REPORT OF GEOTECHNICAL INVESTIGATION
PROPOSED HOSPITAL ADDITION
ABERDEEN, WASHINGTON

FOR THE

GRAYS HARBOR COMMUNITY HOSPITAL

14225-001-05
FEBRUARY 28, 1985

Dames & Moore

Dames & Moore

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February 28, 1985

Decker/Hobbs, Fukui, Davison
300 East Pike Street
Seattle, Washington 98122

Attention: Mr. Howard Meeks

Gentlemen:

We submit herewith nine copies of our "Report of Geotechnical Investigation, Proposed Hospital Addition, Aberdeen, Washington," for the Grays Harbor Community Hospital. The scope of our services was outlined in our proposal dated December 21, 1984 and our proposal addendum dated January 3, 1985. The investigation was authorized by Grays Harbor Community Hospital on January 17, 1985.

The results of the explorations and our preliminary recommendations were discussed with Messrs. Ichiro Ikeda and Jon Magnusson of Skilling, Ward, Rogers, Barkshire, Inc. at their offices on February 20, 1985.

We appreciate the opportunity of serving you in conducting this investigation. Should you have any questions concerning our findings or recommendations, please contact us. We are available to provide further consultation during design and inspection and monitoring services during site excavation and foundation construction.

Yours very truly,

DAMES & MOORE

By Joseph Lamont, Jr.
Joseph Lamont, Jr.
Senior Partner

JL:VJP:emw
9 copies submitted
14225-001-005

cc: Skilling, Ward, Rogers, Barkshire, Inc.
Attention: Mr. Ichiro Ikeda

REPORT OF GEOTECHNICAL INVESTIGATION

PROPOSED HOSPITAL ADDITION

ABERDEEN, WASHINGTON

for the

GRAYS HARBOR COMMUNITY HOSPITAL

SCOPE

We present in this report the results of our geotechnical investigation for a proposed addition to the Grays Harbor Community Hospital in Aberdeen, Washington. The addition will be constructed adjacent to the north side of the existing building as shown on the Plot Plan, Plate 1.

The purpose of our investigation is to explore subsurface soil and ground water conditions underlying the proposed site and provide geotechnical recommendations for foundation support of the structure. Specifically, the scope of our services includes:

1. Drilling three borings within the area of the proposed building addition.

2. Laboratory testing of selected samples for classification purposes and evaluation of relevant engineering properties.

3. Providing recommendations for the following items:

   a. Foundation support of the planned building addition with estimates of foundation settlement.

   b. Lateral earth pressures for design of walls supporting earth loads.

   c. Resistance to earthquake-induced lateral loads.

   d. Temporary excavation slopes with particular emphasis on their impact on adjacent building foundations.

e. Slab support and pavement design parameters.

f. Under slab drainage and basement wall drainage.

g. Site preparation, foundation subgrade preparation, fill gradation and compaction requirements, and use of on-site soils as fill.

h. Consideration of potential corrosion problems for underground utilities.

During our studies, we made reference to the following drawings:

1. Decker/Hobbs Fukui Davison, Additions and Alterations to Grays Harbor Community Hospital Design Drawings CN-1, CN-2, CN-4, CN-6, CN-7, CN-10, dated September 1983 and a preliminary foundation plan, undated.

2. Berglund Schmidt and Associates, Preliminary Topographic Survey, Grays Harbor Community Hospital, Aberdeen, Washington, dated February 1985.

3. Berglund Schmidt and Associates, Plot Plan for Grays Harbor Community Hospital, dated April 1958.

DESIGN CONSIDERATIONS

We understand that the new structure will have two stories above ground level, a basement in the northwest portion of the building, and tunnel corridors at basement level at locations shown on the Plot Plan, Plate 1. The first floor elevation in the building addition will match the existing first floor at approximately Elevation 207.[1] Only a small portion of the total building area will have a first-floor level; the remaining area at first-floor level will function as an access driveway with two additional floors above. We are advised that interior column loads will vary from 100 to 400 kips and that wall loads will not exceed 12 kips per lineal foot.

[1]Elevations in this report refer to City of Aberdeen datum.

SITE CONDITIONS

SURFACE CONDITIONS

The hospital is located at the crest of a hill in the northwest sector of Aberdeen. The site area for the planned addition is currently paved with asphaltic concrete and slopes gently to the north as shown on the Plot Plan. Site grades vary from about Elevation 207 at the north side of the existing hospital to about Elevation 202 at the north edge of the building site. The site topography steepens at the west edge of the proposed addition. The existing slope drops to the west at about 2.5 to 1 (horizontal to vertical) for a distance of about 20 feet beyond the limits of the proposed structure, and then increases in steepness to about 1.75 to 1. There is an existing retaining wall in the southwest corner of the addition site which extends about 36 feet north from the existing building. We also observed a spoils pile in the northwest corner of the site.

SUBSURFACE CONDITIONS

Subsurface conditions at the site were explored by drilling three borings with truck-mounted hollow-stem auger drilling equipment to a depth of approximately 30 feet at the locations shown on Plate 1. A detailed description of the field explorations along with boring logs and a discussion of soil testing procedures is presented in the appendix.

Our explorations indicate that the site is covered with varying thicknesses of fill material above native silts and sandstone. The silt probably derived from the weathered sandstone formation immediately underlying the silt layer. Idealized subsurface profiles are presented on Plates 2 and 3.

The fill observed in Boring 2 generally consists of loose light brown fine sandy silt and soft to medium stiff black organic silt to a depth of about 12 feet; concrete rubble or a boulder was also encountered at a depth of about 3 feet. At Boring 1, the fill is about 2 feet thick and consists of medium stiff brown clayey silt. It is

apparent that some variation in the fill soils occurs across the proposed building area. Original site topography survey information provided to us by Berglund Schmidt Associates, Inc. (1958) suggest that fill thicknesses vary from 0 to 2 feet along the east edge of the site and increase to 14 feet at the west end of the site as shown on the Fill Thickness Map, Plate 4.

The native silt layer below the fill has a relatively uniform thickness of 3 to 6 feet at the boring locations and consists of medium stiff to stiff clayey silt and medium dense fine sandy silt. The underlying weathered sandstone is a silty fine sand which increases in density with depth. The upper 3 to 5 feet of weathered sandstone is loose to medium dense followed by a 6- to 10-foot zone of dense material, the top of which occurs at about Elevation 195 at Borings 1 and 3, and approximately Elevation 185 at Boring 2. The sandstone becomes very dense at about Elevation 185 at the east end of the site, sloping to about Elevation 178 at the west end of the site.

Ground water was not encountered in any of the borings during our site exploration program. We do not anticipate the presence of any ground water from natural causes within the depth of the proposed excavations.

CONCLUSIONS AND RECOMMENDATIONS

FOUNDATION SUPPORT

General: We recommend that the new hospital addition be supported by drilled caissons or spread footings bearing on the dense or very dense weathered sandstone formation. Shallow spread footings founded on the native stiff silts above the weathered sandstone may be used for support of lightly loaded walls or columns in the eastern portion of the addition where the fill thickness is minimal. The existing on-site fill is unsuitable for supporting building foundations.

Caissons: Drilled caissons founded within the dense weathered sandstone stratum may be designed using the allowable loads shown on

Plate 5. For a given caisson diameter, founded on the dense sandstone, the allowable load will depend on the depth of caisson embedment below the lowest adjacent floor grade. Accordingly, caissons installed beneath or immediately adjacent to basement or tunnel areas should reference basement subgrade elevations in determining embedment depths, whereas caissons beneath first-floor areas should reference first-floor subgrade elevations. The allowable load values apply to total design load (dead and live) and may be increased by one-third when transient winds and/or seismic loads are included.

