SOILS INVESTIGATION

PROPOSED KALANI IKI CONDOMINIUM DEVELOPMENT
HONOLULU, OAHU, HAWAII

FOR

ISLAND FEDERAL SAVINGS & LOAN ASSOCIATION
OF HONOLULU

Dames & Moore Job No. 3038-004-11

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October 14, 1971

Hogan & Chapman Architects
1210 Auahi Street
Suite 115
Honolulu, Hawaii 96814

Attention: Mr. Donald D. Chapman, AIA

Gentlemen:

Six copies of our report, "Soils Investigation, Proposed Kalani Iki Condominium Development, Honolulu, Oahu, Hawaii, for Island Federal Savings & Loan Association of Honolulu", are herewith submitted.

The scope of our work was defined in our confirming proposal dated September 1, 1971, and this investigation has generally conformed to the scope described in the proposal. The area of our consideration was enlarged somewhat during the investigation when the westernmost section of the project site was included in the area for possible development. However, no subsurface exploration was conducted in that area.

Our findings and recommendations are presented in the body of the report. For convenient reference, a summary is given on the first page. Our recommendations have been discussed with Messrs. Chapman and Johnson of your firm and Mr. David Dawes of Island Federal Savings & Loan Association. Also, draft copies of this report were furnished to you on October 6, 1971.

Selected soil samples were used in laboratory testing, and the remaining soil samples and rock cores will be kept for a period of time for possible inspection and examination. Unless requested otherwise, they will be discarded six months from this date.

It has been a pleasure performing this assignment for you. If you have any questions regarding this report, please feel free to contact us for clarification.

Yours very truly,

DAMES & MOORE

David C. Liu

David C. Liu

DCL DRR mw

SOILS INVESTIGATION

PROPOSED KALANI IKI CONDOMINIUM DEVELOPMENT

HONOLULU, OAHU, HAWAII

FOR

ISLAND FEDERAL SAVINGS & LOAN ASSOCIATION OF HONOLULU

SUMMARY

Subsurface conditions were explored by three borings and four test pits.

Most of the project site is underlain by a mixture of expansive clayey silts and weathered boulders. Previously placed fill apparently covers the surface of much of the western portion of the site. Basaltic bedrock is exposed on the steeper slopes and generally was found in the borings within 20 feet of the surface.

The expansive soils and recent fill pose problems regarding slope stability and foundation design: Low-rise structures generally should not be placed on existing slopes steeper than 5:1 and all footings should rest on natural soils, not fill. Design bearing pressures of 4500 psf are recommended for footings on expansive soil, but differential vertical movement between footings should be anticipated. Earthwork should be kept to a minimum.

Diverting all surface drainage away from structures is critical to minimizing problems regarding stability and footing movements.

INTRODUCTION

This report presents the results of our soils investigation at the site of a proposed new condominium development in Kalani Iki Valley in Honolulu, Oahu, Hawaii. Our recommendations regarding the development are presented in the last section of this report. Details regarding our field work and laboratory testing are presented in the attached Appendix.

SCOPE OF WORK

Our investigation was conducted with the understanding that the development would consist of low-rise multiple-unit townhouse structures located entirely on the eastern side of the existing north-south paved road on the property. It was suspected that a scar on the western side of this road might have resulted from a landslide, and it was believed advisable not to consider that area for development. However, present information indicates that the scar is from previous excavation, and the area west of the road may be considered for development. Additionally, the construction of one or two medium-rise structures may be considered as an alternative development approach to the low-rise scheme.

The scope of our field work was based on the original development approach and consisted of three borings and four test pits plus surface geological reconnaissance. A review of previous development plans and drawings for the area also was conducted in an attempt to establish the extent of earthwork which had been performed previously at the site.

Appropriate laboratory soil tests were made and evaluated to assist in developing our recommendations which are presented later in the report.

A preliminary draft of this report was submitted to Hogan and Chapman Architects on October 6, 1971 for their advance information.

SITE CONDITIONS

SETTING

The project site is located in Kalani Iki Valley immediately upstream from the existing developed area. The general location is shown on the Map of Area, Plate 1, and a more detailed site plan is presented on Plate 2.

The area lies between elevation +310 and +210 in a zone receiving an approximate average annual rainfall of 40 inches. A large percentage of this precipitation occurs as a few large storms each year during the late fall, winter and early spring. During the remainder of the year and between storms, the site is much drier.

SURFACE CONDITIONS

As shown on Plate 2, most of the site lies within the main valley, but the southeastern corner occupies the mouth of a tributary valley. The main channel is dry except during heavy rainfall. It has been confined to a rectangular section about 10 to 15 feet wide and about 10 feet deep. Sides of this channel consist of large boulders with a gunite facing and possibly concrete mortar between the boulders. The bottom of the channel is poured concrete. Upstream from the development site, the City and County of Honolulu has constructed a low wier, apparently for the purposes of decreasing velocity and volume of flow and possibly for containing boulders being washed down the channel. These drainage structures indicate that anticipated runoff during heavy storms is considerable. The tributary from the northeast apparently has been replaced by a buried 72-inch corrugated metal pipe culvert.

The gradient within the main channel is between 10 and 15 percent, which reflects the general slope of the ground surface along the axis of the main valley. Slopes perpendicular to the drainage are relatively flat for a width of about 250 feet in the lowest section of the site, but the flat area tapers out about three-fourths of the way upstream toward the property line.

The eastern boundary of the main valley is a steep cliff formed of basalt bedrock. The western slope below the existing paved road is believed to be composed of recent fill over bouldery alluvium. The slope below the road has an average inclination of about 2:1 (horizontal: vertical). Above the road is a scar which has some of the appearances of a landslide, but which we believe to be an old cut area, possibly for a roadway which was not completed.

The upper end of the property within the main valley is a relatively flat zone about 20 feet above the bottom of the channel. This flat probably was formed when material removed from the area of construction of the wier upstream was dumped here.

The section of the property at the mouth of the tributary valley is composed primarily of bouldery alluvium with slopes averaging approximately 4:1 downward in a westerly direction. At the base of this area, a steep road cut at least ten feet high has been excavated during previous earthwork operations.

Some signs of soil creep were observed on the western slopes above the exavation scar beyond the property boundaries. Indications of slow downward movement of surface soils are also suggested by longitudinal cracking in the asphaltic concrete of the north-south road. This can be explained as consolidation of fill materials under the outer edge of the road and thus would be an indication of relatively poor soil conditions in that slope.

SUBSURFACE CONDITIONS

Detailed subsurface conditions were explored by three borings and four test pits at the locations indicated on Plate 2. Additionally, geological reconnaissance was conducted to evaluate overall subsurface conditions. A generalized cross-section across the main valley is presented on Plate 3.

Along the eastern edge of the main drainage channel and in the area between the channel and the north-south paved road, most of the existing surface soil appears to be recently placed fill. The approximate limits of the area underlain by more than two or three feet of fill are shown on Plate 2. We believe that this fill was placed during road construction, during drainage channel construction and as waste from the City and County wier upstream. Thus, the fill material probably was derived from adjacent areas. It consists of a mixture of weathered basaltic boulders and clayey silt fines which is similar but possibly rockier than in-place material. Overall density of the fill material is suspected to be low, and the material is not recommended for support of structures unless reworked.

Undisturbed soils consist primarily of dark brown expansive clayey silts of the type which locally is referred to as "adobe". This material surrounds weathered basaltic boulders up to at least one cubic yard in volume. Normally, the soil constitutes about two-thirds of the material and the boulders about one-third. On the east side of the main channel at Boring 3, this material extended to a depth of about 17 feet at which point basaltic bedrock was encountered. On the west side of the channel at Boring 1, decomposed basalt was encountered at about 11 feet, but unweathered bedrock was not encountered to the total depth explored of 25 feet.

In the southeastern corner of the property, the soils also consist of weathered basalt boulders and clayey silt fines. The fine material appears to be less expansive in this area. The boulders generally comprise the bulk of the material, although much of the granular material is very weathered or decomposed. Basaltic bedrock was encountered in this area in Boring 2 at a depth of 19 feet.

