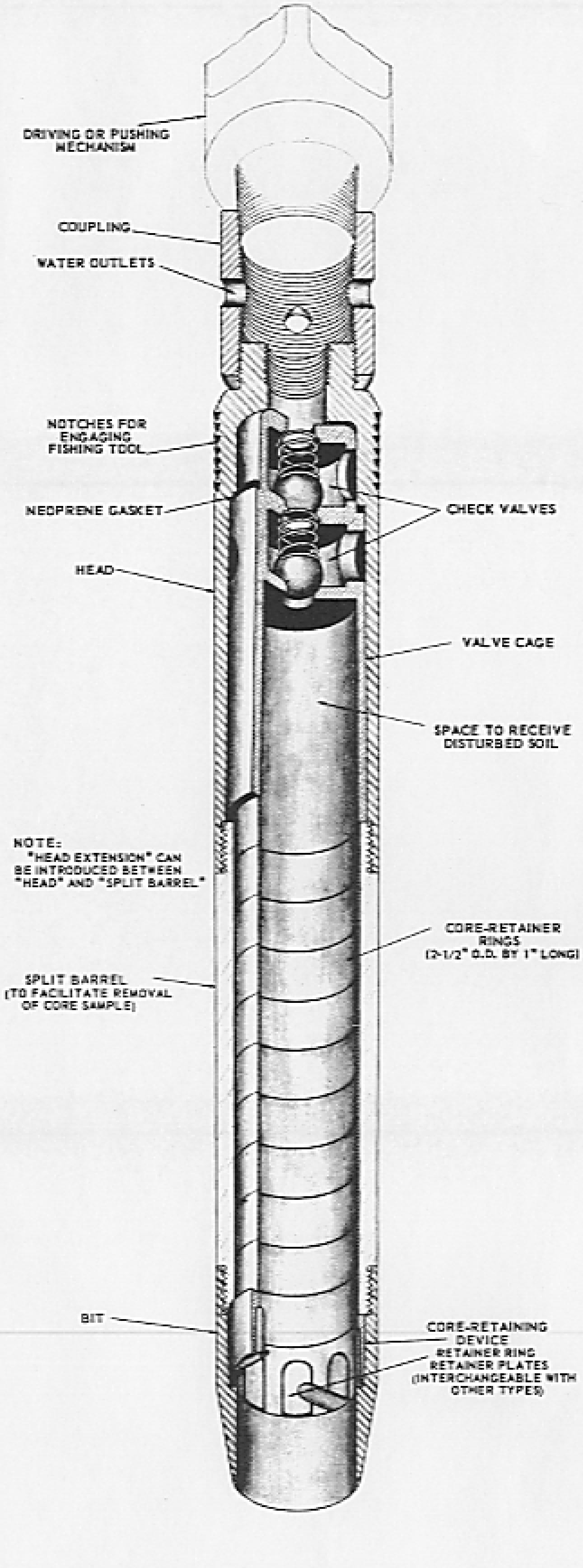
The test borings were drilled approximately four inches in diameter. Undisturbed soil samples, suitable for laboratory testing, were extracted from the test borings using the Dames & Moore Soil Sampler illustrated on Page A-2 of this Appendix. The Dames & Moore Soil Sampler is three and one-quarter inches in outside diameter and approximately two and one-half inches in inside diameter.

The test borings for this investigation were drilled utilizing two different types of drilling equipment. Where boring locations were accessible to truck-mounted equipment, the borings were drilled utilizing truck-mounted Failing 1500 rotary drilling rigs. Where boring locations were in areas inaccessible to truck-mounted equipment, the borings were drilled utilizing a skid-mounted wash-boring rig.

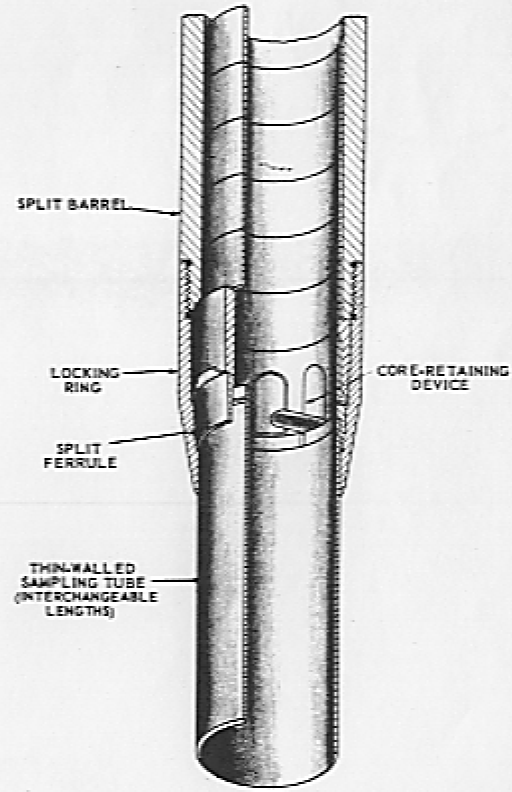
The test borings were drilled utilizing an expansive clay slurry. This slurry (driller's mud) prevents caving of the boring during the drilling operations.

SOIL SAMPLER TYPE U

FOR SOILS DIFFICULT TO RETAIN IN SAMPLER
U. S. PATENT NO. 2,318,062



ALTERNATE ATTACHMENTS



DAMES & MOORE

417.9(6-61)

The field explorations were performed under the technical direction and supervision of Dames & Moore Soil Mechanics Engineers. A description of the soil conditions encountered in each boring and detailed observations of surface conditions, ground water conditions, and other factors of importance in this investigation were recorded by our representatives.

Detailed descriptions of the soils encountered in the borings are presented on the Logs of Borings in this Appendix, Plates A-1A through A-10. The soils were classified in accordance with the Unified Soil Classification System as described on Plate A-2.

A discussion of site conditions, including topographic features, subsurface soil conditions and ground water conditions is presented in the text of this report.

The locations of the test borings were referenced to the station of the transmission line route. The borings were located in the field by our representatives using Plan and Profile survey drawings provided by the Virginia Electric and Power Company in conjunction with survey stakes in the field and topographic maps published by the U. S. Geologic Survey.

For the portion of the line west and south of Tanner Point, the ground surface elevations were obtained from the Plan and Profile drawings. For the portion of the line north of Tanner Point, Plan and Profile drawings were not available at the time of our investigation. However, ground surface elevations were generally between Elevations 0 and +10.

The ground surface elevations at the boring locations, where available, are presented above each log of boring. Data concerning the observed depth to ground water in each boring and the date on which the boring was completed are presented beneath each boring log.

LABORATORY TESTS

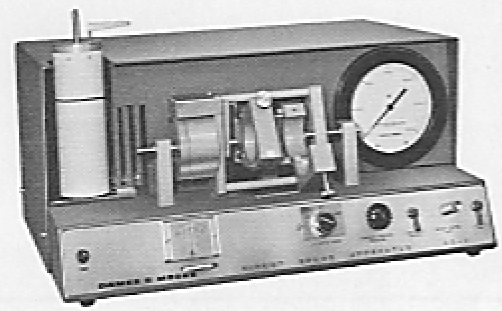
Strength Tests: Selected representative soil samples recovered from the borings were tested to evaluate their strength characteristics. Direct shear tests were performed on samples of granular soils in order to evaluate their angle of internal friction. These tests were performed in the manner described on Page A-5. Unconfined compression tests were performed on samples of cohesive and semi-cohesive soils in order to evaluate their shearing strength. The method of performing these tests is described on Page A-6. Determinations of the field moisture content and dry density of the soil were made in conjunction with each strength test. The results of the strength tests and the corresponding moisture and density determinations are presented on the Log of Borings. The method of presenting test data is described in the Key to Test Data on Plate A-2.

Moisture and Density Determinations: In addition to the field moisture content and dry density determinations made in conjunction with the strength tests, independent moisture and density tests were performed on undisturbed soil samples for correlation purposes. The results of all moisture and dry density determinations are presented on the Log of Borings.

-oOo-

Method Of Performing Direct Shear and Friction Tests

Direct shear tests are performed to determine the shearing strengths of soils. Friction tests are performed to determine the frictional resistances between soils and various other materials such as wood, steel, or concrete. The tests are performed in the laboratory to simulate anticipated field conditions.



DIRECT SHEAR TESTING
& RECORDING APPARATUS

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

Direct Shear Tests

A three-inch length of the sample is tested in direct double shear. A constant pressure, appropriate to the conditions of the problem for which the test is being performed, is applied normal to the ends of the sample through porous stones. A shearing failure of the sample is caused by moving the center ring in a direction perpendicular to the axis of the sample. Transverse movement of the outer rings is prevented.

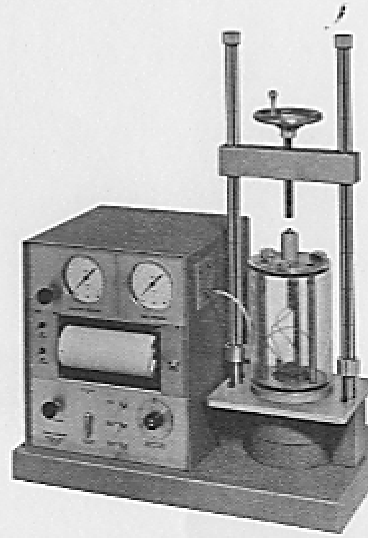
The shearing failure may be accomplished by applying to the center ring either a constant rate of load, a constant rate of deflection, or increments of load or deflection. In each case, the shearing load and the deflections in both the axial and transverse directions are recorded and plotted. The shearing strength of the soil is determined from the resulting load-deflection curves.

Friction Tests

In order to determine the frictional resistance between soil and the surfaces of various materials, the center ring of soil in the direct shear test is replaced by a disk of the material to be tested. The test is then performed in the same manner as the direct shear test by forcing the disk of material from the soil surfaces.

METHODS OF PERFORMING UNCONFINED COMPRESSION AND TRIAXIAL COMPRESSION TESTS

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



TRIAXIAL COMPRESSION TEST UNIT

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

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

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

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

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

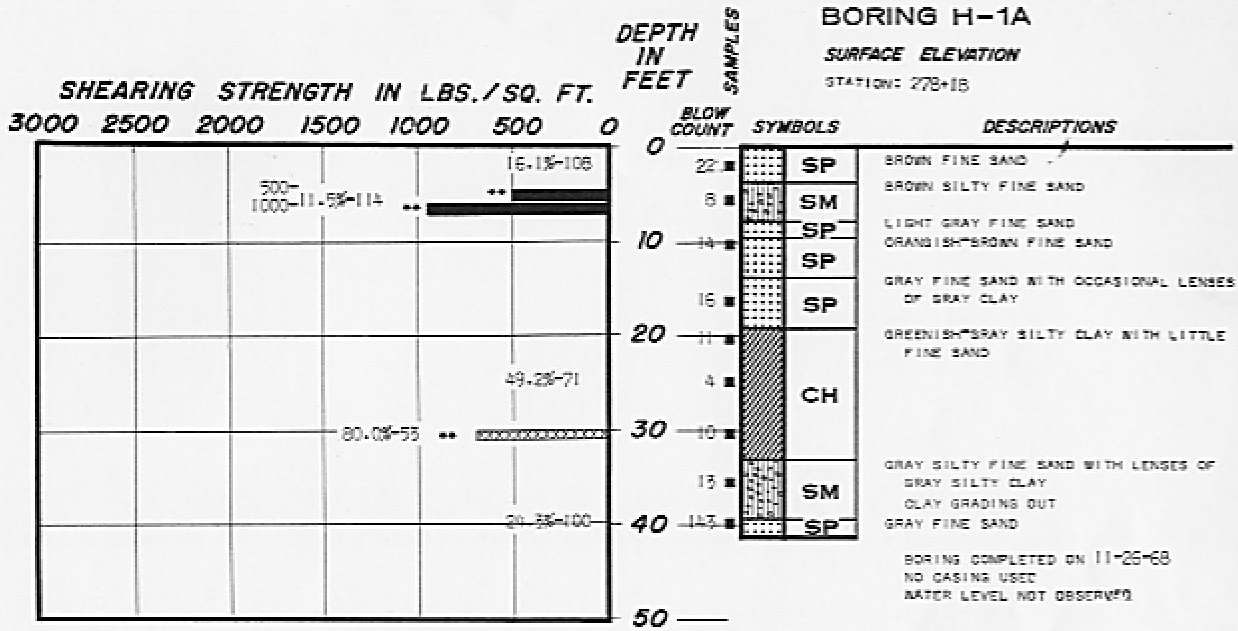
<u>CONSOLIDATED-UNDRAINED:</u> THE SAMPLE IS ALLOWED TO CONSOLIDATE FULLY UNDER THE APPLIED CONFINING PRESSURE PRIOR TO THE START OF THE TEST. THE VOLUME CHANGE IS DETERMINED BY MEASURING THE WATER AND/OR AIR EXPELLED DURING CONSOLIDATION. NO DRAINAGE IS PERMITTED DURING THE TEST AND THE STRESSES WHICH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