We expect that drilled caisson construction can generally be accomplished without installation of casing. However, depending on the condition of the on-site fill soils at specific caisson locations, occasional casing of the upper fill soils may be required. In addition, the presence of concrete rubble or other large pieces of debris in the fill may necessitate backhoe excavation of the fill soils at some foundation locations. Close coordination among the contractor, structural engineer, and soils engineer will be required to check that the caissons are founded within the dense sandstone layer and properly constructed.

We estimate that drilled caissons designed and installed in accordance with the above recommendations and supporting design loads of 100 to 400 kips would experience settlements not exceeding about 1/2 inch. The settlements will essentially occur as the loads are applied. Post-construction settlement should be negligible.

Spread Foundations: The undisturbed dense weathered sandstone will provide adequate support for wall and column footings. The native silts above the weathered sandstone will support column loads not exceeding about 100 kips and the estimated wall loads of 12 kips per foot. We recommend designing isolated spread footings and continuous strip footings founded on the dense sandstone using a maximum bearing pressure of 10,000 pounds per square foot (psf). These footings should be founded at least 2 feet below the lowest adjacent grades and have minimum footing widths of 2 feet. Isolated and strip footings founded on the native silts may be designed for an allowable soil bearing pressure of 4,000 psf. The footings should have at least 2 feet of

embedment below the lowest adjacent grade and a width of at least 2 feet. All bearing pressures refer to the total dead and live loads exclusive of the weight of the footings and backfill over the footings. The recommended bearing values may be increased by one-third when wind and/or seismic loads are included.

We estimate that isolated spread footings founded on the dense sandstone and supporting column loads of 400 kips will experience settlements of less than about 1/2 inch. Footings founded on the native silts and designed for column loads not exceeding 100 kips and 12 kips per lineal foot would experience settlements of approximately 1/2 to 3/4 inch. Settlement of foundations placed on the dense sandstone or native silts should occur essentially as the loads are applied. Post-construction differential settlements between columns and exterior walls founded on both dense sandstone and native silts with the loadings described above will be very small, generally less than 1/4 inch. All settlement estimates assume support on undisturbed materials.

FLOOR SLABS

The new addition will have floor slabs at about Elevation 195 in basement and tunnel areas and Elevation 207 in first-floor areas as shown in the Plot Plan, Plate 1. Both the first floor and basement areas are underlain by fill soil of variable thickness. Due to the soft organic nature of the fill soils observed in the electrical room area, we recommend removing and replacing these soils with granular structural fill as described in the earthwork section of this report. The granular fill should be placed in 8-inch lifts and compacted to 95 percent of maximum density. If the first floor is to be constructed as a slab-on-grade, the existing fill materials should be approved by the soils engineer during construction. Removal of existing organic fill soils, wherever they occur within the building area, would be appropriate.

We recommend placing a minimum 8-inch layer of free-draining well-graded sand and gravel directly below floor slabs placed on grade. This material should have less than 5 percent by weight passing the No. 200 sieve and be compacted to at least 95 percent of its maximum dry density as determined by ASTM Test Designation D-1557.

BASEMENT WALLS

Basement walls should be designed for either active or at rest earth pressures depending upon the amount of wall movement. For walls which allow small movements, we recommend a lateral pressure equal to an equivalent fluid density of 35 pounds per cubic foot (pcf). Where no wall movement is allowed, the lateral pressure equal to an equivalent fluid density of 60 pcf may be used. These design pressures assume that hydrostatic pressures do not develop behind the walls. In this regard, we recommend backfilling adjacent to basement walls with clean, free-draining sand and gravel having less than 5 percent passing the No. 200 sieve. This drainage material should be at least 12 inches thick against the full height of the wall and extend to a perforated drain pipe placed along the exterior base of the basement walls. The wall drains should discharge into a tight line connected to proper disposal systems. Such backfill should be compacted sufficiently to eliminate voids and possible future backfill settlement. However, care should be taken to avoid exerting undue pressure on the adjacent wall. A hand-operated vibratory plate compactor would be appropriate.

SEISMIC DESIGN RESISTANCE

Dynamic seismic loads will be resisted by passive soil pressure against the basement walls and foundations. Allowable passive resistance on basement walls, spread footings, and buried grade beams with a level ground surface may be evaluated using an equivalent fluid density of 400 pcf. Where the soil surface slopes downward away from the wall at a 2 to 1 slope such as occurs along the west side of the proposed electrical room, the passive resistance would be reduced to an equivalent fluid density of 100 pcf.

For footings founded in the sandstone, a base friction coefficient of 0.5 may be used. For footings supported on the silt, a base friction coefficient of 0.4 is recommended.

Where caissons are used, lateral resistance will be provided by passive soil pressure against the embedded portion of the caissons.

Lateral caisson capacities will depend upon permissible structural deflections of the caisson. For analysis purposes we recommend using a lateral spring-constant K which varies with depth such that

$$K = \frac{(N)(Z)}{B}$$

where

K = lateral spring-constant

N = constant of horizontal subgrade reaction

Z = depth below the surface

B = caisson diameter

The constant of horizontal subgrade reaction may be taken as 20 pounds per cubic inch in the sandstone and native silt and 10 pounds per cubic inch in the existing fill. The lateral caisson resistance for a given deflection may be determined by elastic analysis using the calculated spring constant.

EXCAVATIONS

The proposed 12-foot deep excavations for basement and tunnels will encounter existing fill and loose to medium dense native silty sand. Due to the soft nature of the fill soils observed at Boring 2, we recommend sloping excavations in these existing fill soils to 2 to 1 or flatter. At existing moisture contents the native sands and silts should support a 1 to 1 slope. If the slopes become saturated, sloughing could occur and slope flattening or shoring may be necessary. Excavations located adjacent to existing building foundations should be designed so that the excavation slopes are to be at least 5 feet horizontally from the edge of the existing footing at the footing level. If this is not possible, shoring should be used to protect the existing footings.

PAVEMENT DESIGN

Based on the quality of the existing fill soils observed at Borings 1 and 2, we recommend designing pavements using a CBR value of 5. If the existing fill soils are replaced with compacted granular material, a CBR value of 15 may be used.

EARTHWORK RECOMMENDATIONS

Site Preparation: We recommend preparing the site by removing the asphaltic concrete pavement that covers most of the site, and removing the spoils pile from the northwest corner of the site. The spoil materials were not sampled, but based on cursory observations these soils appeared to consist of native silty fine sand, suitable for reuse as backfill.

Foundation Subgrade Preparation: Spread foundation subgrades will consist of native silt and silty sand. These soils can become disturbed during wet weather. We recommend that the contractor protect the foundation-bearing surface either by pouring a thin slab of lean concrete or by placing an 8-inch thick layer of clean sandy gravel with less than 5 percent by weight passing the No. 200 sieve. This material should be compacted to 95 percent of maximum density (ASTM D-1557) with a hand-operated vibratory plate compactor. Prior to placing such a working pad, the contractor should remove all ponded water and zones of soft soil that cannot readily be compacted.

Use of On-Site Native Soils: During drier weather, the weathered sandstone removed from basement excavations may be used as structural fill but not as drainage material beneath slabs or behind basement walls. However, we anticipate that during wet or rainy weather the native soils will be difficult to properly place and compact as structural fill. Accordingly, in inclement weather we recommend using clean granular material with less than 5 percent passing the No. 200 sieve. Due to the high organic content of the existing fill soils observed in Boring 2, these soils will not be suitable for use as compacted fill and should be removed from the site.

MISCELLANEOUS CONSIDERATIONS

The native soils do not exhibit expansive or chemically active properties. Based on conversations with the City of Aberdeen's Director of Public Works, we have learned that corrosion of underground pipe lines is not a concern in the Aberdeen area. This opinion was corroborated by the Grays Harbor Community Hospital plant engineer.