Weathered basaltic bedrock has been exposed along the western edge of the property by excavation at those locations shown on Plate 2. Similar material is exposed naturally on the eastern margin of the property along the cliff line also shown on Plate 2.

No water was encountered in any of the borings or test pits. The ground water table can be safely assumed to be within the basaltic bedrock well below the ground surface and will not be a factor in design considerations.

PROJECT CONSIDERATIONS

The exact layout of the development has not yet been determined. At the start of our investigation, the development was to consist of approximately 60 condominium units, possibly in duplex and quadruplex configurations. The contemplated design was to use "pole" structures up to four stories high with staggered levels paralleling existing slopes. Wooden poles would be the principal vertical members, and frame construction would generally be used. We understood that the results presented in this report would assist in determining final siting of the structures.

At the time of writing, we understand that fewer structures of taller configurations may be considered as an alternative to achieve the same density of units on the site.

DISCUSSIONS AND RECOMMENDATIONS

SITING

Because of the unknown, but probably poor, quality of fills placed in the slopes below the existing roadway and near the upper end of the site, we recommend that structures be located so that they do not lie on these slopes. This recommendation is made for two reasons: 1) footings founded within the fill materials could be subject to large settlements, and 2) the expansive soils underlying the fills could exhibit a marked decrease in strength if ever saturated. Since the original ground surface under the fill was a slope at about a 3:1 inclination, decreased soil strength along this surface could create a slope stability problem.

Within that portion of the site adjacent to and on either side of the main drainage channel, we recommed that all low-rise structures be constructed on existing slopes no steeper than 5:1 if spread footings on soils are used. For steeper areas, footings should be extended to intact bedrock.

In the southeastern corner of the property, low-rise structures may be sited on the existing slopes, although minor regrading may be appropriate.

Structures several stories high could be located on the eastern side of the main drainage channel where topography is relatively flat, and material adequate for supporting heavier loads is anticipated within about 20 feet of the surface. The southeastern area of the property also appears to be underlain by material suitable for support of taller structures within about 20 feet of the surface.

EARTHWORK

We recommend that all earthwork undertaken at the site be as light as possible. As a guide, we recommend that cuts or fills higher than five feet generally be avoided. All fill slopes should be no steeper than 2:1, but cut slopes in existing soils may be as steep as 1:1 up to ten feet high if the final plan calls for no structures within horizontal distances of 20 feet above the cut or 10 feet below the cut. Cuts in basaltic bedrock may be made at ½:1.

Cut and fill soil slopes at 2:1 can be revegetated so as to avoid erosion; steeper slopes will be more difficult to revegetate and cut slopes at 1:1 or steeper probably will erode.

When used for fill, the on-site materials should have all boulders greater than six inches removed, particularly if the fill will be in support of structures or pavements. We recommend such fills be compacted, depending on purpose and location, to a minimum relative compaction of 85 or 90 percent of the maximum dry density determined by AASHO Test Method T-180.

Excavation can be accomplished with normal earthmoving equipment, but the larger boulders may pose some difficulties to excavation, particularly for footings and utility lines. On-site fill materials should be placed with footed or grid-type rollers, and addition of moisture to the materials may be required to achieve adequate compaction during dry periods. Earthwork during periods of heavy rainfall should be avoided, because compaction would be very difficult, and loss of soil by erosion would occur.

SLOPE STABILITY

The unfortunate occurrence of a few landslides in Honolulu in soil conditions similar to those found at the project site has led to the supposition that hillsides underlain by "adobe" soils are undergoing a slow downslope movement referred to as "soil creep". This continuing movement may be accelerated by cuts and fills made in association with development and, in the worst cases, become active landslides.

Normally, the soils at the site will lose strength if given access to a sufficient amount of water to expand. This condition is not anticipated under normal circumstances, but excessive irrigation of lawns, poor surface runoff drainage patterns, or water leaking from utility lines all can introduce more moisture into these materials than they now possess. It is likely that most of the landslides which have occurred in Honolulu have been triggered by similar changes in soil moisture conditions.

The only signs of existing slope instability noted at the site were on the western slopes above the excavation scar where the top two or three feet of soil appeared to be slowly moving downward, probably as a reaction to the removal of support by excavation. Cracking on the north-south roadway pavement may indicate that the roadway fill is moving slightly relative to underlying in-place soils. To avoid potential slope stability problems as much as possible and still make use of the property, we recommend that no low-rise structures be founded on existing slopes steeper than 5:1 which are underlain by expansive soils. This recommendation applies primarily to that section of the site on both sides of the main drainage channel and east of the existing north-south paved road.

The slight possibility of a boulder rolling from outside the property down into the development will always exist. Low walls along the property line could provide some protection. Also, a flat parking area above any structure would also provide a potential catchment area for a boulder. Erosion and minor sliding of soil immediately above cut slopes also are possible, particularly if the soil is poorly vegetated.

FOUNDATIONS

The primary consideration for foundation design of low-rise structures at the project site is to minimize the effects of potential expansion and contraction in the expansive soils. Two primary recommendations can be made in this regard. These are:

1) To impose as heavy bearing pressures as is possible under conscientious engineering design and
2) To place the base of the footings on material where moisture changes will be minimal.

We believe that design bearing pressures of 4500 psf may be used for footings on the bouldery alluvial soils at the project site. All such footings should be founded on in-place materials, not on fill, and should rest a minimum of three feet below final outside grade. Plate 4 is an illustration of our recommended footing configuration. Additionally, the material at the base of all footings should be as homogeneous as possible, i.e. not part boulder and part soil. In this regard, we recommend that all footing excavations be inspected and approved by a soils engineer during construction. Backfill around the footings should be compacted on-site soils with no rock fragments larger than two inches.

Since the soils at the site appear to be variable in their expansive properties, it is possible that some slight settlement could occur under footings loaded to 4500 psf. Such settlement would be most probable if the underlying soil were saturated. Thus, we believe that differential movement between footings could be at least as great as one inch, depending on actual loads, soil variability and subsequent changes in soil moisture.

A design feature which can be considered to minimize the effects of differential movement is that of adjustable column-to-footing connections. Under the best design and construction techniques, it is still possible that some expansion and/or contraction may take place under the footings with resulting distress to the overlying structure. The ability to adjust the structure supports will help to correct this situation, should it occur. Two approaches are possible:

1) Leaving a break between the column and footing where shimming may be inserted or removed in order to level a structure, and

2) Providing adjustment capability in the form of screw jacks attached to either the footings or the columns in order to adjust the structure to a level configuration.

For footings of any low-rise structure founded on basaltic bedrock, we recommend that they be embedded a minimum of 12 inches into in-place rock as illustrated on Plate 4.

Should design bearing pressures higher than 4500 psf be required, we recommend that footings rest on basaltic bedrock. This material is recommended in any event as the bearing medium under higher structures in order to avoid the problems associated with the expansive soils. On the bedrock, design bearing pressures of at least 10,000 psf are feasible provided at least five feet of overburden is available above the base of the footings. Driven piles could not penetrate the bouldery alluvium and thus are not recommended. Instead, pre-cast piles in pre-drilled holes or cast-in-place piers are recommended. The preferred construction method should be determined only after vertical and horizontal design loading conditions have been determined. Inspection of footing excavations by a soils engineer also is recommended for pier foundations.

DRAINAGE

We understand that slope runoff can be considerable at the site and has caused some problems in the developed area futher down the valley. Diverting such runoff away from all footings is extremely important to minimize erosion and moisture changes in the underlying soil. Therefore, all grading should be designed with the intention of diverting surface drainage away from the structures. Small concrete-lined ditches at the foot of any slopes above structures should be considered to carry away slope runoff. Special provision to carry roof runoff away from buildings should also be provided.

We particularly recommend that the condominium organization institute procedures to check periodically for leakage in water and sewer lines so that any such leakage can be detected quickly and remedies effected immediately.

