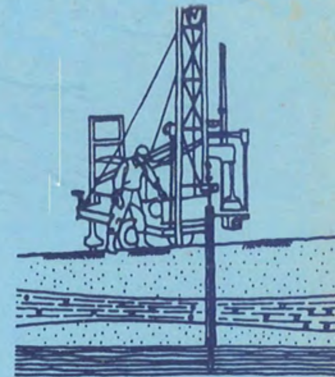
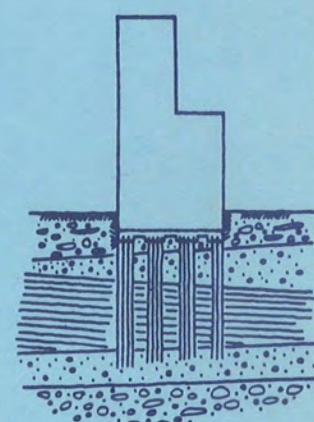
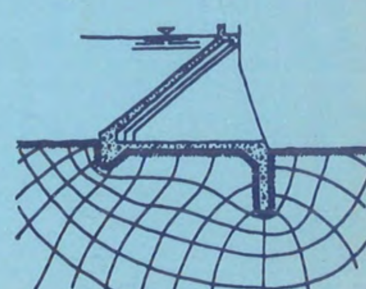
FOUNDATION INVESTIGATION

JESSIE STREET OFFICE BUILDING

SAN FRANCISCO, CALIFORNIA







Lee and Praszker

CONSULTING GEOTECHNICAL ENGINEERS
AND GEOLOGISTS

147 Natoma Street at New Montgomery
San Francisco, CA 94105 (415) 392-4866

Member

FOUNDATION INVESTIGATION

JESSIE STREET OFFICE BUILDING

SAN FRANCISCO, CALIFORNIA

LEE AND PRASZKER
A CALIFORNIA CORPORATION
CONSULTING GEOTECHNICAL ENGINEERS AND GEOLOGISTS
147 NATOMA ST. AT NEW MONTGOMERY
SAN FRANCISCO 94105
(415) 392-4866

CHARLES H. LEE (1883-1967)
F. ASCE

MICHAEL PRASZKER
F. ASCE

March 9, 1981

Perini Land and Development Co.
One Maritime Plaza
Suite 1320
San Francisco, CA 94111

Attention: Mr. Randall Verreau

Re: Jessie Street Office Building
San Francisco, California

Gentlemen:

We are pleased to submit herewith five (5) copies of the above-captioned report. The work described therein is in fulfillment of our proposal dated January 12, 1981 and your subsequent authorization of the same date.

Conclusions and recommendations contained in the report are predicated upon limited subsurface exploration and laboratory testing programs. As a result, variations between anticipated and actual soil conditions may be found in localized areas during construction. We would, therefore, appreciate the opportunity of being engaged to observe the installation of pile foundations, during which time we may have to make changes in our recommendations if such are deemed necessary.

We will continue our endeavor to be of service to you on this project, and will be looking forward to working with you during its construction.

Very truly yours,

Michael Praszker

MP/RR/eb

Encls.

cc: Jorge de Quesada, Inc. (1)
Raj Desai Associates (1)

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LIST OF DIAGRAMS

APPENDICES

FOUNDATION INVESTIGATION
JESSIE STREET OFFICE BUILDING
SAN FRANCISCO, CALIFORNIA

1. INTRODUCTION AND SCOPE

This report presents the results of our foundation investigation for a proposed office building to be constructed on the east corner of the intersection of Jessie and Ecker Streets in San Francisco, California. The scope of work for the investigation described herein included drilling three test borings to an aggregate depth of 286-1/2 feet; taking of undisturbed samples of the subsurface soils and performing laboratory tests on same; interpretation and evaluation of test results; and engineering analyses of foundations based on field and laboratory test results.

The Architect for the project is Jorge de Quesada, Inc., and the Structural Engineers are Raj Desai Associates, Inc., both firms are from San Francisco.

2. SITE DESCRIPTION

The project site is bounded on the northwest by Jessie Street with a frontage of 105 feet, on the southwest by Ecker Street with a frontage of 76 feet, on the southeast by Elim Alley with a frontage of 103.5 feet, and on the northeast by an existing five story concrete building. The building along the northeast property line has a basement which has a floor elevation of about -2 feet (San Francisco City Datum). The site is relatively level with ground surface elevations ranging from about 6 feet to about 7 feet (SFCD) and is presently occupied by an asphalt paved parking lot. It appears as though a building, with a basement, once occupied the site and was razed and the basement filled in. It is not known what type of building occupied the site or what kind of foundation it was supported on.

3. REGIONAL GEOLOGY

Geologically the San Francisco Peninsula is a range of hills composed of rocks of the Franciscan Formation. These consist of sedimentary arkosic sandstones and shales with igneous rock intrusions of basalt and serpentine. During the glacial age the level of the ocean throughout the world varied greatly, and for a long period was some 350 feet lower than at present. San Francisco Bay was then a broad inland valley extending from San Jose northward to Mare Island and thence by narrow canyon through Carquinez Straits to the great Central Valley of California. The drainage outlet of the valley of San Francisco Bay was the channel of the Sacramento-San Joaquin River which followed a deep canyon between Angel Island and Tiburon, now known as Raccoon Straits, and thence outward through the Golden Gate to its mouth westward of the present coast line. The valley was surrounded by hills of the Franciscan rock series.

Local storm runoff from the hills surrounding the valley of San Francisco Bay eroded ravines, valleys, and deep canyons into the then exposed Franciscan bedrock and deposited alluvial debris upon the lower hill slopes and over the valley floor. Vegetation grew, with grass and tules forming peat beds in moist low areas. On the higher valley slopes and hillsides there were forests.

Toward the end of the glacial period, after the Wisconsin glacial advance, the valley and the lower levels of these hills were submerged by the rise in ocean level caused by the melting of the earth's ice caps.

As the ocean level rose, salt water entered through the Golden Gate, flooding the valley and lower hill slopes. The coarser erosional debris which was carried by local streams, and which was formerly deposited out on the valley floor, now accumulated at the water's edge in deltas and bars, while the fines were carried out into salt water and formed bluish grey marine mud, which in time became firm and stiff clay. The ensuing fluctuations in the ocean level during the glacial period often caused the earlier clay deposits to be covered by coarser erosional material. The extensive beds of soft bay clay which overlie these coarse materials, and which are the first encountered in sub-aqueous borings throughout San Francisco Bay, have accumulated during the current prolonged period of the existing stage of the ocean.