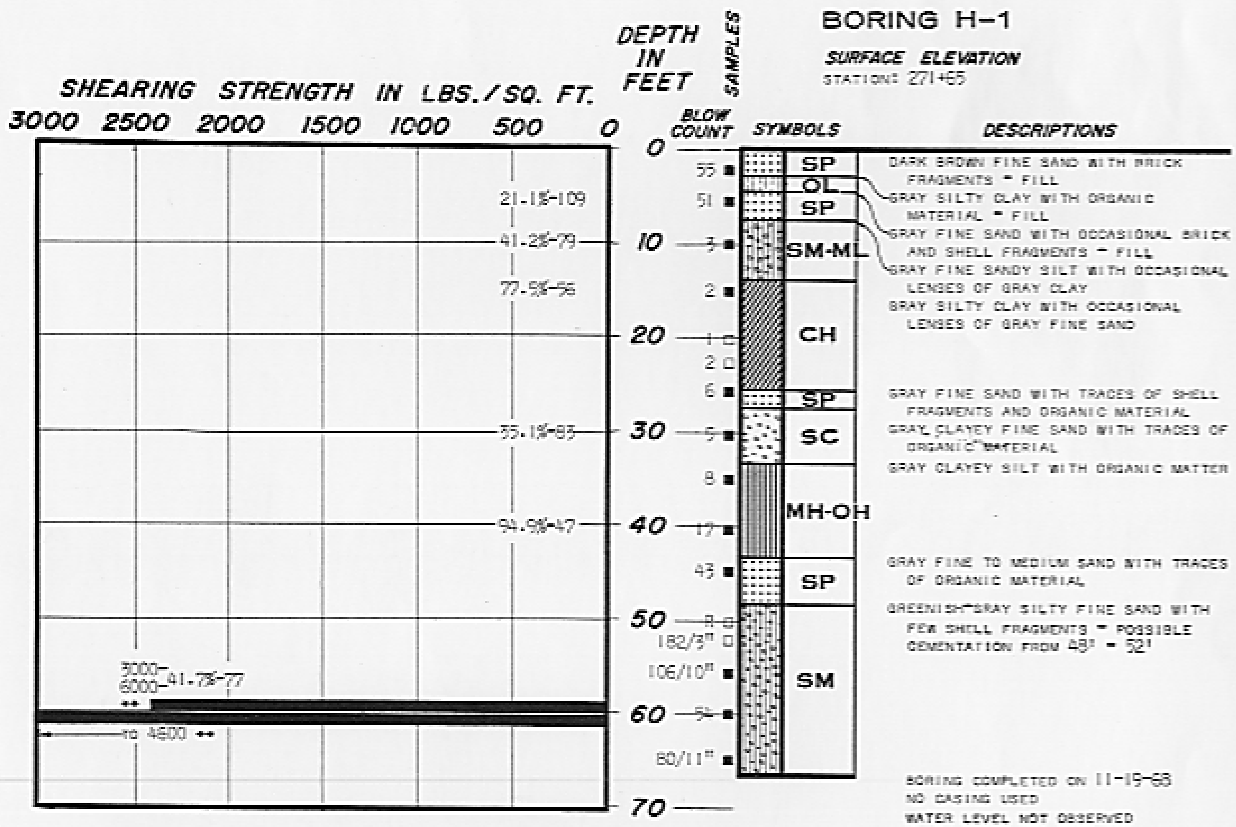
<u>DRAINED:</u> THE INTERGRANULAR STRESSES IN A SAMPLE MAY BE MEASURED BY PERFORMING A DRAINED, OR SLOW, TEST. IN THIS TEST THE SAMPLE IS FULLY SATURATED AND CONSOLIDATED PRIOR TO THE START OF THE TEST. DURING THE TEST, DRAINAGE IS PERMITTED AND THE TEST IS PERFORMED AT A SLOW ENOUGH RATE TO PREVENT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

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

The following Plates are attached and complete this Appendix:

Plate A-1A	-	Log of Borings (Borings H-1A and H-1)
Plate A-1B	-	Log of Boring (Boring H-2)
Plate A-1C	-	Log of Boring (Boring H-3)
Plate A-1D	-	Log of Boring (Boring H-4)
Plate A-1E	-	Log of Boring (Boring H-5)
Plate A-1F	-	Log of Boring (Boring H-6)
Plate A-1G	-	Log of Boring (Boring H-7)
Plate A-1H	-	Log of Borings (Borings H-8 and H-9)
Plate A-1I	-	Log of Boring (Boring H-25)
Plate A-1J	-	Log of Boring (Boring H-24)
Plate A-1K	-	Log of Borings (Borings H-23 and H-22)
Plate A-1L	-	Log of Borings (Borings H-21 and H-20)
Plate A-1M	-	Log of Borings (Borings H-19A and H-19)
Plate A-1N	-	Log of Borings (Borings H-18 and H-17)
Plate A-1O	-	Log of Boring (Boring H-16)
Plate A-2	-	Unified Soil Classification System





LOG OF BORINGS

NOTES:

THE FIGURES UNDER THE COLUMN LABELLED "BLOW COUNT" INDICATE THE NUMBER OF BLOWS REQUIRED TO ADVANCE THE DAMES AND MOORE SAMPLER A DISTANCE OF ONE FOOT USING A 140 POUND WEIGHT FALLING 30 INCHES. THE DAMES AND MOORE SAMPLER IS 3¼" O.D. AND APPROXIMATELY 2½" I.D. THE LETTER "P" INDICATES THAT THE SAMPLER AND RODS WERE ADVANCED BY BEING PUSHED BY HAND OR THAT THE SAMPLER ADVANCED UNDER THE WEIGHT OF THE HAMMER AND RODS.

ELEVATIONS REFER TO THE USC & GS MEAN SEA LEVEL DATUM.

THE DISCUSSION IN THE TEXT OF THE REPORT IS NECESSARY FOR A PROPER UNDERSTANDING OF THE SUBSURFACE CONDITIONS.

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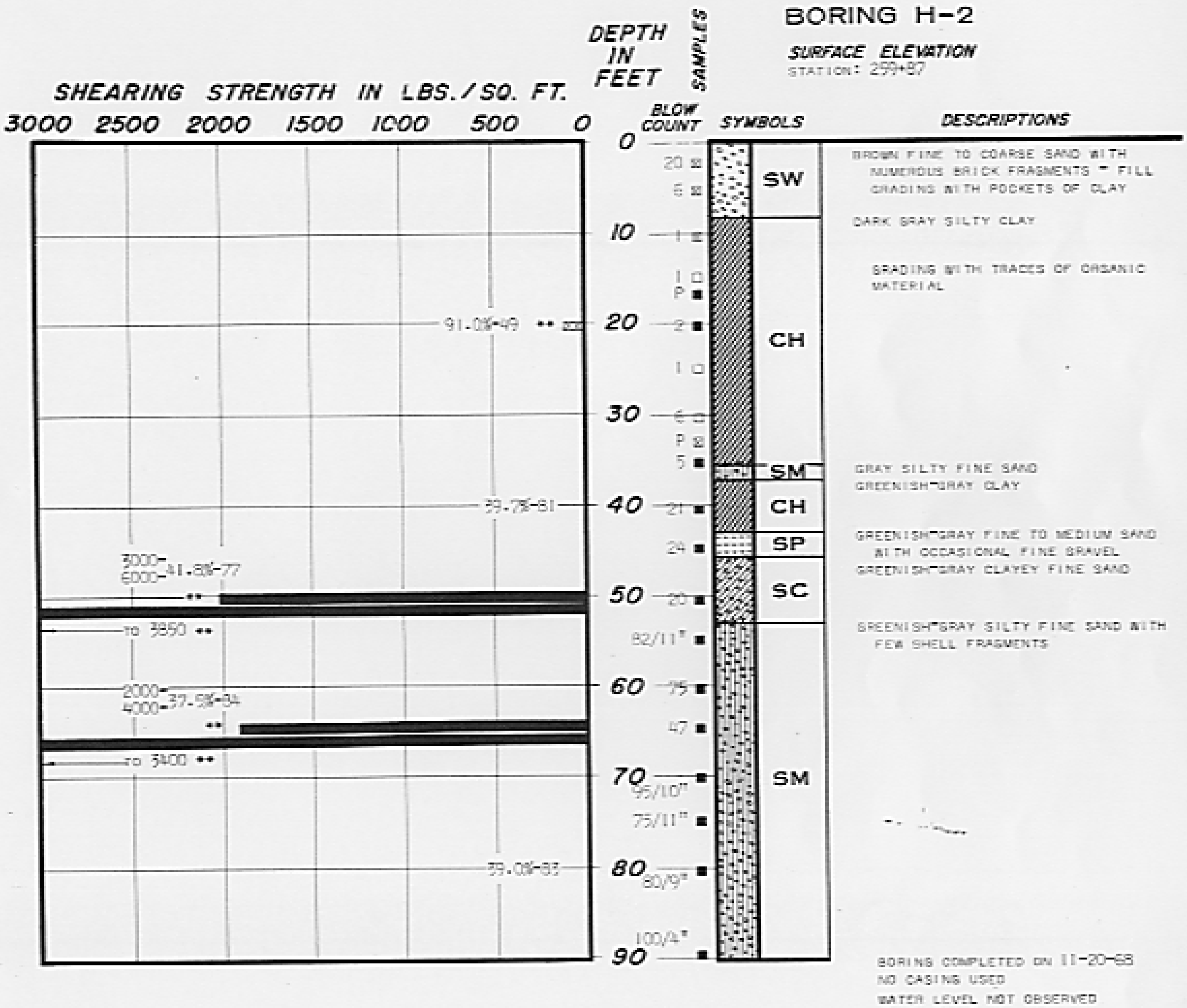
PLATE A-1A