CONSTRUCTION OBSERVATIONS

Because of the nature of the soil conditions underlying the project area and the potential adverse circumstances arising from construction activity and weather conditions, we recommend that a geotechnical engineer observe site preparation, excavation, selection and placement of structural fill and drainage fill, construction of caisson foundations and backfilling of basement walls. Special attention will be required to verify the location of the dense weathered sandstone bearing layer for spread foundations or drilled caisson construction. Dames & Moore will be pleased to provide these construction monitoring services.

CLOSURE

The recommendations presented in this report are provided for design purposes and are based on soil conditions disclosed by field observations and the exploration borings. Subsurface information presented herein does not constitute a direct or implied warranty that the soil conditions between boring locations can be directly interpolated or extrapolated or that subsurface conditions and soil variations different from those disclosed by the borings will not be revealed. The recommendations outlined in this report are based also on the assumption that the plan location and development details of the building are consistent with that shown on Plate 1 and the description provided in this report.

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The following plates and appendix are attached and complete this report:

Plate 1 - Plot Plan

Plate 2 - Subsurface Profile Section A-A'

Plate 3 - Subsurface Profile Section B-B'

Plate 4 - Fill Thickness Map

Plate 5 - Caisson Design Capacities

Appendix A - Site Exploration and Laboratory Tests

Plates A-1 and A-2 - Log of Borings

Plate A-3 - Unified Soil Classification System

Plate A-4 - Consolidation Test Data

Plate A-5 - Summary of Triaxial Test Data

Respectfully submitted,

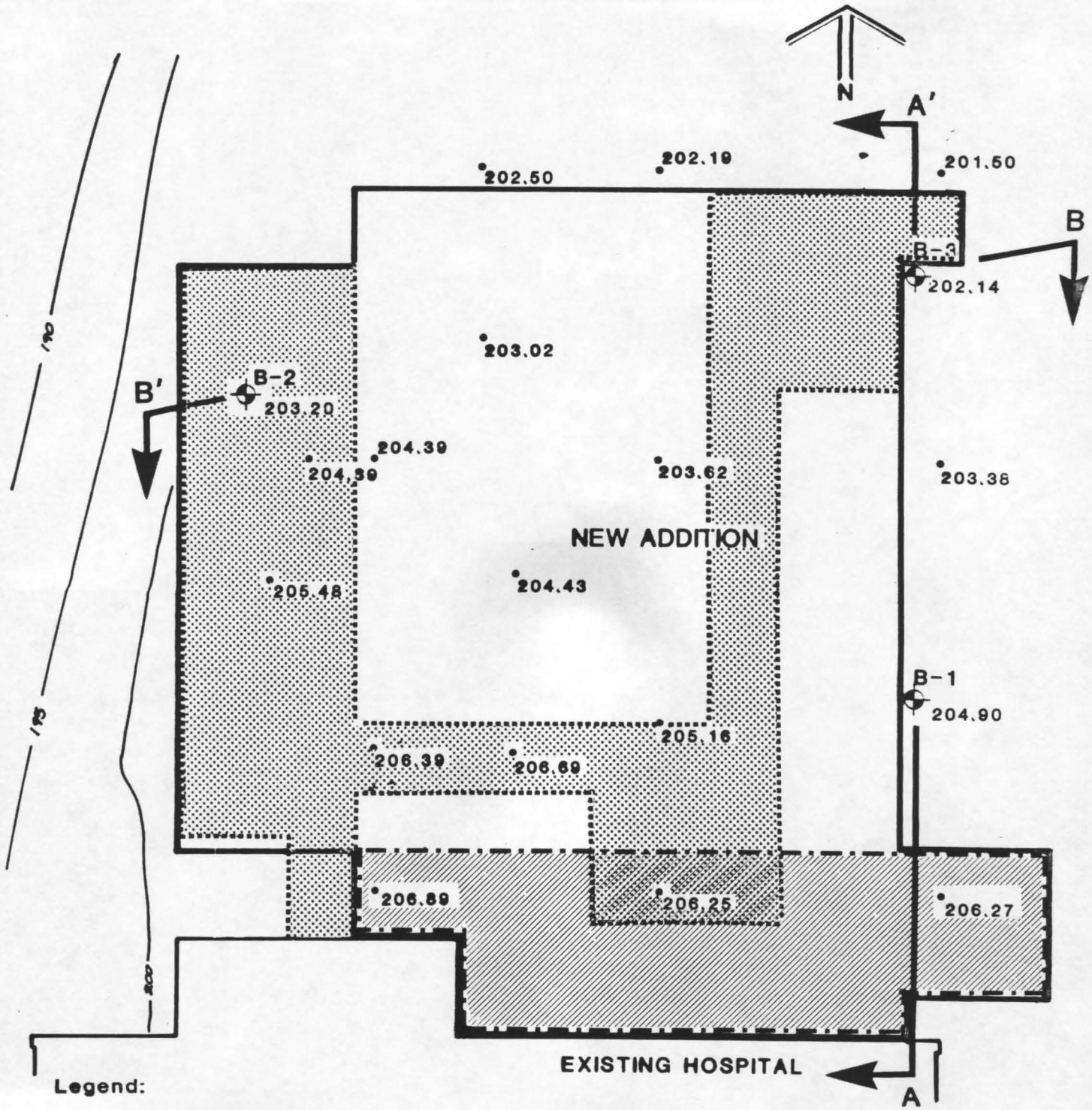
DAMES & MOORE

By Joseph Lamont Jr.
Joseph Lamont, Jr.
Senior Partner

And Vincent J Perrone
Vincent J. Perrone, P.E.
Project Engineer

February 27, 1985

Dames & Moore



Legend:

B-1 Boring Location

203.38 Existing Ground Surface Spot Elevations (Aberdeen City Datum)

First Floor Area

Basement Area

Plot Plan

0 20

Horizontal Scale (in feet)