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The following Plates and Appendix are attached and complete this report.

Plate 1	-	Map of Area
Plate 2	-	Site Plan
Plate 3	-	Generalized Cross-Section
Plate 4	-	Recommended Footing Details
Appendix	-	Field Exploration and Laboratory Testing

Respectfully submitted,

DAMES & MOORE

David C. Liu

David C. Liu

DCL DRR mw

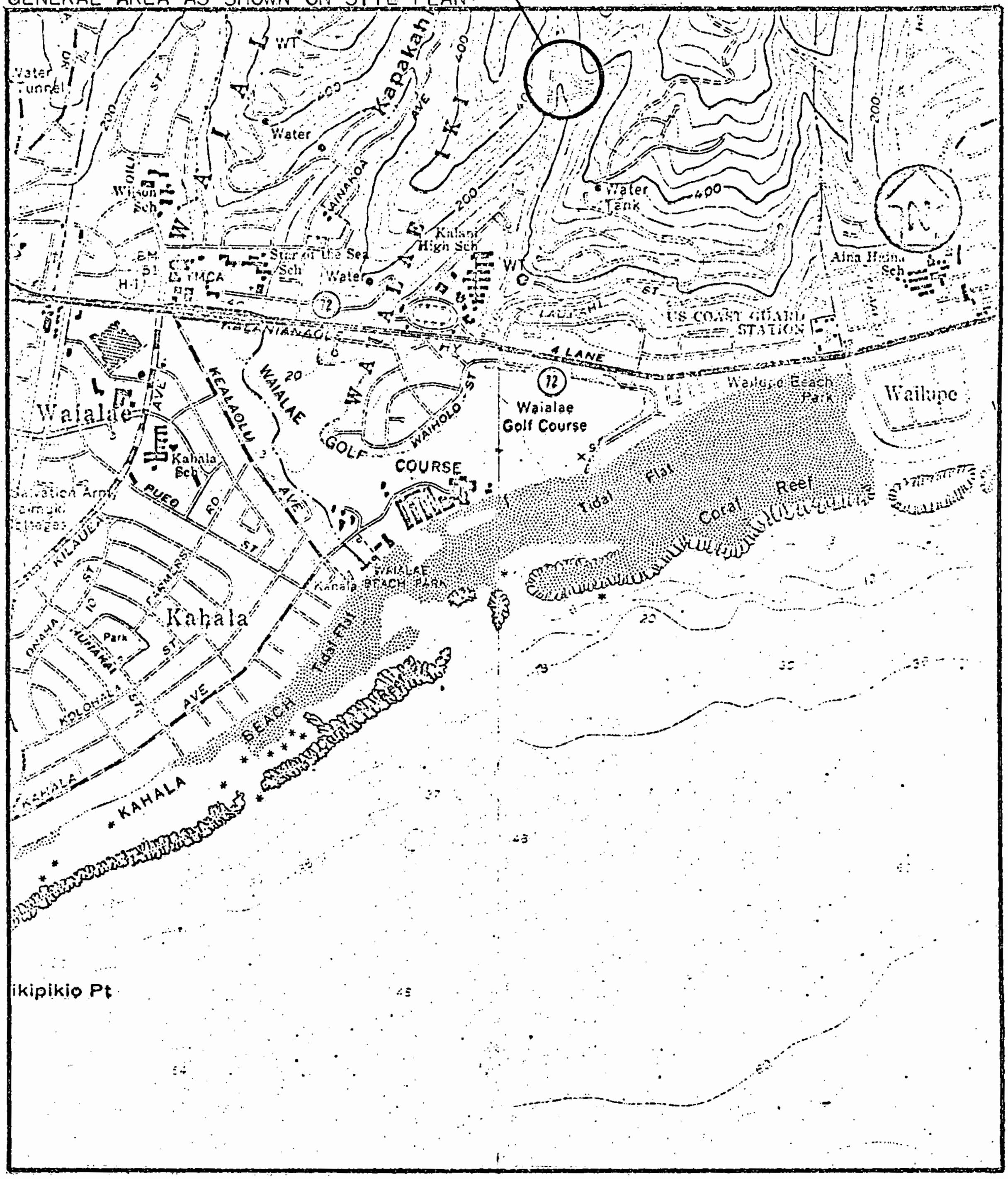
DAVID C. LIU
REGISTERED PROFESSIONAL ENGINEER
No. 1265
HAWAII, U.S.A.

THIS WORK WAS PREPARED BY ME OR UNDER MY SUPERVISION.

David C. Liu

38273

GENERAL AREA AS SHOWN ON SITE PLAN



MAP OF AREA

SCALE 1:24000

1 ½ 0 1 MILE

1000 0 1000 2000 3000 4000 5000 6000 7000 FEET

1 .5 0 1 KILOMETER

REFERENCE:
U.S.G.S. QUADRANGLE MAP
KOKO HEAD, HAWAII
DATED 1969

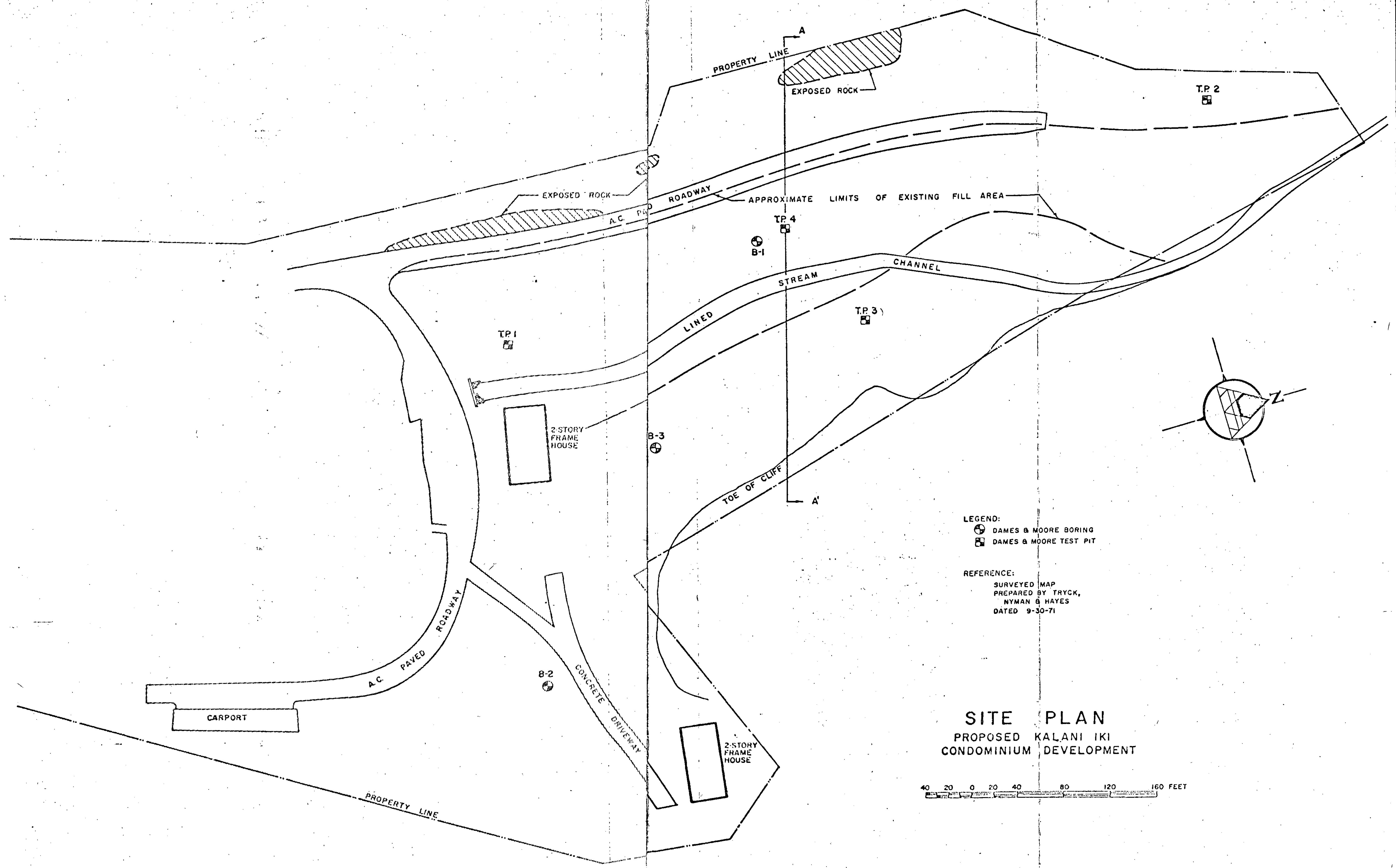
DAMES & MOORE

REVISIONS BY ____ DATE ____

FILE 3038-004

BY ____ DATE ____ CHECKED BY ____

(REV. 6-61)

