REPORT OF SOILS INVESTIGATION

PROPOSED PRINTING PLANT

ABERDEEN, WASHINGTON

for

THE DAILY WORLD

EARTH SCIENCES

DAMES & MOORE

CONSULTING ENGINEERS IN THE APPLIED EARTH SCIENCES

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June 1, 1972

Buttress, McClellan & Markwith, Inc.
6456 York Boulevard
Los Angeles, California 90042

Attention: Mr. R. L. Smullens

Gentlemen:

We are submitting five copies of our "Report of Soils Investigation, Proposed Printing Plant, Aberdeen, Washington," for The Daily World.

This work has been accomplished in accordance with the terms of our proposal dated April 17, 1972. Authorization for the work was received from Mr. Dave Caffoe on April 19, 1972. Our preliminary conclusions were discussed with Mr. Smullens during the investigation.

If you have any questions on the application of our recommendations or if we can be of assistance during foundation installation, please contact us.

Yours very truly,

DAMES & MOORE

By Jack K. Tuttle

Jack K. Tuttle
Partner

JKT-JPS:fm

REPORT OF SOILS INVESTIGATION

PROPOSED PRINTING PLANT

ABERDEEN, WASHINGTON

for

THE DAILY WORLD

SCOPE

The results of our soils investigation for a proposed printing plant to be constructed in Aberdeen, Washington, for The Daily World are presented in this report. The site is located on the south half of the block which is bounded by West State Street, South Alder Street, West Heron Street, and South Michigan Street, as shown on the Plot Plan, Plate 1.

The purpose of this investigation is to evaluate the best means for supporting the structure and major equipment. The detailed scope of our investigation includes:

1) Exploring subsurface stratification and groundwater conditions by drilling test borings.
2) Evaluating pertinent engineering characteristics of the foundation soils by laboratory testing.
3) Evaluating the penetration requirements for wood piles for support of the building, major equipment, and floor slabs supporting heavy storage loads.
4) Estimating settlements of pile foundations and soil-supported floor slabs.
5) Providing recommendations against any unusual construction problems.

DESIGN CONSIDERATIONS

The proposed structure will be 90 by 190 feet in plan dimensions and will be 2 stories in height. We understand that a large portion of the interior of the building will have a clear height of 20 feet. A mezzanine level will be constructed in the remainder. Major loads in the structure will consist of six press units and the paper storage area shown on the Plot Plan, Plate 1, where the floor loads will be approximately 800 pounds per square foot. We understand that the floors will be supported on grade beams resting on 6 to 12 inches of gravel replacing the stripped topsoils.

SITE CONDITIONS

The site of the proposed plant lies near the confluence of the Wishkah and Chehalis Rivers near the east end of Grays Harbor. We understand that originally the ground surface in the area of the site ranged from about Elevation 2 to Elevation 12, Port of Grays Harbor Datum. At sometime in the past, the site was filled to bring the general site grade up to about Elevation 15.

The soils underlying the site were explored by drilling three borings with truck-mounted, cable-tool drilling equipment at locations shown on the Plot Plan. Detailed logs of the borings and a description of the field exploration procedures are presented in the Appendix to this report.

Our borings indicate that the fill overlying the site is very mixed in composition and is generally about 5 feet in thickness. In the area of Boring 3, it was difficult to distinguish the boundary between the fill and the natural soils due to apparent mixing of the materials that occurred during filling. Except for a surface layer of granular material approximately 1 foot thick at Boring 1, the fill is composed of a varied

mixture of silt, sand, sawdust, wood chips, and assorted debris. Generally, the fill is soft and compressible, especially in those areas where it is clayey and has a greater accumulation of organic matter.

The fill material is underlain by a deep layer of soft gray clayey silt with layers of loose silty fine sand. This material is quite compressible and contains appreciable quantities of organic material in the upper portions of the layer. These soils extend to elevations ranging from -13 in Boring 3 to -19 in Boring 1.

The soft silts are underlain by medium dense silty sands that contain layers of soft clayey silt and clayey sand. This deposit extends to elevations of about -58 in Borings 1 and 2. Boring 3 was terminated in this layer. The silty sands are, in turn, underlain by medium stiff to stiff silty clays and medium dense sandy silts which overlie dense sands and clayey gravel. Borings 1 and 2 were terminated in clayey gravels at elevations of -89 and -102 after penetrating 3 and 6 feet, respectively.

At the time the borings were drilled, the water table was encountered at depths of 4½ to 7½ feet. The water level is expected to fluctuate with seasonal variations in rainfall and runoff.

LABORATORY EVALUATIONS

Our analyses of foundation support requirements are based on the results of laboratory tests performed on representative samples of the soils encountered. A detailed description of the test procedures and test results is presented in the Appendix.

CONCLUSIONS AND RECOMMENDATIONS

GENERAL

The presence of the significant thickness of weak compressible soils precludes the possibility of shallow foundations for the building. Evaluation of settlement behavior of lightly loaded soil-supported floor slabs indicates that ultimate settlements of 1 to 2 inches can be expected. Differential settlements of up to 1 inch across the slab and in excess of 1 inch between the slab and the structural frame would be experienced. Settlement of the floor in the paper storage area would be on the order of 6 to 9 inches if soil supported. Thus, we conclude that both the structural frame and the floor slab should be pile supported.

FOUNDATION SUPPORT

We recommend that timber piles be used for support of the proposed building frame and floor slab. These piles are readily available in the Pacific Northwest and, for the range of loading and type of subsurface conditions which prevail at this site, should prove to be the most practical and economical type of foundation support. We recommend that timber piles have a minimum tip diameter of 7 inches. Penetration requirements for various design capacities are tabulated below.

ALLOWABLE PILE CAPACITY IN TONS	PILE PENETRATION BELOW EXISTING GROUND SURFACE (FEET)
10	40
15	45
20	50
25	55

If the tops of the piles are below the lowest possible water table so that the upper sections would not be subjected to alternate cycles of wetting and drying, untreated timber piles may be used. Such piles should have all bark removed. We recommend that no untreated timber piles extend above Elevation +5. Any timber piles extending above this elevation should be fully protected by pressure-treating with creosote. All timber piles, whether treated or untreated, should meet normal standards for straightness and soundness. As an alternate, composite piles consisting of an untreated lower section and a concrete upper section would also be permissible, provided the concrete section down extends to at least Elevation +5.