FILE 4718-036
BY FT DATE 1/6/69
CHECKED BY CM DATE
REVISIONS
BY DATE
BY DATE
PLATE OF



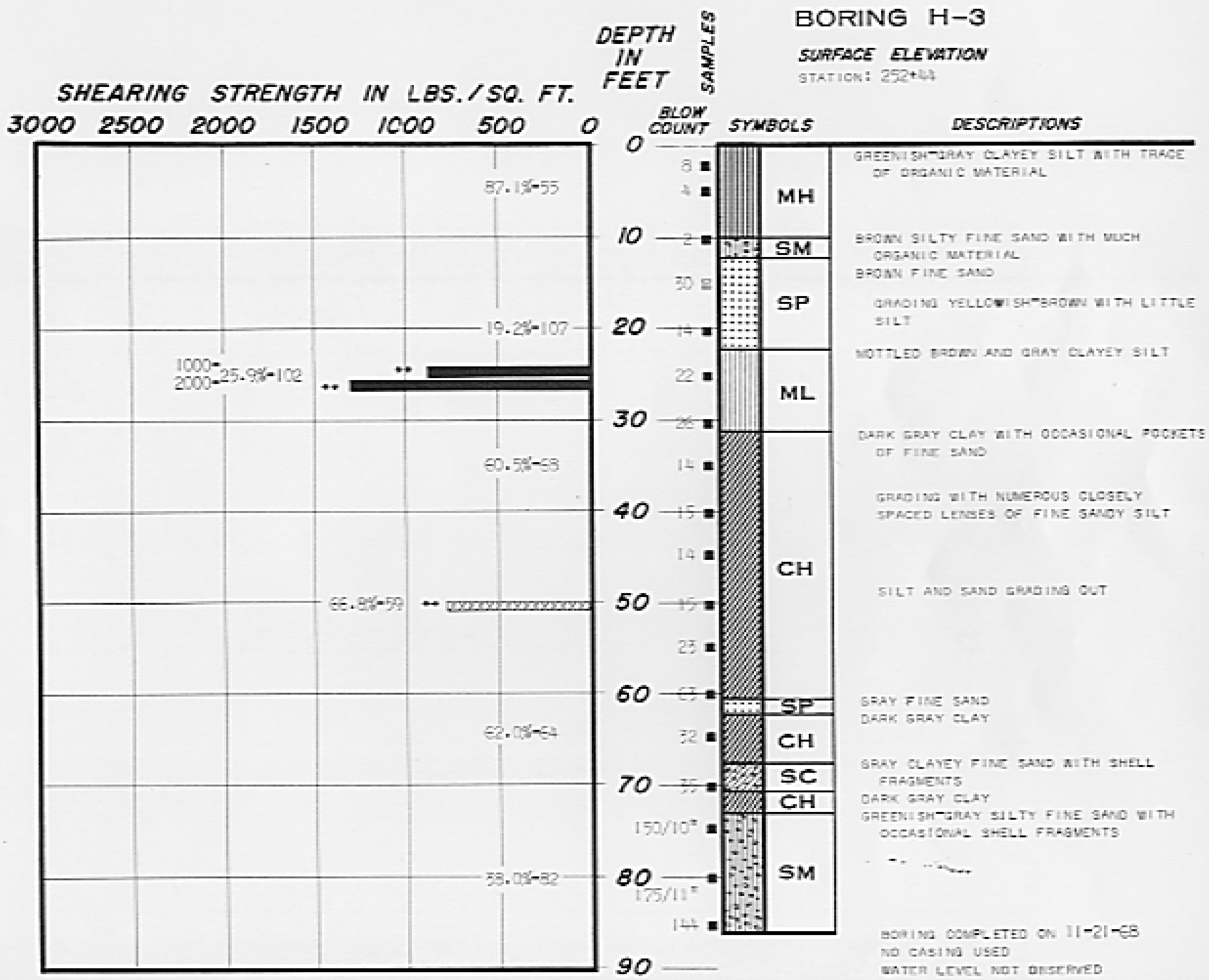
LOG OF BORING

DAMES & MOORE

PLATE A-1B

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

FILE 4718-036
BY F.F. DATE 1/6/69
CHECKED BY C.M. DATE 1/7/69



LOG OF BORING

DAMES & MOORE

PLATE A-1C

FILE 4718-034
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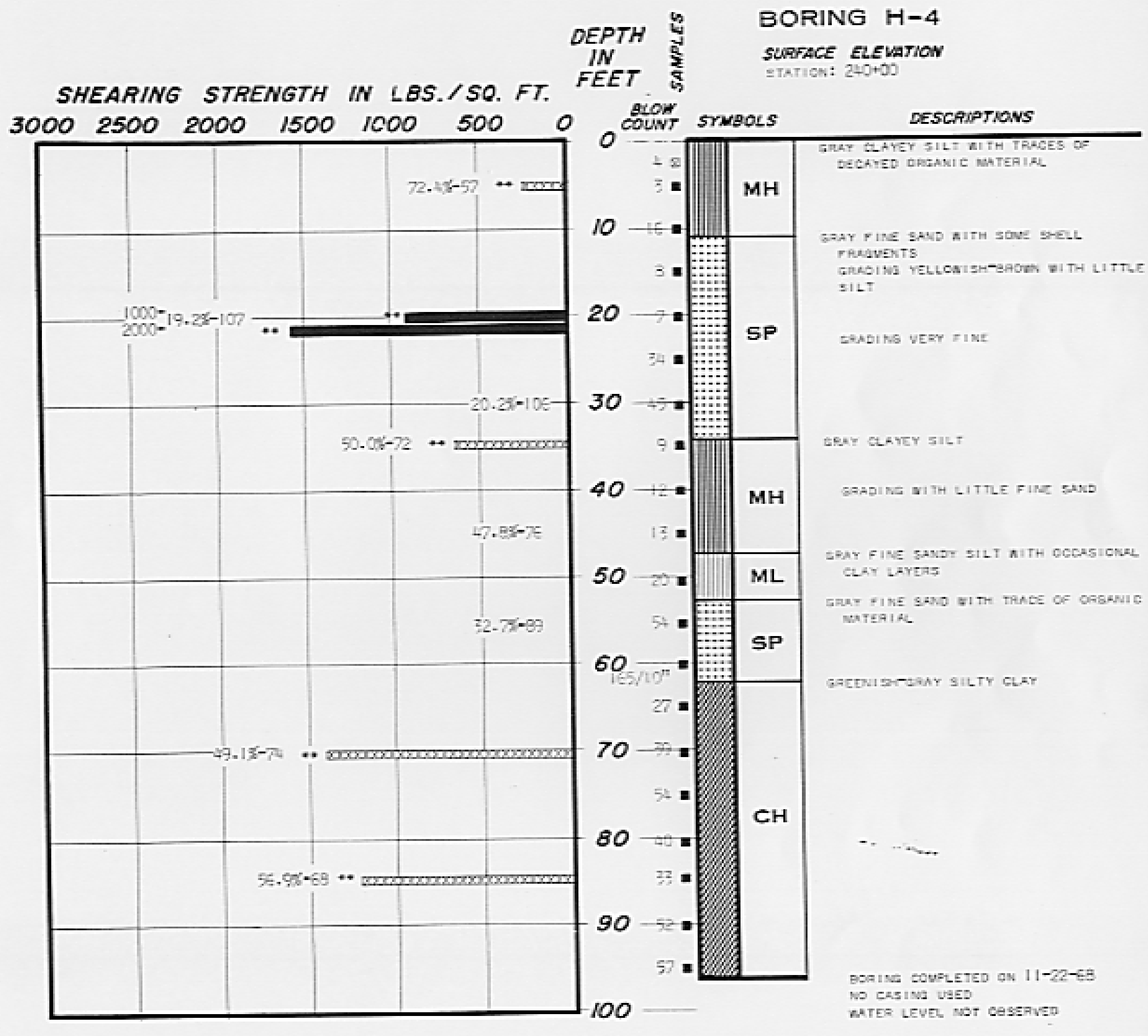
CHECKED BY CMM DATE 1/8/69
REVISIONS
BY ____ DATE ____
BY ____ DATE ____
PLATE ____ OF ____



