APPENDIX D

GEOLOGY

Jerry Kovacs and Associates, Inc.
Consulting Geotechnical Engineers

439 WESTERN AVENUE, GLENDALE, CALIFORNIA 91201-2837
(818) 240-9600 (818) 980-0825 FAX (818) 240-9675

RECEIVED
MAR 06 2000
By______________

GEOTECHNICAL ENGINEERING INVESTIGATION
PROPOSED MIXED-USE DEVELOPMENT
SOUTH SIDE OF WEYBURN AVENUE
BETWEEN TIVERTON AND GLENDON AVENUES
WESTWOOD, CALIFORNIA

FOR

THE CASDEN COMPANY

FILE NO. 17526-S FEBRUARY 25, 2000

TABLE OF CONTENTS

TABLE OF CONTENTS - Continued

TABLE OF CONTENTS - Continued

APPENDIX

GEOTECHNICAL ENGINEERING INVESTIGATION

PROPOSED MIXED-USE DEVELOPMENT

SOUTH SIDE OF WEYBURN AVENUE,

BETWEEN TIVERTON AND GLENDON AVENUES

WESTWOOD, CALIFORNIA

INTRODUCTION

This report presents the results of the geotechnical engineering investigation performed on the subject property. The purpose of this investigation was to evaluate the nature of the soils underlying the site, to ascertain their engineering properties, and to provide recommendations for site preparation, foundation design, resistance to lateral loading, basement wall design, concrete slabs-on-grade, expansive soils, temporary excavations, and shoring. In addition, the geologic/seismic environment of the site was defined and an evaluation of the potential seismic hazards was performed.

This investigation included excavating six exploratory borings, obtaining representative samples, laboratory testing including corrosivity testing by the office of M.J. Schiff and Associates, engineering analysis, review of available geologic literature and geotechnical engineering reports, review of a dewatering report, and the preparation of this report. Geotechnical reports reviewed included two previous geotechnical investigations of the subject site, conducted in 1990 and 1996, as discussed in the "Previous Reports" section of this report. The exploratory boring locations from the current and previous geotechnical investigations are shown on the enclosed Plot Plan. The results of the exploration and the laboratory tests are shown in the Appendix of this report.

INTENT

The purpose of this report is to aid in the design and completion of the described project. Implementation of the advice presented in this report is intended to reduce certain risks associated with construction projects. The professional opinions and geotechnical advice contained in this report are sought because of special skill in engineering and geology. Jerry Kovacs and Associates has a duty to exercise the ordinary skill and competence of members of the engineering profession. Those who hire Jerry Kovacs and Associates are not justified in expecting infallibility, but can expect reasonable care and competence.

STRUCTURAL CONSIDERATIONS

Information concerning the proposed development consisted of conceptual information provided by the offices of Van Tilberg, Banvard & Soderbergh, Architects. A topographic survey prepared by Psomas and Associates, dated April 27, 1989, was used as a reference for the exploration program and the preparation of the enclosed Plot Plan. Structural loading information is not yet available. It is anticipated the proposed development will consist of a three to six-story mixed use development. The entire complex will be underlain by a 5 level subterranean garage. According to information provided by the architect, the garage will slope or step down to the south to southwest, roughly following the site contours, maintaining the lower garage level at approximately 50 feet below the current site grade.

Due to the preliminary nature of the design at this time, wall and column loads are not yet available. For preliminary purposes, it will be estimated that column loads will be a maximum of 2,000 kips and wall loads will be a maximum of 25 kips per linear foot. Once structural loads are finalized, the recommendations within this report should be reviewed and revised, if necessary.

Grading will consist of excavations up to approximately 55 feet for the proposed subterranean parking garage and foundations.

In the event of any changes in the design or location of any structure, as outlined in this report, the recommendations contained herein should not be considered valid unless the changes are reviewed and recommendations are modified or reaffirmed after such review.

SITE CONDITIONS

The subject property is located within the Westwood area of the City of Los Angeles, California. The site is bounded by Weyburn Avenue along the north, Tiverton Drive along the east, a public alley along the west, and existing buildings and Glendon Avenue to the south. Glendon Avenue currently runs north-south through the site, however, it will terminate at the southern property boundary upon project completion. The Vicinity Map included in the Appendix shows the location of the site.

The eastern two-thirds of the site, east of Glendon Avenue, is currently occupied by paved, at-grade parking, with the exception of a four-story building at the extreme south-central portion of the site. The portion of the site west side of Glendon Avenue is occupied by a two-story building over a one-level basement, a two-story theater and paved parking. The site ascends gently towards the northeast. The total topographic relief across the site is approximately 19 feet.

The site is bounded by buildings along the majority of the south property line. The extent of basement levels beneath these buildings is not currently known.

Vegetation on the site consists of shrubs and trees in isolated planters scattered across the site. Drainage is by sheetflow along the existing contours to the south and southwest.

PREVIOUS GEOTECHNICAL REPORTS

As part of this investigation, two previous geotechnical investigations of the site were reviewed. The first of these investigations was by LeRoy Crandall and Associates, entitled "Report of Geotechnical Investigation, Proposed Westwood Village Mixed Use Project," dated January 25, 1990. The second report was by this office (Jerry Kovacs and Associates), and was entitled "Geotechnical Engineering Investigation, Proposed Village Center Mixed-Use Development, South Side of Weyburn Avenue, Between Tiverton and Glendon Avenues, Westwood, California," dated July 15, 1996. The locations of borings performed during these investigations are shown on the enclosed Plot Plan, and logs of these borings are included in the Appendix of this report. The results of laboratory testing included in the 1996 report by this office have been incorporated into this current investigation.

EXPLORATION AND TESTING

The site was most recently explored on January 10, 2000 by drilling two exploratory borings to depths of 100 feet below existing site grade. The borings were excavated with the aid of a truck-mounted, hollow-stem auger drilling machine, and were approximately 8 inches in diameter. The boring locations from the current and previous site exploration are shown on the Plot Plan and the soils encountered are logged on Plates A-1 through A-21.

Undisturbed samples of the soils encountered in the borings were obtained and transported to the laboratory. The results of the laboratory tests, along with a description of the exploration and laboratory test procedures used are given in the Appendix.

EARTH MATERIALS

Existing Fill

Fill material was encountered in one of the borings to a depth of 7 feet, and in the remaining 15 borings excavated by this firm and Crandall to a maximum depth of approximately 2 feet. The fill consists of various mixtures of silt, sand and clay, which were gray to reddish brown, generally dense, fine to coarse grained, and contained various amounts of gravel.

Native Soils

The native soils consist mostly of various combinations of sand, silt and clay, which are generally dense to very dense and stiff, fine to coarse grained, slightly moist to moist except below the water level, and contain varying amounts gravel. Occasional very gravelly zones were encountered. According to Dibblee (1991), the native soils consist of older alluvial sediments typical to this area of Los Angeles County. More detailed soil profiles may be obtained from individual boring logs.

The subsurface conditions described herein have been projected from excavations on the site as indicated and should not be construed to reflect any variations which may occur between these excavations or which may result from changes in subsurface conditions.

GROUNDWATER

Approximately 30 minutes after completion of the recent Borings 5 and 6, water levels were measured at depths of 64 and 62 feet, respectively. These depths correspond to depths of elevations of 272.5 to 273 feet above mean sea level, respectively. Wells were set in each of the borings, and the water level was subsequently measured at the following depths and elevations, as reported by the firm California Environmental:

Boring	Date	Depth to Water (Feet)	Elevation* (Feet Above Mean Sea Level)
5	2/8/00	42.56	293.9
	2/24/00	43.42	293.1
6	2/8/00	44.87	290.1
	2/24/00	44.66	290.3

* Approximate - Boring Elevations Not Surveyed

It should be noted that these elevations reflect the long term, stabilized level of wells which penetrate into much deeper water bearing lenses.

Water levels were not determined during the 1995 exploration by this firm, due to the nature of the equipment utilized. During Crandall's exploration between December 1988 and September 1989, groundwater was encountered in bucket-auger borings at depths of 53 to 70 feet below ground surface, corresponding to elevations between 262 and 274 feet. According to Crandall, water seepage was encountered at shallower depths in other borings excavated for nearby properties between 1981 and 1988.

According to Mendenhall (1905), historic groundwater levels in the vicinity of the site have been at an elevation of approximately 225 feet, corresponding to a depth of about 110 feet below existing site grade. It is suspected that the shallower water encountered is indicative of localized perched water contained within granular layers.

Fluctuations in the level of groundwater may occur due to variations in rainfall, temperature, and other factors not evident at the time of the measurements reported herein. Fluctuations also may occur across the site. High groundwater levels can be hazardous.

DESCRIPTION OF EXPLORATION AND LABORATORY TESTING

Exploration

Field exploration is performed with the aid of a truck-mounted, rotary drilling machine. The soil is continuously logged by the field engineer and classified by visual examination in accordance with the Unified Soil Classification system.

The location of borings is determined by property lines furnished by the client. Elevations of borings are determined by hand level or interpolation between plan contours. The location and elevation of the borings should be considered accurate only to the degree implied by the method used.

Undisturbed samples of soil are obtained at frequent intervals. Unless noted on the boring logs as an SPT sample, samples acquired while utilizing a hollow-stem auger drill rig are obtained by driving a thin-walled, California Standard Sampler with successive 30-inch drops of a 140-pound hammer. Samples from bucket-auger drilling are obtained utilizing a California Standard Sampler with successive 12-inch drops of a kelly bar, whose weight is noted on the boring logs. The soil is retained in brass rings of 2.50 inches inside diameter and 1.00 inches in height. The central portion of the samples are stored in close fitting, waterproof containers for transportation to the laboratory. Samples noted on the boring logs as SPT samples are obtained in accordance with ASTM D 1586.

Classification

The field classification is verified in the laboratory, also in accordance with the Unified Soil Classification System. Laboratory classification may include visual examination, Atterberg Limit Tests and grain size distribution. The final classification is shown on the boring logs.

Moisture-Density

The field moisture content and dry unit weight are determined for each of the undisturbed soil samples, and the moisture content is determined for SPT samples. The information is useful in providing a gross picture of the soil consistency between borings and any local variations. The dry unit weight is determined in pounds per cubic foot and shown on the "Boring Logs," A-Plates. The field moisture content is determined as a percentage of the dry unit weight.

Shear Tests

Shear tests are performed with a strain controlled, direct shear machine manufactured by Soil Test, Inc. The rate of deformation is approximately 0.025 inches per minute. Each sample is sheared under varying confining pressures in order to determine the Mohr-Coulomb shear strength parameters of the cohesion intercept and the angle of

DESCRIPTION OF EXPLORATION AND LABORATORY TESTING - continued

internal friction. Samples are generally tested in an artificially saturated condition. Depending upon the sample location and future site conditions, samples may be tested at field moisture content. The results are plotted on the "Shear Test Diagram," B-Plates.

Consolidation

Settlement predictions of the soil's behavior under load are made on the basis of the consolidation tests. The consolidation apparatus is designed to receive a single one-inch high ring. Loads are applied in several increments in a geometric progression, and the resulting deformations are recorded at selected time intervals. Porous stones are placed in contact with the top and bottom of each specimen to permit addition and release of pore fluid. Samples are generally tested at increased moisture content to determine the effects of water on the bearing soil. The normal pressure at which the water is added is noted on the drawing. Results are plotted on the "Consolidation Test," C-Plates.

Expansion Tests

In order to determine the expansiveness of the soil, two tests are generally performed. The swell test is performed on natural or recompacted soil within two rings. Each ring is confined by a normal pressure of 60 pounds per square foot. One ring is inundated with water and allowed to expand over a 24-hour period. The total vertical rise is measured and the swell determined as a percent of vertical height. The second ring is air-dried and the shrinkage measured. The total expansion is determined as the difference between the air-dry and the saturation measurements. The expansion character is often determined by the Expansion Index Method.

Remolded Tests

Compaction tests are performed in accordance with ASTM D 1557. Remolded samples for shear, swell, and consolidation are then prepared at densities corresponding to 90 or 95 percent of the maximum dry density. Compaction tests are tabulated on Plate D. Shear tests are shown on B-Plates and Consolidation results on C-Plates.