Reference drawings noted in text

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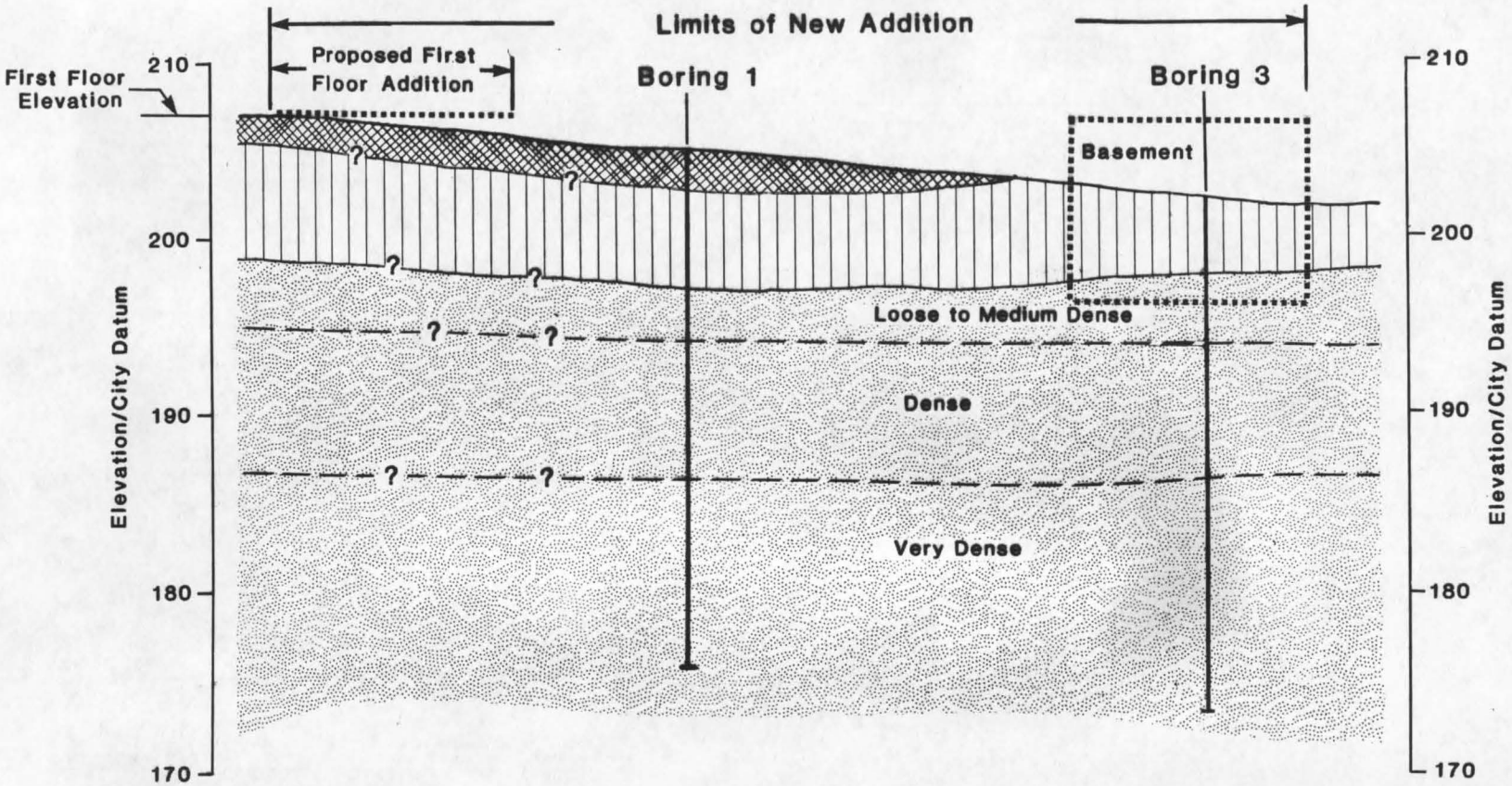
Plate 1



Notes:

1. Section lines are located on Plate 1
2. Subsurface profile is accurate only at boring locations and is an approximate interpretation elsewhere

Legend:

- Fill
- Sandy or Clayey Silt
- Weathered Sandstone

Subsurface Profile
Section A–A'

0 20
Horizontal Scale
(in feet)

Job No. 14225-001

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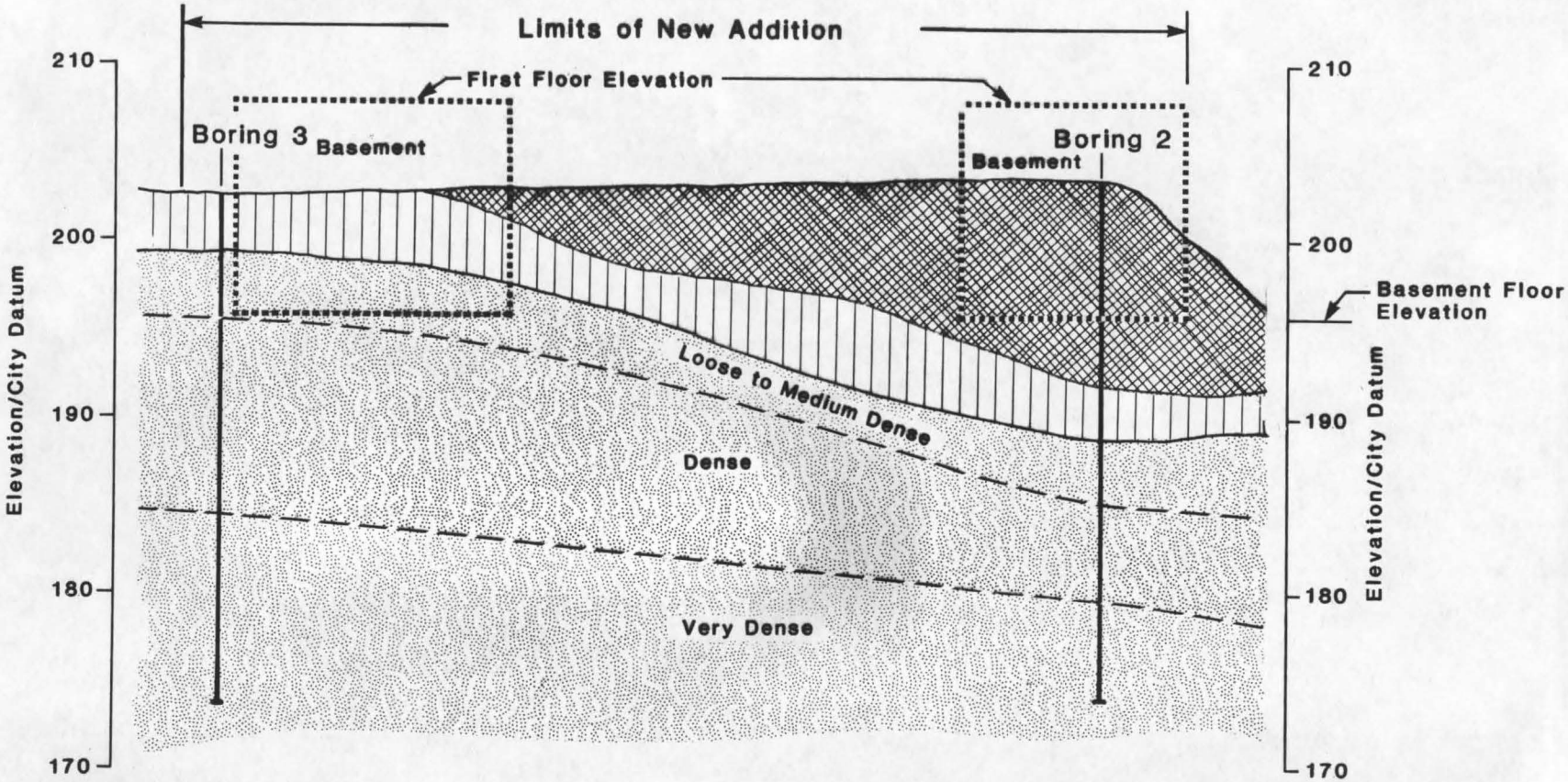
Plate 2



Notes:

1. Section lines are located on Plate 1
2. Subsurface profile is accurate only at boring locations and is an approximate interpretation elsewhere

Legend:

- Fill
- Sandy or Clayey Silt
- Weathered Sandstone

Subsurface Profile
Section B–B'

0 20
Horizontal Scale
(in feet)