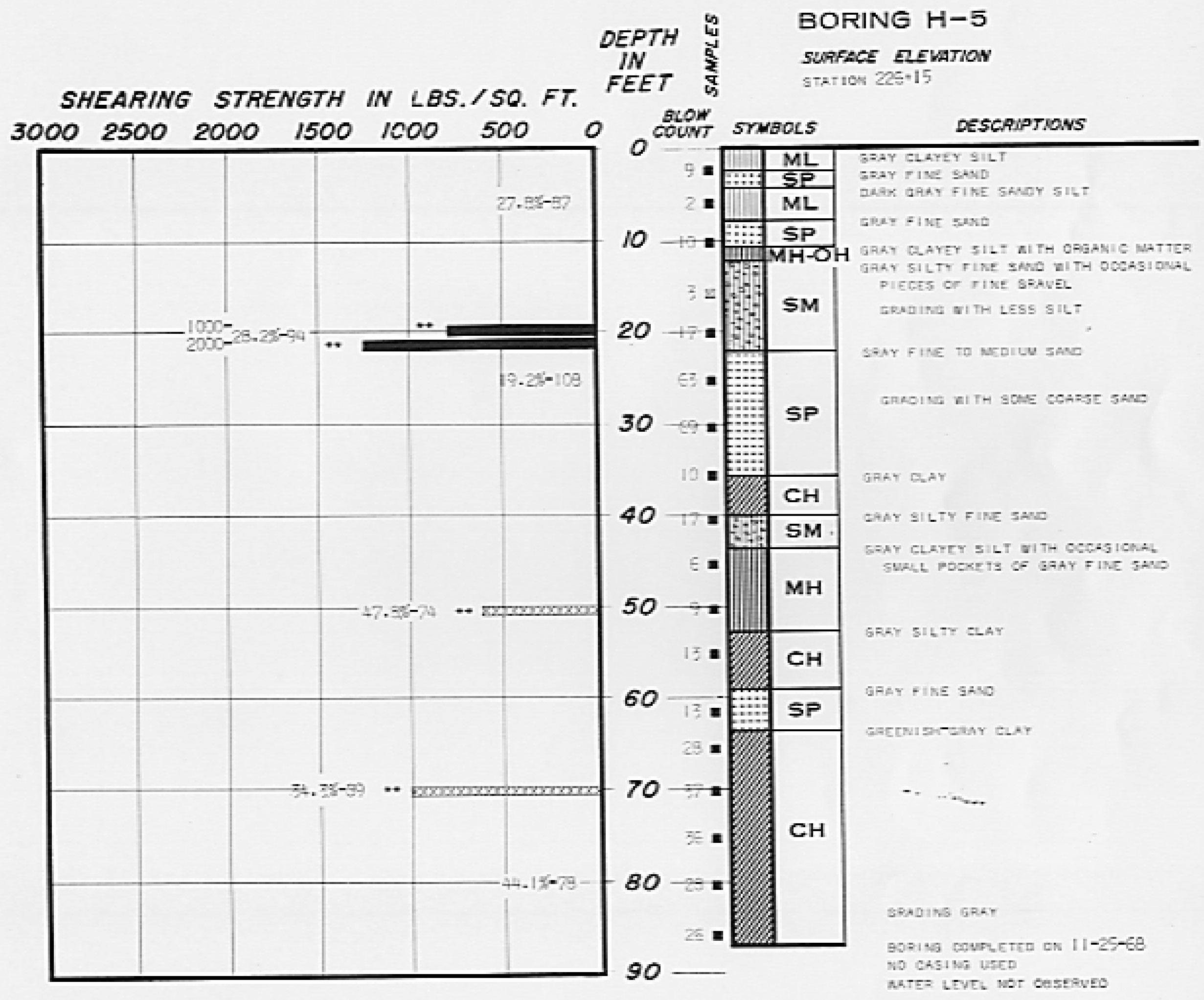
LOG OF BORING

DAMES & MOORE

PLATE A-1D

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

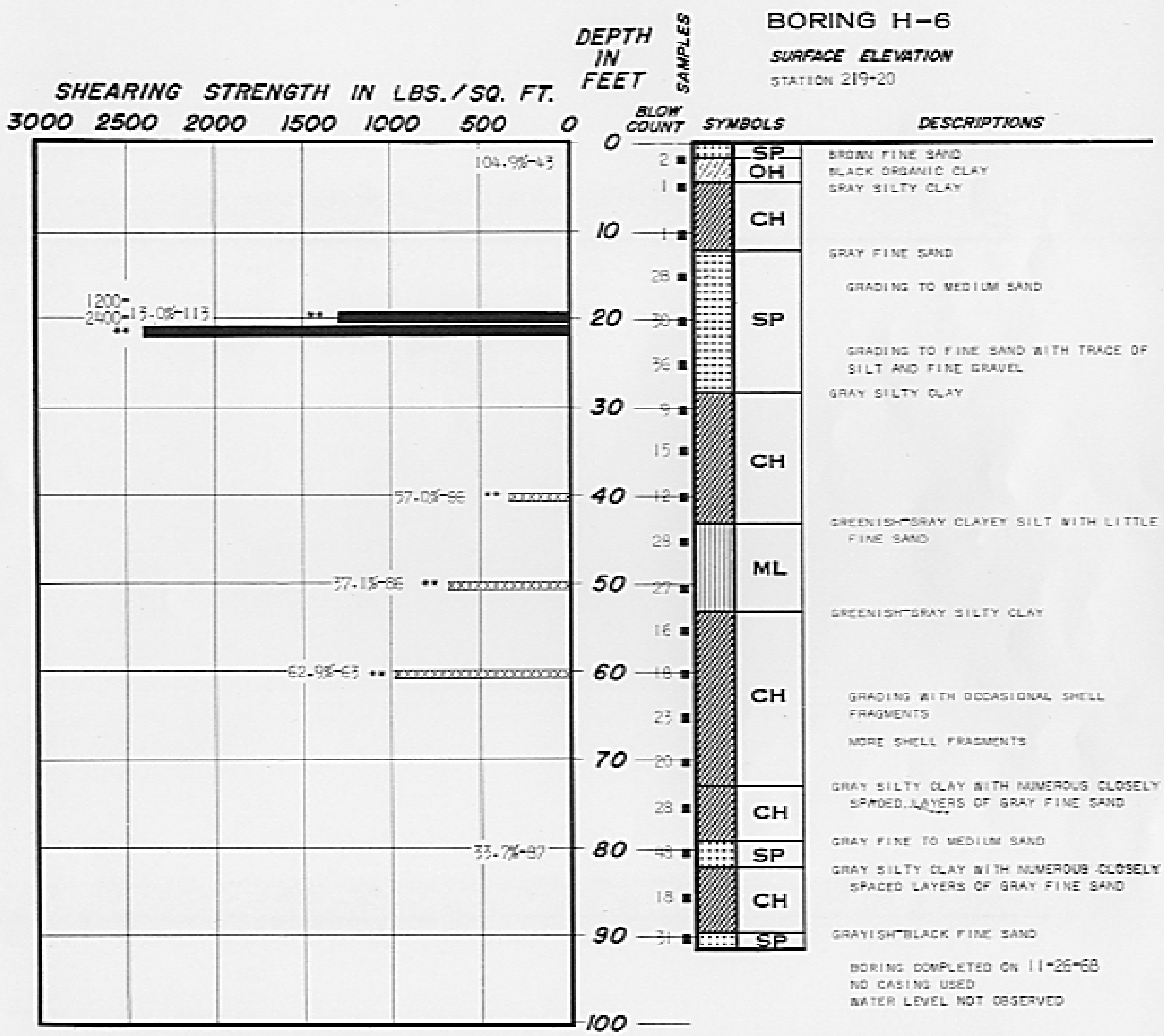
FILE 4118-030
BY FF DATE 1/6/69
CHECKED BY CMH DATE 1/7/69