During the period preceding the flooding of the valley of San Francisco Bay, runoff from a large valley in the Mission Dolores and Civic Center vicinity flowed to the north, and its stream eroded a deep bedrock canyon which meanders through the low area of San Francisco and whose lower end is located beneath the east end of the Ferry Building where it entered the valley of San Francisco Bay. The stream designated as Yerba Buena Creek passed just southeast of the project site. Yerba Buena Creek drained a portion of the watershed of the modern Islais Creek, including Noe Valley, most of the present watershed Mission Creek, together with Eureka and Hayes Valleys, and the areas south of lower Market Street.

Elevations and slopes of the deeply furrowed bedrock surface beneath lower Market Street were controlled by the hydraulic grade of Yerba Buena Creek. Base level of erosion of this creek was, in turn, controlled by water level in Sacramento River at the Golden Gate. Available geologic and subsurface data indicate a depth to bedrock under the subject site of approximately 265 feet, or about elevation -260 feet (SFCD).

4. SEISMICITY AND NATURAL SITE PERIOD

No known faults pass beneath the site. However, the San Francisco Bay Area lies within a seismically active region and the active San Andreas Fault lies approximately 8.5 miles southwest of the site and the active Hayward Fault lies approximately 10 miles to the northeast. Therefore it must be assumed that the proposed building will be subjected to significant shaking during its life. The dynamic characteristics of the ground motions at the site will be typical of sites in San Francisco where soft bay clays overlay deep stiff soils and bedrock. On the basis of an idealized soil profile with bedrock at a depth of about 265 feet, the natural period of the site is on the order of 1.4 seconds.

5. PROPOSED DEVELOPMENT

The proposed structure will be an 18 story steel frame building with a penthouse and a partial basement. The first floor of the building will be open except for a small lobby and the service core on the northeast side of the building as shown on Diagram 1. The basement will be under the core only, extending about 11 feet below street grade. The open area will be occupied by a fountain, tables and shrubbery.

Building loads for the structure will be supported by exterior and interior columns and will range in magnitude from about 1550 kips on exterior columns to 1750 kips on interior columns for dead plus adjusted live loads. Typical column bays will be approximately 23 by 24 feet.

6. TEST BORINGS AND GROUNDWATER OBSERVATION

Test Borings

Three test borings were made by means of a rotary wash rig at locations shown on Diagram 1. The maximum depth drilled was 121-1/2 feet and the total footage drilled was 286-1/2 feet.

The drilling operations were observed by a qualified geotechnical engineer of this office on a full time basis and all the soils penetrated by the borings were logged by him in the field. Soil samples were taken at critical horizons or at regular intervals, as directed by our engineer, and transported to our laboratory for testing. Test boring logs were prepared from field logs supplemented by soil identification in the lab and are presented in this report as Diagrams A-2 through A-5.

A summary of test borings with some pertinent data is as follows:

Boring No.	Approximate* Ground Elev.	Bottom Elev.	Depth Drilled	Undisturbed Samples
1	6-1/2'	-115'	121-1/2'	10
2	7'	-76'	83'	6
3	7'	-75'	82'	6
		Total:	286-1/2	22

Groundwater Observations

Groundwater observations were made in Test Boring 2. An open standpipe consisting of a 1-inch diameter slotted plastic pipe embedded in coarse sand was installed in TB-2. Groundwater readings were made several days after drilling when the groundwater level had stabilized. On the basis of these observations, the groundwater level for design purposes should be taken at a maximum of elevation -13 feet (SFCD).

* San Francisco City Datum

7. LABORATORY TESTING

For the purpose of foundation evaluation, soil testing was confined to the following:

A. Volumetric Tests

Twenty-one samples were subjected to unit weight and moisture content tests, results of which aid in the identification of the soils and their characteristics. The results are entered on the respective boring logs alongside the appropriate sample locations.

B. Shear Tests

Two direct shear tests were performed on representative samples of the bearing stratum soils to establish strength characteristics for pile design. The results of these tests are presented on Diagrams B-1 and B-2.

C. Consolidation Tests

A consolidation test was made on a sample of the sandy clay encountered in Test Boring 1 below elevation -70 feet to determine its compressibility under various superimposed loads. Test results are presented in graphic form on Diagram B-3, which also shows other pertinent data.

A summary of all laboratory test results is presented in Appendix B.

8. SUBSURFACE CONDITIONS

An idealized subsurface profile prepared by interpolation between available test borings is presented in plan on Diagram 1 and in Section on Diagram 2. The profile shows that the site is underlain by sand and rubble fill to a depth of about 10 feet. Beneath the fill is a layer of clean medium dense wind blown sand some 14 to 25 feet in thickness. Underlying the sand is a soft compressible clay, known locally as upper bay clay, extending to 45 to 50 feet below the ground surface. The upper bay clay layer is underlain by 25 to 30 feet of dense sand below which are alternating strata of very stiff clays and dense sands, extending to bedrock some 265 feet below the ground surface.

9. CONCLUSIONS AND RECOMMENDATIONS

A. General

The presence at the site of the relatively thick layer of uncontrolled fill and the underlying compressible upper bay clay precludes the use of shallow foundations, if large total and differential settlements between columns must be avoided. Since only a partial basement is contemplated, the possibility of a floating structural mat has not been given consideration. A deep foundation taking support by means of piles and extending through the upper bay clay into the firmer strata below is therefore recommended.

With regard to the use of the existing fill to support a slab-on-grade it must be pointed out that such a slab-on-grade may be subject to future uneven settlements as a result of internal degeneration in the existing fill during earthquakes. The partial removal of the existing uncontrolled fill and its replacement with select compact fill would partially alleviate such undesirable future settlements, and it is recommended that this be done.

B. Pile Foundation

1. Type of Pile and Pile Load Capacities for Vertical and Horizontal Load

Based on preliminary analyses and discussions with the Structural Engineer, it appears that, not to the exclusion of other pile types, 12 inch square pre-cast prestressed concrete piles driven into the sand below the upper bay clay are most economical for this project. These piles may be designed for a safe capacity of 100 tons dead load plus adjusted live load with a minimum embedment of approximately 15 feet in the bearing sand layer, tip elevation about -55 to -60 feet (SFCD). From the idealized soil profile it can be seen that to achieve this penetration, piles will have to be on the order of 65 feet long, except in the partial basement area where they will be about 10 feet shorter. It is possible that during driving the piles will encounter greater resistance than anticipated, thus preventing penetration of the full 15 feet and resulting in shorter piles. Therefore selection of design pile lengths should await the probe pile results discuss below.

Because local variations in soil strength characteristics are likely to exist in the sand layer, the supporting capacity of each pile must be verified in the field during driving. This will be accomplished by using a driving resistance-bearing capacity relationship such as the Engineering News Record (ENR) formula, or the more sophisticated Wave Equation. The driving criteria will be developed after the selection of the pile driving equipment and the performance of a probe pile driving program.

The allowable lateral load for a 12 inch pre-cast, pre-stressed concrete pile, for a horizontal deflection of 1/2 inch at the ground surface, has been calculated to be 20 kips. Additional lateral load resistance may be obtained from passive resistance acting against the face of pile caps as illustrated on Diagram C-1.