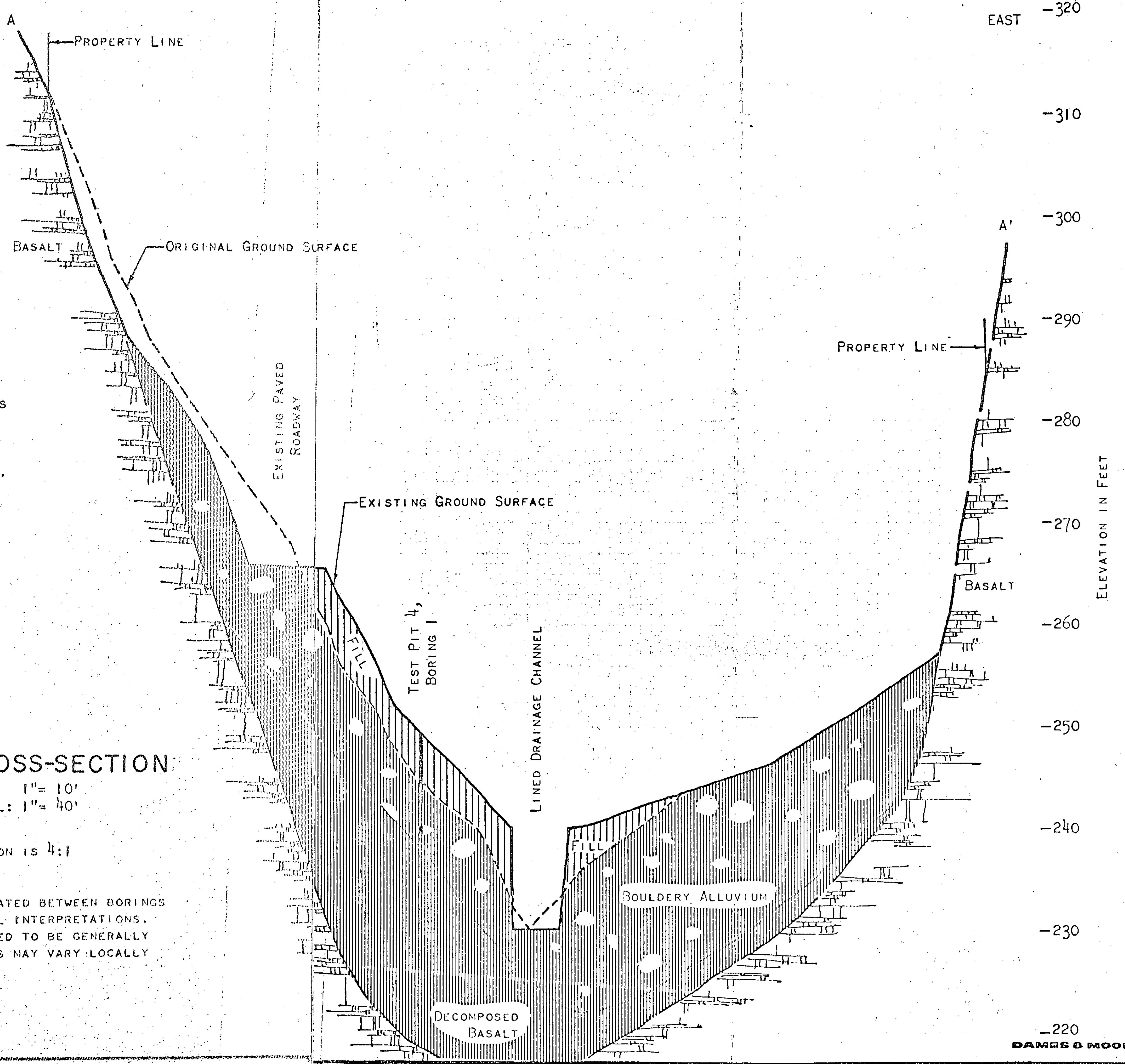


LEGEND:
- DAMES & MOORE BORING
- DAMES & MOORE TEST PIT

REFERENCE:
SURVEYED MAP PREPARED BY TRYCK, NYMAN & HAYES DATED 9-30-71

SITE PLAN
PROPOSED KALANI IKI CONDOMINIUM DEVELOPMENT

40 20 0 20 40 80 120 160 FEET



REFERENCES:

1. TOPOGRAPHICAL MAP
BY TRYCK, NYMAN, & HAYES
DATED 9-30-71

2. GRADING PLAN
BY MURODA & TANAKA, INC.
DATED 11-18-64

GENERALIZED CROSS-SECTION

SCALES - VERTICAL: 1"= 10'
HORIZONTAL: 1"= 40'

VERTICAL EXAGGERATION IS 4:1

NOTE: THE CONDITIONS ILLUSTRATED BETWEEN BORINGS ARE BASED ON GEOLOGICAL INTERPRETATIONS. WHILE THESE ARE BELIEVED TO BE GENERALLY CORRECT, THE CONDITIONS MAY VARY LOCALLY FROM THOSE INDICATED.

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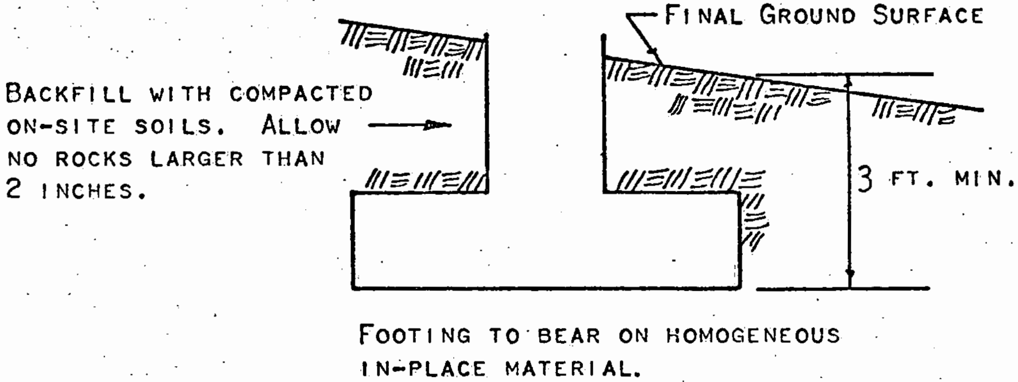
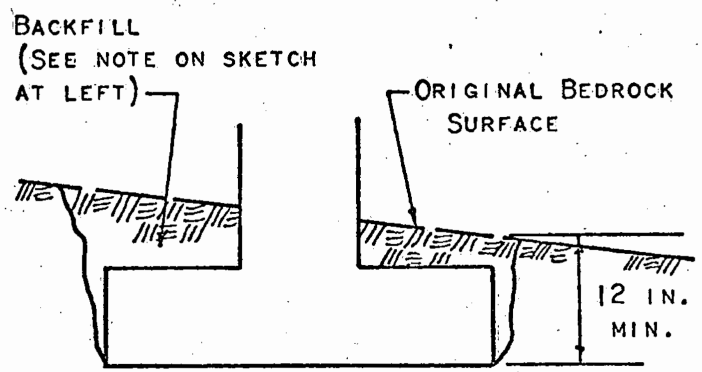
PLATE 3





ALL FOOTING EXCAVATIONS TO BE INSPECTED AND APPROVED BY SOILS ENGINEER.