Job No. 14225-001

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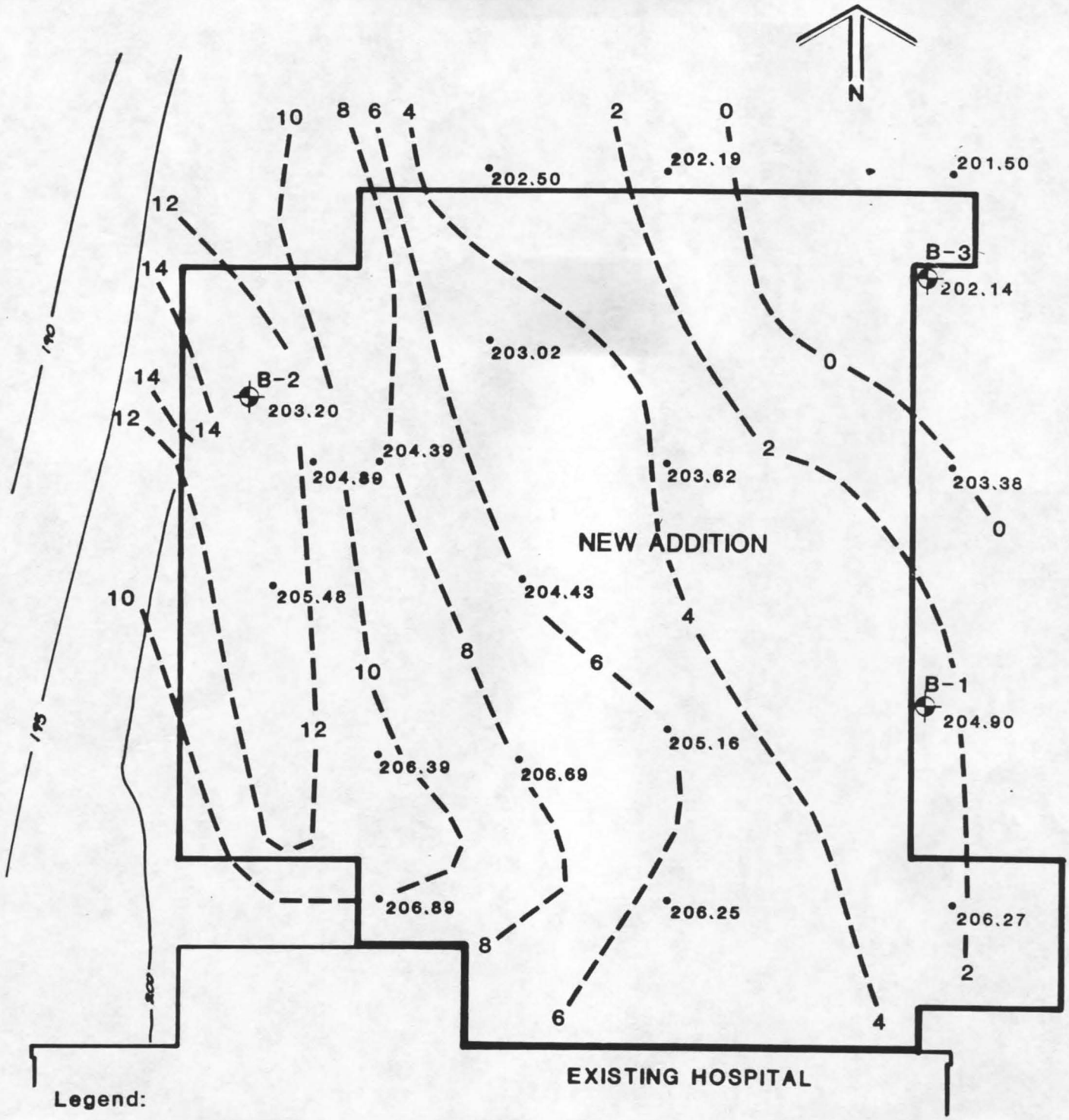
Plate 3



Legend:

B-1 Boring Location

203.38 Existing Ground Surface Spot Elevations (Aberdeen City Datum)

6 Fill Thickness in Feet

Fill Thickness Map

0 20

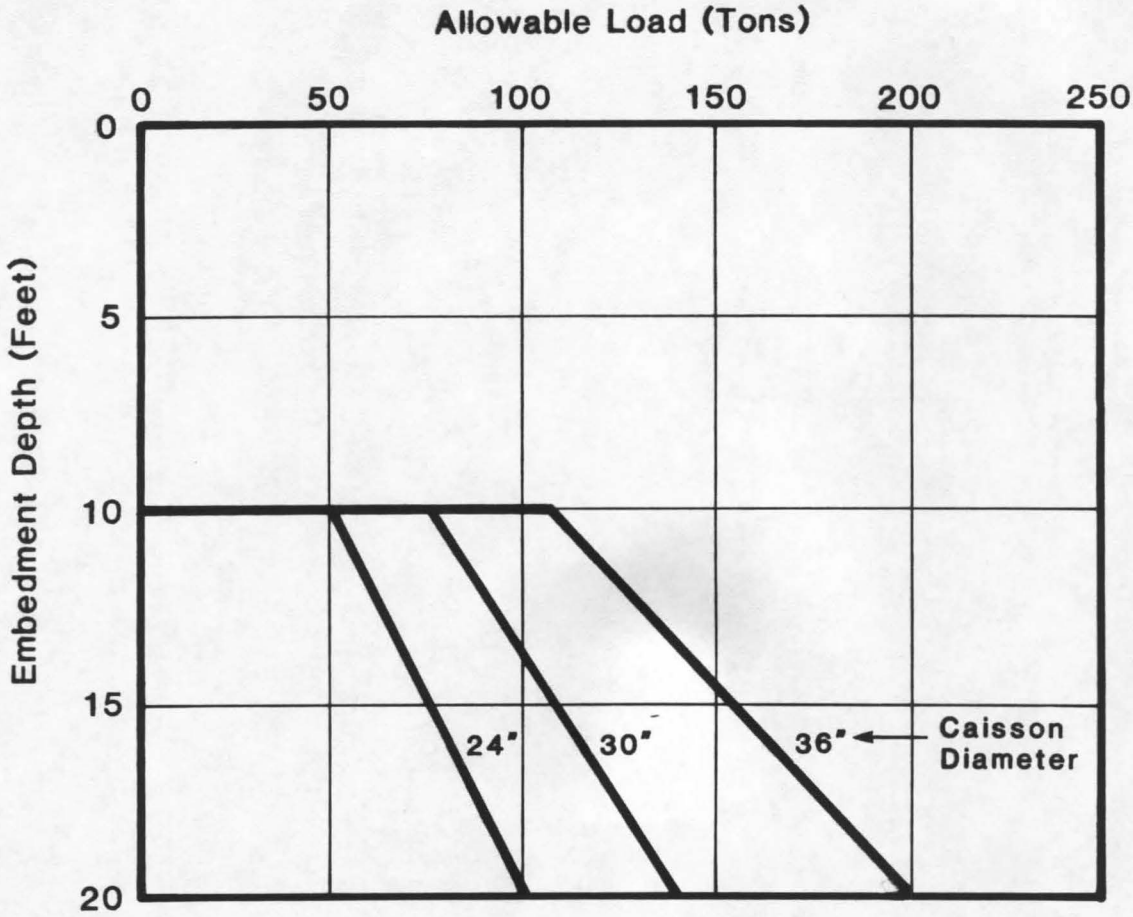
Horizontal Scale (in feet)

Reference drawings noted in text

Dames & Moore

Job No. 14225-001

Plate 4



Notes:

1. Values apply for caissons bearing on dense sand.
2. Zero depth refers to first floor subgrade elevation if no basement is present and basement floor subgrade elevation if basement is present.
3. Refer to text for complete discussion of design capacities shown on this plate.

Caisson Design Capacities

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APPENDIX

SITE EXPLORATIONS AND LABORATORY TESTS

SITE EXPLORATIONS

The subsurface and ground water conditions at the site were investigated by drilling three exploratory borings at the locations shown on Plate 1, utilizing truck-mounted hollow stem auger drilling equipment. The borings were drilled to depths of approximately 30 feet.

A Dames & Moore field representative located the borings, classified the soils encountered, maintained a log of each boring, and obtained soil samples for visual examination and laboratory testing. The soils penetrated at the boring locations are described on the Log of Borings, Plates A-1 and A-2. The soils have been classified in accordance with the Unified Soil Classification System described on Plate A-3.

Soil samples were obtained from the borings using a Dames & Moore Type U Sampler, illustrated on page A-3. The sample was advanced with a downhole hammer weighing 325 pounds and having a free-fall of 30 inches. The number of blows required to drive the sampler are presented on the Log of Borings.