BORING LOG NUMBER 1

Drilling Date: 12/4/95
Project: File No. 16315-S

Elevation: 338.0'
Village Center

Sample Depth Ft	Blows per ft	Moisture content %	Dry Unit Weight p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: 2 to 3 inches A.C. paving
				0		FILL: Clayey Sand, reddish brown, dense
				1		
2	15	13.4	113.5	2	SC	Clayey Sand, reddish brown, dense, fine grained, trace gravel
				3		
				4	CL	Sandy Clay, reddish brown, firm, trace gravel
5	12	17.2	116.2	5		
				6		
				7		
				8	SC	Clayey Sand, reddish brown, dense
				9		
10	20	12.9	113.9	10		
				11		
				12		
				13		
				14		abundant slate gravel with fine to medium grained clayey sand matrix
15	35	12.3	125.6	15		
				16		
				17		
				18		
				19		
	40	11.5	127.6	20		
				21		
				22		
				23	GC	Clayey Gravel, brown and reddish brown, dense, angular clasts
				24		
25	17	12.2	114.7	25		
				26		
				27		
				28		
				29		
30	6	16.2	112.0	30		

BORING LOG NUMBER 1 (continued)

Drilling Date:
Project: File No. 16315-S

Elevation:
Village Center

Sample Depth Ft	Blows per ft	Moisture content %	Dry Unit Weight p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions
				31		
				32		
				33		
				34		
35	40	12.3	123.6	35		Sandy Gravel, medium to coarse grained sand, dense
				36		
				37		
				38	SM	Silty Sand, orange-brown, dense, fine grained
				39		
40	35	8.9	118.5	40		
				41		
				42		
				43		brown
				44		
45	30	13.0	110.8	45		
				46		
				47	CH	Silty Clay, reddish brown, firm, trace coarse grained sands
				48		
				49		
50	22	27.3	101.4	50		
				51		
				52		
				53		
				54		
55	45	20.6	113.3	55		
				56		
				57	SC	Clayey Sand, brown, dense, fine grained
				58		
				59		
60	40	20.3	111.7	60		

BORING LOG NUMBER 1 (continued)

Drilling Date:
Project: File No. 16315-S

Elevation:
Village Center

Sample Depth Ft	Blows per ft	Moisture content %	Dry Unit Weight p.c.f.	Depth in feet	USCS Class.	Surface Conditions Description
				61		
				62		
				63		
				64	CL	Silty Clay, reddish brown, firm, some slate clasts
65	80	13.5	125.1	65		
				66		
				67	SM	Silty Sand, orange-brown, dense, fine grained
				68		
				69		
70	30	21.6	110.0	70		
				71		
				72		
				73	CL	Silty Clay, reddish brown, firm, some slate gravel
				74		
75	40	21.4	105.9	75		
				76	SM	Silty Sand, orange-brown, dense, fine grained
				77		
				78		
				79		
80	55	23.8	105.4	80		
				81		
				82	CL	Silty Clay, reddish brown, firm
				83		
				84		
85	35	28.3	99.7	85		
				86		
				87	SC	Clayey Sand, brown, dense, fine grained
				88		
				89		
90	40	20.6	110.7	90		
						brown with gray mottling, fine to medium grained

BORING LOG NUMBER 1 (continued)

Drilling Date:
Project: File No. 16315-S

Elevation:
Village Center

Sample Depth Ft	Blows per ft	Moisture content %	Dry Unit Weight p.c.f.	Depth in feet	USCS Class.	Surface Conditions Description
				91		
				92		
				93		
				94		gray, very dense
95	60	22.5	111.1	95		
				96		
				97		
				98	CH	Silty Clay, black, hard
				99		
100	40	28.2	100.0	100		
				101		
				102		
				103		
				104		
105	35	23.1	105.1	105		
				106		
				107		
				108	SC	Clayey Sand, banded orange-brown, gray, brown, very dense, fine to medium grained
				109		
110	50	21.1	110.8	110		
				111		Total Depth: 110 feet Depth to groundwater not measured Fill to 1 foot
				112		
				113		
				114		
				115		
				116		
				117		
				118		Note: The stratification lines represent the approximate boundary between earth types; the transition may be gradual.
				119		
				120		

Jerry Kovacs and Associates

BORING LOG NUMBER 2

Drilling Date: 12/5/95
roject: File No. 16315-S

Elevation: 336.0'
Village Center

Sample Depth Ft	Blows per ft	Moisture content %	Dry Unit Weight p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: 1 inch A.C. paving, poor condition
				0	SC	Clayey Sand, dark brown, dense, fine grained, trace coarse grains
				1		
2	12	11.1	120.3	2		
				3		
				4		
5	15	21.5	115.7	5	CL	Sandy Clay, reddish brown, firm, some slate clasts
				6		
				7		
				8		
				9	SP	Gravelly Sand, dense, medium to coarse grained sand, angular clasts
10	23	9.6	127.3	10		
				11		
				12		Sand, fine to medium grained, some slate gravel
				13		
				14		
15	25	10.0	118.3	15		
				16		
				17		
				18	CL	Silty Clay, reddish brown, firm
				19		
20	11	18.9	110.8	20		
				21		
				22	SC	Clayey Sand, dark reddish brown, dense, fine to medium grained, trace slate gravel
				23		
				24		
25	15	14.1	116.4	25		
				26		
				27		
				28		sandier, orange-brown
				29		
30	20	14.9	115.2	30		

BORING LOG NUMBER 2 (continued)

Drilling Date:
Project: File No. 16315-S

Elevation:
Village Center

Sample Depth Ft	Blows per ft	Moisture content %	Dry Unit Weight p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions
				31		
				32		
				33		
				34		
35	40	8.5	136.3	35	GC	Sandy, Clayey Gravel, dense, angular clasts
				36		
				37		
				38	SC	Clayey Sand, gray and orange-brown, dense, fine grained
				39		
40	22	16.6	115.6	40		
				41	CL	Sandy Clay, gray, firm
				42		
				43		
				44		
45	25	15.9	112.0	45		
				46		
				47		
				48	CH	Silty Clay, orange-brown, stiff, blocky structure
				49		
50	25	23.9	104.4	50		
				51		
				52	GP	Sandy Gravel, dense
				53		
				54		
55	50	30.1	101.1	55	SC	Clayey Sand, brown, dense, fine grained
				56		
				57	CL	Sandy Clay, reddish brown, stiff
				58		
				59		
60	25	19.0	105.9	60	SC	Clayey Sand, brown, dense, fine grained

BORING LOG NUMBER 2 (continued)

Drilling Date:
Project: File No. 16315-S

Elevation:
Village Center

Sample Depth Ft	Blows per ft	Moisture content %	Dry Unit Weight p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions
				61		
				62		
				63		
				64		
65	20	23.4	105.1	65	CL	Silty Clay, reddish brown, stiff
				66		
				67		
				68		
				69	SC	Clayey Sand, orange-brown, dense, medium grained, some slate gravel
70	45	15.5	119.3	70		
				71		
				72		decrease in slate gravel
				73		
				74		
75	50	21.9	113.6	75		
				76		
				77		
				78		
				79		
80	40	22.4	110.2	80		
				81		
				82		
				83		
				84	CL	Sandy Clay, orange-brown and gray, stiff
85	40	18.4	112.9	85		
				86		
				87	SC	Clayey Sand, reddish brown, dense
				88		
				89		
90	60	21.0	111.0	90		
						Total Depth: 90 feet; Depth to groundwater not measured

Jerry Kovacs and Associates

BORING LOG NUMBER 3

Drilling Date: 12/5/95
Project: File No. 16315–S

Elevation: 330.0'
Village Center

Sample Depth Ft	Blows per ft	Moisture content %	Dry Unit Weight p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: 1 to 2 inches A.C. paving, poor condition
				0	CL	Sandy Clay, reddish brown, stiff, trace slate gravel
				1		
2	15	15.3	117.9	2		
				3		
				4		
5	17	14.4	120.9	5		sandier
				6	SC	Clayey Sand, reddish brown, dense, fine grained
				7		
				8		
				9		
10	12	14.1	119.9	10		
				11		
				12		
				13		clayier
				14		
15	12	16.3	113.4	15		
				16		
				17	CL	Sandy Clay, reddish brown, firm, some slate gravel
				18		
				19		
20	18	14.9	117.6	20		
				21		
				22		
				23		
				24		
25	20	18.8	110.9	25		
				26		
				27		less sandy
				28		
				29		
30	20	18.0	114.1	30		

BORING LOG NUMBER 3 (continued)

Drilling Date:
Project: File No. 16315-S

Elevation:
Village Center

Sample Depth Ft	Blows per ft	Moisture content %	Dry Unit Weight p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions
				31		
				32		
				33		
				34	SC	Clayey Sand, brown, dense, some slate gravel
35	40	16.9	121.0	35		
				36		
				37		
				38		
				39		
40	35	18.2	116.8	40		
				41		
				42	SP	Sand, brown, dense, poorly graded, medium grained
				43		
				44		
45	20	22.4	105.3	45	CH	Silty Clay, reddish brown, stiff
				46		
				47		
				48		
				49	CL	Sandy Clay, reddish brown, firm
50	25	21.7	108.9	50		
				51		
				52		
				53		
				54	SC	Clayey Sand, gray-brown, dense, medium grained, some slate
55	40	23.5	106.9	55		
				56		
				57		increase in slate gravel
				58		
				59	SP	Gravelly Sand, gray-brown, dense, angular clasts
60	80	10.9	123.8	60		

BORING LOG NUMBER 3 (continued)

Drilling Date:
Project: File No. 16315-S

Elevation:
Village Center

Sample Depth Ft	Blows per ft	Moisture content %	Dry Unit Weight p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions
				61		
				62		
				63		
				64	CL	Silty Clay, reddish brown, hard
65	30	24.4	105.2	65		
				66		
				67		
				68		
				69		
70	30	22.2	106.2	70		
				71		
				72		
				73		sandy clay
				74		
75	40	14.9	123.8	75		
				76		
				77	SP	Gravelly, coarse sand, dense, angular clasts
				78		
				79		
80	60	14.5	114.1	80		
				81		Total Depth: 80 feet Depth to groundwater not measured
				82		
				83		
				84		
				85		
				86		
				87		
				88		
				89		
				90		

BORING LOG NUMBER 4

Drilling Date: 12/5/95
Project: File No. 16315-S

Elevation: 333.0'
Village Center

Sample Depth Ft	Blows per ft	Moisture content %	Dry Unit Weight p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions:
				0	CL	Sandy Clay, reddish brown, stiff, fine grained, trace slate gravel
				1		
2	22	13.4	118.4	2		
				3		
				4		
5	17	15.5	115.7	5		
				6		
				7		
				8		
				9		
10	30	15.4	119.2	10		
				11		
				12		
				13	SC	Clayey Sand, reddish brown, dense, increase in slate gravel
				14		
15	20	13.7	116.1	15		
				16		
				17		
				18	SW	Gravelly Sand, dense, coarse grained sand
				19		
20	45	9.8	127.9	20		
				21		
				22		
				23	SC	Clayey Sand, orange-brown, dense, fine grained
				24		
25	35	13.6	114.7	25		
				26		
				27		
				28		
				29	CL	Silty Clay, gray and reddish brown, hard
30	20	24.0	103.0	30		

BORING LOG NUMBER 4 (continued)

Drilling Date:
Project: File No. 16315-S

Elevation:
Village Center

Sample Depth Ft	Blows per ft	Moisture content %	Dry Unit Weight p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions
				31		
				32		dark reddish brown, some slate gravel
				33		
				34		
35	25	18.7	91.5	35		
				36		
				37		
				38		
				39	SC	Clayey Sand, brown, dense, fine grained
40	25	17.4	113.5	40		
				41		
				42		
				43	CL	Sandy Clay, brown, firm
				44		
45	30	22.2	99.1	45		
				46		
				47	SC	Clayey Sand, brown, dense, fine grained
				48		
				49		
50	75	8.4	134.0	50		gravelly, medium grained
				51		
				52		
				53	CL	Sandy Clay, orange-brown, stiff
				54		
55	45	21.0	113.8	55		
				56		
				57		
				58		
				59		
60	65	20.0	119.5	60	SC	Clayey Sand, orange-brown, dense, fine to medium grained, some slate gravel