In designing pile groups, piles should be spaced at a minimum center-to-center distance of at least 3 feet. We estimate that the settlements of the building, supported on piles installed as recommended, will be on the order of 3/4 inch or less and essentially uniform.

Although our recommendations are presented in detail for timber piles, other types of piles, such as concrete or steel piles, are also feasible from a technical point of view. However, in our opinion, such piles would be more expensive than timber piles and would have to be limited in their load-carrying capability because of the soil conditions, unless they were driven into the dense gravels encountered at depth in Borings 1 and 2. Should you so desire, we can evaluate other types of piles in detail, but feel that the timber piles represent the best solution for this particular installation.

PILE INSTALLATION

We recommend that timber piles be driven with a power-operated steam hammer, such as a Vulcan No. 1 or equivalent, developing at least 15,000 foot-pounds of energy per blow. Using the Engineering News Formula for a hammer of this size, the estimated driving resistance for various capacity piles is as follows:

ALLOWABLE PILE CAPACITY (TONS)	DRIVING RESISTANCE FOR LAST SIX INCHES BLOWS/FOOT
10	9
15	13
20	18
25	24

This should not be taken as the sole criterion for acceptance of the pile during installation. Our previous experience with pile driving in the Aberdeen area has indicated that the penetration resistance computed by dynamic formulas may not be reached in the clayey or silty soils during continuous driving. However, the silty and sandy soils will "set up" around the pile quite rapidly after driving. We recommend that selected piles be redriven a short distance 24 hours after initial installation to assess the "set up" which occurs.

Difficulty may be encountered in driving piles through the overlying fill inasmuch as others have encountered large pieces of granular material, debris, or tree stumps within the fill in this general area. Should such conditions be encountered, it may be necessary to predrill short lead holes or use steel driving tips in order to reduce the risk of damage to these piles. Predrilling or steel tip attachment should only be used where necessary and as approved by the inspecting engineer. All pile driving

should be accomplished under the supervision of qualified personnel who can interpret the driving records as work progresses and evaluate the implications of significant variations in penetration resistance with respect to driving criteria. We recommend that the installation be accomplished under the supervision of a soils engineer and would be pleased to provide these services.

We recommend that three or four test piles be driven at different locations before driving the production piles. A continuous record of driving resistance, including locations and extent of any predrilling necessary should be maintained for each pile. These data should be reviewed by us to finalize the appropriate driving criteria and installation techniques for the foundation pile driving. The test piles may be driven at locations where they will serve as permanent foundation piles.

DOWNDRAG LOADING

We understand that no significant change in site grade is planned. Therefore, downdrag loads due to areal settlement are not included in the pile capacity analysis. However, should any fill be placed adjacent to the structure in the future, this would induce non-uniform downdrag effects on the foundation piles. This would result in differential settlements across the building. In order to eliminate this potential problem, piles would have to be driven to the deep layer of dense granular soil.

LATERAL RESISTANCE

Seismic or wind loads can be resisted by the pile caps, grade beams, and/or the foundation piles. In determining available lateral resistance, an equivalent fluid density of 100 pounds per cubic foot may be used. This can be assumed to act over the exposed lateral surface of pile caps and grade

beams and one and one-half times the projected area of foundation piles. This value is based on a factor of safety of one; an appropriate safety factor should be incorporated into your evaluation of required lateral resistance.

SITE PREPARATION

We understand that site preparation will include stripping of the surface soils to depths of 6 inches to 1 foot and replacing them with gravel. This replacement will result in a slight increase in the load applied to the compressible soils, and thus induce minor settlements. We expect that the major portion of this settlement will occur fairly rapidly after the load is applied.

The consolidation accompanying minor stress increases, in our opinion, is not liable to produce any significant downdrag loads. The factors of safety which are incorporated into the recommendations presented for supporting capacity of piles will be ample to take care of the very limited amount of downdrag loads.

We recommend that the site preparation be carried out as soon as possible so that any settlements which may occur will be largely complete when pile installation is started. We also recommend that asphaltic concrete be planned for any paved areas in order to minimize post-construction maintenance difficulties and expenditures.