Elevations of the ground surface at the boring locations were determined by Berglund, Schmidt & Associates, Inc., surveyors of this project site. The elevations are presented on the boring logs and refer to the City of Aberdeen datum.

LABORATORY TESTS

Laboratory tests were performed on selected samples obtained from the borings in order to aid classification of the soils and assist in evaluating engineering characteristics. The laboratory testing program consisted of several tests described as follows:

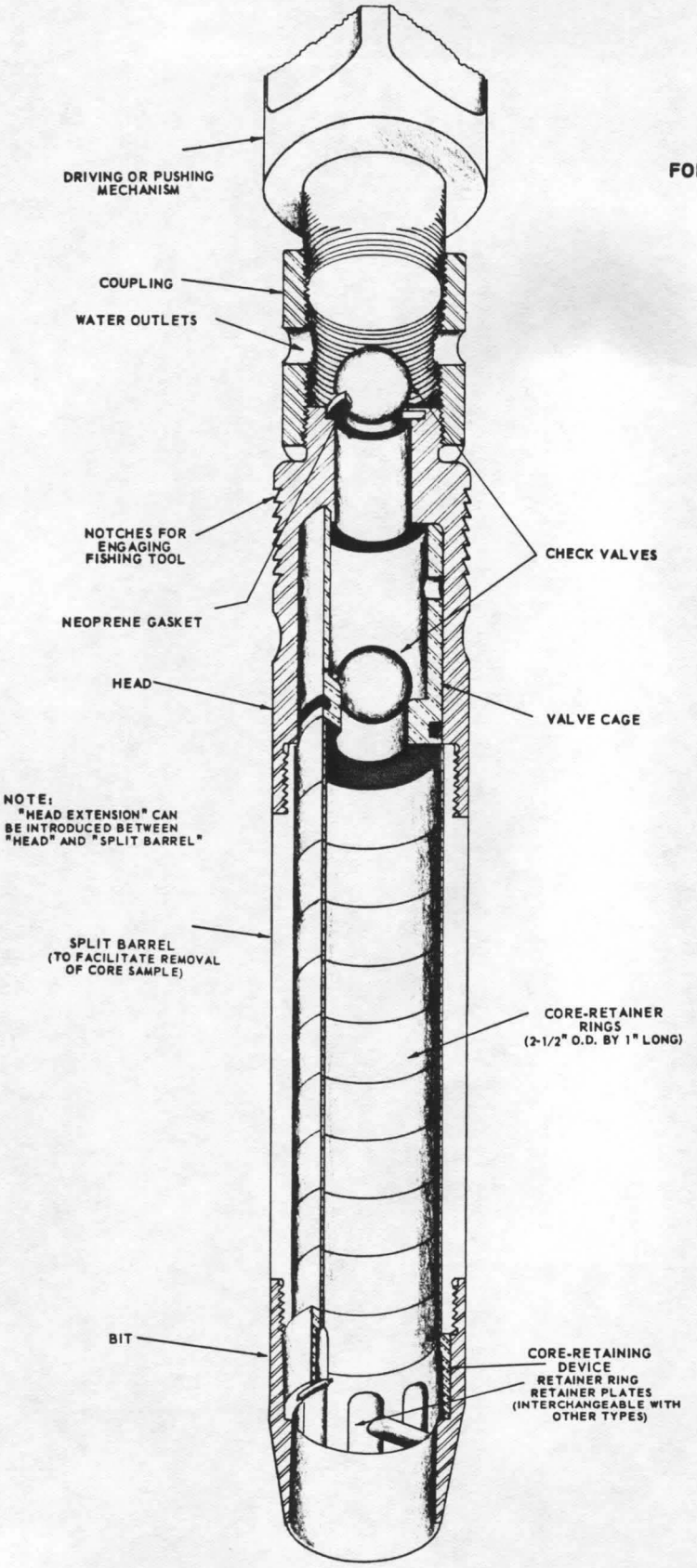
Moisture Density Tests: Moisture and density tests were performed on selected samples in accordance with ASTM Test Designation D 2216-80. Results of these tests appear adjacent to the appropriate sample notations on the Log of Borings, Plates A-1 and A-2.

Consolidation Test: A consolidation test was performed on a representative sample of sandy silt to provide data for evaluating the compressibility characteristics of this soil. The consolidation test was performed in accordance with the method described on page A-4. Results of the consolidation test appear on Plate A-4.

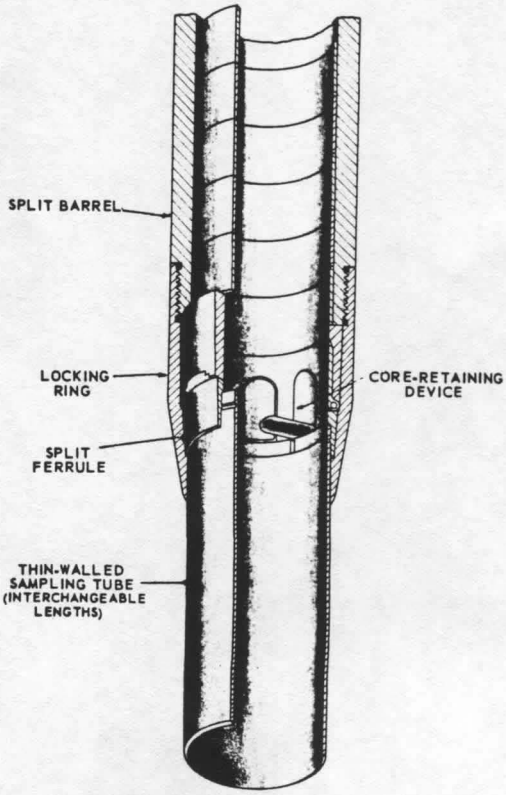
Triaxial Compression Tests: Unconsolidated, undrained triaxial compression tests were performed to evaluate the undrained shear strength and Youngs Modulus for the native silts and weathered sandstone soils. The tests were performed as described on page A-5 and the results appear on Plate A-5.

SOIL SAMPLER TYPE U

FOR SOILS DIFFICULT TO RETAIN IN SAMPLER



ALTERNATE ATTACHMENTS



417.9 (5-68)

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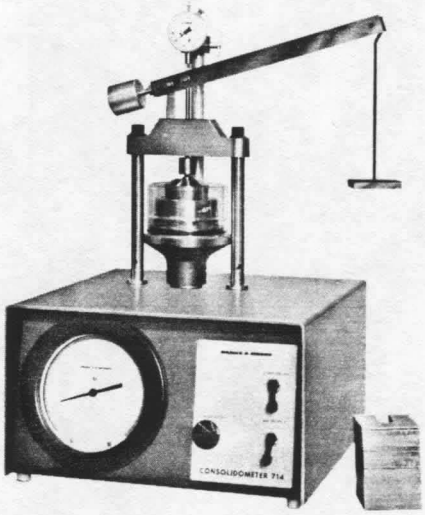
METHOD OF PERFORMING CONSOLIDATION TESTS

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.



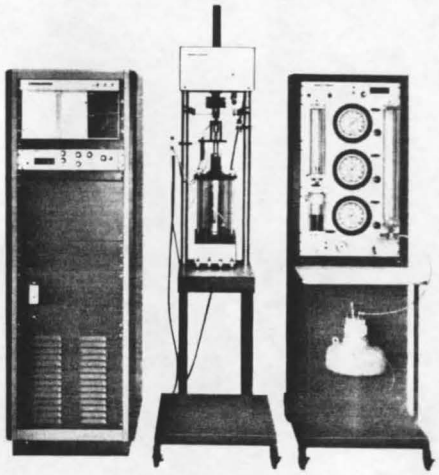
DEAD LOAD-PNEUMATIC CONSOLIDOMETER

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.