LOG OF BORING

DAMES & MOORE

PLATE A-1E



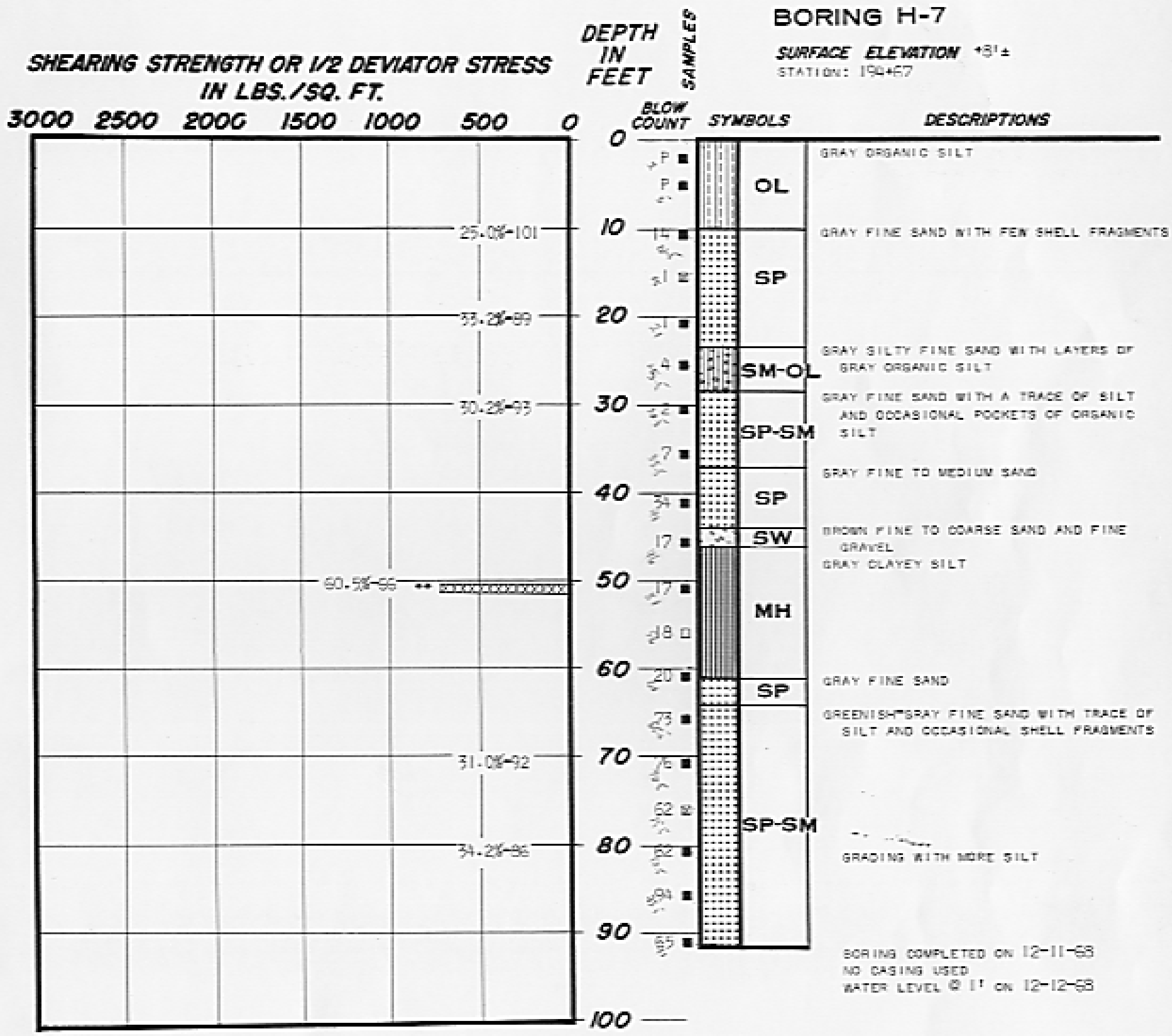
LOG OF BORING

DAMES & MOORE

PLATE A-1F

FILE 4714-034
BY F.T. DATE 1/6/69
CHECKED BY C.W. DATE 1/7/69

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



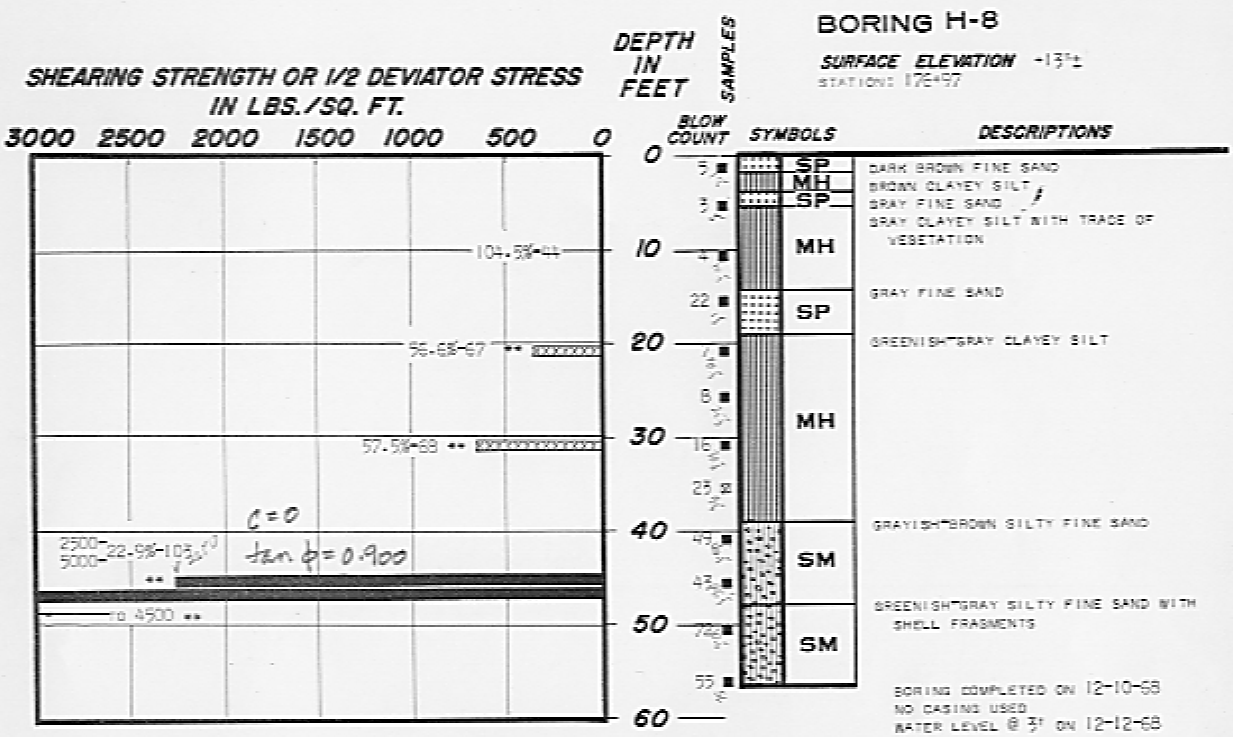
LOG OF BORING

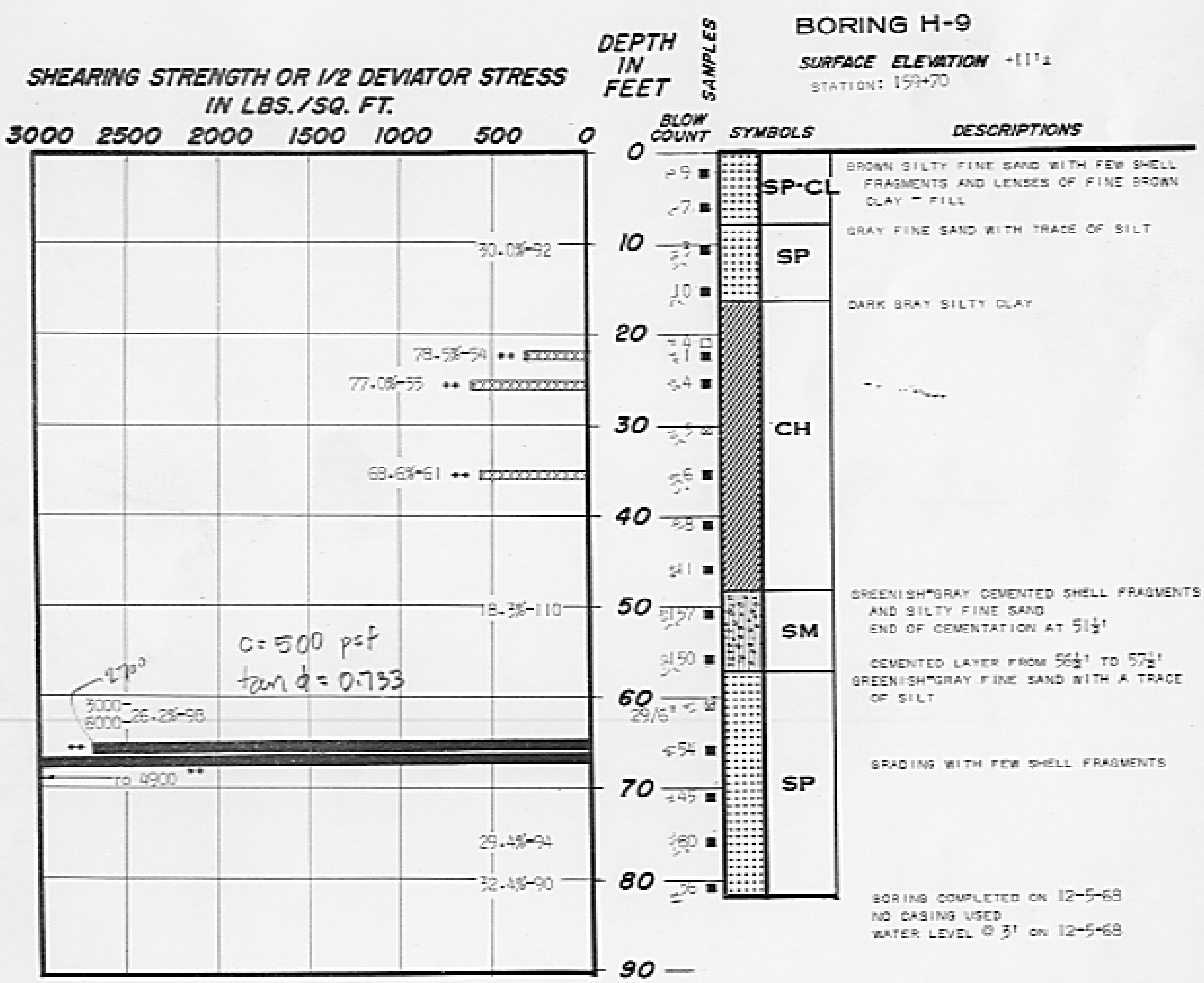
DAMES & MOORE

PLATE A-16

FILE 4718-036
BY F.F. DATE 11/6/67
CHECKED BY C.B.M. DATE 11/1/67

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





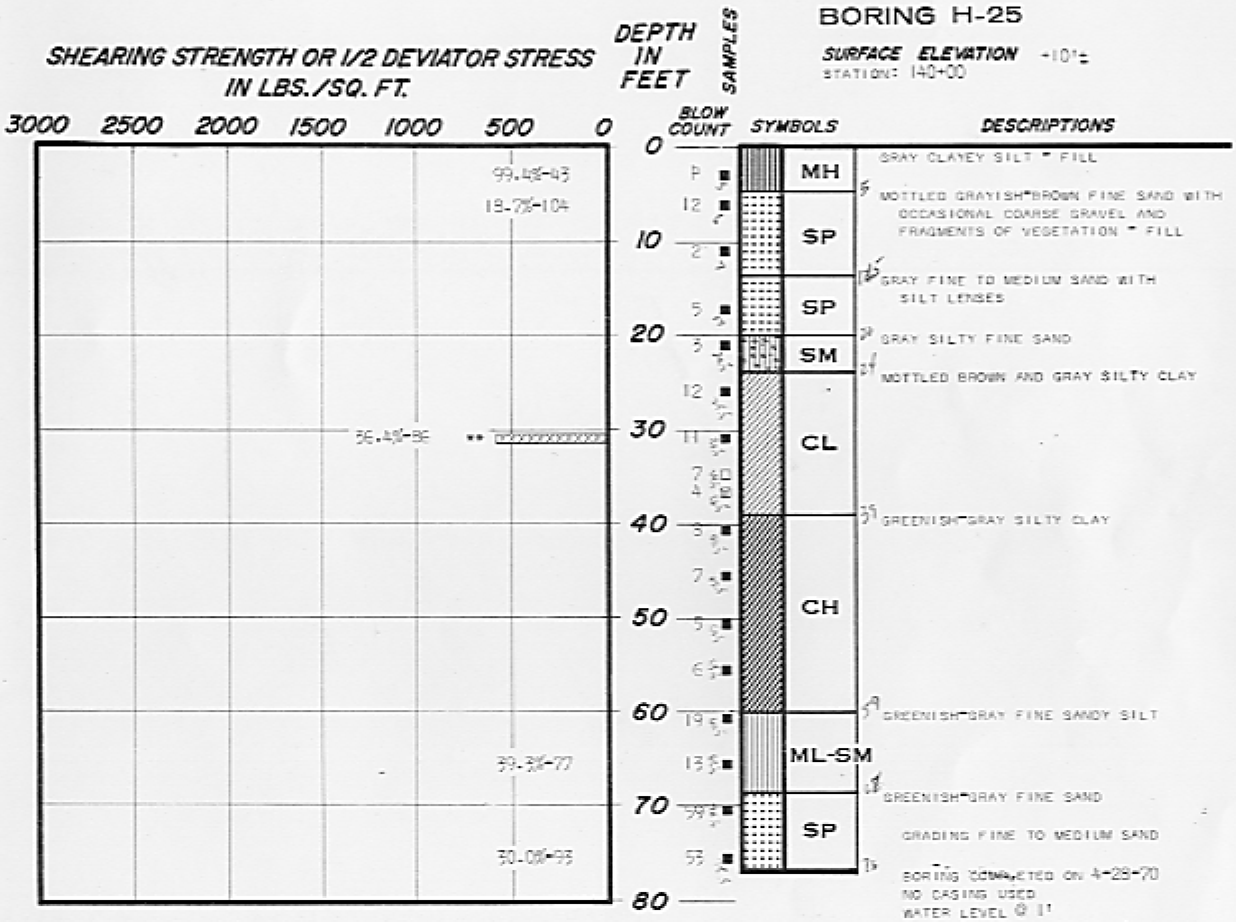
LOG OF BORINGS

DAMES & MOORE

PLATE A-1H

FILE 4778-034
BY F.I. DATE 1/6/69
CHECKED BY C.D.M. DATE 1/7/69

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



LOG OF BORING

DAMES & MOORE

PLATE A-11

FILE 9718-036
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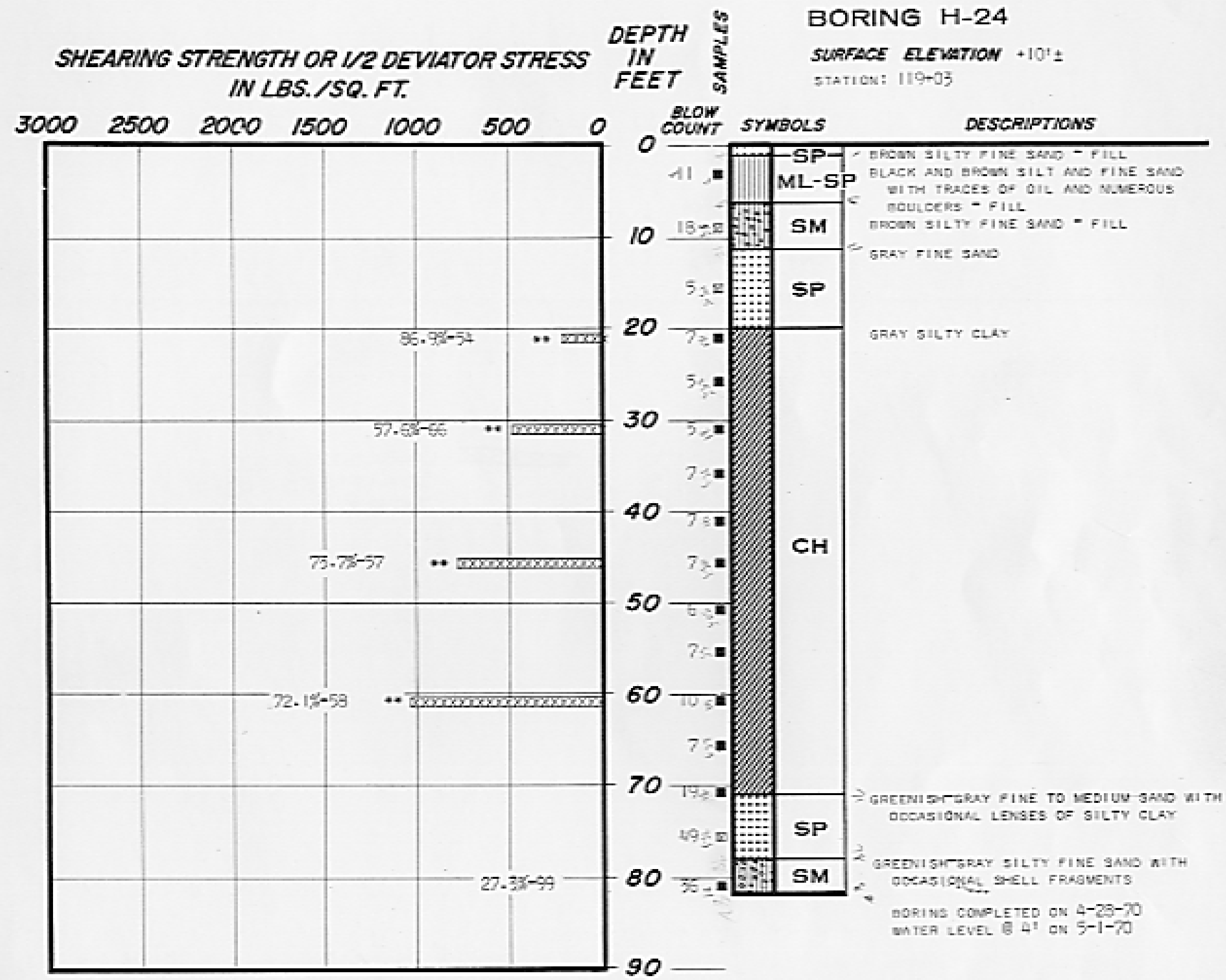
CHECKED BY ____ DATE 5-25-70