Respectfully submitted,

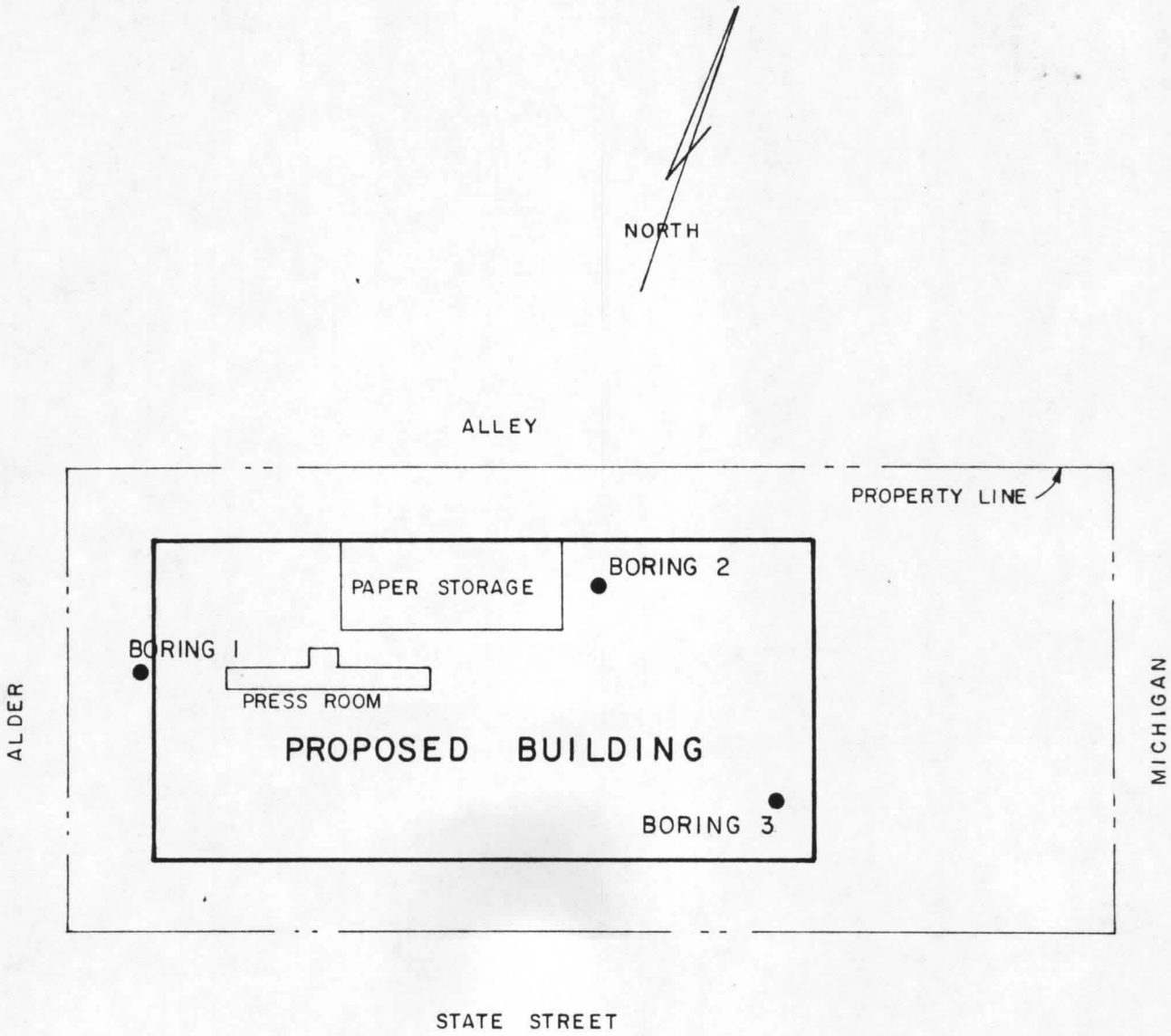
DAMES & MOORE

By

Jack K. Tuttle
Partner

June 1, 1972

JACK K. TUTTLE
STATE OF WASHINGTON
REGISTERED
PROFESSIONAL ENGINEER



PLOT PLAN

SCALE: 1" = 50'

REFERENCES:
1. DRAWING ENTITLED "THE DAILY WORLD - ABERDEEN, WASHINGTON" DATED 4-26-72 BY BUTTRESS, McCLELLAN & MARKWITH, INC.
2. PRELIMINARY STUDY 1 - FLOOR PLAN - NEWSPAPER PLANT, ABERDEEN, WASHINGTON BY BUTTRESS, McCLELLAN & MARKWITH — UNDATED.

DAMES & MOORE

PLATE 1

REVISIONS BY ____ DATE ____

FILE 9377001

BY JCB DATE 5/17/72

CHECKED BY ____

946.7 (REV. 6-61)

APPENDIX

FIELD AND LABORATORY TESTS

FIELD EXPLORATIONS

The soil stratigraphy underlying the site of the proposed plant was explored by drilling 3 borings ranging from 64 to 114 feet deep with truck-mounted, cable-tool drilling equipment. The borings were 6 inches in diameter and located as shown on the Plot Plan, Plate 1.

Drilling of the borings was supervised by an engineer from our office who classified the soils and prepared a continuous log of the conditions encountered in each boring. The boring logs are presented on Plates A-1 through A-3. The soils were classified by visual and textural examination in accordance with the Unified Soil Classification System described on Plate A-4.

Undisturbed samples of the various soil strata were obtained at frequent intervals with a Dames & Moore Sampler of the type illustrated on page A-3. The sampler was driven with a weight of 400 pounds falling 24 inches. The number of blows required to drive the sampler a distance of 12 inches into the soil is shown above the sample notations on the boring logs.

Observations of the water levels in each boring were made during the field work. The highest water level observed is indicated on the boring log.

LABORATORY TESTS

Direct shear tests were performed on representative samples obtained from the borings to evaluate the strength of the soils. The tests were performed at a continuous rate of shearing deflection in the manner described on page A-4. The test results are presented in a tabular form on Plates A-5 and A-6.