Due to the probable existence of an old buried basement at the site, with associated foundations and slabs, pile driving could prove difficult. To avoid damage to piles it may be necessary to pre-drill all pile locations.

2. Probe Piles

Prior to specifying the lengths of the "Production Piles", it is advisable to drive at least 10 probe piles across the site in order to observe the driving characteristics of the piles and the ability of the driving equipment to drive them. The driving criteria and optimum length or range of lengths of the "Production Piles" will be established from these probe piles, and it is mandatory that the contractor use the same driving equipment for production piles as he used for driving the probe piles. Probe pile lengths should be established using a tip elevation of -64 feet, which is 5 feet longer than deemed necessary but will allow for flexibility in the probe pile program in the event the piles drive easier than anticipated. All pile driving operations should be observed by a representative of this office to ensure the adequacy of penetration into the sand and to determine practical "refusal" of piles therein.

3. Settlements

The future settlements under the piles will take place due to the presence of the clay layers beneath the sand bearing layer. It is estimated that over the life of the building total settlements will be approximately 1-1/4 inches under the center of the building, but differential settlements between exterior and interior columns will not exceed about 1/2 inch.

C. Removal of Existing Fill, Placement of Engineered Fill, and Treatment Under Slab-on-Grade

The existing loose sand and rubble fill underlying the site should be removed to a depth of at least 2 feet and be replaced with compacted granular fill. The existing fill may be reused if it does not contain excessive quantities of debris. Such a fill layer will not eliminate settlements in the underlying fill and soft clay but it will reduce differential settlements by acting as a soil mat.

Material to be used for fill should be predominantly granular, free of organic matter and large rocks, and have a plasticity index of 12 or less. Fill should be placed in 8-inch lifts and compacted to a minimum relative compaction of 90 percent of maximum as determined by ASTM Test Designation D1557-70.

It is recommended that any existing buried basement walls encountered at the site be removed to a depth of at least 3 feet below finished grade so as not to interfere with proposed slab-on-grade. Where grade beams cross any existing walls, at least 6 inches of clearance should be maintained.

It may be desirable to moisture proof interior slabs-on-grade at the ground floor level. This may be accomplished by placing beneath the slabs a 4-inch coarse sand capillary break, an impermeable membrane such as visqueen, and a 2-inch layer of protective sand.

10. LIMITATIONS

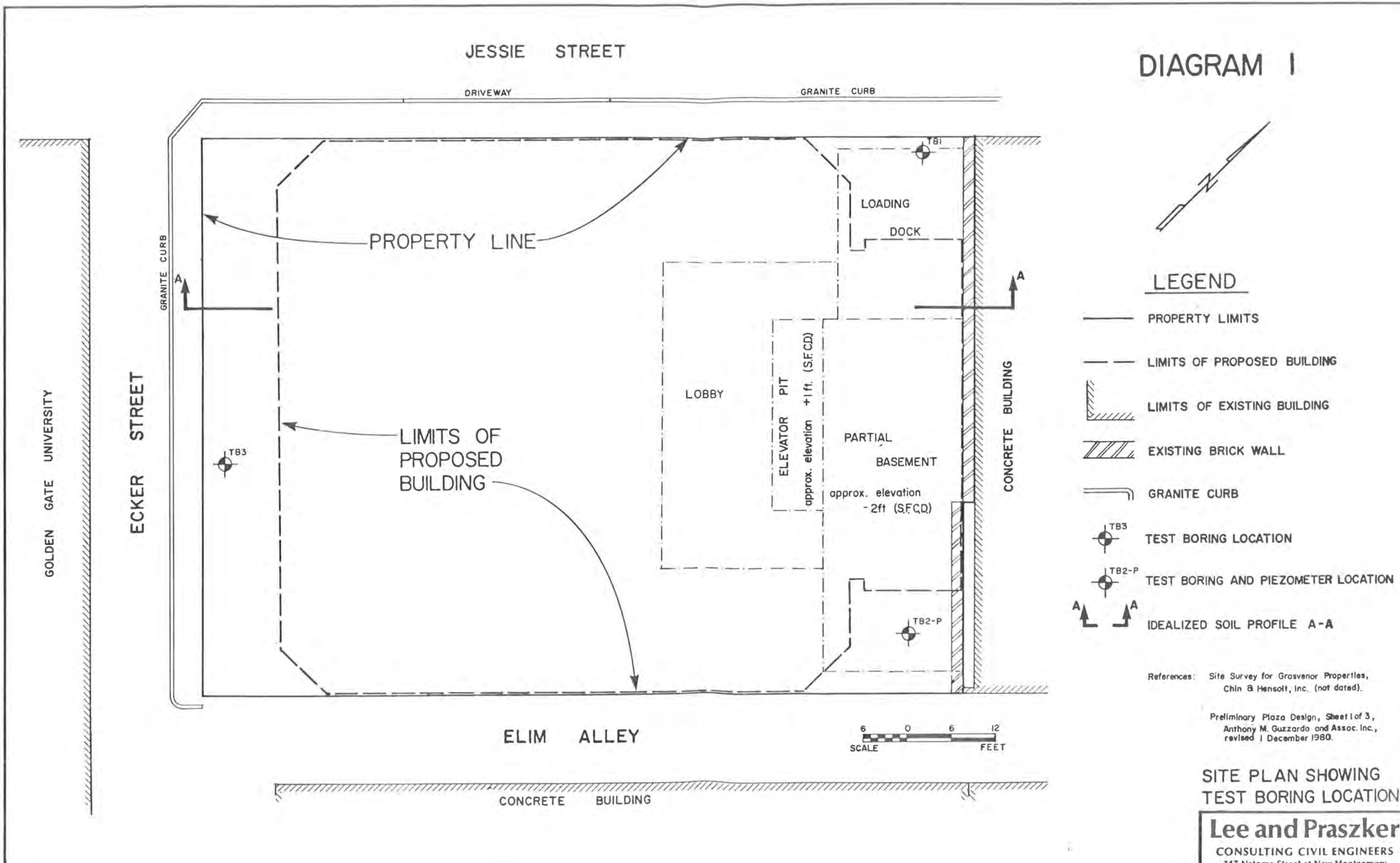
This report necessarily assumes uniform variation of soils between test borings. If any unusual conditions, such as soft pockets or unusually high groundwater are encountered when making excavations, the owner or his representative should notify the Soil Engineer immediately, so that supplementary recommendations can be made.

This report is issued with the understanding that it is the responsibility of the owner or his representative to ensure the applicable provisions of the recommendations contained herein are called to the attention of the Architect and the Structural Engineers and incorporated into the plans and that the necessary steps are taken to see that the contractor and sub-contractors carry out such provisions in the field.

The use of this report, its contents, or any part thereof by a party, or its agents, other than the one to whom this report is addressed, is herewith disallowed.