BORING LOG NUMBER 4 (continued)

Drilling Date:
Project: File No. 16315-S

Elevation:
Village Center

Sample Depth Ft	Blows per ft	Moisture content %	Dry Unit Weight p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions
				61		increasing slate gravel
				62		
				63		
				64	GP	Sandy Gravel, dark gray, dense, medium to coarse grained sand, angular gravel clasts
65	35	14.3	126.6	65		
				66		
				67		
				68		
				69	CL	Sandy Clay, reddish brown, stiff
70	35	15.4	117.5	70		
				71		
				72		
				73		
				74		silty clay, orange-brown, stiff
75	35	23.2	105.8	75		
				76		
				77		
				78		
				79		
80	35	20.2	110.8	80		
				81		Total Depth: 80 feet Depth to groundwater not measured
				82		
				83		
				84		
				85		
				86		
				87		
				88		
				89		
				90		

BORING LOG NUMBER 5

Drilling Date: 1/10/00
Project: File No. 17526-S

Elevation: 336.5'
Casden Company

Sample Depth Ft	Blows per ft	Moisture content %	Dry Unit Weight p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: 1½-inch asphalt, poor condition, 1-inch sand and gravel base
				0		FILL: Clayey Sand, brown, moist, dense, fine grained
				1		
2	65	10.0	129.3	2		very dense, fine to coarse grained, abundant gravel
				3		
4	74	8.2	122.8	4		Silty Sand, brown to reddish brown, slightly moist, very dense, fine to coarse grained, gravel
				5		
				6		
7	50 50/4"	10.0	129.0	7	SM	Silty Sand, brown, slightly moist, very dense, fine grained, trace coarse grained
				8		
				9		
10	44 50/5"	9.5	118.6	10		
				11		
				12		
				13		
				14		
				15		
				16		
				17		
				18		
				19		
20	81	17.8	112.3	20	ML	Clayey Silt, orange-brown, moist, stiff
				21		
				22		
				23		
				24		
				25		
				26		
				27		
				28		
				29		
30	67	16.0	119.7	30	SC	Clayey Sand, orange-brown, moist, dense, fine to medium grained, some gravel

BORING LOG NUMBER 5 (continued)

Project: File No. 17526-S **Casden Company**

Sample Depth Ft	Blows per ft	Moisture content %	Dry Unit Weight p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions
				31		
				32		
				33		
				34		
				35		
				36		
				37		
				38		
				39		
40	68	14.4	105.7	40	SM	Silty Sand, orange-brown to tan, moist, very dense, fine grained
				41		
				42		
				43		
				44		
				45		
				46		
				47		
				48		
				49		
50	40 50/3"	21.2	111.9	50	SC	Clayey Sand, orange-brown, moist, very dense, fine grained
				51		
				52		
				53		
				54		
55	25 50/3"	4.4	115.6	55	SW	Sand, tan, moist, very dense, fine to coarse grained, abundant gravel
				56		
				57		
				58		
				59		
60	66	20.7	109.1	60	CL	Sandy Clay, orange to red-brown with gray, moist, stiff

BORING LOG NUMBER 5 (continued)

Project: File No. 17526-S **Casden Company**

Sample Depth Ft	Blows per ft	Moisture content %	Dry Unit Weight p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions
				61		
				62		
				63		
				64		water
65	34 50/3"	10.4	128.0	65	SM	Silty Sand, orange-brown, wet, very dense, fine to coarse grained, some gravel
				66		
				67		
				68		
				69		
70	50/6"	20.1	113.0	70		tan
				71		
72.5	28 50/2"	16.6	SPT	72	SC	Clayey Sand, orange-brown, moist, very dense, fine grained
				73		
				74		
75	30 50/3"	14.6	121.7	75	SM	Silty Sand, tan, wet, very dense, fine to coarse grained
				76		
				77		
				78		
				79		
80	16 50/5"	25.4	SPT	80	CL	Sandy Clay, brown, moist, stiff
				81		
				82		
				83		
				84		
85	15 50/5"	19.8	109.2	85	SC	Clayey Sand, gray, very moist, very dense, fine grained, trace medium and coarse grained
				86		
				87		
				88		
				89		
90	73	20.4	SPT	90		orange-brown lenses

Jerry Kovacs and Associates

BORING LOG NUMBER 6 (continued)

Project: File No. 17526-S **Casden Company**

Sample Depth Ft	Blows per ft	Moisture content %	Dry Unit Weight p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions
				94		
				92		
				93		
				94		
95	87	25.1	102.9	95		
				96		grades more clayey, gray
				97		
				98		
				99		
100	70	16.7	SPT	100		dark gray
				101		Total depth: 100 feet Water at 64 feet Fill to 7 feet
				102		
				103		
				104		
				105		
				106		
				107		
				108		
				109		
				110		
				111		
				112		
				113		
				114		
				115		
				116		
				117		
				118		
				119		
				120		

BORING LOG NUMBER 6

Drilling Date: 1/10/00
Project: File No. 17526-S

Elevation: 335.0'
Casden Company

Sample Depth Ft	Blows per ft	Moisture content %	Dry Unit Weight p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: 2-inch asphalt, poor condition, no base
				0		FILL: Clayey Sand, brown, moist to very moist, dense, fine grained
				1		
2	34	15.2	113.2	2		
					SC	Clayey Sand, orange-brown, moist, dense, fine grained
				3		
				4		
5	25	12.7	112.8	5		
					SM	Silty Sand, orange-brown, slightly moist, dense, fine grained
				6		
				7		
				8		
				9		
10	41	16.3	111.1	10		
						grades siltier, very moist
				11		
				12		
				13		
				14		
				15		
				16		
				17		
				18		
				19		
20	35	21.1	107.4	20		
					SC	Clayey Sand, orange-brown, slightly moist to moist, dense, fine grained
				21		
				22		
				23		
				24		
				25		
				26		
				27		
				28		
				29		
30	76	7.0	126.8	30		
					SM	Silty Sand, brown, moist, very dense, fine to coarse grained, some gravel

BORING LOG NUMBER 6 (continued)

Project: File No. 17526-S **Casden Company**

Sample Depth Ft	Blows per ft	Moisture content %	Dry Unit Weight p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions
				31		
				32		
				33		
				34		
				35		
				36		
				37		
				38		
				39		
40	75	16.5	114.6	40		
				41	SC	Clayey Sand, brown, slightly moist, very dense, fine grained
				42		
				43		
				44		
				45		
				46		
				47		
				48		
				49		
50	50 50/5"	12.7	125.3	50		
				51	SM	Silty Sand, grayish brown, slightly moist to moist, very dense, fine to coarse grained
52	66	29.6	96.3	52	CL	Sandy Clay, mottled orange-brown and gray, moist, stiff
				53		sandy lens
				54		
55	33 50/4"	28.5	101.7	55	ML	Sandy Silt, tan, moist, stiff
				56	SM	Silty Sand, orange to tan, moist, very dense, fine to coarse grained
				57		
				58		mostly fine grained, some medium and coarse grained
				59		
60	55	27.1	100.6	60		
				-	CL	Sandy Clay, orange-brown, moist, stiff

BORING LOG NUMBER 6 (continued)

Project: File No. 17526-S **Casden Company**

Sample Depth Ft	Blows per ft	Moisture content %	Dry Unit Weight p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions
				61		
				62		
				63		water
				64		
65	59	29.4	SPT	65		
				66		mottled orange-brown and gray, moist to very moist, stiff
				67		
				68		
				69		
70	67	22.5	105.5	70		
				71	SM/ML	Silty Sand to Sandy Silt, orange-brown, very moist, very dense to stiff, fine grained
				72		
				73		
				74		
75	82	15.8	SPT	75		
				76	SM	Silty Sand, tan to brown, very dense, saturated, mostly fine grained, some coarse grained
				77		
				78		
				79		
80	40 50/5"	20.1	109.5	80		
				81	CL	Sandy Clay, orange-brown, moist, stiff
				82		
				83		
				84		
85	16 50/5"	16.3	SPT	85		
				86	SC	Clayey Sand, orange-brown, moist to very moist, very dense, mostly fine and coarse grained
				87		
				88		
				89		
90	81	17.6	114.5	90		
				-	CL	Sandy Clay, orange-brown, moist, very stiff

BORING LOG NUMBER 6 (continued)

Project: File No. 17526-S **Casden Company**

Sample Depth Ft	Blows per ft	Moisture content %	Dry Unit Weight p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions
				94		
				92		
				93		
				94		
95	63	24.3	SPT	95		
				96	SM	Silty Sand, tan and gray, wet, very dense, fine to coarse grained
				97	CL	Sandy Clay, red and gray, slightly moist, very stiff
				98		
				99		
100	72	no recovery		100		
				101		Total depth: 100 feet Water at 62 feet Fill to 2 feet
				102		
				103		
				104		
				105		
				106		
				107		
				108		
				109		
				110		
				111		
				112		
				113		
				114		
				115		
				116		
				117		
				118		
				119		
				120		

JOB L89421.AEB DATE 9/21/89 F.T. BG/FH DR. [illegible] O.E. MT W.P. Ip CHKD

Note : The log of subsurface conditions shown hereon applies only at the specific boring location and at the date indicated. It is not warranted to be representative of subsurface conditions at other locations and times.

BORING 1

DATE DRILLED: September 6 to 8, 1989
EQUIPMENT USED: 18" - Diameter Bucket to 73'
5" - Diameter Rotary Wash to 112'

ELEVATION 340*

ELEVATION	DEPTH (ft.)		MOISTURE (% of dry wt.)	DRY DENSITY (lbs./cu. ft.)	DRIVE ENERGY (ft.-kips/ft.)	SAMPLE LOC.
			10.5	105	8	■
			14.0	105	8	■
335	5					
			8.9	118	7	■
			13.0	121	15	■
330	10					
			10.3	129	15	■
325	15		11.9	126	13	■
320	20		7.5	130	20	■
315	25		6.6	122	18	■
310	30		6.7	124	18	■
305	35		19.5	110	7	■
300	40		18.0	111	7	■

SM 2" Asphaltic Paving
SILTY SAND - fine, light brown

Some Gravel

ML CLAYEY SILT - some Sand, reddish brown

ML SANDY SILT - reddish brown

SM SILTY SAND - fine to medium, about 25% Gravel, light greyish brown

Few Cobbles

* Elevations refer to datum of reference survey; see Plate 1.

ML CLAYEY SILT - reddish brown

CL SILTY CLAY - reddish brown

(LL = 32; PI = 12)

(CONTINUED ON FOLLOWING PLATE)

LOG OF BORING

LeROY CRANDALL AND ASSOCIATES

JOB L89421.AEB DATE 9/21/89 F.T. BG/FH DR. lp O.E. MT W.P. lp CHKD

Note : The log of subsurface conditions shown hereon applies only at the specific boring location and at the date indicated. It is not warranted to be representative of subsurface conditions at other locations and times.

BORING 1 (Continued)

DATE DRILLED: September 6 to 8, 1989
EQUIPMENT USED: 18" - Diameter Bucket to 73'
5" - Diameter Rotary Wash to 112'

ELEVATION	DEPTH (ft.)	"N" VALUE STD.PEN.TEST	MOISTURE (% of dry wt.)	DRY DENSITY (lbs./cu. ft.)	DRIVE ENERGY (ft.-kips/ft.)	SAMPLE LOC.		
							SM	SILTY SAND - fine, few Gravel, light brown
295	45		8.9	110	8	■		
290	50		21.3	107	7	■		Layer of Clayey Silt, brown
							ML	SILT - grey
285	55		14.9	119	11	■		
			11.6	120	13	■	SM	SILTY SAND - fine, light brown
280	60							
			19.6	108	12	■	ML	Brown CLAYEY SILT - reddish brown
			19.3	111	6	■		
275	65							
			20.1	111	6	■		
270	70		19.6	111	10	■		
							CL	SILTY CLAY - brown
			20.6	108	7	■		
265	75							Some Sand
							SP	SAND - fine to medium, some Silt, few Gravel, brown
			15.0	119	62	■		
							CL	SILTY CLAY - brown
260	80							

(CONTINUED ON FOLLOWING PLATE)

LOG OF BORING

LeROY CRANDALL AND ASSOCIATES

JOB L89421.AEB DATE 9/21/89 F.T. BG/FH DR. lp O.E. MT W.P. lp CHKD

Note : The log of subsurface conditions shown hereon applies only at the specific boring location and at the date indicated. It is not warranted to be representative of subsurface conditions at other locations and times.