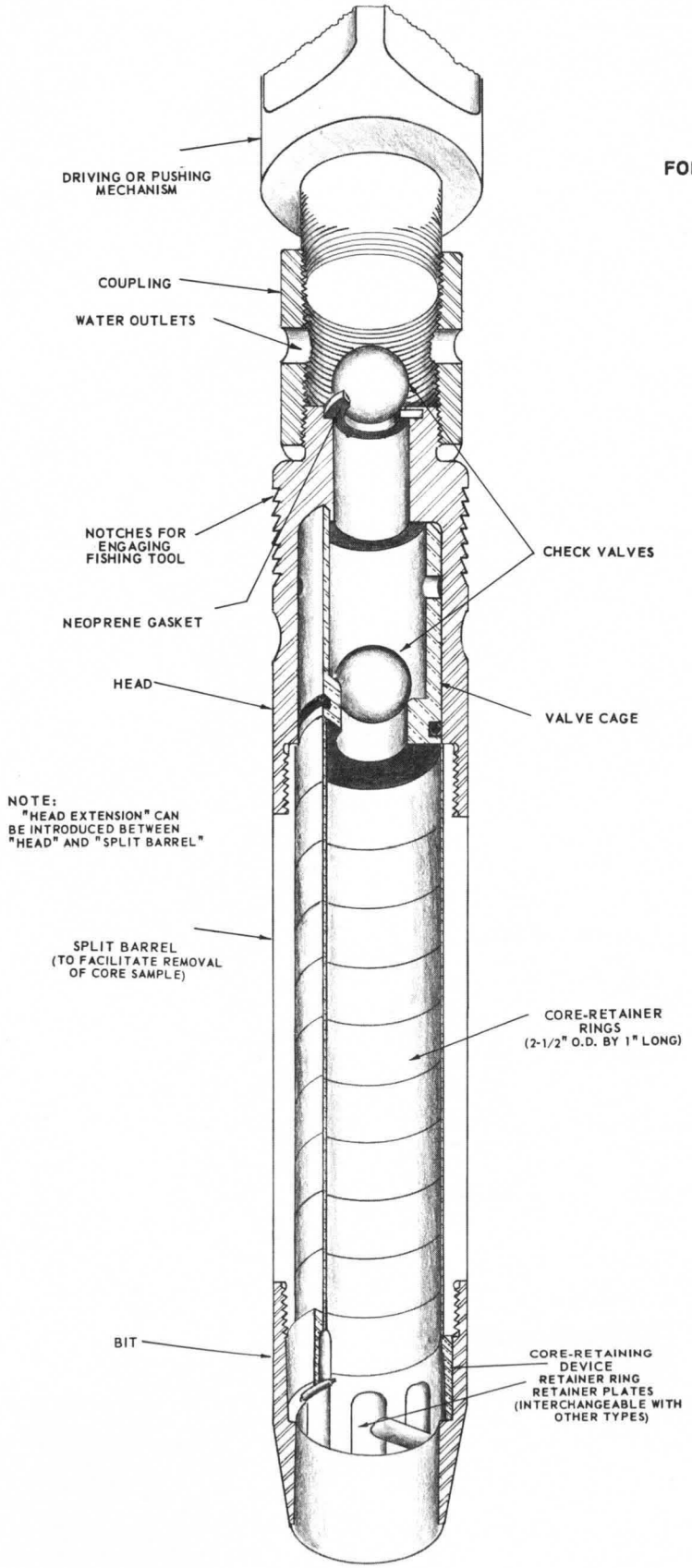
Unconsolidated, undrained triaxial compression tests were also performed on representative samples obtained from the borings to evaluate strength characteristics. The tests were performed at a continuous rate of shearing deflection in the manner described on page A-5. The results are presented in tabular form on Plate A-7.

Consolidation tests were performed on selected samples of the softer soils to develop information for estimating the magnitude and rate of settlements. The test procedure is described on page A-6, and the results of the tests are presented on Plate A-8.

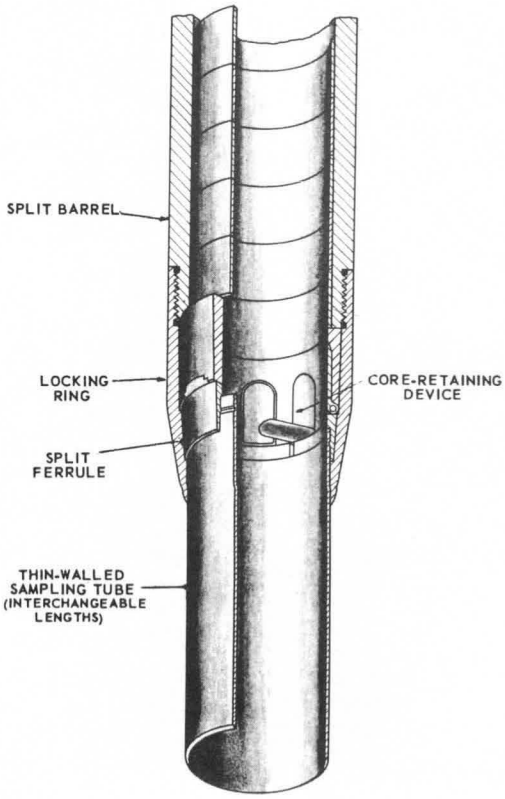
Identification tests consisting of moisture and density and Atterberg Limit determinations were performed on selected samples for correlation purposes. The moisture and density data are presented adjacent to the appropriate sample notations on the boring logs and the Atterberg Limit results are presented on Plate A-8.

SOIL SAMPLER TYPE U

FOR SOILS DIFFICULT TO RETAIN IN SAMPLER



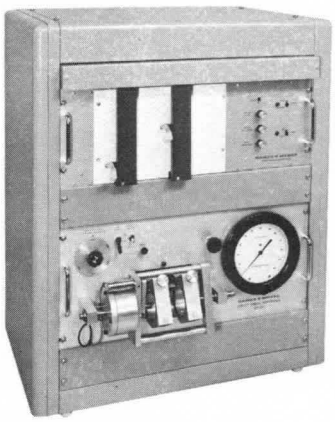
ALTERNATE ATTACHMENTS



417.9 (5-68)

METHOD OF PERFORMING DIRECT SHEAR AND FRICTION TESTS

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



DIRECT SHEAR APPARATUS WITH ELECTRONIC RECORDER (IN SEATTLE OFFICE)

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.

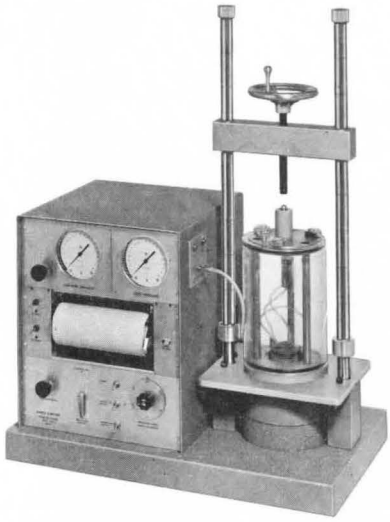
FRICTION TESTS

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.