The use of any information contained in this report for purposes other than those expressedly stated in the report is at the user's own risk.

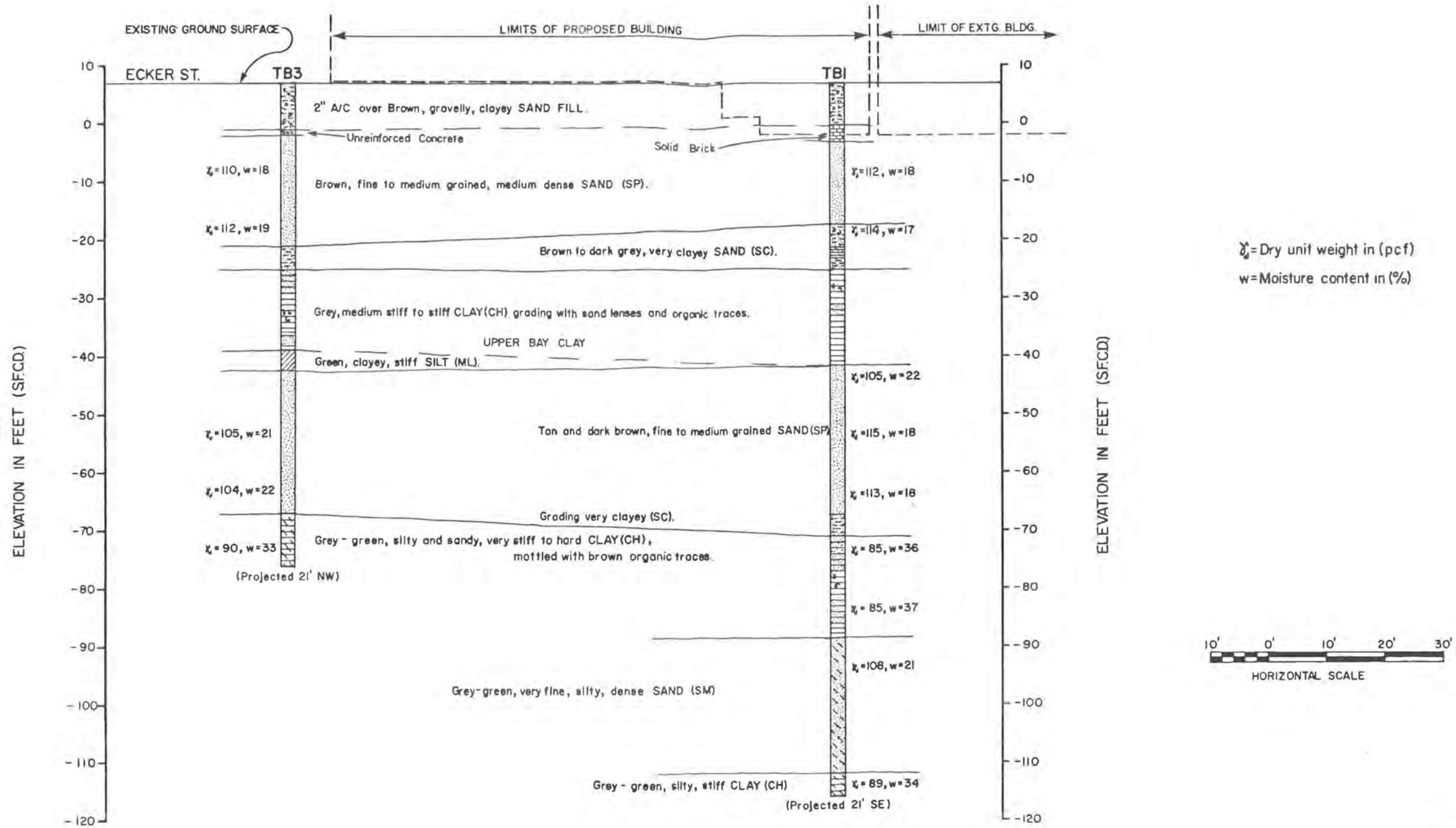
DIAGRAMS



JESSIE STREET OFFICE BUILDING - SAN FRANCISCO, CALIFORNIA

IDEALIZED SOIL PROFILE A-A

DIAGRAM 2



IDEALIZED SOIL PROFILE A-A

Lee and Praszker
CONSULTING CIVIL ENGINEERS
147 Natoma Street at New Montgomery
San Francisco, 94105 (415) 392-4866

DRAWN BY BSM	DATE 27 JAN 81	REVISION
CHECKED BY	JOB NO. L-726	SHEET NO.
APPROVED		DIAGRAM NO. 2

JESSIE STREET OFFICE BUILDING - SAN FRANCISCO, CALIFORNIA

APPENDICES

APPENDIX A

FIELD INVESTIGATION

DIAGRAM A-1

GRAVEL

SAND, SANDY

SILT

SILTY

CLAY

CLAYEY

ORGANIC MATERIAL

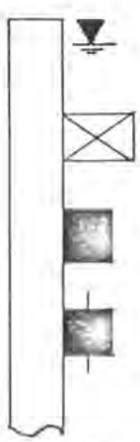
WEATHERED TO DECOMPOSED BEDROCK

BEDROCK

CLASSIFICATION	RANGE OF GRAIN SIZES U.S. Standard Sieve Size	RANGE OF GRAIN SIZES Grain Size in mm
BOULDERS	Above 12"	Above 305
COBBLES	12" to 3"	305 to 76.2
GRAVEL	3" to No.4	76.2to4.76
Coarse	3"to3/4"	76.2to19.1
Fine	3/4"toNo.4	19.1to4.76
SAND	No.4toNo.200	4.76to0.074
Coarse	No.4toNo.10	4.76to2.00
Medium	No.10toNo.40	2.00to0.420
Fine	No.40toNo.200	0.42to0.074
SILT & CLAY	Below No.200	Below .074

GRAIN SIZE CHART

METHOD OF SOIL CLASSIFICATION

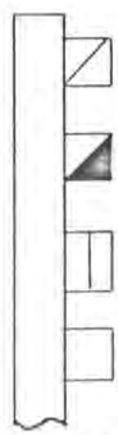


Indicates Ground Water Level

Indicates Depth of Disturbed Jar Sample

Indicates Depth of Undisturbed Split Tube Sample (2½" I.D.)

Indicates Depth of Undisturbed Shelby Tube Sample (3" O.D.)