FOOTING ON SOIL

FOOTING ON BEDROCK

RECOMMENDED FOOTING DETAILS

NO SCALE

APPENDIX

FIELD EXPLORATION AND LABORATORY TESTING

FIELD EXPLORATION

Exploration of subsurface conditions was conducted by drilling three borings and excavating four test pits at the locations indicated on Plate 2, Site Plan, included with the body of this report.

The borings were drilled on September 2 and 3, 1971, by subcontracted personnel using a truck-mounted drill rig. Each boring was advanced by means of a continuous flight auger in soils and an NX core bit in bedrock. Relatively undisturbed samples of the soils were obtained by using a Dames & Moore type U sampler, described on Exhibit A-1. NX core samples were recovered from the coring operations. All samples were returned to our laboratory for subsequent examination and testing.

The test pits were excavated on September 16, 1971 using a subcontracted backhoe and operator. Relatively undisturbed samples of the soils were obtained by using a Dames & Moore hand-drive sampler. Samples were returned to our laboratory for testing.

All borings were drilled and test pits excavated under the general supervision of one of our engineering geologists who maintained a field log of each boring and test pit. These logs are presented as Plates A-1A to A-1C and A-2A to A-2B. Our geologist also assisted in obtaining the relatively undisturbed samples of soils and classified these samples in accordance with the Unified Soil Classification System presented in Plate A-3.

Field reconnaissance of the area was conducted on several dates by our personnel while preparing the proposal for our services, while selecting the boring and test pit locations and subsequent to subsurface exploration in order to verify certain details of our observations.

The exact locations of Boring 1 and Test Pits 1, 3 and 4 were determined by the survey crew from Tryck, Nyman and Hayes. The other borings and test pit were located by pace-and-compass methods by our personnel.

LABORATORY TESTING

General - Laboratory tests were conducted on relatively undisturbed samples of soils obtained from the borings in order to evaluate the engineering properties of the soils with regard to strength and expansiveness. Laboratory tests performed included the following: moisture-density determinations, unconfined and unconsolidated/undrained triaxial shear strength tests, double direct shear tests, expansion tests, and Atterberg limits determinations. These tests are described individually in the following subsections.

Moisture-Density Tests - In-place moisture content and dry density determinations were conducted on most of the samples obtained. The test results are presented on the Log of Borings, Plates A-1A through A-1C, and Log of Test Pits, Plates A-2A through A-2B.

Unconfined and Unconsolidated/Undrained Triaxial Shear Strength Tests - Four unconfined and three unconsolidated/undrained triaxial shear strength tests were performed on selected samples of the soils. The test procedures are described on Exhibit A-2. These tests were performed to evaluate the shear strength of the various soils at the site in order to calculate allowable design bearing pressures and to evaluate possible slope stability problems. Results of the unconfined and triaxial tests are presented below:

Boring No.	Depth (ft)	Confining Pressure (psf)	Peak Shear Strength (psf)
1	9	0	1870
1	11.5	1000	2810
1	19	2000	6805
1	24.5	3000	7750
3	2.1	0	8085
T.P.1	4	0	2390
T.P.3	3	0	2460

Direct Shear Test - One direct shear test was conducted on a soil sample which was of insufficient height to perform an unconfined or trixial test. The purpose of the test was the same as for the other strength tests. The test method is described on Exhibit A-3. Results of the test were:

Boring No.	Depth (ft)	Normal Pressure (psf)	Peak Shear Strength (psf)
T.P.4	7	2000	4100

Expansion Tests - Four expansion tests were conducted on selected samples in order to evaluate the amount of expansion which could be anticipated from the soils if thoroughly saturated under a range of pressures. All samples were chosen from relatively shallow depths to represent those soils which would be found below footings. Results of these tests are presented on the curves shown on Plate A-4. These curves show the relationship between the amount of expansion and the logarithm of applied pressure. The test procedure is similar to that of consolidation tests as described on Exhibit A-4. However, for the expansion tests, the material was allowed to expand under 100 pounds per square foot until about 5 percent expansion was reached. Further expansion was inhibited by increasing the applied pressure in order to minimize any loss of sample.

The dashed portions of the curves shown on Plate A-4 represent the range of pressure in which the sample was not allowed to expand fully.

Atterberg Limits Determinations - Atterberg limits were determined for each of the samples on which expansion tests were performed. The results of these tests were to determine the plasticity characteristics of the soils and the relationship, if any, between these characteristics and the potential for expansion. The results of the tests were also used for classifying samples in the Unified Soil Classification System. Results are presented below.

Boring No.	Depth (ft)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Unified Soil Classi-cation	Expansion Under 400 psf (%)
B3	2.6	71	33	38	CH/MH	7.9
TP1	2.5	58	35	23	MH	4.7
TP2	4.7	90	37	53	CH/MH	3.9
TP3	5.0	71	38	33	MH	3.2

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The following Plates and Exhibits are attached and complete this Appendix.

Plate A-1A	-	Log of Borings, Boring 1
Plate A-1B	-	Log of Borings, Boring 2
Plate A-1C	-	Log of Borings, Boring 3
Plate A-2A	-	Log of Test Pits, Test Pits 1 and 2
Plate A-2B	-	Log of Test Pits, Test Pits 3 and 4
Plate A-3	-	Unified Soil Classification System
Plate A-4	-	Expansion Curves
Exhibit A-1	-	Soil Sampler Type U
Exhibit A-2	-	Method of Performing Unconfined Compression and Triaxial Compression Tests
Exhibit A-3	-	Method of Performing Direct Shear and Friction Tests
Exhibit A-4	-	Method of Performing Consolidation Tests

BORING 1

SURFACE ELEVATION +246 FEET

Moisture Content in %	Dry Density in PCF	Blows/Ft. on Sampler	Core and % Recovery	Samples and/or Cores	Depth in Feet	Letter Symbol	Description
						ML	RED-BROWN CLAYEY AND SANDY SILT TOPSOIL
						BD-MH	WEATHERED BASALTIC BOULDERS WITH SILTY FINES
					5		
25.0		45		▣		ML	BROWN SANDY SILT WITH A FEW SMALL ROOTS, MEDIUM STIFF
						MH	BROWN CLAYEY SILT, VERY STIFF
						BD	BASALTIC BOULDER
38.8	78.3	20		▣		MH	MOTTLED BROWN CLAYEY SILT WITH SMALL WEATHERED TO DECOMPOSED BASALT FRAGMENTS, STIFF
					10		
42.4	68.6	40		▣		ML-MH	MULTI-COLORED CLAYEY SILT, VERY STIFF TO HARD. (VERY WEATHERED TO DECOMPOSED BASALT, PROBABLY IN-PLACE BUT POSSIBLY BOULDERS)
					15		LESS WEATHERED 14.5 TO 18.0 FEET
40.6	71.5	30		▣			
					20		
42.6	74.8	50		▣			
					25		

NO WATER ENCOUNTERED IN BORING
BORING COMPLETED AT 25.0 FEET ON 9-2-71

LOG OF BORINGS

NOTES:

- ▣ -DEPTH AT WHICH UNDISTURBED SAMPLE WAS TAKEN
- ☒ -DEPTH AT WHICH DISTURBED SAMPLE WAS TAKEN
- ☐ -DEPTH AT WHICH SAMPLE WAS LOST DURING EXTRACTION
- I -DEPTH AND LENGTH OF CORE RUN