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



TRIAXIAL COMPRESSION TEST UNIT

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

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

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

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

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

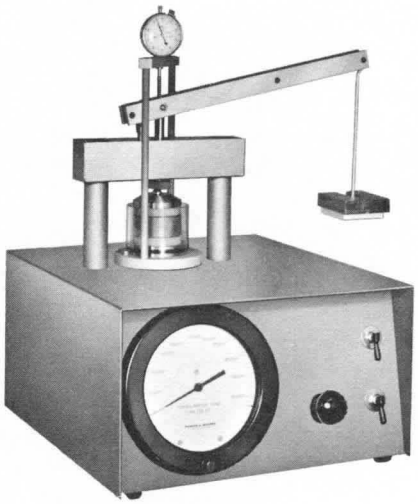
DRAINED: THE INTERGRANULAR STRESSES IN A SAMPLE MAY BE MEASURED BY PERFORMING A DRAINED, OR SLOW, TEST. IN THIS TEST THE SAMPLE IS FULLY SATURATED AND CONSOLIDATED PRIOR TO THE START OF THE TEST. DURING THE TEST, DRAINAGE IS PERMITTED AND THE TEST IS PERFORMED AT A SLOW ENOUGH RATE TO PREVENT THE BUILDUP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

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

METHOD OF PERFORMING CONSOLIDATION TESTS

CONSOLIDATION TESTS ARE PERFORMED TO EVALUATE THE VOLUME CHANGES OF SOILS SUBJECTED TO INCREASED LOADS. TIME-CONSOLIDATION AND PRESSURE-CONSOLIDATION CURVES MAY BE PLOTTED FROM THE DATA OBTAINED IN THE TESTS. ENGINEERING ANALYSES BASED ON THESE CURVES PERMIT ESTIMATES TO BE MADE OF THE PROBABLE MAGNITUDE AND RATE OF SETTLEMENT OF THE TESTED SOILS UNDER APPLIED LOADS.

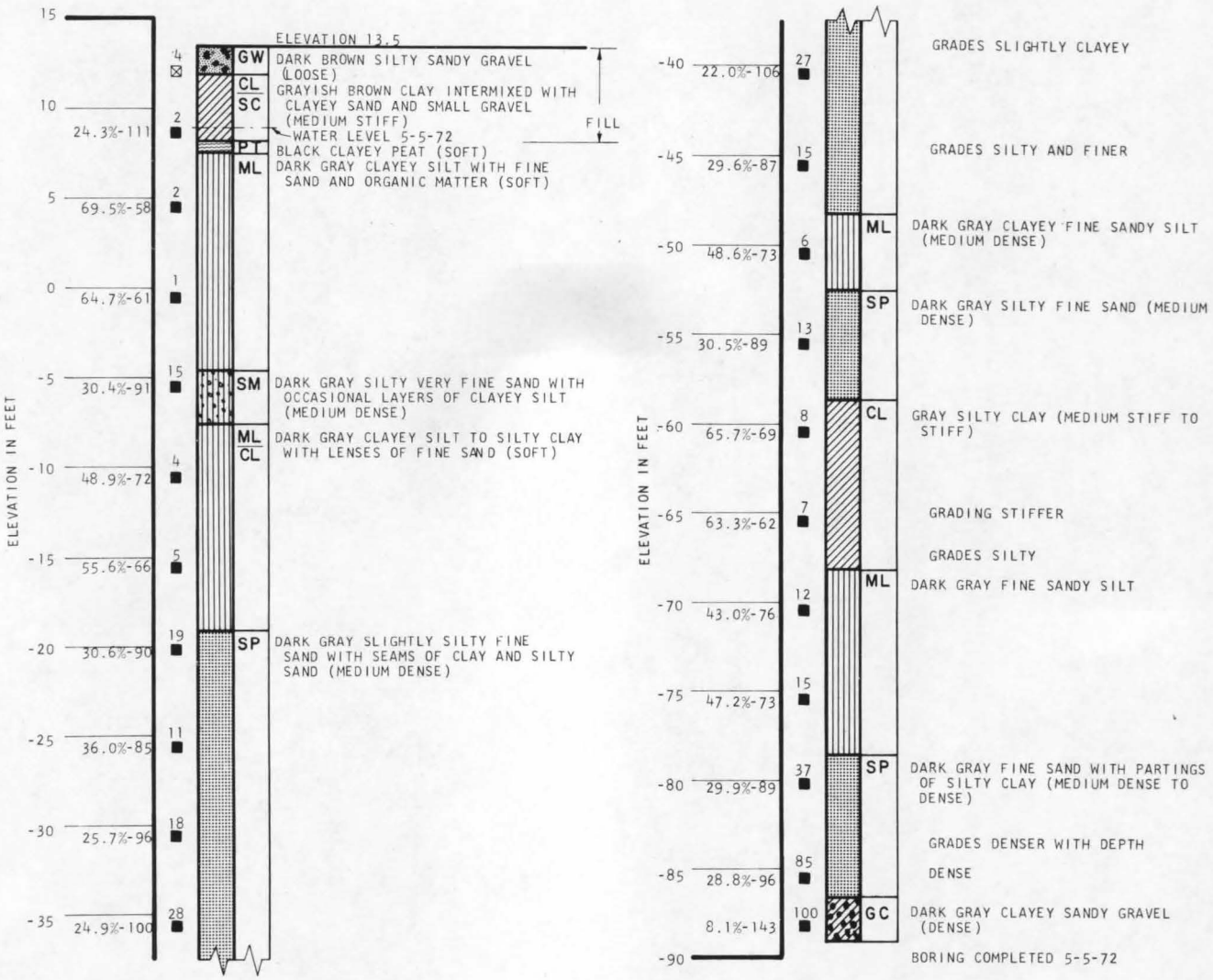
EACH SAMPLE IS TESTED WITHIN BRASS RINGS TWO AND ONE-HALF INCHES IN DIAMETER AND ONE INCH IN LENGTH. UNDISTURBED SAMPLES OF IN-PLACE SOILS ARE TESTED IN RINGS TAKEN FROM THE SAMPLING DEVICE IN WHICH THE SAMPLES WERE OBTAINED. LOOSE SAMPLES OF SOILS TO BE USED IN CONSTRUCTING EARTH FILLS ARE COMPACTED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



DEAD LOAD-PNEUMATIC CONSOLIDOMETER

IN TESTING, THE SAMPLE IS RIGIDLY CONFINED LATERALLY BY THE BRASS RING. AXIAL LOADS ARE TRANSMITTED TO THE ENDS OF THE SAMPLE BY POROUS DISKS. THE DISKS ALLOW DRAINAGE OF THE LOADED SAMPLE. THE AXIAL COMPRESSION OR EXPANSION OF THE SAMPLE IS MEASURED BY A MICROMETER DIAL INDICATOR AT APPROPRIATE TIME INTERVALS AFTER EACH LOAD INCREMENT IS APPLIED. EACH LOAD IS ORDINARILY TWICE THE PRECEDING LOAD. THE INCREMENTS ARE SELECTED TO OBTAIN CONSOLIDATION DATA REPRESENTING THE FIELD LOADING CONDITIONS FOR WHICH THE TEST IS BEING PERFORMED. EACH LOAD INCREMENT IS ALLOWED TO ACT OVER AN INTERVAL OF TIME DEPENDENT ON THE TYPE AND EXTENT OF THE SOIL IN THE FIELD.