FORM NO. 417.3 (Rev. 1-75)

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:

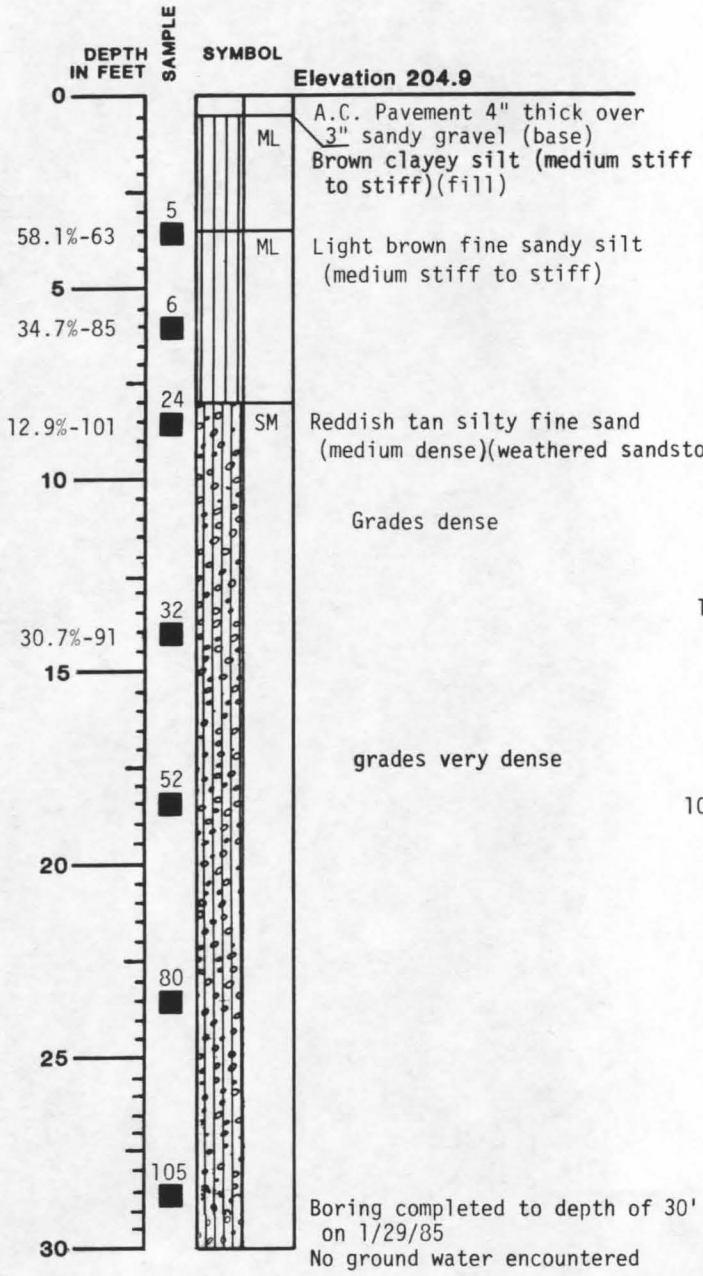
UNCONSOLIDATED-UNDRAINED: THE CONFINING PRESSURE IS IMPOSED ON THE SAMPLE AT THE START OF THE TEST. NO DRAINAGE IS PERMITTED AND THE STRESSES WHICH ARE MEASURED REPRESENT THE SUM OF THE INTERGRANULAR STRESSES AND PORE WATER PRESSURES.

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.

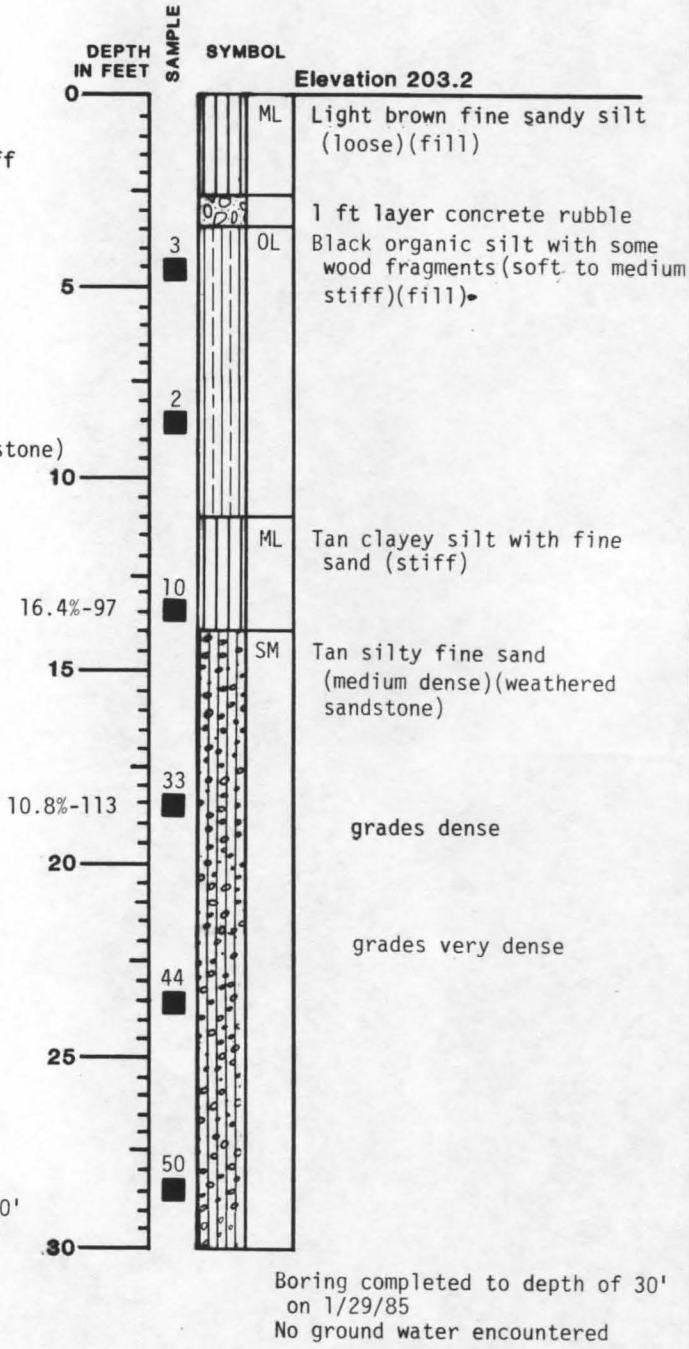
DRAINED: THE INTERGRANULAR STRESSES IN A SAMPLE MAY BE MEASURED BY PERFORMING A DRAINED, OR SLOW, TEST. IN THIS TEST THE SAMPLE IS FULLY SATURATED AND CONSOLIDATED PRIOR TO THE START OF THE TEST. DURING THE TEST, DRAINAGE IS PERMITTED AND THE TEST IS PERFORMED AT A SLOW ENOUGH RATE TO PREVENT THE BUILDUP 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.

Boring 1



Boring 2



Key:

MOISTURE CONTENT → 12.9%-101 ← DRY DENSITY IN PCF

BLOWS REQUIRED TO DRIVE DAMES & MOORE SAMPLER ONE FOOT WITH A HAMMER WEIGHT OF 325 LBS. AND A STROKE OF 30 INCHES.