Indicates Depth of Undisturbed Pitcher Tube Sample

Indicates Depth of Undisturbed Modified Pitcher Tube Sample

Indicates Depth and Length of Run with NX Core Barrel

Indicates Depth of Sampling Attempt with NO Recovery

KEY TO SAMPLES SHOWN ON BORING LOGS

SOIL CLASSIFICATION CHART & KEY TO SAMPLES

Lee and Praszker
CONSULTING CIVIL ENGINEERS

JOB NO. L - 726
BY T T
DATE 1/26/81

LOG OF BORING TB-1

FILE NO L-726

DIAGRAM A-2

JOB Jessie Street Office Building DATE DRILLED 1/19/81 INSPECTOR TT CHECK RR

G.S. ELEV +6.6' SFCD DRILLER J. N. Pitcher RIG Failing 750 SAMPLER See A-1

DEPTH FT	ELEV FT	LOG	SAMPLE	BLOWS/FT WT 400# DROP 15" 1st	2nd	W. S. ELEV. / ELEV.	DESCRIPTION	DRY UNIT WEIGHT PCF	MOISTURE % DRY WT.	VOID RATIO e
0	+7			6"	12"		0-7': 2" A/C over Brown Gravely Clayey SAND FILL			
7	0						7-10': Solid Brick			
10	-3						10-24': Brown Fine to Medium Grained, Medium Dense SAND (SP)	112	18	.51
15			■	13	45					
25	-17		■	13	37		24-32': Brown to Dark Grey SAND mixed with Grey Stiff CLAY (SP-CH)	114	17	.48
30	-25									
32							32-48': Grey Medium Stiff to Stiff CLAY (CH) grading with Brown Organic Traces and Sand Lenses UPPER BAY CLAY			
48	-41									
50			■	20	62		48-77': Tan and Dark Brown Fine to Medium Grained Dense SAND (SP)	105	22	.61
60			■	36	90/6"			115	18	.47
70			■	24	96			113	18	.49
75							- Grading Very Clayey (SC)			
77	-70									
78			■	10	40		77-95': Grey Green Sandy Very Stiff CLAY (CH)	85	36	.98

LOG OF BORING TB-1

FILE NO L-726 DIAGRAM A-3

JOB Jessie Street Office Building DATE DRILLED 1/19/81 INSPECTOR II
CHECK RR

G.S. ELEV +6.6' SFCD DRILLER J. N. Pitcher RIG Failing 750 SAMPLER See A-1

DEPTH FT	ELEV FT	LOG	SAMPLE	BLOWS/FT WT 400# DROP 18" 1st	2nd	W. S. ELEV. / ELEV.	DESCRIPTION	DRY UNIT WEIGHT PCF	MOISTURE % DRY WT.	VOID RATIO e
80				6"	12"		77-95': Grey Green Sandy Very Stiff to Hard CLAY (CH) Mottled w/ Brown Organic Traces			
85										
90				7	57			85	37	.98
95	-88						95-118': Green Grey Very Fine Silty Dense SAND (SM)			
100				30	50/4"			108	21	.56
105										
110										
115										
	-111									
120				11	42		118-121': Grey Green Silty Stiff CLAY (CH)	89	34	.89

Bottom of Test Boring -1
121-1/2' Below Ground Surface
at -115' SFCD

LOG OF BORING TB-2

FILE NO L-726 DIAGRAM A-4

JOB Jessie Street Office Building DATE DRILLED 1/20/81 INSPECTOR TT CHECK RR

G.S. ELEV +6.7' SFCD DRILLER J. N. Pitcher RIG Failing 750 SAMPLER See A-1

DEPTH FT	ELEV FT	BLOWS/FT WT 400# DROP 15" 1st 6"	2nd 12"	DESCRIPTION	DRY UNIT WEIGHT PCF	MOISTURE % DRY WT.	VOID RATIO e
0	+7			0-8': 2" A/C Over Brown Gravelly Clayey SAND FILL			
	-2			8-9': Unreinforced Concrete			
15		27	66/6"	9-28': Brown Fine to Medium Grained, Medium Dense SAND (SP)	110	18	.53
25		24	46/6"		112	19	.51
	-21			28-32': Brown Very Clayey SAND (SC)			
	-25			32-46': Grey Medium Stiff CLAY (CH) grading with Sand Lenses and Organic Traces UPPER BAY CLAY	52	79	.94
	-39			46-49': Green Clayey Stiff SILT (ML)			
50	-42	34	60/5"	49-74': Tan and Dark Brown Fine to Medium Grained, Dense SAND (SP)			
60		42	50/5"		105	21	.60
70		29	90		104	22	.62
	-67			74-83': Green Grey Silty Very Hard CLAY (CH)	90	33	.88
				Bottom TB-2 83' Below Surface at -76' SFCD			

LEE & PRASZKER

FILE NO L-726

LOG OF BORING TB-3

DIAGRAM A-5

JOB Jessie Street Office Building DATE DRILLED 1/21/81 INSPECTOR TT

CHECK RR

G.S. ELEV +7.3' SFCD DRILLER J. N. Pitcher RIG Failing 750 SAMPLER See A-1

DEPTH FT	ELEV FT	LOG	SAMPLE	BLOWS/FT WT 400# DROP 15" 1st	2nd	W. S. ELEV. / ELEV.	DESCRIPTION	DRY UNIT WEIGHT PCF	MOISTURE % DRY WT.	VOID RATIO e
0	+7			6"	12"		0-9': 2" A/C over Brown Gravelly Clayey SAND FILL			
10	-3						9-10': Bricks and Concrete			
15			■	14	68		10-35': Brown Fine to Medium Grained, Slightly Silty Medium Dense SAND (SP)	113	17	.49
25			■	28	96			112	18	.50
35	-28						35-46': Grey Very Silty and Sandy Medium Stiff CLAY (CH) Mottled with Organic Traces UPPER BAY CLAY			
45	-39									
50			■	15	24		46-55': Green Very Silty and Clayey Fine Grained, Dense SAND (SM)	113	18	.49
55	-48									
60			■	48	50/3"		55-77': Tan and Dark Brown Fine to Medium Grained, Dense SAND (SP)	106	20	.59
70			■	35	50/5"			114	18.5	.48
75	-70									
80			■	28	80		77-82': Grey Green Fine Grained, Dense SAND (SP)	109	20	.54
							Bottom TB-3 82' Below Surface at -75' SFCD			

LEE & PRASZKER

APPENDIX B

LABORATORY TEST RESULTS

SUMMARY OF LABORATORY TEST RESULTS

Test Boring		Material	Physical Constants					Atterberg Limits			Unconfined Compression		Direct Shear			Cons Test	Mechanical Analysis % Passing Sieve No.			
No.	Depth Feet		γ_w	γ_D	w	e	G *	LL	PI	SL	Qu	α	c	ø	S	Cc	4	40	200	
1	15	Dense SAND	131	112	18	.51	2.7													
1	25	Med. Dense SAND	133	114	17	.48	2.7													
1	50	Dense SAND	128	105	22	.61	2.7													
1	60	Dense SAND	136	115	18	.47	2.7						730	34^0						
1	70	Dense SAND	133	113	18	.49	2.7													
1	80	Stiff CLAY	116	85	36	.98	2.7													
1	90	Stiff CLAY	116	85	37	.98	2.7													
1	100	Dense SAND	130	108	21	.56	2.7													
1	120	Stiff CLAY	119	89	34	.89	2.7													
2	15	Med. Dense SAND	130	110	18	.53	2.7													
2	25	Med. Dense SAND	133	112	19	.51	2.7													
2	40	Med. Stiff CLAY	94	52	79	.94	2.7													
2	50	No Sample																		
2	60	Dense SAND	128	105	21	.60	2.7													
2	70	Dense SAND	127	104	22	.62	2.7													
2	80	Stiff CLAY	120	90	33	.88	2.7													