BORING I



KEY:

MOISTURE CONTENT — 100 — BLOWS REQUIRED TO DRIVE SAMPLER ONE FOOT WEIGHT= 400 LBS., STROKE= 24 INCHES.

8.1%-143 ■ INDICATES DEPTH AT WHICH UNDISTURBED SAMPLE WAS EXTRACTED.

DRY DENSITY IN PCF

⊠ INDICATES DEPTH AT WHICH DISTURBED SAMPLE WAS EXTRACTED.

THE DISCUSSION IN THE TEXT OF THIS REPORT IS NECESSARY TO A PROPER UNDERSTANDING OF THE NATURE OF THE SUBSURFACE MATERIALS.

LOG OF BORINGS

NOTE: ELEVATIONS ARE REFERENCED TO PORT OF GRAYS HARBOR DATUM.

REVISIONS BY DATE BY DATE PLATE OF

BY DCB DATE 5/16/72 CHECKED BY DATE

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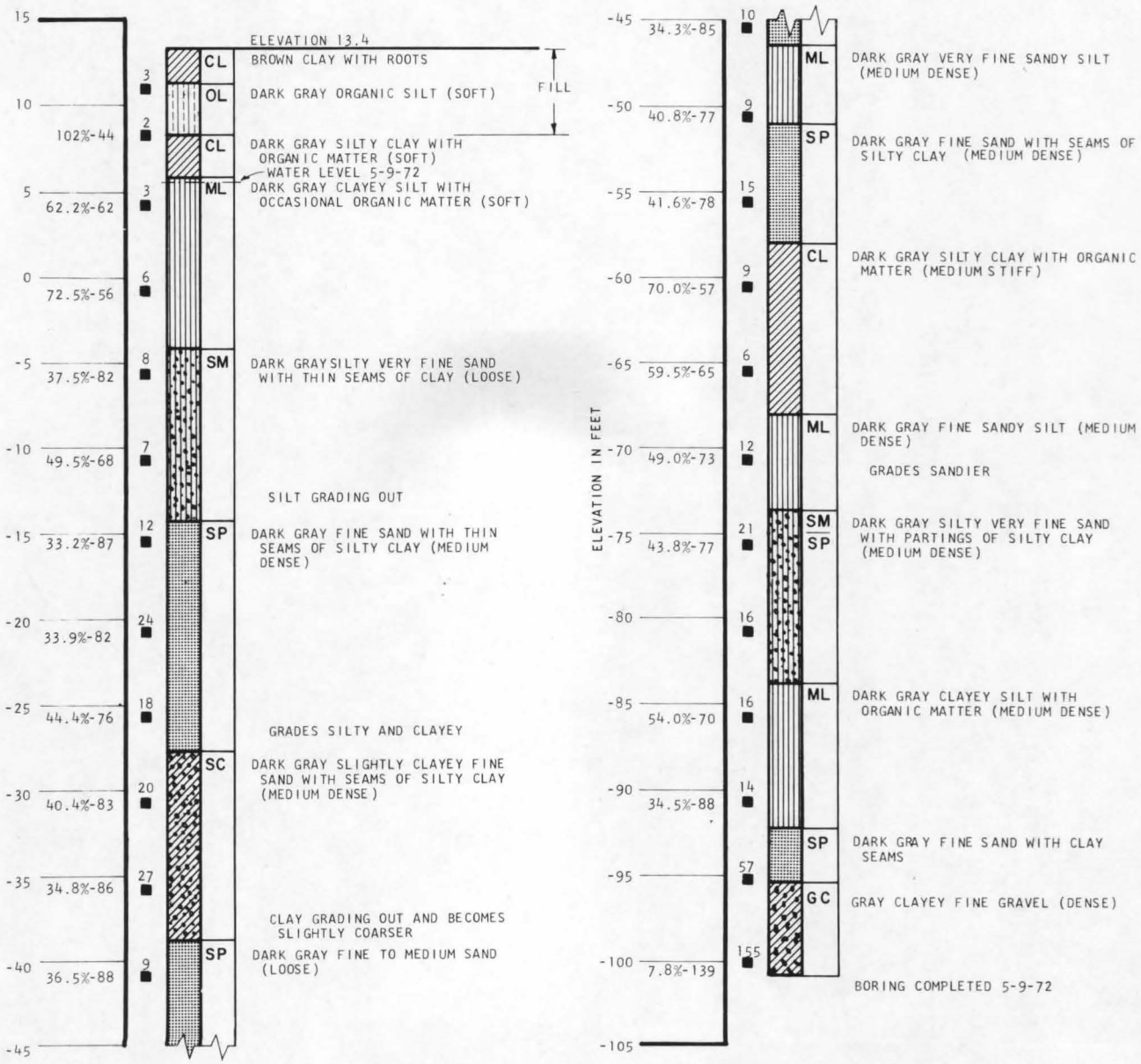
PLATE A-1