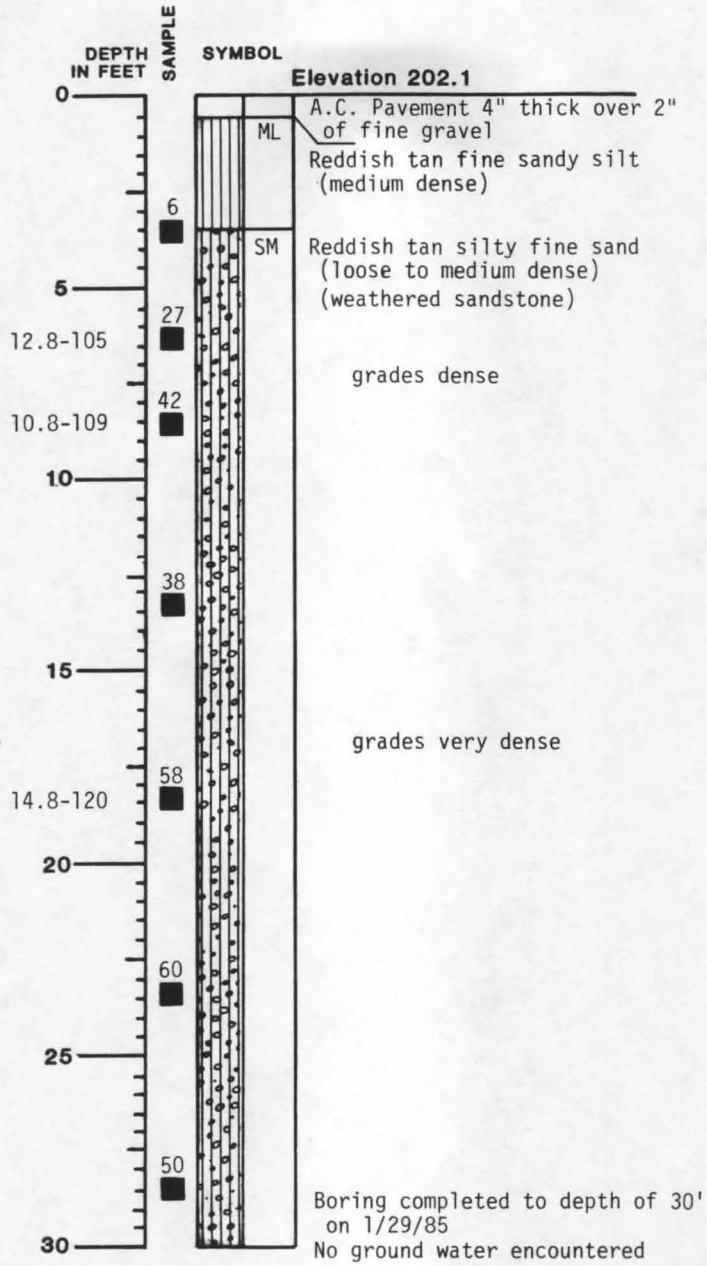
■ INDICATES DEPTH AT WHICH UNDISTURBED DAMES & MOORE SAMPLE WAS EXTRACTED.

Note: Elevations refer to City of Aberdeen datum.

Log Of Borings

Dames & Moore

Boring 3



Log of Borings

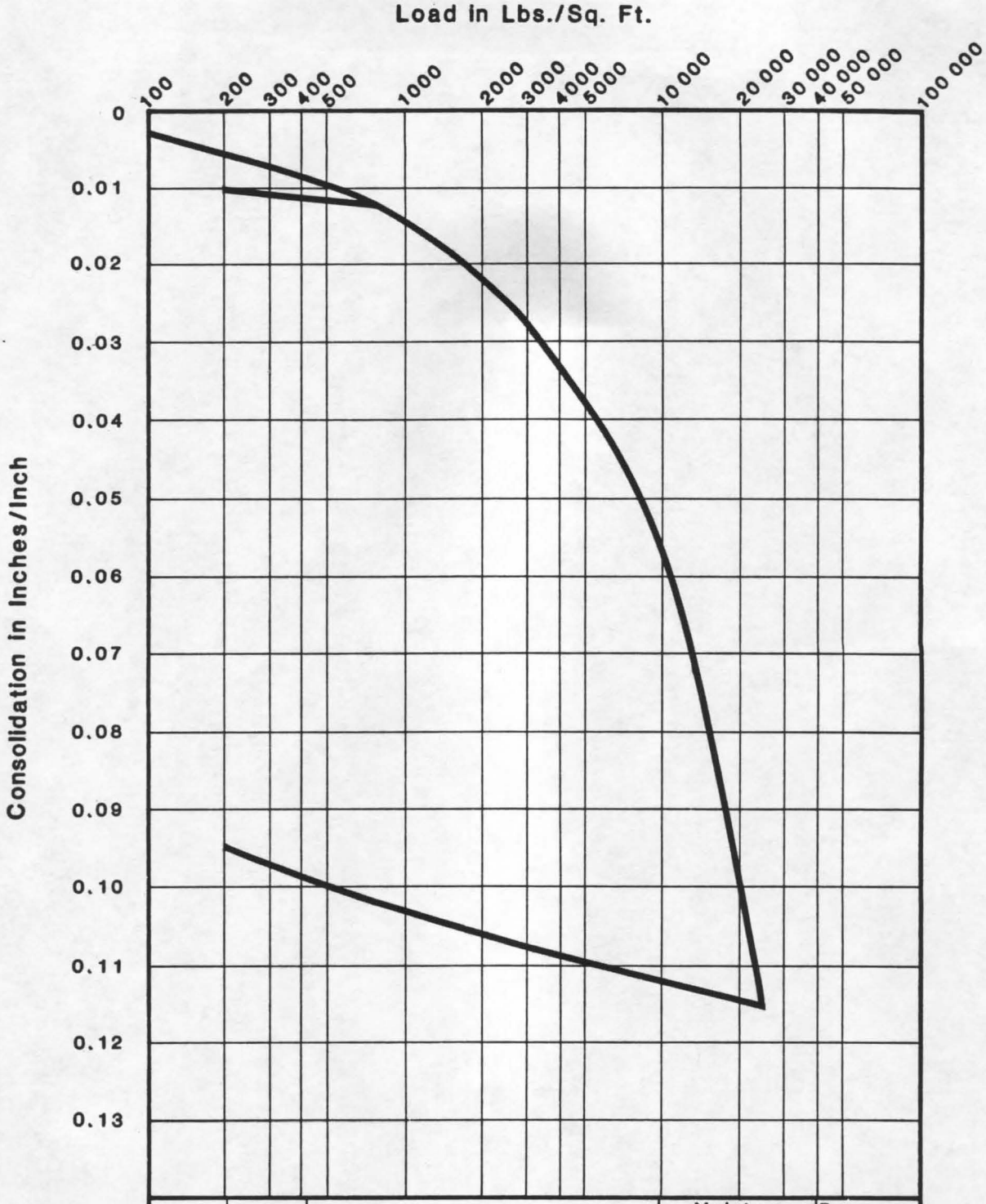
Dames & Moore

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 silt 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 plasticity, organic silts.
Highly Organic Soils				PT	Peat, humus, swamp soils with high organic contents.

Note: Dual symbols are used to indicate borderline soil classifications.

Unified Soil Classification System

Dames & Moore



Boring	Depth	Soil Type	Moisture Content		Dry Density in Lbs/Cu Ft
			Before	After	
1	6'	Sandy silt	34.7	31.9	85

Consolidation Test Data

Dames & Moore

Boring	Depth (Feet)	Soil Type	Moisture Content % of Dry Weight	Dry Density Lbs./Cu. Ft.	Cell Pressure in Lbs./Sq. Ft.	1/2 Deviator Stress in Lbs./Sq. Ft.	Youngs Modulus Lbs./Sq. Ft.
1	3.5	Clayey silt	58.1	63	500	6300	500,000
1	13.5	Silty fine sand	30.7	91	1600	5280	810,000
3	18.5	Silty fine sand	14.8	120	2500	9960	1,750,000

Summary of Triaxial Test Data