DRIVING ENERGY - 300-LB WEIGHT DROPPING 30 INCHES

DAMES & MOORE
PLATE A-1A

REVISIONS BY ____ DATE ____
FILE 3038-006
CHECKED BY DEC DAT

BORING 2

SURFACE ELEVATION +244 FEET

Moisture Content in %	Dry Density in PCF	Blows/Ft. on Sampler	Core and % Recovery	Samples and/or Cores	Depth in Feet	Letter Symbol	Description
					0	BD	BASALTIC BOULDER
		30/5"		⊠		SM	YELLOW-BROWN SILTY MEDIUM TO COARSE SAND, DENSE, SLIGHTLY CEMENTED
		20/2"		☐	5	BD-ML	WEATHERED BASALTIC BOULDERS OF VARIOUS SIZES WITH SILTY FINES
27.7	73.0	30/6"		▣	10	ML-MH	DARK BROWN SANDY AND CLAYEY SILT, HARD (DECOMPOSED BASALT BOULDER)
						BD-ML	BASALTIC BOULDERS WITH SILTY FINES
37.0	79.8	50		▣	15	ML-MH	MULTI-COLORED SILTS: SOFT PINK CLAYEY SILT, HARD YELLOW ASHY SILT, AND HARD DARK BROWN SANDY SILT. (PROBABLY ALL DECOMPOSED BASALT)
29.5	90.0	70/11"		▣		ML-MH	DARK BROWN SANDY AND CLAYEY SILT, HARD (DECOMPOSED TO VERY WEATHERED BASALT) HARDER ZONE 16.5 TO 17.5 FEET
			NX 65%	I	20		GRAY SLIGHTLY VESICULAR BASALT. SLIGHTLY WEATHERED ON JOINTS FOUND AT 4" TO 12" INTERVALS
					25		

NO WATER ENCOUNTERED IN BORING
BORING COMPLETED AT 25.0 FEET ON 9-3-71

LOG OF BORINGS

NOTES:

▣ -DEPTH AT WHICH UNDISTURBED SAMPLE WAS TAKEN
⊠ -DEPTH AT WHICH DISTURBED SAMPLE WAS TAKEN
☐ -DEPTH AT WHICH SAMPLE WAS LOST DURING EXTRACTION
I -DEPTH AND LENGTH OF CORE RUN
DRIVING ENERGY - 300-LB WEIGHT DROPPING 30 INCHES

DAMES & MOORE
PLATE A-1B

REVISIONS BY ____ DATE ____
FILE 3031
BY ____ CHECKED BY DRR DATE ____

BORING 3

SURFACE ELEVATION +232 FEET

Moisture Content in %	Dry Density in PCF	Blows/Ft. on Sampler	Core and % Recovery	Samples and/or Cores	Depth in Feet	Letter Symbol	Description
						BD	Basaltic boulders
26.8	92.0	15		▩		MH	Dark brown clayey silt with a few basalt pebbles and small roots, hard
						BD-MH	Basaltic boulders with stiff to hard sandy and clayey silt fines
					5		
32.3	79.4	22/6"		▩			
					10		
22.6	92.7	30/6"		☒			
		31/3"		☒	15		
							Gray slightly vesicular basalt. Slightly weathered on joints fround at 2" to 12" intervals
			NX 95%	I	20		Very vesicular zone 20.5' to 21.5'

No water encountered in boring

Bpring completed at 22.5 feet on 9-3-71

LOG OF BORINGS

NOTES:

▩ -DEPTH AT WHICH UNDISTURBED SAMPLE WAS TAKEN

☒ -DEPTH AT WHICH DISTURBED SAMPLE WAS TAKEN

☐ -DEPTH AT WHICH SAMPLE WAS LOST DURING EXTRACTION

I -DEPTH AND LENGTH OF CORE RUN

DRIVING ENERGY - 300-LB WEIGHT DROPPING 30 INCHES

DAMES & MOORE

PLATE A-1C

REVISIONS BY ____ DATE ____

FILE 3038.004

CHECKED BY ____ DATE ____

TEST PIT 1

SURFACE ELEVATION +219.6 FEET

MOISTURE CONTENT IN %	DRY DENSITY IN PCF	SAMPLES	DEPTH IN FEET	GRAPH SYMBOL	LETTER SYMBOL	DESCRIPTION
					MH-BD	DARK BROWN CLAYEY SILT (VERY STIFF TO HARD) AND SLIGHTLY WEATHERED BASALTIC BOULDERS TO 3 FEET IN DIAMETER. ABOUT 50% BOULDERS. (POSSIBLY FILL)
23.3	90.8	▣				
34.1	85.2	▣	5			BELOW 3 FEET SILT BECOMES MORE PLASTIC AND BOULDERS DECREASE TO ABOUT 35%. (PROBABLY IN-PLACE)

NO WATER ENCOUNTERED IN TEST PIT
TEST PIT COMPLETED AT 7.8 FEET ON 9-17-71

TEST PIT 2

SURFACE ELEVATION +298 FEET

MOISTURE CONTENT IN %	DRY DENSITY IN PCF	SAMPLES	DEPTH IN FEET	GRAPH SYMBOL	LETTER SYMBOL	DESCRIPTION
					MH-BD	DARK BROWN TO GRAY-BROWN CLAYEY SILT (VERY STIFF TO HARD) WITH SLIGHTLY WEATHERED BASALTIC BOULDERS TO 2 FEET IN DIAMETER. FEW SMALL ROOTS IN TOP 2.5 FEET. ABOUT 50% BOULDERS (POSSIBLY FILL)
		▣				BETWEEN 2.5 AND 6.0 FEET, SILT BECOMES MORE PLASTIC AND BOULDERS DECREASE TO ABOUT 30%. (PROBABLY IN-PLACE)
40.6	78.5	▣	5			
42.1	60.5	▣			BD-MH	ABOUT 65% BOULDERS TO 3 FEET IN DIAMETER WITH CLAYEY SILT FINES AS ABOVE

NO WATER ENCOUNTERED IN TEST PIT
TEST PIT COMPLETED AT 7.7 FEET ON 9-16-71

LOG OF TEST PITS

NOTES:

- ▣ - DEPTH AT WHICH UNDISTURBED SAMPLE WAS TAKEN
- ☒ - DEPTH AT WHICH DISTURBED SAMPLE WAS TAKEN
- ☐ - DEPTH AT WHICH SAMPLE WAS LOST DURING EXTRACTION

REVISIONS BY ____ DATE ____

FILE 3039.0

BY [illegible] DATE 10.11.71

CHECKED BY DEP

REVISIONS BY ____ DATE ____

FILE 3038.004

BY ___ CHECKED BY DBE DATE ___

TEST PIT 3

SURFACE ELEVATION +253 FEET

MOISTURE CONTENT IN %	DRY DENSITY IN PCF	SAMPLES	DEPTH IN FEET	LETTER SYMBOL	DESCRIPTION
				MH-BD	DARK BROWN TO GRAY BROWN PLASTIC CLAYEY SILT (VERY STIFF) WITH SLIGHTLY WEATHERED BASALTIC BOULDERS. FEW SMALL ROOTS. ABOUT 35% BOULDERS
31.6	83.5	▩			
38.4	76.8	▩	5		
				BD-MH	ABOUT 65% BASALTIC BOULDERS TO 5 FEET IN DIAMETER WITH CLAYEY SILT FINES. SILT IS STIFF TO HARD AND LESS PLASTIC THAN ABOVE. FEW SMALL ROOTS

NO WATER ENCOUNTERED IN TEST PIT
TEST PIT COMPLETED AT 9.0 FEET ON 9-17-71

TEST PIT 4

SURFACE ELEVATION +249.5 FEET

MOISTURE CONTENT IN %	DRY DENSITY IN PCF	SAMPLES	DEPTH IN FEET	LETTER SYMBOL	DESCRIPTION
				MH-BD	BROWN TO DARK BROWN CLAYEY SILT (VERY STIFF TO HARD) WITH MANY SLIGHTLY WEATHERED BASALTIC BOULDERS. FEW SMALL ROOTS (POSSIBLY FILL)
		☐			
		☒	5		
					BELOW 6.0 FEET, SILT BECOMES MORE PLASTIC AND NUMBER OF BOULDERS DECREASES (PROBABLY IN-PLACE)
33.8	91.6	▩			
			10		MANY WEATHERED BOULDERS AT 10.0 FEET

NO WATER ENCOUNTERED IN TEST PIT
TEST PIT COMPLETED AT 10.5 FEET ON 9-16-71

LOG OF TEST PITS

NOTES:

▩ - DEPTH AT WHICH UNDISTURBED SAMPLE WAS TAKEN
☒ - DEPTH AT WHICH DISTURBED SAMPLE WAS TAKEN
☐ - DEPTH AT WHICH SAMPLE WAS LOST DURING EXTRACTION

DAMES & MOORE

PLATE A-2B

$\ldots$7 (REV. 6-61)

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
MORE THAN 50 % OF MATERIAL IS SMALLER THAN NO 200 SIEVE SIZE	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

NOTES:

1. DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE CLASSIFICA
2. WHEN SHOWN ON THE BORING LOGS, THE FOLLOWING TERMS ARE TO DESCRIBE THE CONSISTENCY OF COHESIVE SOILS AND THE RELATIVE COMPACTNE COHESIONLESS SOILS.