BORING 2



LOG OF BORINGS

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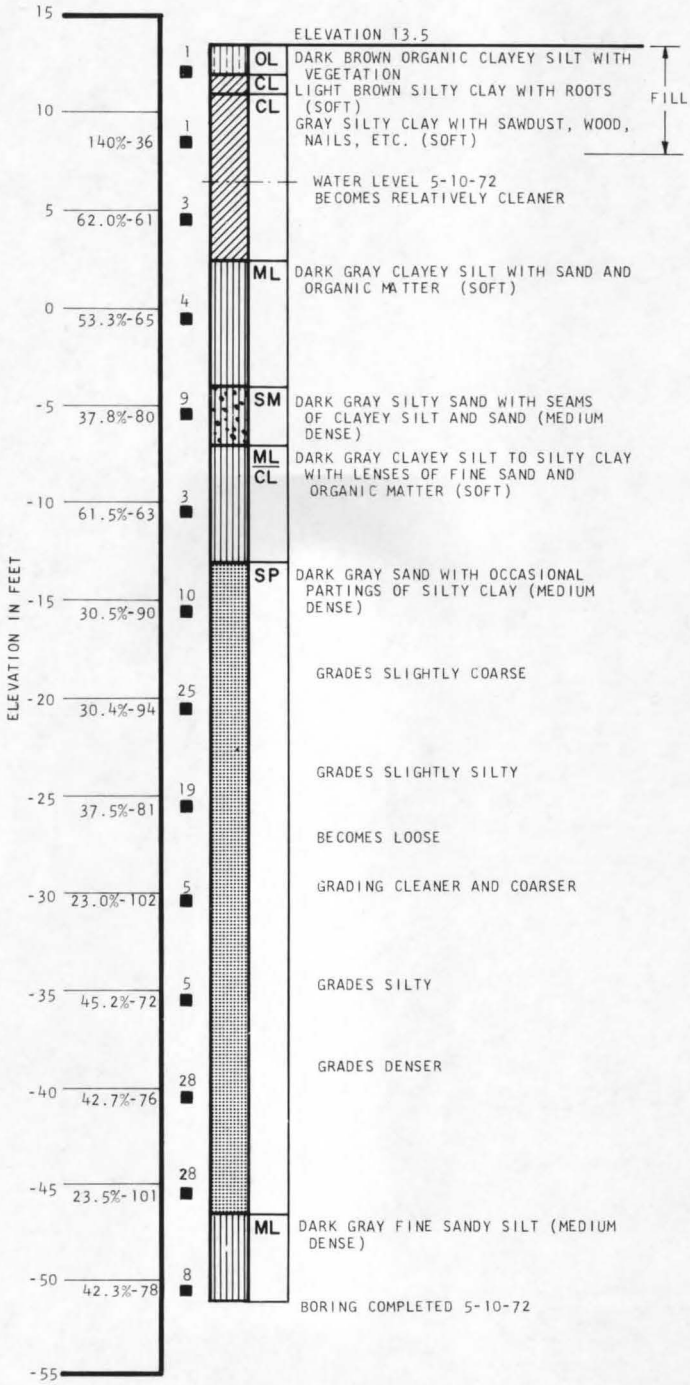
BORING 3



LOG OF BORINGS

DAMES & MOORE

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

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

SOIL CLASSIFICATION CHART

UNIFIED SOIL CLASSIFICATION SYSTEM

DAMES & MOORE

BY DLB DATE 5-24-72 FILE 9377-001 Daily World REVISIONS

CHECKED BY BY DATE

BORING	ELEVATION	SOIL TYPE	MOISTURE CONTENT % OF DRY WEIGHT	DRY DENSITY LBS./CU.FT.	NORMAL PRESSURE LBS./SQ.FT.	YIELD PT. SHEAR STRENGTH LBS./SQ.FT.
1	+5.0	CLAYEY SILT WITH ORGANIC MATTER	69.5*	58	1000	340
1	-5.0	SILTY FINE SAND	30.4	91	1000	800
1	-5.0	SILTY FINE SAND	32.3*	90	3000	2000
1	-19.5	FINE SAND WITH SEAMS OF CLAY AND SILTY SAND	30.6	90	2000	1500
1	-19.5	FINE SAND WITH SEAMS OF CLAY AND SILTY SAND	39.4	85	4000	2600
1	-30.5	FINE SAND WITH SEAMS OF CLAY AND SILTY SAND	25.7	96	2500	1500
1	-30.5	FINE SAND WITH SEAMS OF CLAY AND SILTY SAND	49.5	81	4000	1800
1	-35.5	FINE SAND WITH SEAMS OF CLAY AND SILTY SAND	24.9	100	2500	1700
1	-35.5	FINE SAND WITH SEAMS OF CLAY AND SILTY SAND	26.3	99	5000	2900
1	-40.5	SILTY FINE SAND	29.6	87	3000	1200
2	0.5	CLAYEY SILT WITH ORGANIC MATTER	72.5	56	1000	500
2	0.5	CLAYEY SILT WITH ORGANIC MATTER	73.0*	56	3000	1250
2	-10.5	SILTY FINE SAND WITH SEAMS OF CLAY	49.5*	68	1500	750
2	-10.5	SILTY FINE SAND WITH SEAMS OF CLAY	52.5*	67	3000	1200

* FIGURES INDICATE MOISTURE CONTENT AND DRY DENSITY AT END OF TEST.

SUMMARY OF DIRECT SHEAR TEST DATA

DAMES & MOORE

BY DLB DATE 5-24-72 FILE 9377-001 REVISIONS

CHECKED BY BY DATE

BORING	ELEVATION	SOIL TYPE	MOISTURE CONTENT % OF DRY WEIGHT	DRY DENSITY LBS./CU.FT.	NORMAL PRESSURE LBS./SQ.FT.	YIELD PT. SHEAR STRENGTH LBS./SQ.FT.
2	-20.5	FINE SAND WITH SEAMS OF CLAY	33.9	88	2000	1400
2	-20.5	FINE SAND WITH SEAMS OF CLAY	33.9	88	4000	2900
2	-35.5	CLAYEY FINE SAND	34.8	86	2500	1750
2	-35.5	CLAYEY FINE SAND	38.8	84	4000	2400
2	-50.5	FINE SAND WITH SEAMS OF SILTY CLAY	40.8*	77	3000	2000
2	-50.5	FINE SAND WITH SEAMS OF SILTY CLAY	40.3*	77	5000	2350
3	0.5	CLAYEY SILT WITH ORGANIC MATTER	53.3*	65	1000	450
3	0.5	CLAYEY SILT WITH ORGANIC MATTER	47.4	67	3000	1600
3	-15.5	FINE SAND	30.5	90	2000	1450
3	-15.5	FINE SAND	39.0	84	3000	1600
3	-35.5	SILTY FINE SAND	45.2*	72	2500	1450
3	-35.5	SILTY FINE SAND	47.3*	71	4000	2350
3	-40.5	SILTY FINE SAND	42.7	76	2500	1620
3	-40.5	SILTY FINE SAND	40.5*	78	4000	2550

* FIGURES INDICATE MOISTURE CONTENT AND DRY DENSITY AT END OF TEST.