BORING 1 (Continued)

DATE DRILLED: September 6 to 8, 1989

EQUIPMENT USED: 18" - Diameter Bucket to 73'
5" - Diameter Rotary Wash to 112'

ELEVATION	DEPTH (ft.)	"N" VALUE STD.PEN.TEST	MOISTURE (% of dry wt.)	DRY DENSITY (lbs./cu. ft.)	DRIVE ENERGY (ft.-kips/ft.)	SAMPLE LOC.		
			17.9	113	30	■		
255	85						SM	SILTY SAND - fine, brown
			24.1	102	72	■		
							CL	SILTY CLAY - brown
250	90							
			19.6	111	27	■		
							SM	SILTY SAND - fine, light greyish brown
245	95							
			20.5	109	43	■		
							ML	SANDY SILT - brownish grey
240	100							
			24.9	101	30	■		
							CL	SILTY CLAY - brownish grey
235	105							
			23.8	103	43	■		
230	110							
			21.2	112	33	■		
225	115							

NOTE: <u>BUCKET BORING:</u>

Water not encountered. No caving.

<u>ROTARY WASH BORING:</u>

Drilling mud used in drilling process. Mud removed at completion of drilling. Water level measured at 43' on 9/11/89 and at 48' on 9/14/89.

LOG OF BORING

LeROY CRANDALL AND ASSOCIATES

BORING 2

DATE DRILLED: September 11, 1989
EQUIPMENT USED: 5" - Diameter Rotary Wash

ELEVATION 329

ELEVATION	DEPTH (ft.)		MOISTURE (% of dry wt.)	DRY DENSITY (lbs./cu. ft.)	DRIVE ENERGY (ft.-kips/ft.)	SAMPLE LOC.		
							CL	2" Asphaltic Paving FILL - SILTY CLAY - few Gravel, grey and brown
			16.5	110	3	■		
							ML	CLAYEY SILT - brown
325			18.4	103	2	■		
	5						CL	SILTY CLAY - dark brown
			17.6	114	3	■		
			21.8	97	3	■		
320								
	10						ML	CLAYEY SILT - dark brown
			24.8	98	4	■		
315								Some Sand
	15		14.4	116	5	■		
310								
	20		15.3	120	7	■		
305							SM	SILTY SAND - fine, brown
	25		19.1	108	15	■		Some Gravel
								Layer of Sandy Silt
300								
	30		14.0	118	20	■		
							ML	SANDY SILT - brown
295								
	35		17.0	114	12	■		
							ML	CLAYEY SILT - light brown
290								
	40		17.4	110	17	■		

(CONTINUED ON FOLLOWING PLATE)

Note : The log of subsurface conditions shown hereon applies only at the specific boring location and at the date indicated. It is not warranted to be representative of subsurface conditions at other locations and times.

JOB L89421.AEB · DATE 9/21/89 F.T. ___ FH ___ DR. lp O.E. MT W.P. lp CHKD ___

LOG OF BORING

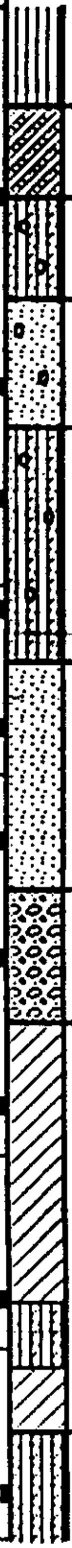
LeROY CRANDALL AND ASSOCIATES

JOB L89421.AEB DATE 9/21/89 F.T. FH DR. lp O.E. MT W.P. lp CHKD

Note : The log of subsurface conditions shown hereon applies only at the specific boring location and at the date indicated. It is not warranted to be representative of subsurface conditions at other locations and times.

BORING 2 (Continued)

DATE DRILLED: September 11, 1989
EQUIPMENT USED: 5" - Diameter Rotary Wash



ELEVATION	DEPTH (ft.)	"N" VALUE STD.PEN.TEST	MOISTURE (% of dry wt.)	DRY DENSITY (lbs./cu. ft.)	DRIVE ENERGY (ft.-kips/ft.)	SAMPLE LOC.		
							SC	CLAYEY SAND - fine to medium, reddish brown
285								
	45		14.3	120	43	■		
							SM	SILTY SAND - fine to medium, few Gravel, reddish brown
							SP	SAND - fine to medium, some Gravel, brown
280								
	50		5.7	128	62	■		
							SM	SILTY SAND - fine to medium, few Gravel, brown
			7.1	121	72	■		
275								
								Dark greyish brown
	55		7.0	124	72	■		
							SP	SAND - fine to medium, dark greyish brown
270			15.6	119	43	■		
	60							
			10.6	116	43	■		
							GW	GRAVEL - well graded, some Sand, grey and brown
265	65		12.5	120	48	■		
							CL	SILTY CLAY - reddish brown
260			17.8	113	27	■		
	70							
255			24.7	104	43	■		
								Layer of Silty Sand
	75							
							SM	SILTY SAND - fine, light brown
250			21.5	100	24	■		
	80							

(CONTINUED ON FOLLOWING PLATE)

LOG OF BORING

LeROY CRANDALL AND ASSOCIATES

JOB L89421.AEB . DATE 9/21/89 F.T. BG/FH DR. lp O.E. MT W.P. lp CHKD

Note : The log of subsurface conditions shown hereon applies only at the specific boring location and at the date indicated.
It is not warranted to be representative of subsurface conditions at other locations and times.

BORING 2 (Continued)

DATE DRILLED: September 11, 1989
EQUIPMENT USED: 5" - Diameter Rotary Wash

ELEVATION	DEPTH (ft.)	"N" VALUE STD.PEN.TEST	MOISTURE (% of dry wt.)	DRY DENSITY (lbs./cu. ft.)	DRIVE ENERGY (ft.-kips/ft.)	SAMPLE LOC.
245			15.8	111	33	■
	85					
240			22.5	106	27	■
	90					
235			32.0	90	14	■
	95					
230			37.6	88	14	■
	100					

CL	SILTY CLAY - brown
SM	SILTY SAND - fine, grey
ML	SANDY SILT - greyish brown Layer of Silty Clay Some Clay
SM	SILTY SAND - fine, grey
ML	CLAYEY SILT - greyish brown
CH	CLAY - brownish grey

NOTE: Drilling mud used in drilling process. Mud removed at completion of drilling. Water level measured at 35' on 9-14-89.

LOG OF BORING

LeROY CRANDALL AND ASSOCIATES

PLATE A - 1.2c

BORING 3

DATE DRILLED: September 6 to 8, 1989
EQUIPMENT USED: 18" - Diameter Bucket to 75'
5" - Diameter Rotary Wash to 111'

ELEVATION 334

ELEVATION	DEPTH (ft.)		MOISTURE (% of dry wt.)	DRY DENSITY (lbs./cu. ft.)	DRIVE ENERGY (ft.-kips/ft.)	SAMPLE LOC.		
							ML	2" Asphaltic Paving CLAYEY SILT - brown
			20.7	99	5	■		
							SM	SILTY SAND - fine, reddish brown
330			12.8	119	7	■		
	5						ML	CLAYEY SILT - some Sand, few Gravel, light brown
			12.4	126	10	■		
			12.8	122	10	■		
325								
	10						ML	SANDY SILT - some Clay, light brown
			17.2	112	5	■		
320								
	15		14.6	117	7	■		Few rootlets, dark reddish brown
							SM	SILTY SAND - fine to medium, some Gravel, occassional Cobbles, grey and brown
315								
	20		3.8	128	11	■		
								About 15% Gravel, few Cobbles
310								
	25		17.1	114	33	■		
							ML	SANDY SILT - grey and brown
							CL	SILTY CLAY - grey (LL = 35; PI = 16)
305								
	30		19.1	108	8	■		
							ML	SANDY SILT - some Clay, brown
300								
	35		16.7	110	8	■		
							SP	SAND - fine to medium, few Gravel, dark grey and brown
295								
	40		5.3	124	20	■		

(CONTINUED ON FOLLOWING PLATE)

LOG OF BORING

LeROY CRANDALL AND ASSOCIATES

JOB L89421.AEB DATE 9/21/89 F.T. BG/FH DR. lp O.E. MT W.P. lp CHKD

Note.: The log of subsurface conditions shown hereon applies only at the specific boring location and at the date indicated. It is not warranted to be representative of subsurface conditions at other locations and times.

JOB L89421.AEB DATE 9/21/89 F.T. BG/FH DR. lp O.E. MT W.P. lp CHKD

Note : The log of subsurface conditions shown hereon applies only at the specific boring location and at the date indicated. It is not warranted to be representative of subsurface conditions at other locations and times.

BORING 3 (Continued)

DATE DRILLED: September 6 to 8, 1989
EQUIPMENT USED: 18" - Diameter Bucket to 75'
5" - Diameter Rotary Wash to 111'

ELEVATION	DEPTH (ft.)	"N" VALUE STD.PEN.TEST	MOISTURE (% of dry wt.)	DRY DENSITY (lbs./cu. ft.)	DRIVE ENERGY (ft.-kips/ft.)	SAMPLE LOC.		
							SM	SILTY SAND - fine, brown
290								
	45		15.0	113	13	■		
285							CL	SILTY CLAY - reddish brown
	50		23.6	101	4	■		
							SM	SILTY SAND - fine, brown
280								
	55		14.4	109	10	■		
			3.2	118	14	■	SP	SAND - fine to medium, few Gravel, light brown
275								
	60							
			18.9	111	13	■	SM	SILTY SAND - fine, brown
							ML	SANDY SILT - some Clay, reddish brown
270			21.4	108	8	■		
	65							
							SC	CLAYEY SAND - fine to medium, reddish brown
			13.4	121	12	■		
							SM	SILTY SAND fine, few Gravel, brown
265								
	70		16.7	116	14	■		
							ML	SANDY SILT - some Clay, few Gravel, brown
			16.7	113	11	■		
260							SM	SILTY SAND - fine to medium, brown
	75		14.2	112	11	■		
255								
	80							

(CONTINUED ON FOLLOWING PLATE)

LOG OF BORING

LeROY CRANDALL AND ASSOCIATES

JOB L89421.AEB DATE 9/21/89 F.T. BG/FH DR. lp O.E. MT W.P. lp CHKD

Note : The log of subsurface conditions shown hereon applies only at the specific boring location and at the date indicated. It is not warranted to be representative of subsurface conditions at other locations and times.

BORING 3 (Continued)

DATE DRILLED: September 6 to 8, 1989
EQUIPMENT USED: 18" - Diameter Bucket to 75'
5" - Diameter Rotary Wash to 111'

ELEVATION	DEPTH (ft.)	"N" VALUE STD.PEN.TEST	MOISTURE (% of dry wt.)	DRY DENSITY (lbs./cu. ft.)	DRIVE ENERGY (ft.-kips/ft.)	SAMPLE LOC.
			18.0	113	33	▮
250						
	85					
			25.4	101	27	▮
245						
	90					
			19.8	107	21	▮
240						
	95					
			16.6	114	33	▮
235						
	100					
			18.8	106	13	▮
230						
	105					
			21.0	107	15	▮
225						
	110					
			17.6	106	72	▮
220						
	115					

ML CLAYEY SILT - greyish brown

ML SANDY SILT - light grey

CL SILTY CLAY - brownish grey

SP SAND - fine, reddish brown

NOTE: <u>BUCKET BORING:</u>

Slight water seepage at 69'. About 1" of water at bottom of boring 20 minutes after completion of drilling. No caving.

ROTARY WASH:

Drilling mud used in drilling process. Mud removed at completion of drilling. Water level measured at 35' on 9-11-89 and at 38' on 9-14-89.

LOG OF BORING

LeROY CRANDALL AND ASSOCIATES

PLATE A - 1.3c

JOB L89421.AEB DATE 9/21/89 F.T. FH DR. lp O.E. MT W.P. lp CHKD

Note : The log of subsurface conditions shown hereon applies only at the specific boring location and at the date indicated. It is not warranted to be representative of subsurface conditions at other locations and times.