File No. 1-726 *Assumed Lee & [illegible]

SUMMARY OF LABORATORY TEST RESULTS

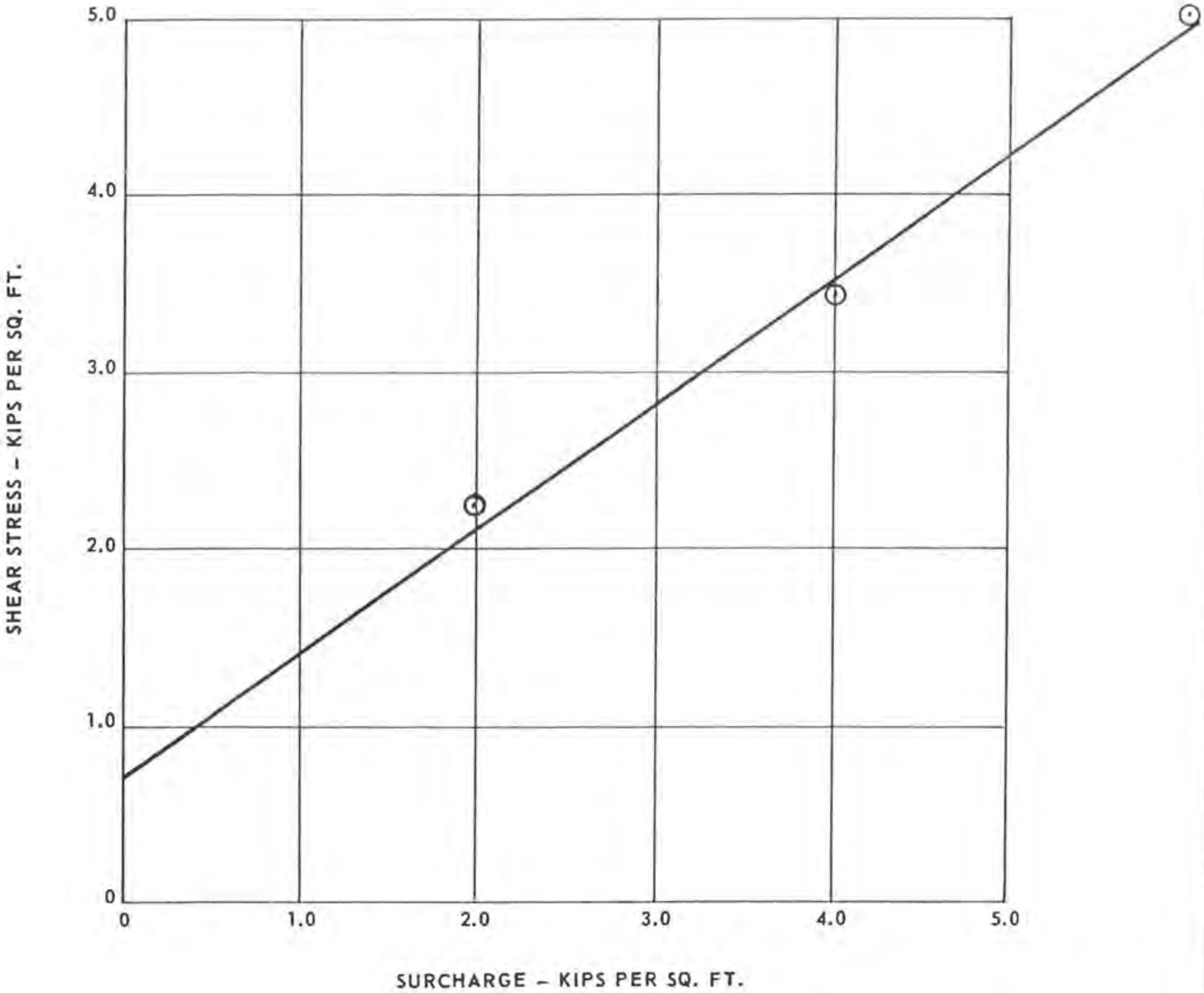
Test Boring		Material	Physical Constants					Atterberg Limits			Unconfined Compression		Direct Shear			Cons Test	Mechanical Analysis % Passing Sieve No.			
No.	Depth Feet		γ_w	γ_D	w	e	G*	LL	PI	SL	Qu	α	c	ø	S	Cc	4	40	200	
3	15	Med. Dense SAND	132	113	17	.49	2.7													
3	25	Med. Dense SAND	133	112	18	.50	2.7													
3	50	Dense SAND	127	106	20	.59	2.7													
3	60	Dense SAND	135	114	19	.48	2.7													
3	70	Dense SAND	135	114	19	.48	2.7						270	41^{0}						
3	80	Dense SAND	131	109	20	.54	2.7													

File No. L-726 * Assumed

JOB # L-726 BY BM DATE 1/21/81

DIAGRAM B-1

DIRECT SHEAR TEST



DESCRIPTION OF SAMPLE	Brown Fine to Medium, Dense SAND (SP)		
BORING: TB-1	DEPTH: 60'	ELEVATION: -53' SFCD	
APPARENT ANGLE OF INTERNAL FRICTION (Ø)	35°	APPARENT COHESION (C)	730 PSF
NATURAL VOID RATIO	0.47	EXISTING OVERBURDEN STRESS (KSF.)	
DRY DENSITY (PCF.)	115	SHEAR RATE (IN/MIN)	0.05
NATURAL MOISTURE CONTENT %	18	PEAK STRENGTH/RESIDUAL STRENGTH	