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



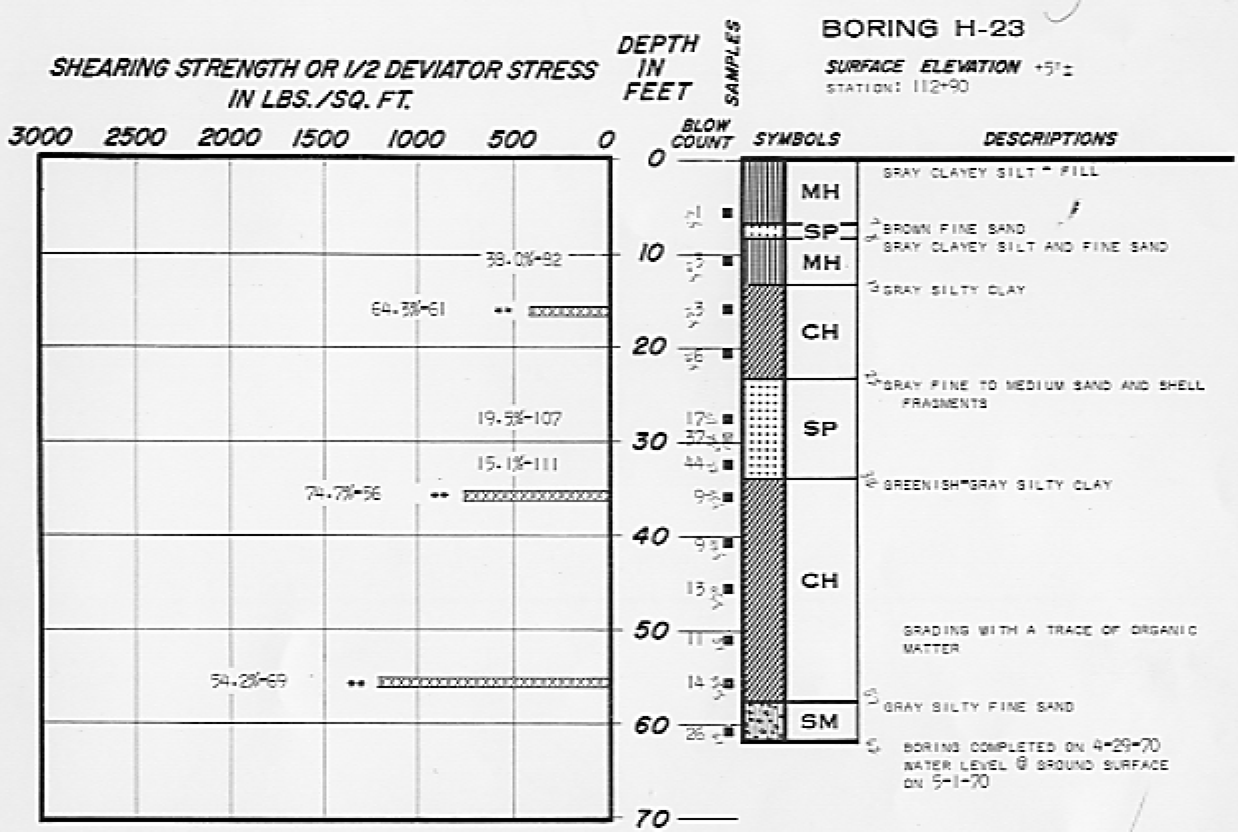
LOG OF BORING

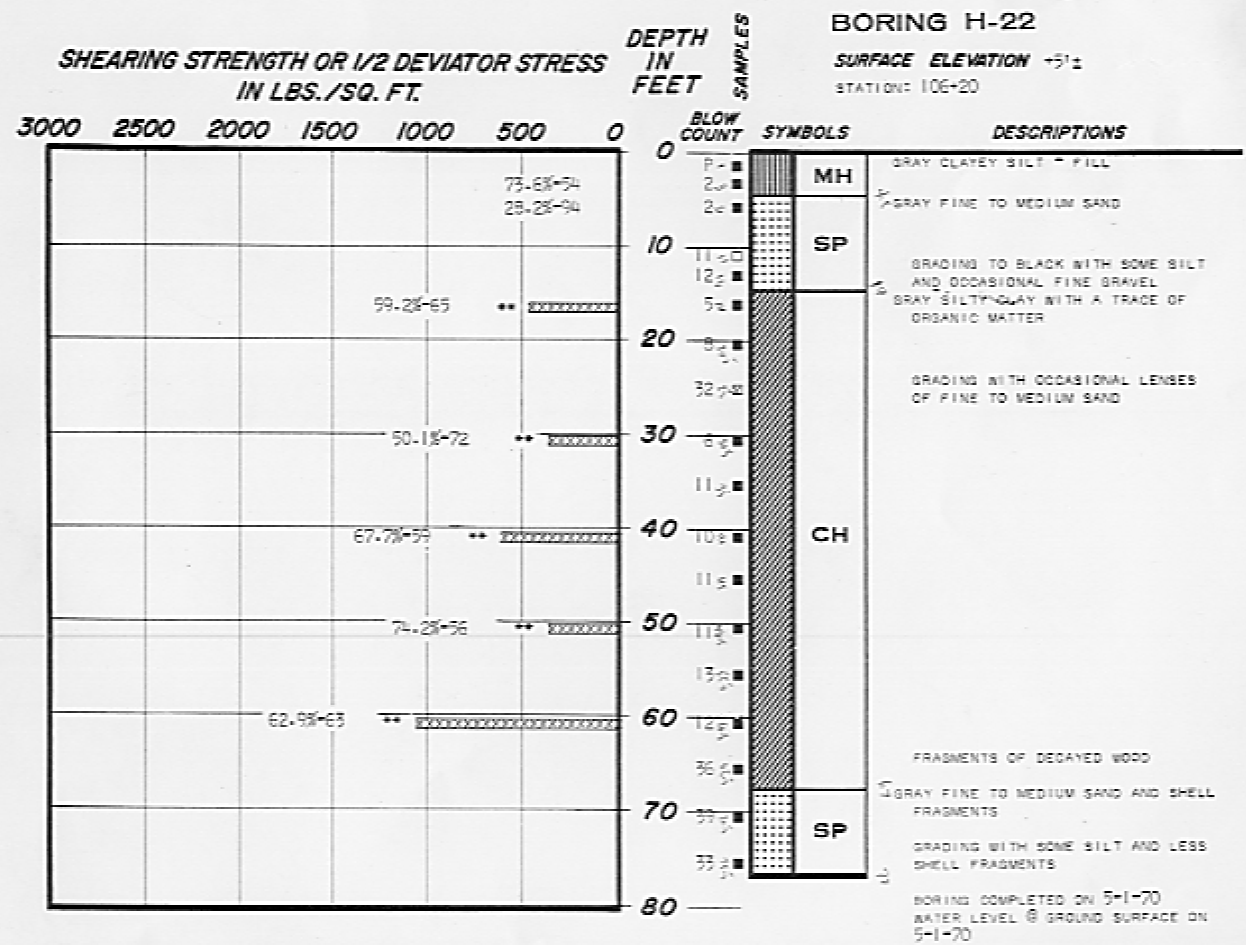
DAMES & MOORE

PLATE A-1 J

FILE 4718-036
BY ____ DATE 5-26-70
CHECKED BY ____ DATE 5-27-70
REVISIONS
BY ____ DATE ____
BY ____ DATE ____
PLATE ____ OF ____

UNCONFINED COMPRESSION TEST





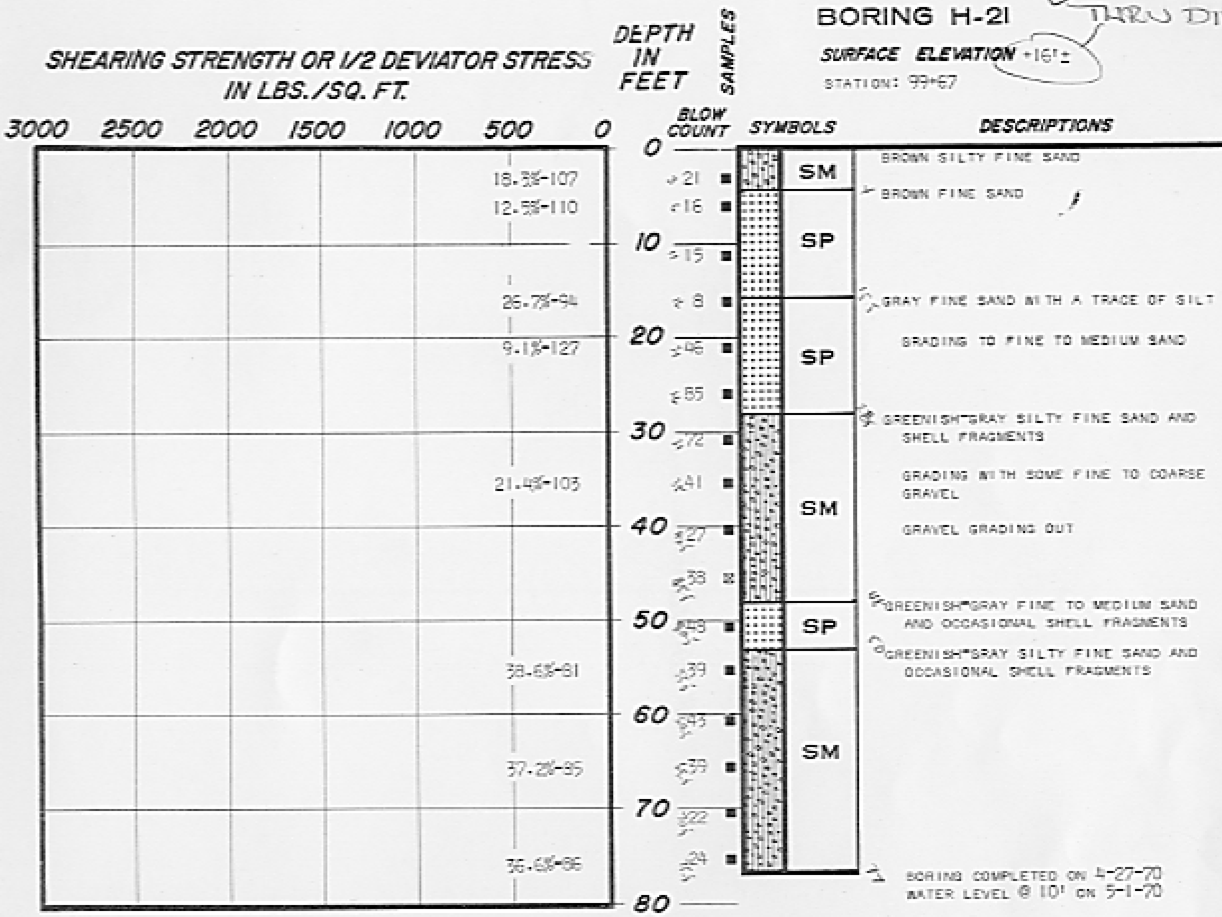
LOG OF BORINGS

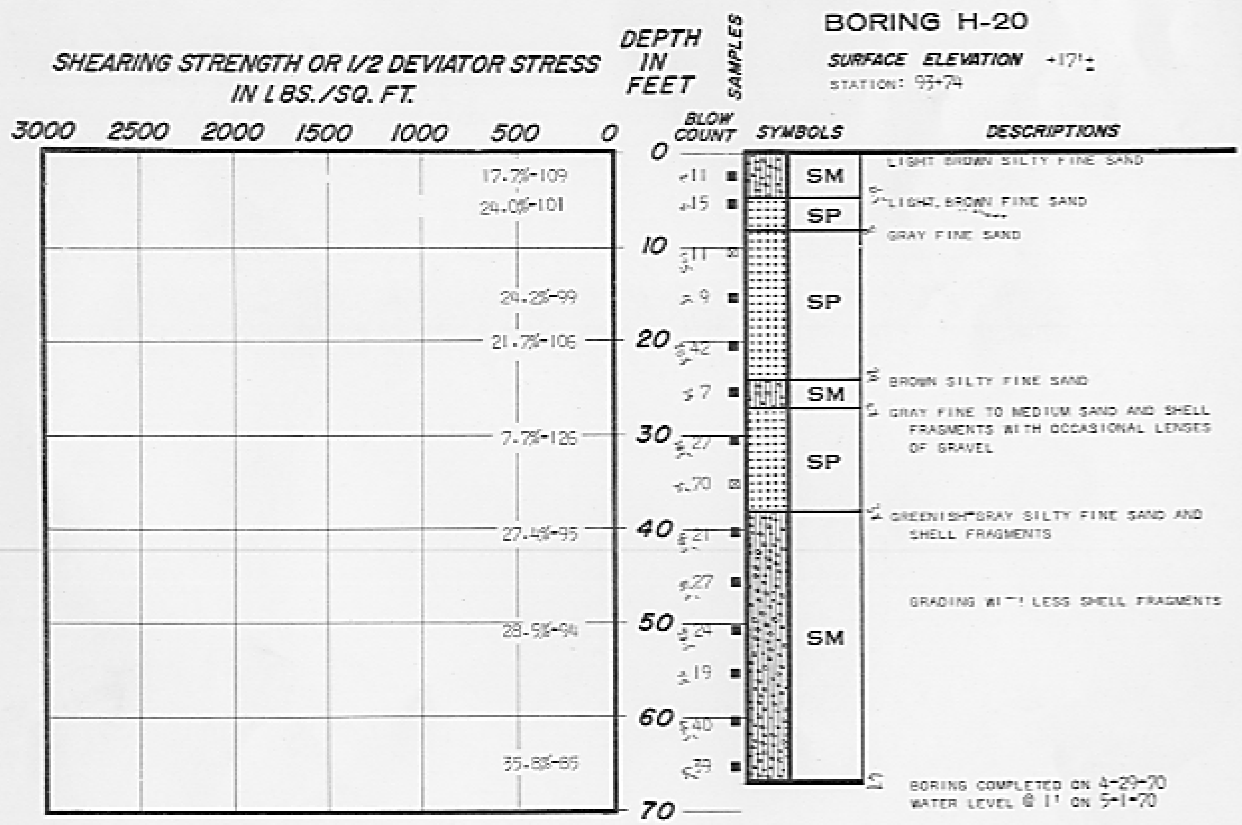
DAMES & MOORE

PLATE A-1K

FILE 4718-035
BY ... DATE 5-25-70
CHECKED BY ... DATE ...