BORING 4

DATE DRILLED: September 6, 1989
EQUIPMENT USED: 5" - Diameter Rotary Wash

ELEVATION 339

ELEVATION	DEPTH (ft.)		MOISTURE (% of dry wt.)	DRY DENSITY (lbs./cu. ft.)	DRIVE ENERGY (ft.-kips/ft.)	SAMPLE LOC.		
							SM	2" - Asphaltic Paving SILTY SAND - fine, brown
			12.9	111	4	▮		
335			10.8	118	3	▮		
	5							
			13.2	114	3	▮		
							ML	SANDY SILT - brown
330			19.7	105	3	▮		
	10						SM	SILTY SAND - fine, brown
			12.3	107	8	▮		
325								
	15		7.0	121	27	▮		Fine to medium, some Gravel, greyish brown
							ML	SANDY SILT - brown
320								
	20		24.4	96	4	▮		
							SM	SILTY SAND - fine to medium, some Gravel, brown
315								
	25		10.1	127	16	▮		
							ML	SANDY SILT - brown
310								
	30		15.4	118	21	▮		
			20.2	107	15	▮		Some Clay
305								
	35						GW	GRAVEL - fine to coarse, some Sand, greyish brown
			6.7	128	72	▮		
							CL	SILTY CLAY - brownish grey
300								
	40		18.7	111	18	▮		

(CONTINUED ON FOLLOWING PLATE)

LOG OF BORING

LeROY CRANDALL AND ASSOCIATES

BORING 4 (Continued)

DATE DRILLED: September 6 , 1989
EQUIPMENT USED: 5" - Diameter Rotary Wash

ELEVATION	DEPTH (ft.)	"N" VALUE STD.PEN.TEST	MOISTURE (% of dry wt.)	DRY DENSITY (lbs./cu. ft.)	DRIVE ENERGY (ft.-kips/ft.)	SAMPLE LOC.
			19.8	107	21	■
295						
	45					
			20.7	106	11	■
290			24.7	101	18	■
	50					
			18.4	110	12	■
285			27.6	94	33	■
	55					
			15.5	108	18	■
280						
	60					
			22.4	103	18	■
275			22.8	104	11	■
	65					
			23.8	102	13	■
270			20.4	109	24	■
	70					
			13.5	118	62	■
265						
	75					
			18.4	113	16	■
260						
	80					

Reddish brown

Layer of Sand

ML SANDY SILT - greyish brown

Some Clay, brown

CL SILTY CLAY - reddish brown

Layer of Silty Sand

SP SAND - fine, few Gravel, brown

CL SILTY CLAY - reddish brown

(CONTINUED ON FOLLOWING PLATE)

LOG OF BORING

LeROY CRANDALL AND ASSOCIATES

PLATE A - 1.4b

JOB L89421.AEB DATE 9/21/89 F.T. FH DR. lp O.E. MT. W.P. lp CHKD

Note : The log of subsurface conditions shown hereon applies only at the specific boring location and at the date indicated. It is not warranted to be representative of subsurface conditions at other locations and times.

BORING 4 (Continued)

DATE DRILLED: September 6, 1989
EQUIPMENT USED: 5" - Diameter Rotary Wash

ELEVATION	DEPTH (ft.)	"N" VALUE STD.PEN.TEST	MOISTURE (% of dry wt.)	DRY DENSITY (lbs./cu. ft.)	DRIVE ENERGY (ft.-kips/ft.)	SAMPLE LOC.
			16.6	115	21	■
255	85		13.1	122	33	■
250	90		18.5	112	18	■
245	95		26.6	98	20	■
240	100		20.3	107	21	■
235	105					

SM SILTY SAND - fine, brown

CL SILTY CLAY - few Gravel, brown

NOTE: Drilling mud used in drilling process. Mud removed at completion of drilling. Water level measured at 57' on 9-11-89 and on 9-14-89.

Note : The log of subsurface conditions shown hereon applies only at the specific boring location and at the date indicated. It is not warranted to be representative of subsurface conditions at other locations and times.

JOB L89421.AEB DATE 9/21/89 F.T. FH DR. lp O.E. MT W.P. lp CHKD

LOG OF BORING

LeROY CRANDALL AND ASSOCIATES

JOB L89421.AEB DATE 9/21/89 F.T. BG/FH DR. lp O.E. MT W.P. lp CHKD

Note : The log of subsurface conditions shown hereon applies only at the specific boring location and at the date indicated. It is not warranted to be representative of subsurface conditions at other locations and times.

BORING 5

DATE DRILLED: September 7 & 8, 1989
EQUIPMENT USED: 18" - Diameter Bucket to 60'
5" - Diameter Rotary Wash to 100'
ELEVATION 328

ELEVATION	DEPTH (ft.)		MOISTURE (% of dry wt.)	DRY DENSITY (lbs./cu. ft.)	DRIVE ENERGY (ft.-kips/ft.)	SAMPLE LOC.		
							SM	2" - Asphaltic Paving SILTY SAND - fine, reddish brown
			16.3	110	3	■		
325							ML	SANDY SILT - reddish brown
			12.0	92	7	■		
	5						SM	SILTY SAND - fine to medium, some Gravel, brown
			11.2	111	7	■		
320			4.9	121	8	■		
	10						ML	SANDY SILT - brown
			15.9	113	5	■		
315								
	15		12.5	117	7	■		
310							SM	SILTY SAND - fine, light brown
	20		18.2	106	5	■		
305							GW	GRAVEL - well graded, some Sand, brown
	25		7.9	105	13	■		
							ML	CLAYEY SILT - reddish brown
300								
	30		18.7	112	10	■		
295							SM	SILTY SAND - fine, light brown
	35		21.9	104	6	■		
290								About 20% Gravel and few Cobbles
	40		5.2	123	14	■		

(CONTINUED ON FOLLOWING PLATE)

LOG OF BORING

LeROY CRANDALL AND ASSOCIATES

PLATE A - 1.5a

JOB L89421.AEB DATE 9/21/89 F.T. BG/FH DR. ip O.E. MT W.P. lp CHKD

Note : The log of subsurface conditions shown hereon applies only at the specific boring location and at the date indicated. It is not warranted to be representative of subsurface conditions at other locations and times.

BORING 5 (Continued)

DATE DRILLED: September 7 & 8, 1989
EQUIPMENT USED: 18" - Diameter Bucket to 60'
5" - Diameter Rotary Wash to 100'

ELEVATION	DEPTH (ft.)	"N" VALUE STD.PEN.TEST	MOISTURE (% of dry wt.)	DRY DENSITY (lbs./cu. ft.)	DRIVE ENERGY (ft.-kips/ft.)	SAMPLE LOC.
285						
	45		7.4	103	13	
280						
	50		20.6	108	4	
275						
	55		17.6	112	14	
270			13.8	104	10	
	60					
			20.1	105	5	
265						
			23.9	101	18	
	65					
			16.9	115	18	
260						
	70					
			20.1	110	16	
255						
	75					
			23.7	104	33	
250						
	80					

CL	SILTY CLAY - reddish brown
SM	SILTY SAND - fine, reddish brown
SP	SAND - fine to medium, light brown
ML	SANDY SILT - brown
CL	SILTY CLAY - brown
SM	SILTY SAND - fine, brown
	Layers of Silty Clay

(CONTINUED ON FOLLOWING PLATE)

LOG OF BORING

LeROY CRANDALL AND ASSOCIATES

PLATE A - 1.5b

BORING 5 (Continued)

DATE DRILLED: September 7 & 8, 1989
EQUIPMENT USED: 18" - Diameter Bucket to 60'
5" - Diameter Rotary Wash to 100'

ELEVATION	DEPTH (ft.)	"N" VALUE STD.PEN.TEST	MOISTURE (% of dry wt.)	DRY DENSITY (lbs./cu. ft.)	DRIVE ENERGY (ft.-kips/ft.)	SAMPLE LOC.		
			20.3	109	48	■		
245							CL	SILTY CLAY - light brown
	85		24.6	103	21	■		S. Some Sand, brown
240								
	90		29.4	97	23	■		
235								
	95		20.9	105	23	■	SM	SILTY SAND - lenses of Silty Clay, brownish grey
230							CL	SILTY CLAY - grey
	100		30.1	94	45	■		
225								
	105							

NOTE: BUCKET BORING:

Water seepage encountered at 58-1/2'. Water level measured at 58-1/2' 20 minutes after completion of drilling. No caving.

ROTARY WASH BORING:

Drilling mud used in drilling process. Mud removed at completion of drilling. Water level measured at 44' on 9-14-89.

JOB L89421.AEB DATE 9/21/89 F.T. BG/FH DR. lp O.E. MT W.P. lp CHKD

Note : The log of subsurface conditions shown hereon applies only at the specific boring location and at the date indicated. It is not warranted to be representative of subsurface conditions at other locations and times.

LOG OF BORING

LeROY CRANDALL AND ASSOCIATES

JOB L89421.AEB DATE 9/21/89 F.T. FH DR. lp O.E. MT W.P. lp CHKD

Note : The log of subsurface conditions shown hereon applies only at the specific boring location and at the date indicated. It is not warranted to be representative of subsurface conditions at other locations and times.

BORING 6

DATE DRILLED: September 5, 1989
EQUIPMENT USED: 5" - Diameter Rotary Wash'

ELEVATION 332

ELEVATION	DEPTH (ft.)		MOISTURE (% of dry wt.)	DRY DENSITY (lbs./cu. ft.)	DRIVE ENERGY (ft.-kips/ft.)	SAMPLE LOC.		
							ML	3" - Asphaltic Paving - 3" Base Course SANDY SILT - brown
330			15.5	116	8	■		
			15.7	111	5	■		
							SM	SILTY SAND - fine to medium, brown
	5							
			7.7	114	10	■		
							SP SM	SAND - fine, some Silt, few Gravel, greyish brown
325								
			4.4	113	16	■		
							CL	SILTY CLAY - brown
	10							
			13.7	121	10	■		
320								
							ML	CLAYEY SILT - few Gravel, brown
			13.1	119	7	■		
	15							
315								
							CL	SILTY CLAY - brown
			17.8	108	10	■		(LL = 30; PI = 8)
	20							
							ML	SANDY SILT - some Clay, brown
310								
			13.5	117	11	■		
	25							
							GW	GRAVEL - well graded, some Sand, grey and brown
305			6.8	130	24	■		
							SM	SILTY SAND - fine to medium, grey and brown
	30							
			7.1	118	27	■		
300								
							CL	SILTY CLAY - brownish grey
			19.9	110	10	■		
	35							
							ML	CLAYEY SILT - brown
295								
			24.1	101	8	■		
	40							

(CONTINUED ON FOLLOWING PLATE)

LOG OF BORING

LeROY CRANDALL AND ASSOCIATES

BORING 6 (Continued)

DATE DRILLED: September 5, 1989
EQUIPMENT USED: 5" - Diameter Rotary Wash

ELEVATION	DEPTH (ft.)	"N" VALUE STD.PEN.TEST	MOISTURE (% of dry wt.)	DRY DENSITY (lbs./cu. ft.)	DRIVE ENERGY (ft.-kips/ft.)	SAMPLE LOC.		
			20.2	105	8	■	ML	SANDY SILT - brown
							CL	SILTY CLAY - brown
290								
	45		22.3	104	10	■		
							ML	CLAYEY SILT - brown
			20.0	109	9	■		
285								
								Layer of Silty Clay
	50							
			17.1	112	11	■		
							SM	SILTY SAND - fine, greyish brown
280			11.1	105	16	■		Layers of Sandy Silt
	55							
			12.5	112	11	■		
							CL	SILTY CLAY - brown
275								
	60		26.7	99	11	■		
			20.8	107	11	■		
270								
	65						SM	SILTY SAND - fine, brown
			13.2	121	30	■		
							ML	CLAYEY SILT - few Gravel, brown
265								
	70		13.2	120	36	■		
							SP	SAND - fine to medium, some Gravel, grey
							SM	SILTY SAND - fine, brown
260								
	75		20.0	110	43	■		
							CL	SILTY CLAY - dark brown
255								
	80		20.0	109	15	■		

(CONTINUED ON FOLLOWING PLATE)

Note : The log of subsurface conditions shown hereon applies only at the specific boring location and at the date indicated. It is not warranted to be representative of subsurface conditions at other locations and times.