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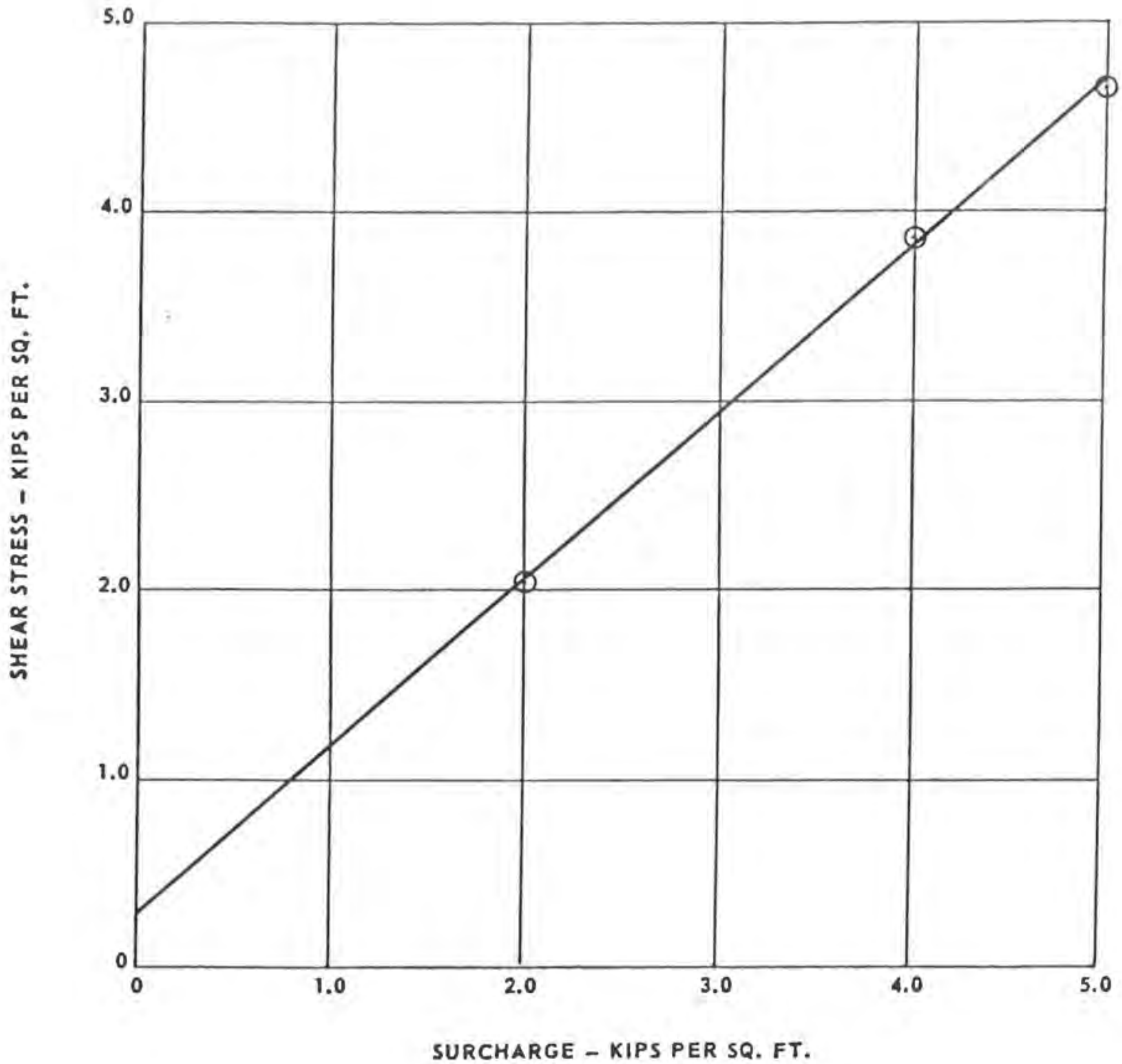
DIAGRAM B-2

DIRECT SHEAR TEST



JOB # L-726 BY BM DATE 1/22/81

DESCRIPTION OF SAMPLE	Grey Brown Medium, Dense SAND (SP)			
BORING: TB-3	DEPTH: 70'		ELEVATION: -63' SFCD	
APPARENT ANGLE OF INTERNAL FRICTION (Ø)	41^{0}	APPARENT COHESION (C)	270 PSF	
NATURAL VOID RATIO	0.48	EXISTING OVERBURDEN STRESS (KSF.)		
DRY DENSITY (PCF.)	114	SHEAR RATE (IN/MIN)	0.05	
NATURAL MOISTURE CONTENT %	19	PEAK STRENGTH/RESIDUAL STRENGTH		

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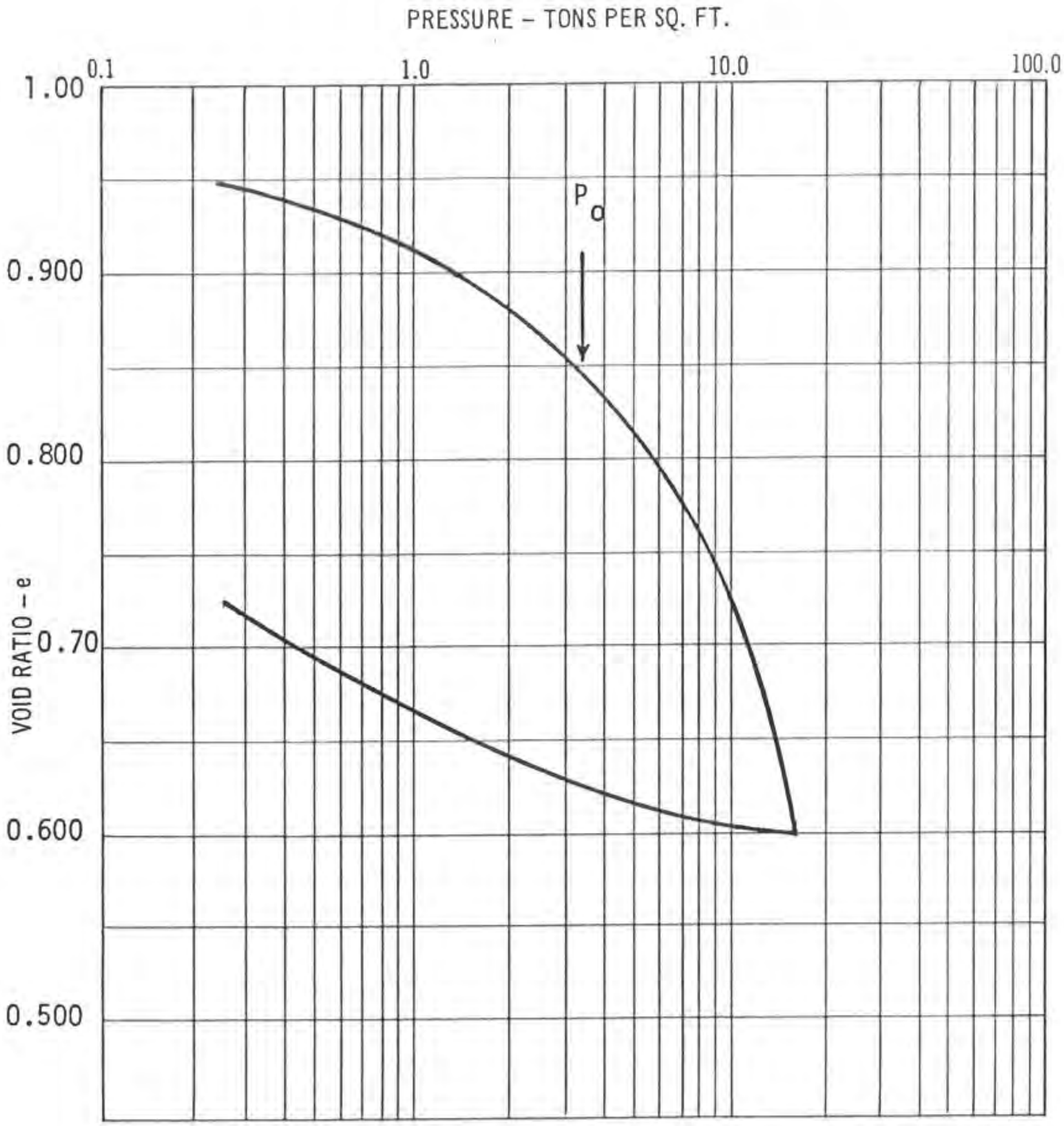
DIAGRAM B-3