SUMMARY OF DIRECT SHEAR TEST DATA

DAMES & MOORE

946.7 (REV. 6-61)

BY DLB DATE 5-25-72
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REVISIONS
BY _____ DATE _____

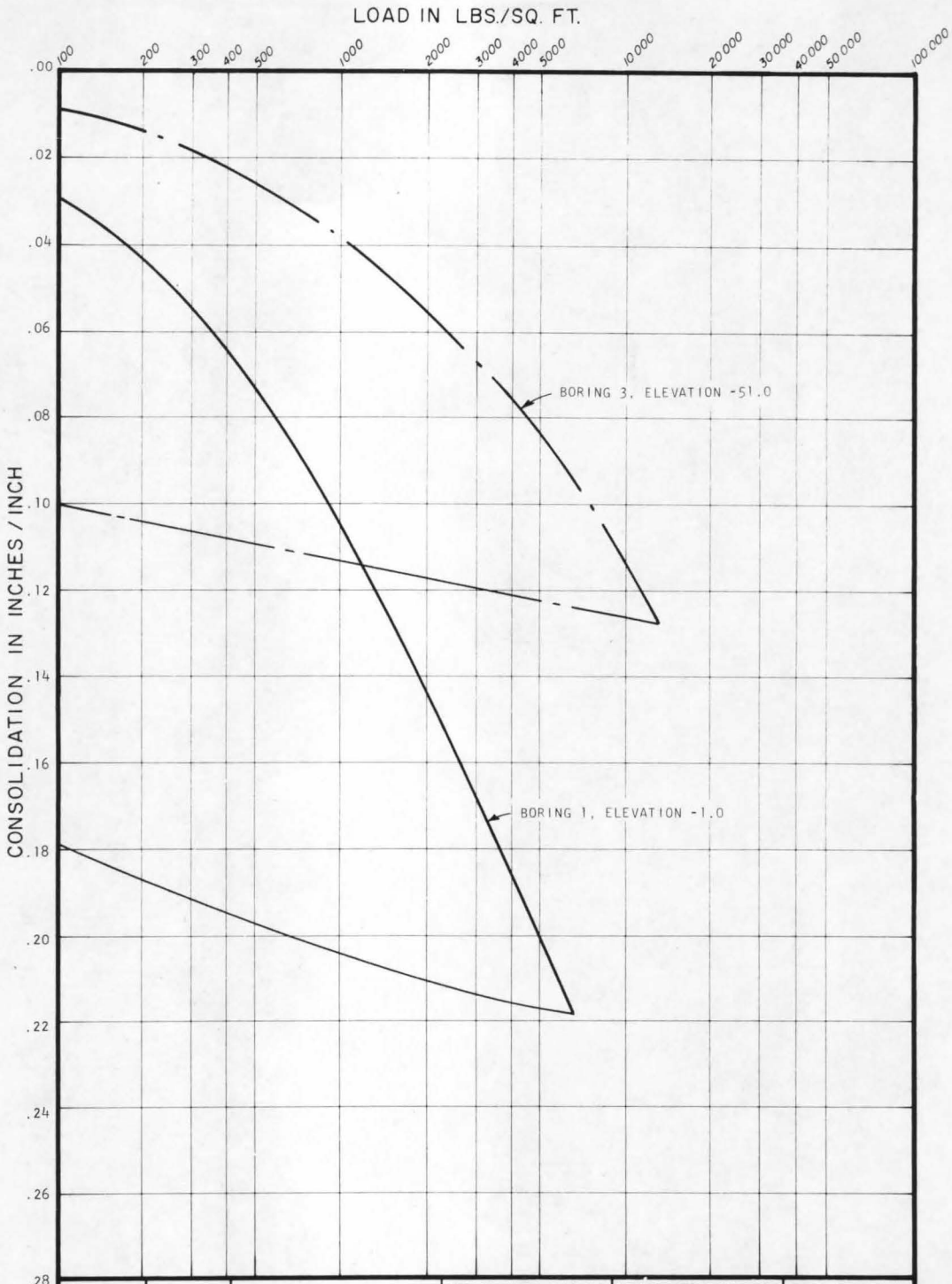
BORING	ELEVATION	SOIL TYPE	MOISTURE CONTENT % OF DRY WEIGHT	DRY DENSITY LBS./CU.FT.	CELL PRESSURE IN LBS./SQ.FT.	DEVIATOR STRESS IN LBS./SQ.FT.	1/2 DEVIATOR STRESS IN LBS./SQ.FT.
1	-15.5	CLAYEY SILT TO SILTY CLAY	55.6	66	1500	860*	430
1	-60.5	SILTY CLAY	65.7	69	4000	1680	840

* DEVIATOR STRESS CORRESPONDS TO 20% AXIAL STRAIN.

SUMMARY OF TRIAXIAL TEST DATA

DAMES & MOORE
APPLIED EARTH SCIENCES

PLATE A-7



BORING	ELEVATION	SOIL TYPE	ATTERBERG LIMITS LL	ATTERBERG LIMITS PL	MOISTURE CONTENT BEFORE	MOISTURE CONTENT AFTER	DRY DENSITY IN LBS./CU. FT.
1	-1.0	CLAYEY SILT WITH ORGANIC MATTER	54	38.5	64.7	50.6	61
3	-51.0	SANDY SILT	40	33.0	42.3	38.3	78

CONSOLIDATION TEST DATA

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SOIL MECHANICS ENGINEERS

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BY DATE
BY DATE
PLATE OF

BY 9377-001 DATE 5-23-72
CHECKED BY DATE