JOB L89421.AEB DATE 9/21/89 F.T. BG/FH DR. lp O.E. MT W.P. lp CHKD

LOG OF BORING

LeROY CRANDALL AND ASSOCIATES

BORING 6 (Continued)

DATE DRILLED: September 5, 1989
EQUIPMENT USED: 5" - Diameter Rotary Wash

ELEVATION	DEPTH (ft.)	"N" VALUE STD.PEN.TEST	MOISTURE (% of dry wt.)	DRY DENSITY (lbs./cu. ft.)	DRIVE ENERGY (ft.-kips/ft.)	SAMPLE LOC.
250						
	85		19.1	110	18	■
245						
	90		12.5	123	33	■
240						
	95		21.9	107	27	■
235						
	100		17.6	114	21	■

SM	SILTY SAND - fine, brown
CL	SANDY CLAY - reddish brown
SC	CLAYEY SAND - fine to medium, some Gravel, greyish brown
SM	SILTY SAND - fine to medium, grey and brown
CL	SILTY CLAY - brown
SM	SILTY SAND - fine, brown
CL	SANDY CLAY - greyish brown

NOTE: Drilling mud used in drilling process. Mud removed at completion of drilling. Water level measured at 47' on 9-11-89 and on 9-14-89.

JOB L89421.AEB DATE 9/21/89 F.T. BG/FH DR. lp O.E. MT W.P. lp CHKD

Note : The log of subsurface conditions shown hereon applies only at the specific boring location and at the date indicated. It is not warranted to be representative of subsurface conditions at other locations and times.

LOG OF BORING

LeROY CRANDALL AND ASSOCIATES

PLATE A - 1.6c.

BORING 7

DATE DRILLED: September 8, 1989
EQUIPMENT USED: 18" - Diameter Bucket

ELEVATION 332

ELEVATION	DEPTH (ft.)		MOISTURE (% of dry wt.)	DRY DENSITY (lbs./cu. ft.)	DRIVE ENERGY (ft.-kips/ft.)	SAMPLE LOC.		
							SM	2" Asphaltic Paving FILL - SILTY SAND - some Clay, brown
330								
							CL	SILTY CLAY - brown
			19.2	107	< 1	■		
							ML	SANDY SILT - light brown
			19.0	107	3	■		
	5							
							SM	SILTY SAND - fine, light brown
			18.1	106	3	■		
325								
			16.4	114	7	■		
	10							
							ML	SANDY SILT - light brown
			21.8	102	2	■		
320								
								Few Gravel
							ML	CLAYEY SILT - some Sand, reddish brown
	15							
			12.8	123	7	■		
315								
							SM	SILTY SAND - fine to medium, few Gravel, reddish brown and grey
			15.5	113	5	■		
	20							
							ML	SANDY SILT - brown
310								
			15.5	111	3	■		
	25							
							SM	SILTY SAND - fine, grey and brown
305								
			13.2	119	7	■		
	30							
			14.0	102	7	■		
300								
			10.9	106	11	■		
	35							
							ML	CLAYEY SILT - reddish brown and grey
			19.8	109	4	■		
295								
			20.9	108	6	■		
	40							

(CONTINUED ON FOLLOWING PLATE)

LOG OF BORING

LeROY CRANDALL AND ASSOCIATES

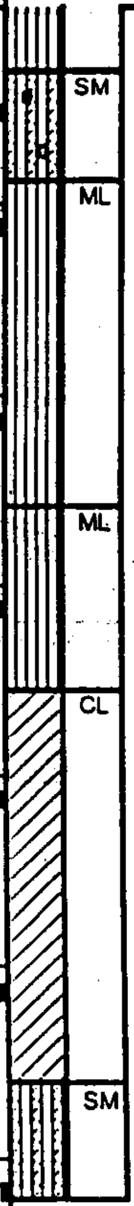
JOB L89421.AEB DATE 9/21/89 F.T. BG/FH DR. lp O.E. MT W.P. lp CHKD ______

Note : The log of subsurface conditions shown hereon applies only at the specific boring location and at the date indicated.
It is not warranted to be representative of subsurface conditions at other locations and times.

BORING 7 (Continued)

DATE DRILLED: September 8, 1989
EQUIPMENT USED: 18" - Diameter Bucket

ELEVATION	DEPTH (ft.)	"N" VALUE STD.PEN.TEST	MOISTURE (% of dry wt.)	DRY DENSITY (lbs./cu. ft.)	DRIVE ENERGY (ft.-kips/ft.)	SAMPLE LOC.
290						
			15.9	111	4	■
	45					
			20.3	108	5	■
285						
	50					
			18.4	108	5	■
280						
	55					
			20.5	105	6	■
275						
	60					
			23.4	103	5	■
270						
	65					
			22.9	105	5	■
265						
	70					
			15.2	120	6	■
260						
	75					



SILTY SAND - fine, few Gravel, reddish brown

CLAYEY SILT - grey and brown

SANDY SILT - grey

SILTY CLAY - brown

SILTY SAND - fine, brown

NOTE: Water seepage encountered at 69-1/2' 20 minutes after completion of drilling. Water level measured at 69-1/2'. No caving.

JOB L89421.AEB DATE 9/21/89 F.T. BG/FH DR. lp O.E. MT W.P. lp CHKD

Note : The log of subsurface conditions shown hereon applies only at the specific boring location and at the date indicated.
It is not warranted to be representative of subsurface conditions at other locations and times.

LOG OF BORING

LeROY CRANDALL AND ASSOCIATES

JOB L89421.AEB DATE 12/15/88 F.T. ___ TC DR. dmh O.E. BW W.P. dmh CHKD ___

Note : The log of subsurface conditions shown hereon applies only at the specific boring location and at the date indicated. It is not warranted to be representative of subsurface conditions at other locations and times.

BORING 8 *

DATE DRILLED: December 9, 1988
EQUIPMENT USED: 16" - Diameter Bucket

ELEVATION 342

ELEVATION (ft.): 340, 335, 330, 325, 320, 315, 310, 305

DEPTH (ft.): 5, 10, 15, 20, 25, 30, 35, 40

MOISTURE (% of dry wt.)	DRY DENSITY (lbs./cu. ft.)	DRIVE ENERGY (ft.-kips/ft.)
12.3	115	7
13.7	110	5
5.8	103	7
3.0	101	5
12.7	118	7
12.2	116	11
13.8	117	8
9.4	116	8
11.0	124	13
10.6	109	13
12.8	93	12
18.1	103	8

4-1/2" Asphaltic Paving

SANDY SILT - lenses of Sandy Clay, few Gravel, brown

SILTY SAND - fine, some Clay, few Gravel, layers of Sandy Silt, brown

Lenses of Sand, light brown

SILTY CLAY - few Gravel, brown

SANDY SILT - lenses of Sandy Clay, few Gravel, brown

Some Gravel

SILTY SAND - fine, some Gravel, lenses of Sandy Clay, brown

* Borings 8, 9 and 10 were previously drilled for the preliminary geotechnical investigation (LCA AE-88383)

Some Clay, layers of Sandy Silt

Greyish brown

Layer of Clayey Silt

Layers of well graded Sand with about 15% Gravel, brownish grey

(CONTINUED ON FOLLOWING PLATE)

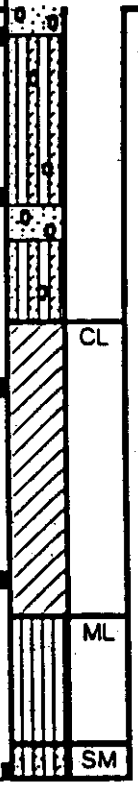
LOG OF BORING

LeROY CRANDALL AND ASSOCIATES

BORING 8 (Continued)

DATE DRILLED: December 9, 1988
EQUIPMENT USED: 16" - Diameter Bucket

ELEVATION (ft.)	DEPTH (ft.)		MOISTURE (% of dry wt.)	DRY DENSITY (lbs./cu. ft.)	DRIVE ENERGY (ft.-kips/ft.)	SAMPLE LOC.
			5.9	98	11	■
300						
	45		4.2	105	11	■
295						
	50		19.7	110	8	■
290						
	55		22.5	103	6	■
285						
	60		17.6	108	8	■



Fine to medium

Fine

SILTY CLAY - brown

SANDY SILT - some Clay, brown

SILTY SAND - fine, greyish brown

NOTE: Water not encountered. No caving.

JOB L89421.AEB DATE 12/15/88 F.T. TC DR. dmh O.E. BW W.P. dmh CHKD

Note : The log of subsurface conditions shown hereon applies only at the specific boring location and at the date indicated. It is not warranted to be representative of subsurface conditions at other locations and times.

LOG OF BORING

LeROY CRANDALL AND ASSOCIATES

JOB L89421.AEB DATE 12/15/88 F.T. TC DR. dmh O.E. BW W.P. dmh CHKD

Note : The log of subsurface conditions shown hereon applies only at the specific boring location and at the date indicated. It is not warranted to be representative of subsurface conditions at other locations and times.

BORING 9

DATE DRILLED: December 12, 1988
EQUIPMENT USED: 16" - Diameter Bucket

ELEVATION 329

ELEVATION (ft.)	DEPTH (ft.)		MOISTURE (% of dry wt.)	DRY DENSITY (lbs./cu. ft.)	DRIVE ENERGY (ft.-kips/ft.)	SAMPLE LOC.		
							ML	6" Asphaltic Paving CLAYEY SILT - lenses of Sandy Silt, brown
			13.3	119	7	■		
325							SM	SILTY SAND - fine, some Clay, few Gravel, brown
	5		11.1	108	10	■		
320			12.9	107	8	■		Lenses of Sand
	10							
			13.4	116	10	■		Lens of Sandy Clay
315							ML	SANDY SILT - some Clay, brown
			10.9	108	5	■		
	15							
							SM	SILTY SAND - fine, some Gravel, brown
			9.7	103	10	■		Layers of well graded Sand with about 15% Gravel, greyish brown
310								
	20							
			6.0	125	15	■		
								About 10% Gravel
305			8.2	121	10	■		Layer of Sandy Silt
	25						CL	SILTY CLAY - some Gravel, brown
			16.5	113	9	■		Lenses of Silty Sand
300								Layers of Sandy Clay
			19.0	103	9	■		
	30							
			23.0	105	9	■		
295								
	35							
			21.9	107	10	■		
290							ML	SANDY SILT - some Clay, brown with grey
	40							

(CONTINUED ON FOLLOWING PLATE)

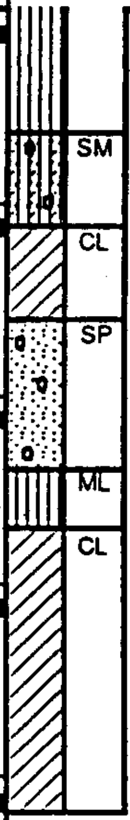
LOG OF BORING

LeROY CRANDALL AND ASSOCIATES

BORING 9 (Continued)

DATE DRILLED: December 12, 1988
EQUIPMENT USED: 16" - Diameter Bucket

ELEVATION (ft.)	DEPTH (ft.)		MOISTURE (% of dry wt.)	DRY DENSITY (lbs./cu. ft.)	DRIVE ENERGY (ft.-kips/ft.)	SAMPLE LOC.
			20.2	107	8	■
285						
	45					
			23.5	103	9	■
280						
	50					
			4.0	110	15	■
275						
	55					
			23.1	105	9	■
270						
	60					
			24.0	104	8	■
265						
	65					



SM SILTY SAND - fine, some Gravel, light greyish brown

CL SILTY CLAY - brown and grey

SP SAND - fine to medium, some Gravel, grey

ML SANDY SILT - brown with grey

CL SILTY CLAY - brown with grey

NOTE: Water seepage encountered at a depth of 59'. Water level measured at 59' 15 minutes after completion of drilling. No caving.

JOB L89421.AEB DATE 12/15/88 F.T. TC DR. dmh O.E. BW W.P. dmh CHKD

Note : The log of subsurface conditions shown hereon applies only at the specific boring location and at the date indicated. It is not warranted to be representative of subsurface conditions at other locations and times.