COHESIVE SOILS

	(APPROXIMATE SHEARING STRENGTH IN KSF)
VERY SOFT	LESS THAN .25
SOFT	0.25 TO 0.5
MEDIUM STIFF	0.5 TO 1.0
STIFF	1.0 TO 2.0
VERY STIFF	2.0 TO 4.0
HARD	GREATER THAN 4.0

COH LESS SOILS

VERY LOOS
LOOSE
MEDIUM DEN
DENSE
VERY DENS

THESE ARE USUALLY BASED ON AN EXAMINATION OF SOIL SAMPLES, PENETRATION RESISTANCE, AND SOIL DENSITY DATA.

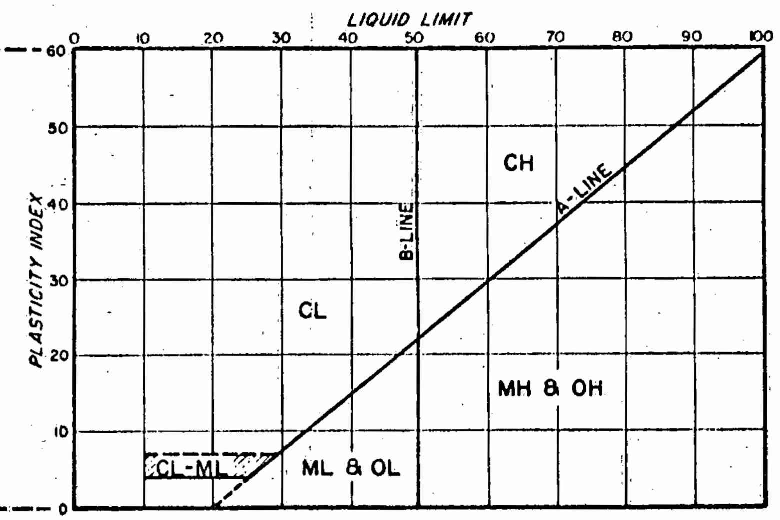
GRADATION CHART

MATERIAL SIZE	PARTICLE SIZE			
	LOWER LIMIT		UPPER LIMIT	
	MILLIMETERS	SIEVE SIZE*	MILLIMETERS	SIEVE SIZE*
SAND				
FINE	.074	#200*	0.42	#40*
MEDIUM	0.42	#40*	2.00	#10*
COARSE	2.00	#10*	4.76	#4*
GRAVEL				
FINE	4.76	#4*	19.1	3/4"•
COARSE	19.1	3/4"•	76.2	3"•
COBBLES	76.2	3"•	304.8	12"•
BOULDERS	304.8	12"•	914.4	36"

* U.S. STANDARD • CLEAR SQUARE OPENINGS

PLASTICITY CHART

FOR LABORATORY CLASSIFICATION OF FINE-GRAINED SOILS



SAMPLES

▣ INDICATES UNDISTURBED SAMPLE
⊠ INDICATES DISTURBED SAMPLE
☐ INDICATES SAMPLING ATTEMPT WITH NO RECOVERY

I INDICATES LENGTH OF CORING RUN

NOTE:
DEFINITIONS OF ANY ADDITIONAL DATA REGARDING SAMPLES ARE ENTERED ON THE FIRST LOG ON WHICH THE DATA APPEAR.

UNIFIED SOIL CLASSIFICATION SYSTEM

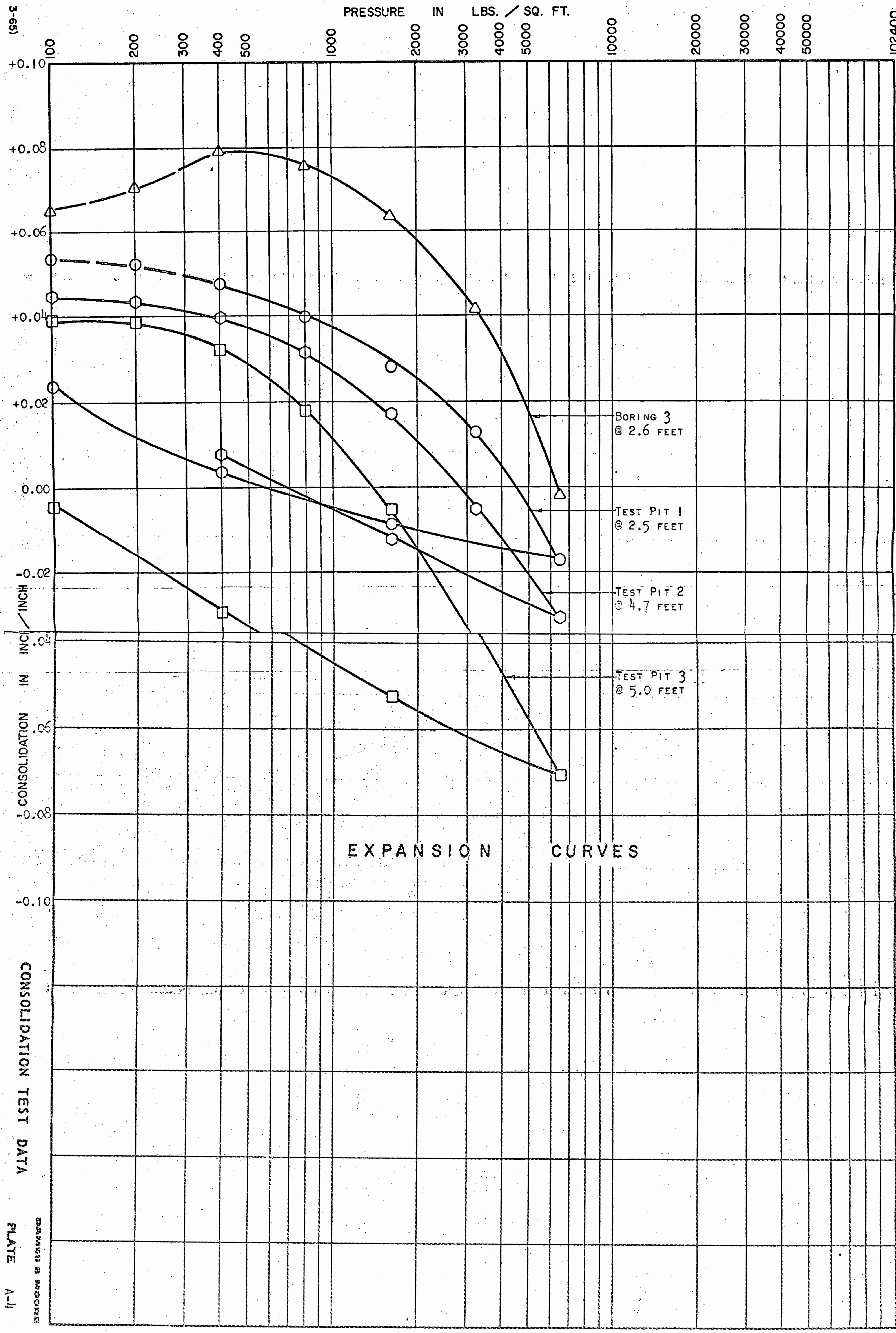
DAMES & MOORE

746.2 (REV. 3-65)