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





LOG OF BORINGS

DAMES & MOORE

PLATE A-1L

FILE 4718-036
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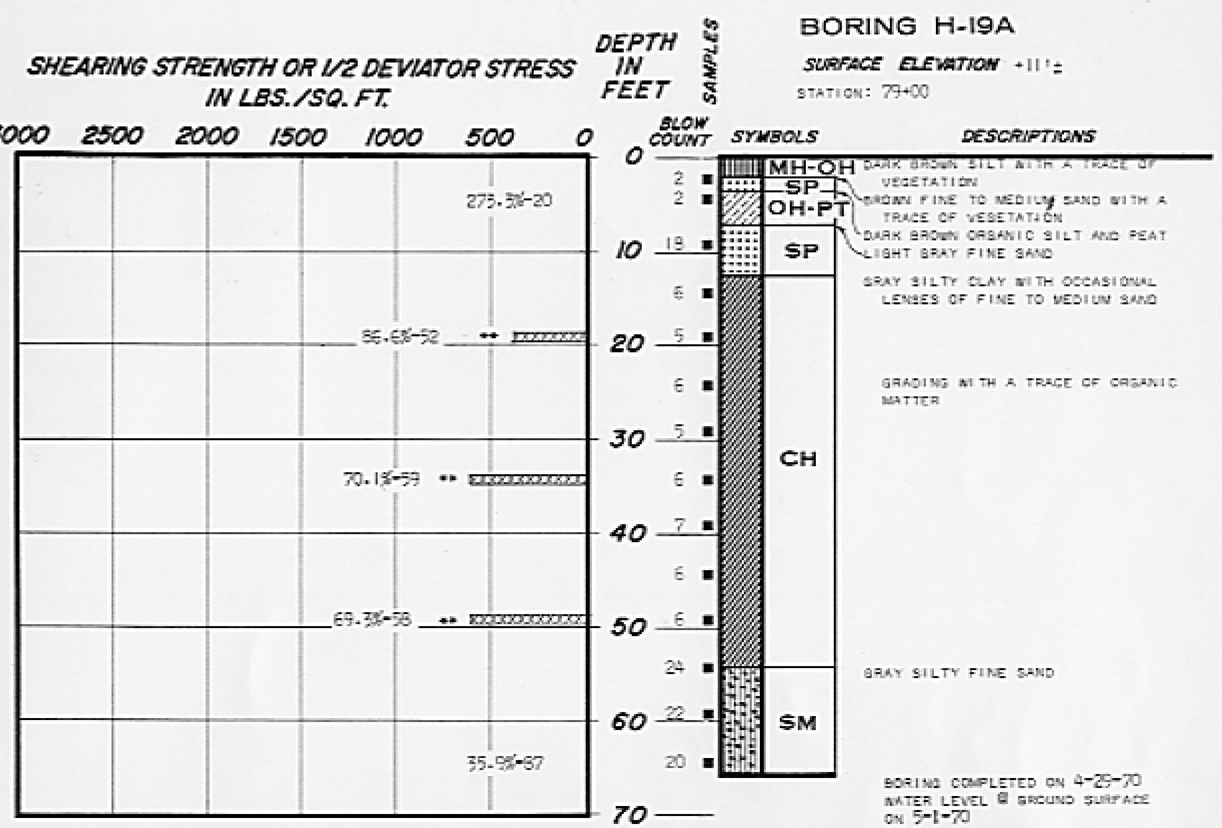
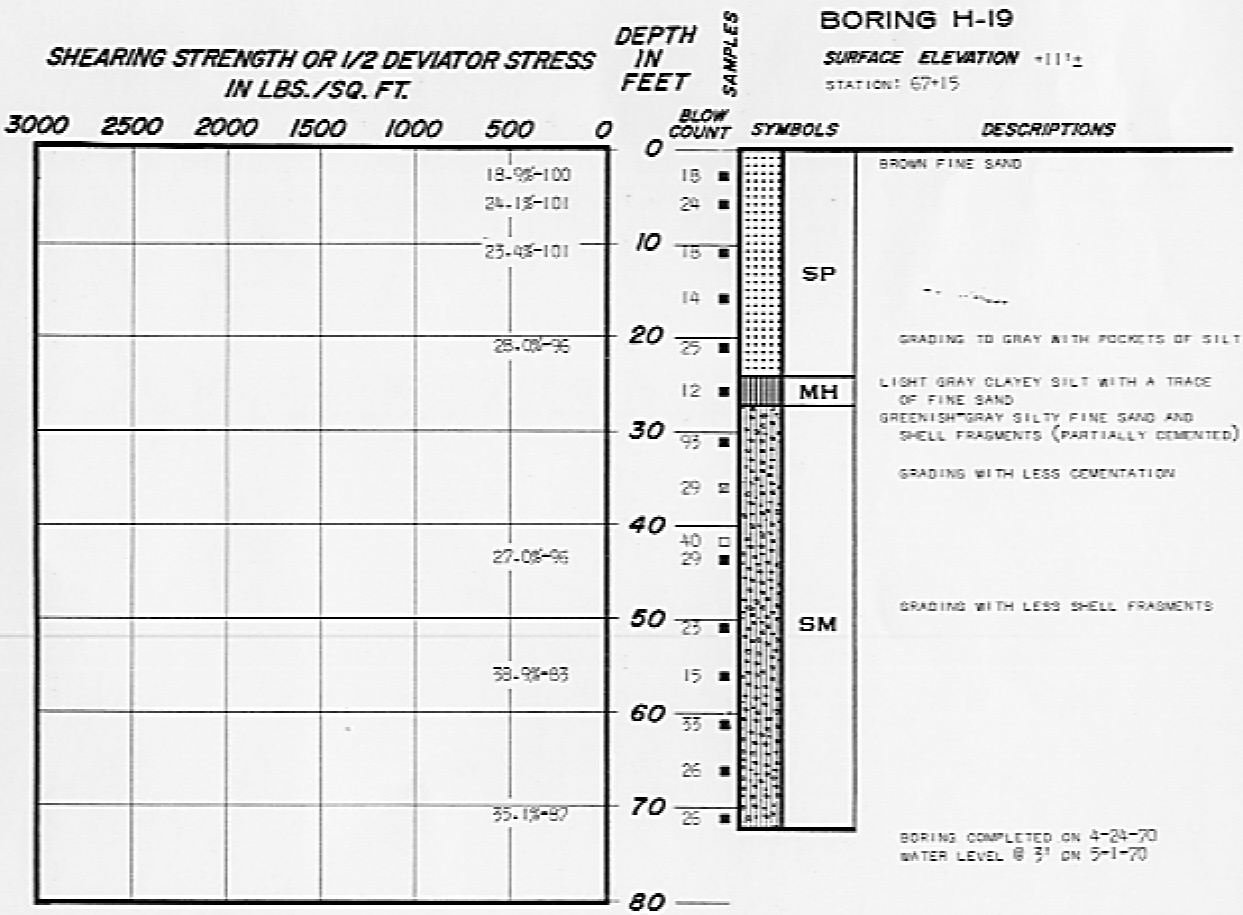
CHECKED BY [illegible] DATE [illegible]