LOG OF BORING

LeROY CRANDALL AND ASSOCIATES

JOB L89421.AEB · DATE 1/5/89 F.T. TC DR. lp O.E. BW W.P. lp CHKD

Note : The log of subsurface conditions shown hereon applies only at the specific boring location and at the date indicated. It is not warranted to be representative of subsurface conditions at other locations and times.

BORING 10

DATE DRILLED: January 4, 1989
EQUIPMENT USED: 20" - Diameter Bucket

ELEVATION 327

ELEVATION	DEPTH (ft.)		MOISTURE (% of dry wt.)	DRY DENSITY (lbs./cu. ft.)	DRIVE ENERGY (ft.-kips/ft.)	SAMPLE LOC.
325						
			18.4	104	2	■
	5					
			19.1	108	< 1	■
320						
			26.1	96	< 1	■
	10					
315			13.3	122	7	■
	15		18.2	105	5	■
310						
			14.0	120	8	■
	20					
			21.7	102	8	■
305						
			16.8	114	8	■
	25					
300			17.4	114	6	■
	30		13.5	117	7	■
295						
	35		6.9	111	12	■
290						
	40		12.7	114	12	■

	4" Asphaltic Paving
SP ML CL	FILL - SAND, SILT and CLAY - pieces of asphaltic paving, brown with grey
CL	SANDY CLAY - few Gravel, lenses of Silty Sand, brown
CL	SILTY CLAY - dark greyish brown
	Layer of Sandy Silt
	Some Sand, few Gravel
	Greyish brown
	About 10% Gravel
ML	SANDY SILT - some Clay, few Gravel, brown with grey
SM	SILTY SAND - fine, few Gravel, greyish brown
	Some Gravel
	Brownish grey
	Layer of Sand

(CONTINUED ON FOLLOWING PLATE)

LOG OF BORING

LeROY CRANDALL AND ASSOCIATES

BORING 10 (Continued)

DATE DRILLED: January 4, 1989
EQUIPMENT USED: 20" - Diameter Bucket

ELEVATION	DEPTH (ft.)		MOISTURE (% of dry wt.)	DRY DENSITY (lbs./cu. ft.)	DRIVE ENERGY (ft.-kips/ft.)	SAMPLE LOC.
285						
	45		24.7	103	6	■
280						
	50		15.0	117	11	■
275						
	55					
270						
	60		24.4	102	13	■

ML	SANDY SILT - some Gravel, grey
CL	SILTY CLAY - brown
	Some Sand and Gravel, lenses of Sandy Silt
	Brown with grey
SM	SILTY SAND - fine to medium, some Gravel, brown
CL	SILTY CLAY - brown

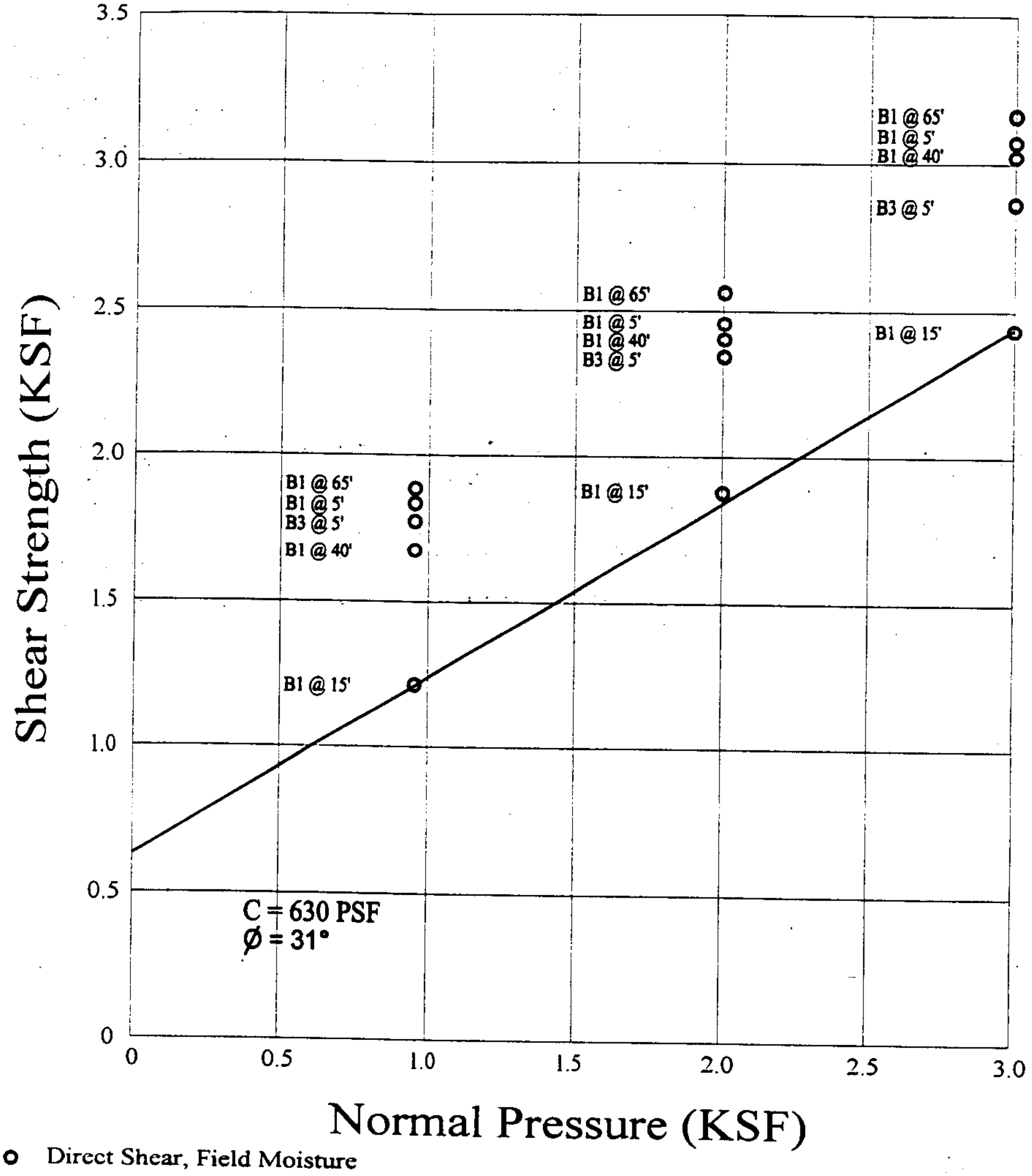
NOTE: Water seepage encountered at a depth of 53'. Water level measured at 53-1/2' at completion of drilling and at 52-1/2' 10 minutes later. No caving.

JOB L89421.AEB DATE 1/5/89 F.T. TC DR. lp O.E. BW W.P. lp CHKD

Note : The log of subsurface conditions shown hereon applies only at the specific boring location and at the date indicated.
It is not warranted to be representative of subsurface conditions at other locations and times.

LOG OF BORING

LeROY CRANDALL AND ASSOCIATES



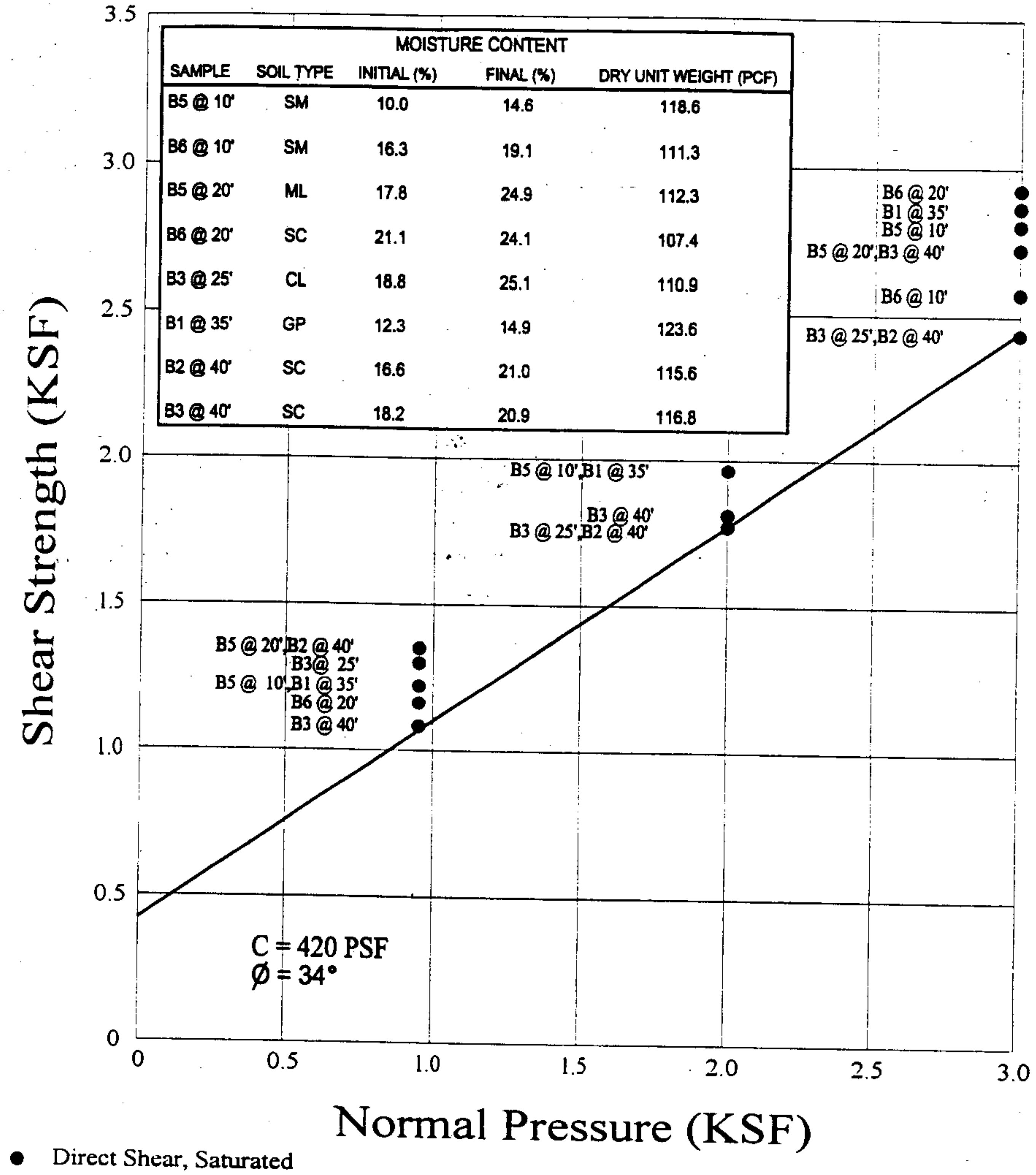
SHEAR TEST DIAGRAM

JERRY KOVACS AND ASSOCIATES, INC.
CONSULTING GEOTECHNICAL ENGINEERS

PROJECT: THE CASDEN COMPANY

FILE NO. 17526-S

PLATE: B-1



		MOISTURE CONTENT		
SAMPLE	SOIL TYPE	INITIAL (%)	FINAL (%)	DRY UNIT WEIGHT (PCF)
B5 @ 10'	SM	10.0	14.6	118.6
B6 @ 10'	SM	16.3	19.1	111.3
B5 @ 20'	ML	17.8	24.9	112.3
B6 @ 20'	SC	21.1	24.1	107.4
B3 @ 25'	CL	18.8	25.1	110.9
B1 @ 35'	GP	12.3	14.9	123.6
B2 @ 40'	SC	16.6	21.0	115.6
B3 @ 40'	SC	18.2	20.9	116.8