BY INV DATE 10-15-71

CHECKED BY: DRR

FILE 3038-004



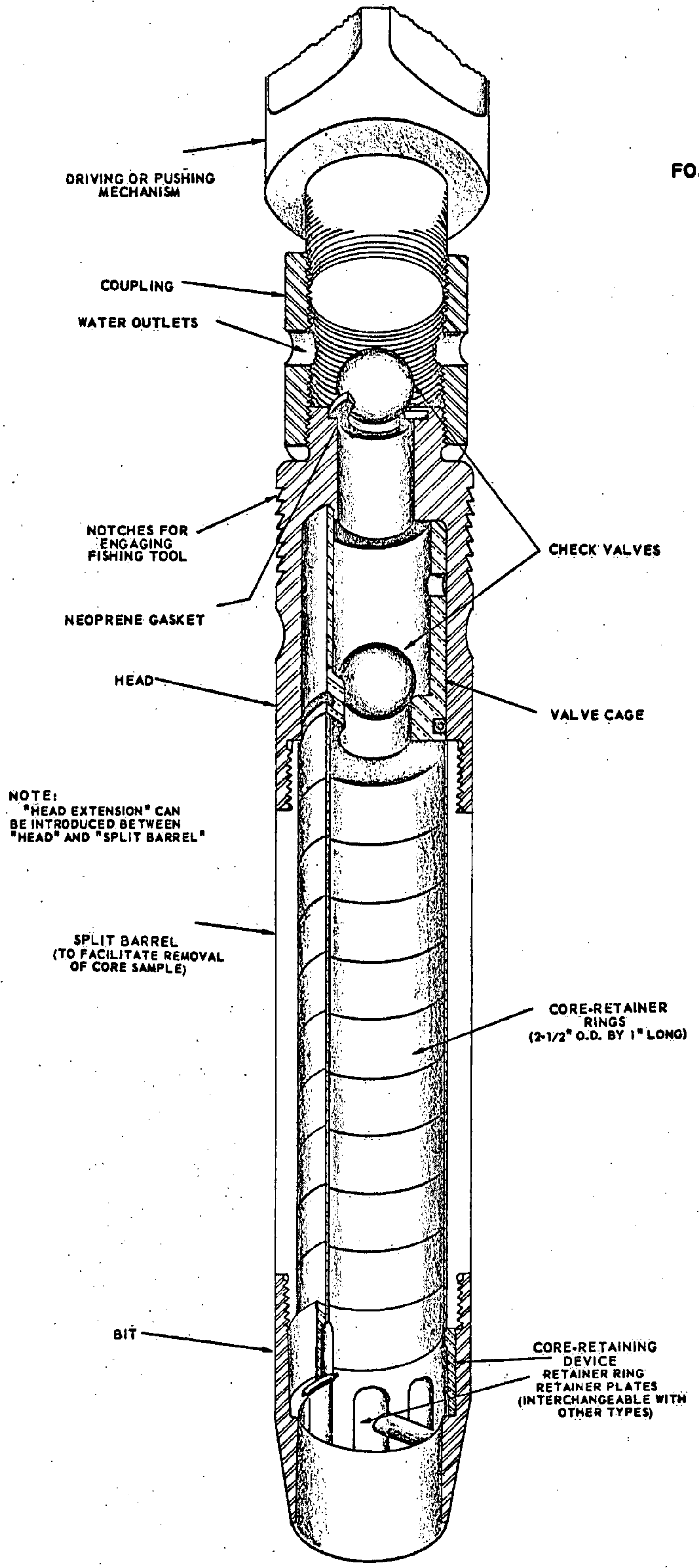
CONSOLIDATION TEST DATA

PLATE A-1

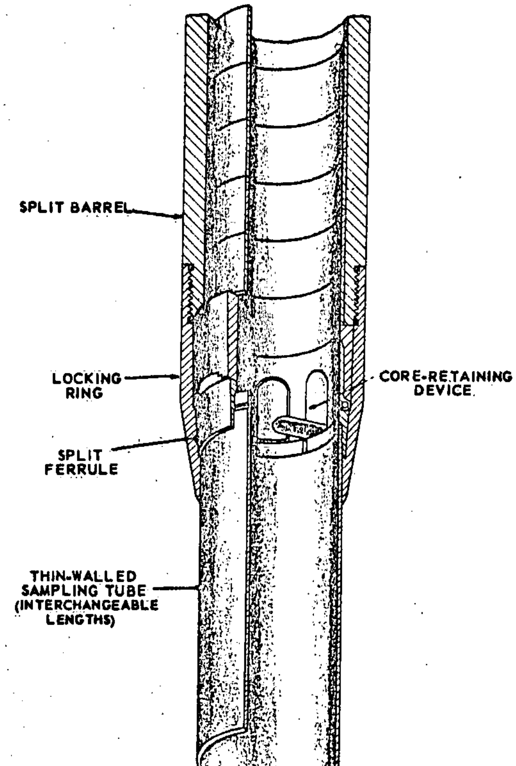
DAMES & MOORE

SOIL SAMPLER TYPE U

FOR SOILS DIFFICULT TO RETAIN IN SAMPLER

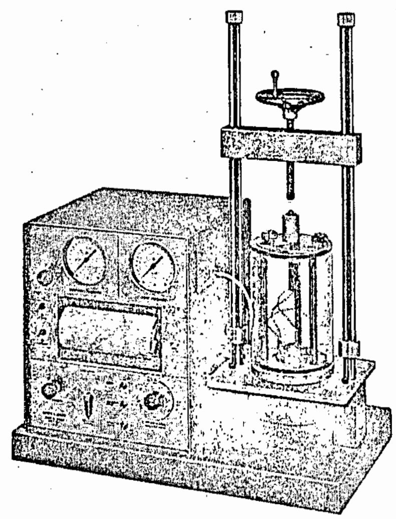


ALTERNATE ATTACHMENTS



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:

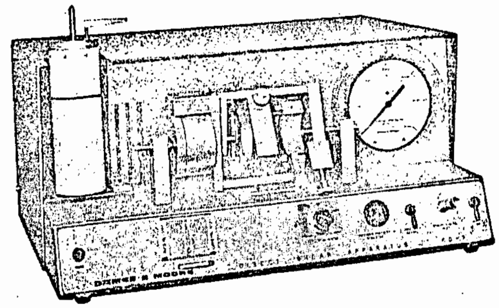
> UNCONSOLIDATED-UNDRAINED: THE CONFINING PRESSURE IS IMPOSED ON THE SAMPLE AT THE START OF THE TEST. NO DRAINAGE IS PERMITTED AND THE STRESSES WHICH ARE MEASURED REPRESENT THE SUM OF THE INTERGRANULAR STRESSES AND PORE WATER PRESSURES.
>
> CONSOLIDATED-UNDRAINED: THE SAMPLE IS ALLOWED TO CONSOLIDATE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

EXHIBIT A-3

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.

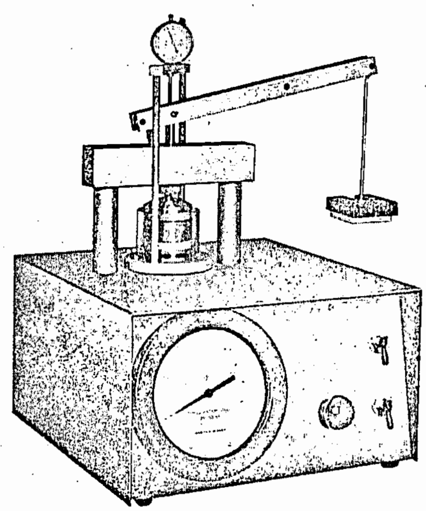
Friction Tests

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.

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.