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





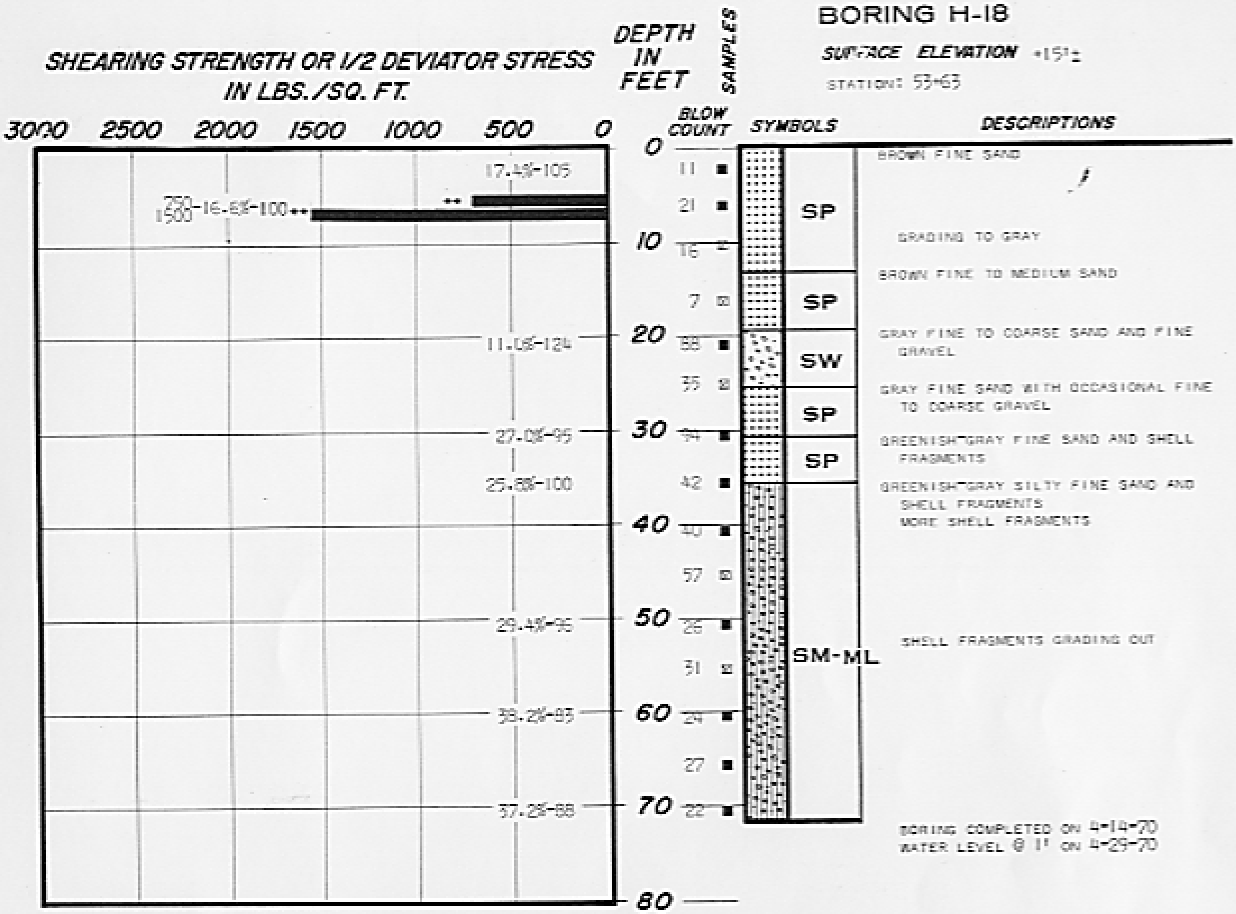
LOG OF BORINGS

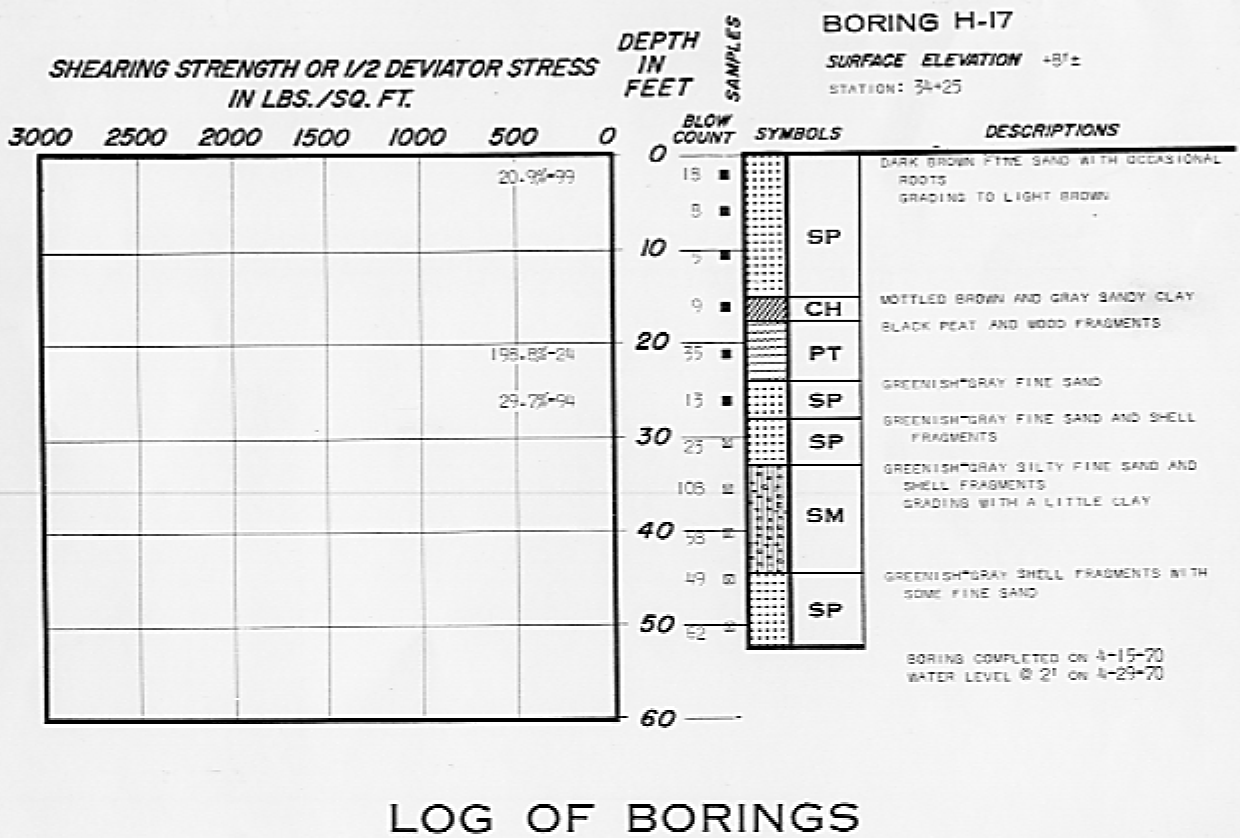
DAMES & MOORE

PLATE A-1M

FILE 4718-036
BY [illegible] DATE 5-26-70
CHECKED BY [illegible] DATE 5-25-70

REVISIONS
BY ______ DATE ______
BY ______ DATE ______
PLATE ______ OF ______





LOG OF BORINGS

DAMES & MOORE

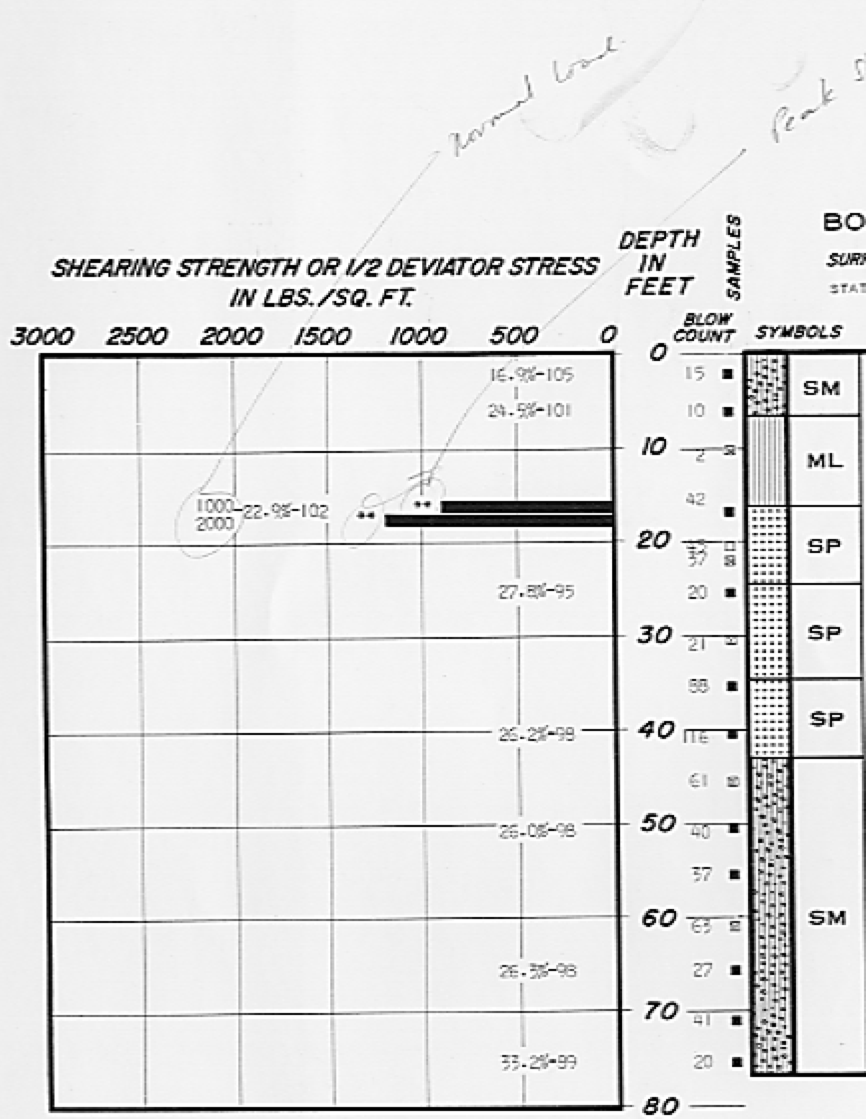
PLATE A-IN

FILE 4718-036
BY [illegible] DATE 5-26-70
CHECKED BY [illegible] DATE 5/25/70
REVISIONS
BY ___ DATE ___
BY ___ DATE ___
PLATE ___ OF ___

BORING H-16

SURFACE ELEVATION +10'±

STATION: 18+28



DESCRIPTIONS

GRAYISH-BROWN SILTY FINE SAND

GRADING WITH LESS SILT

MOTTLED BROWN AND GRAY SILT

GRADING TO GRAY WITH SOME FINE SAND

GRAY FINE TO MEDIUM SAND

GREENISH-GRAY FINE SAND

GRADING WITH SHELL FRAGMENTS AND OCCASIONAL CEMENTED PARTICLES

GREENISH-GRAY PARTIALLY CEMENTED SHELL FRAGMENTS WITH SOME SILT AND FINE SAND

GRADING WITHOUT CEMENTATION

GREENISH-GRAY SILTY FINE SAND AND SHELL FRAGMENTS

GRADING WITH OCCASIONAL CEMENTED PARTICLES

BORING COMPLETED ON 4-14-70
WATER LEVEL @ 1' ON 4-14-70

LOG OF BORING

DAMES & MOORE

PLATE A-10

FILE 4718-036
BY ______ DATE 5-26-70
CHECKED BY ______ DATE ______
REVISIONS
BY ______ DATE ______
BY ______ DATE ______
PLATE ______ OF ______

SOIL CLASSIFICATION CHART

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

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

GRADATION *

% FINER BY WEIGHT	
TRACE	0% TO 10%
LITTLE	10% TO 20%
SOME	20% TO 35%
AND	35% TO 50%

COMPACTNESS *
SAND AND/OR GRAVEL

	RELATIVE DENSITY
LOOSE	0% TO 40%
MEDIUM DENSE	40% TO 70%
DENSE	70% TO 90%
VERY DENSE	90% TO 100%

CONSISTENCY *
CLAY AND/OR SILT

	RANGE OF SHEARING STRENGTH IN POUNDS PER SQUARE FOOT
VERY SOFT	LESS THAN 250
SOFT	250 TO 500
MEDIUM	500 TO 1000
STIFF	1000 TO 2000
VERY STIFF	2000 TO 4000
HARD	GREATER THAN 4000

* VALUES ARE FROM LABORATORY OR FIELD TEST DATA, WHERE APPLICABLE. WHEN NO TESTING WAS PERFORMED, VALUES ARE ESTIMATED.

KEY TO TEST DATA

DIRECT SHEAR AND FRICTION TESTS

TESTS AT FIELD MOISTURE — TESTS AT ARTIFICIALLY CHANGED MOISTURE

2500—2045—104

- TEST NORMAL PRESSURE IN POUNDS PER SQUARE FOOT
- PER CENT FIELD MOISTURE EXPRESSED AS A PERCENTAGE OF THE DRY WEIGHT OF SOIL
- DRY DENSITY EXPRESSED IN POUNDS PER CUBIC FOOT
- PER CENT MOISTURE WHEN TESTED EXPRESSED AS A PERCENTAGE OF THE DRY WEIGHT OF SOIL
- SHEARING STRENGTH IN POUNDS PER SQUARE FOOT
- FRICTION OF SOIL ON STEEL IN POUNDS PER SQUARE FOOT
- FRICTION OF SOIL ON WOOD IN POUNDS PER SQUARE FOOT
- FRICTION OF SOIL ON CONCRETE IN POUNDS PER SQUARE FOOT

UNCONFINED COMPRESSION TESTS

20.4%—104

- PER CENT FIELD MOISTURE EXPRESSED AS A PERCENTAGE OF THE DRY WEIGHT OF SOIL
- DRY DENSITY EXPRESSED IN POUNDS PER CUBIC FOOT
- 1/2 UNCONFINED COMPRESSIVE STRESS AT FAILURE IN POUNDS PER SQUARE FOOT

TRIAXIAL COMPRESSION TESTS

2500—22.9%—104—20.4%

- CELL PRESSURE IN POUNDS PER SQUARE FOOT
- FIELD MOISTURE CONTENT IN PER CENT OF DRY WEIGHT
- DRY DENSITY IN POUNDS PER CUBIC FOOT
- TEST MOISTURE CONTENT IN PER CENT OF DRY WEIGHT
- 1/2 DEVIATOR STRESS AT FAILURE IN POUNDS PER SQUARE FOOT

CONSOLIDATED UNDRAINED — CU
CONSOLIDATED DRAINED — CD
UU — UNCONSOLIDATED UNDRAINED

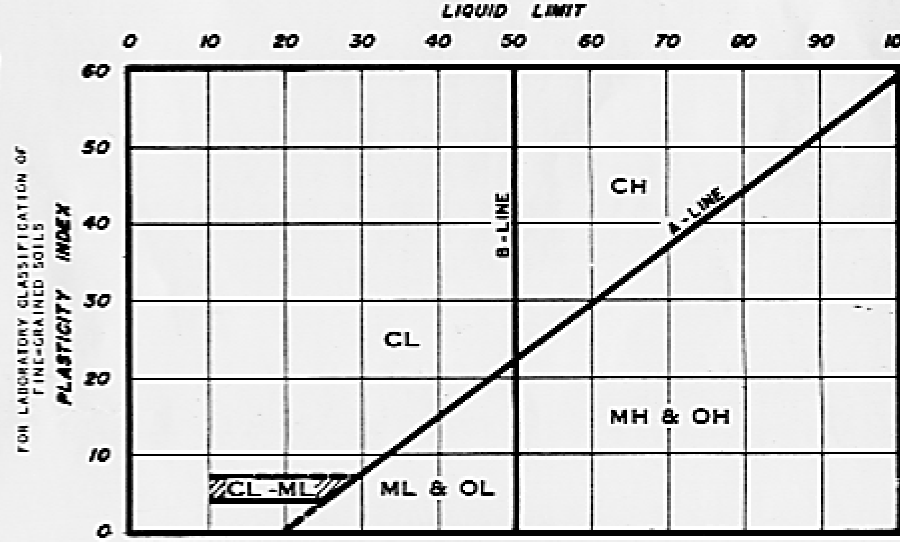
* YIELD STRENGTH
** PEAK STRENGTH
*** ULTIMATE STRENGTH

ROCK COMPRESSION TESTS

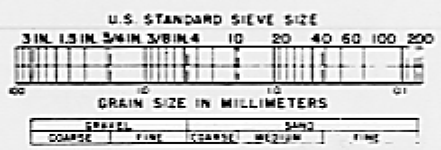
— COMPRESSIVE STRENGTH IN POUNDS PER SQUARE INCH

KEY TO SAMPLES

- ■ INDICATES DEPTH OF UNDISTURBED SAMPLE
- ⊠ INDICATES DEPTH OF DISTURBED SAMPLE
- □ INDICATES DEPTH OF SAMPLING ATTEMPT WITH NO RECOVERY
- INDICATES DEPTH OF SPLIT-SPOON SAMPLE
- I INDICATES DEPTH AND LENGTH OF CORING RUN



PLASTICITY CHART
FOR FINE GRAINED SOILS



PARTICLE SIZE DISTRIBUTION
FOR COARSE GRAINED SOILS

UNIFIED SOIL CLASSIFICATION SYSTEM

DAMES & MOORE
APPLIED EARTH SCIENCES