CONSOLIDATION TEST DATA



DESCRIPTION OF SAMPLE:	Grey-green, stiff CLAY (CH)				
BORING: TB-1		DEPTH: 90'		ELEVATION: -83'	
MOST PROBABLE PRECONSOLIDATION STRESS (TSF) SHOWN THUS: $P_c\downarrow$			EXISTING OVERBURDEN STRESS (TSF) SHOWN THUS: $P_o\downarrow$		3.3
COMPRESSION INDEX (C_c)		0.55	SWELLING INDEX (C_s)		0.09
MOISTURE CONTENT %	INITIAL	37	VOID RATIO	INITIAL - e_o	0.981
	FINAL	26		FINAL - e_F	0.721
DRY DENSITY (PCF.)	INITIAL	85	DEGREE OF SATURATION %	INITIAL	100
	FINAL	100		FINAL	100

JOB NO. L-726 BY BM DATE 2/5/81

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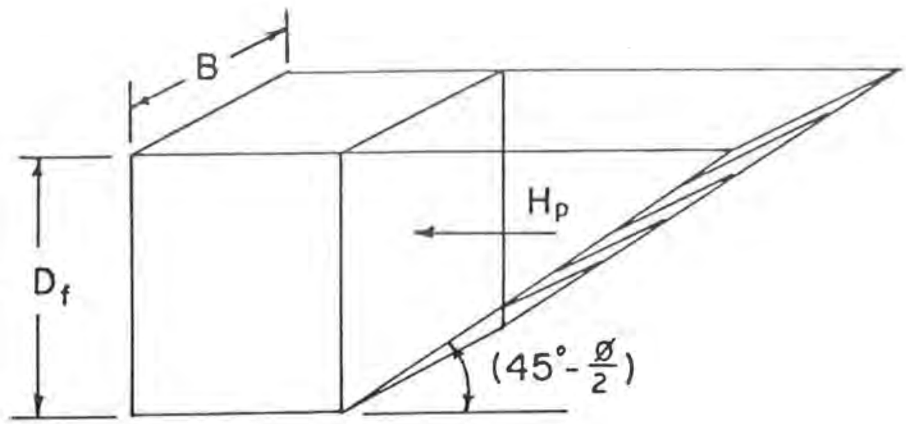
APPENDIX C

LATERAL LOADS

LATERAL LOAD RESISTANCE
ON FRONT SIDE OF PILE CAPS AND GRADE BEAMS



$$H_p = \frac{\gamma K_p}{2} D_f^2 B + 2 C D_f \sqrt{K_p}\ B = 110 D_f^2 B + 566 D_f B \text{ in lbs}$$

Where: γ = Soil Unit Weight = 110 psf

D_f = Depth of Footing in feet

K_p = Coefficient of Passive Pressure = 2.0

B = Width of Footing in feet

ϕ = Angle of Internal Friction = 20°

C = Cohesion = 200 psf

DIAGRAM C-1

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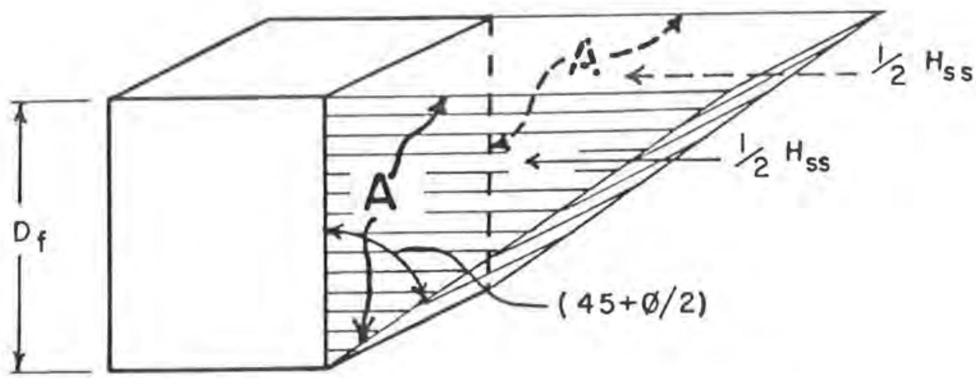
Job No. L-726 | Sheet ___ of ___

By TT | Chk R.R.

Date 2/10/81 | Date ___

LATERAL LOAD RESISTANCE DEVELOPED THROUGH SIDE SHEAR ON PILE CAPS AND GRADE BEAMS



$$H_{ss} = 2A\left[\left(\frac{\gamma K_o D_f}{3}\right)\tan\phi + C\right] = 2A(8D_f + 200) \text{ in lbs.}$$

WHERE:

A = End Area of Failure Wedge
 = $\frac{1}{2} D_f^2 \tan(45 + \phi/2)$ in square feet

D_f = Depth of Footing in feet

ϕ = Angle of Internal Friction = 20°

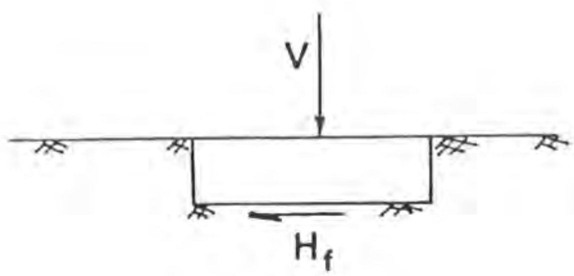
C = Soil Cohesion = 200 psf

γ = Soil Unit Weight = 110 pcf

K_o = Coefficient of At Rest Pressure = 0.6

DIAGRAM C-2

LATERAL LOAD RESISTANCE DEVELOPED THROUGH FRICTION ON THE BOTTOM OF GRADE BEAMS



$$H_f = \mu V = 0.4$$

Where: H_f = Lateral Resistance

μ = Coefficient of Friction Between Rough Concrete and Soil
= 0.4

V = Vertical Load

DIAGRAM C-3