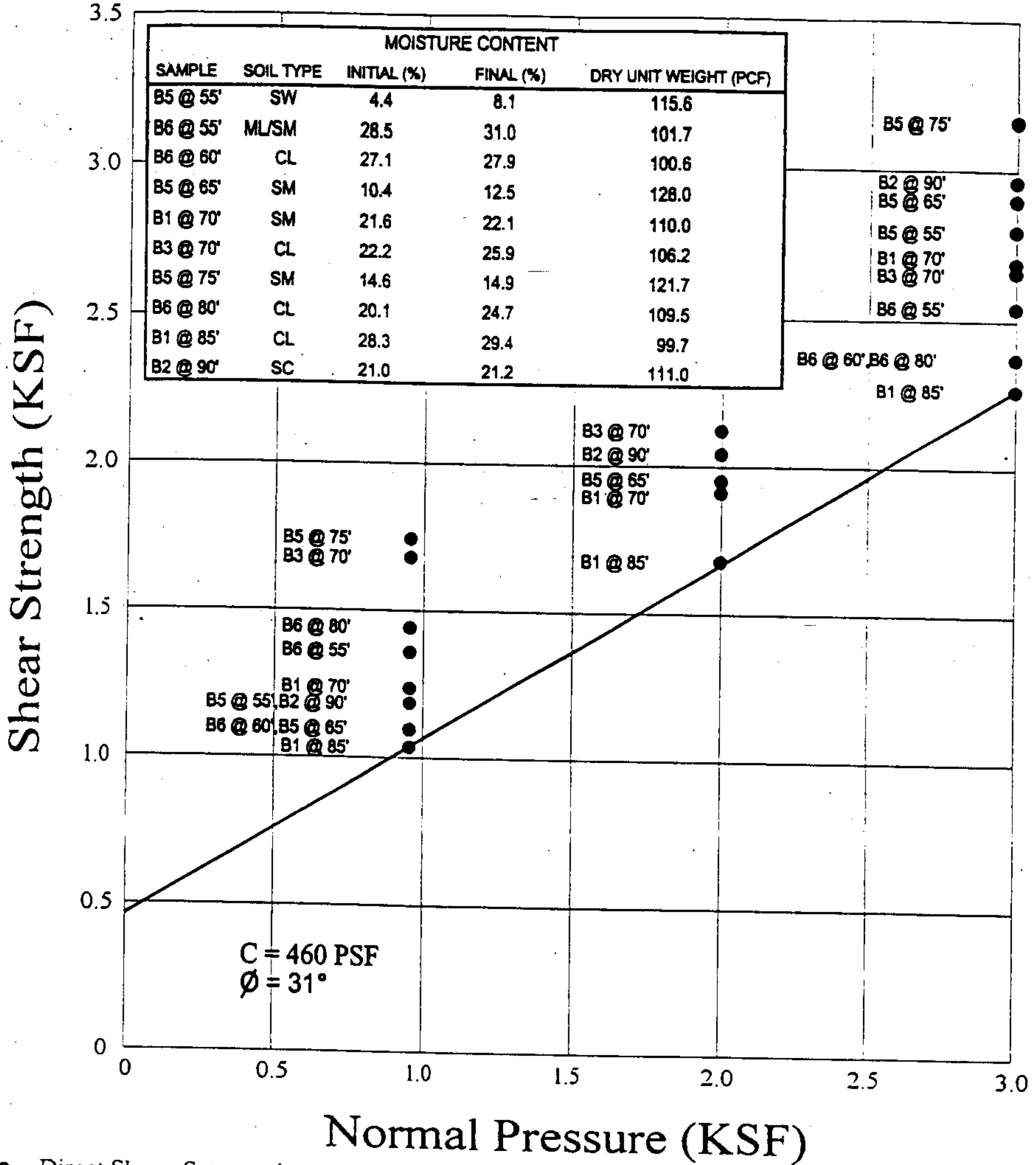
SHEAR TEST DIAGRAM

JERRY KOVACS AND ASSOCIATES, INC.
CONSULTING GEOTECHNICAL ENGINEERS

PROJECT: THE CASDEN COMPANY

FILE NO. 17526-S

PLATE: B-2



		MOISTURE CONTENT		
SAMPLE	SOIL TYPE	INITIAL (%)	FINAL (%)	DRY UNIT WEIGHT (PCF)
B5 @ 55'	SW	4.4	8.1	115.6
B6 @ 55'	ML/SM	28.5	31.0	101.7
B6 @ 60'	CL	27.1	27.9	100.6
B5 @ 65'	SM	10.4	12.5	128.0
B1 @ 70'	SM	21.6	22.1	110.0
B3 @ 70'	CL	22.2	25.9	106.2
B5 @ 75'	SM	14.6	14.9	121.7
B6 @ 80'	CL	20.1	24.7	109.5
B1 @ 85'	CL	28.3	29.4	99.7
B2 @ 90'	SC	21.0	21.2	111.0

● Direct Shear, Saturated

SHEAR TEST DIAGRAM

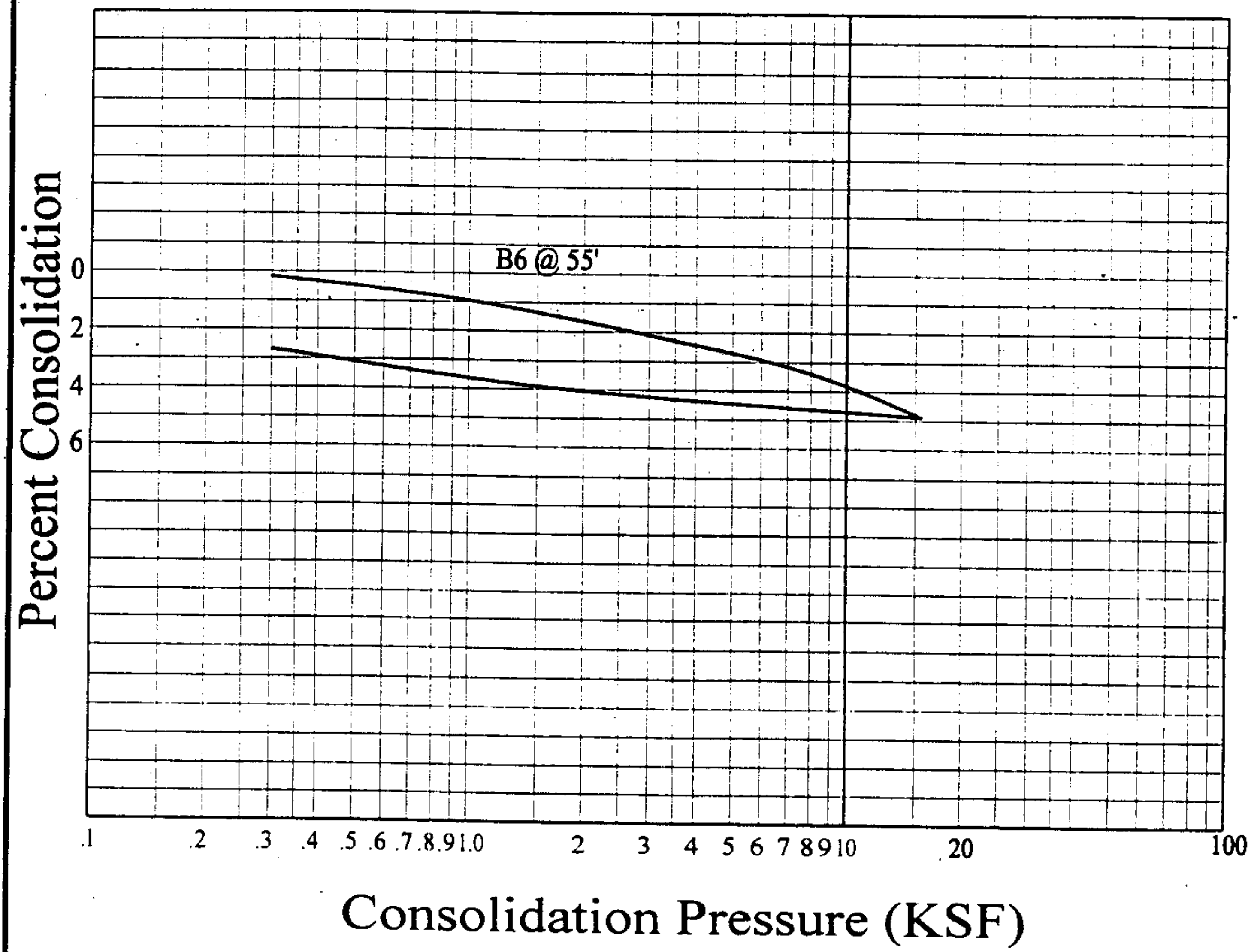
JERRY KOVACS AND ASSOCIATES, INC.
CONSULTING GEOTECHNICAL ENGINEERS

PROJECT: THE CASDEN COMPANY

FILE NO. 17526-S

PLATE: B-3

WATER ADDED AT 4 KSF

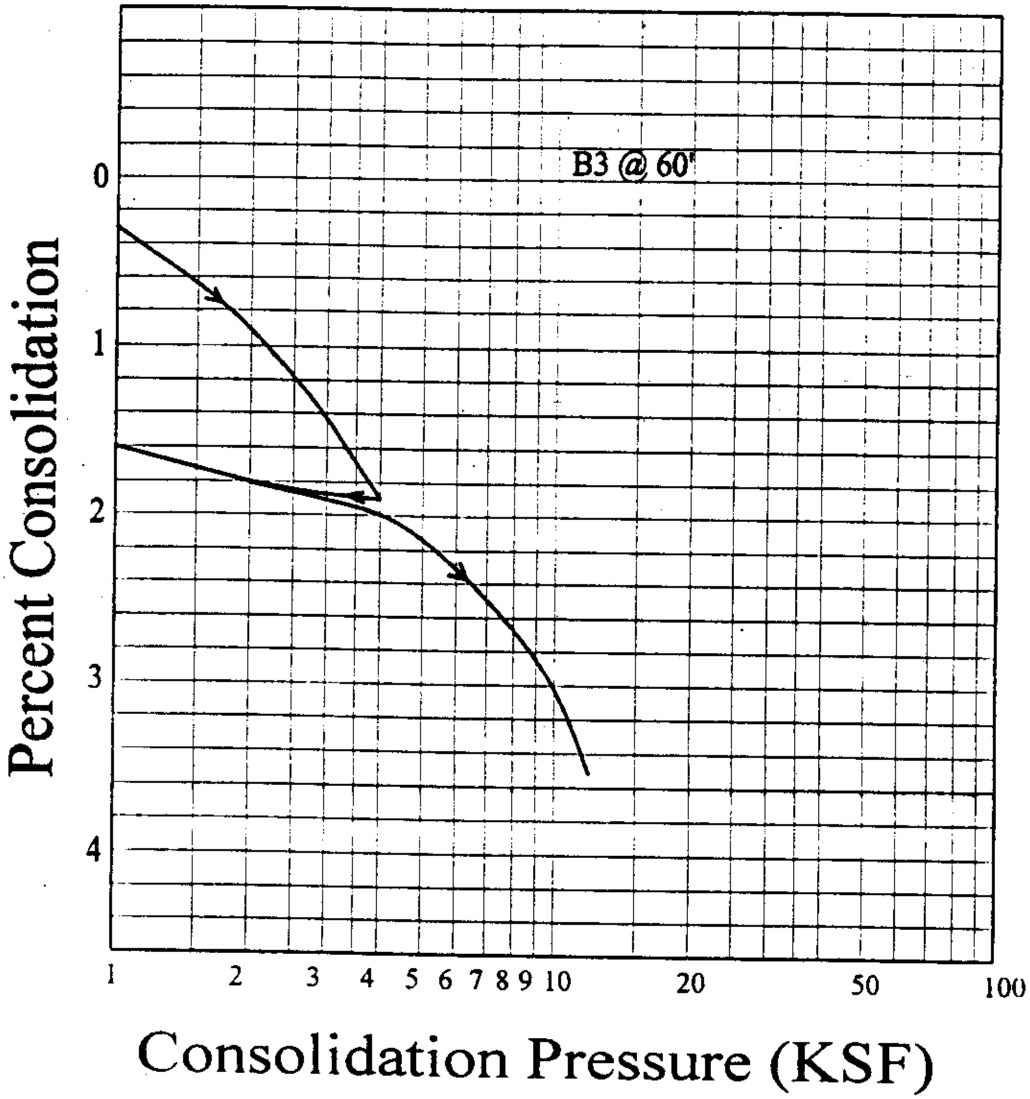


CONSOLIDATION TEST

JERRY KOVACS AND ASSOCIATES, INC.
CONSULTING GEOTECHNICAL ENGINEERS

PROJECT: THE CASDEN COMPANY

FILE NO. 17526-S | PLATE: C-1



WATER ADDED PRIOR TO LOADING

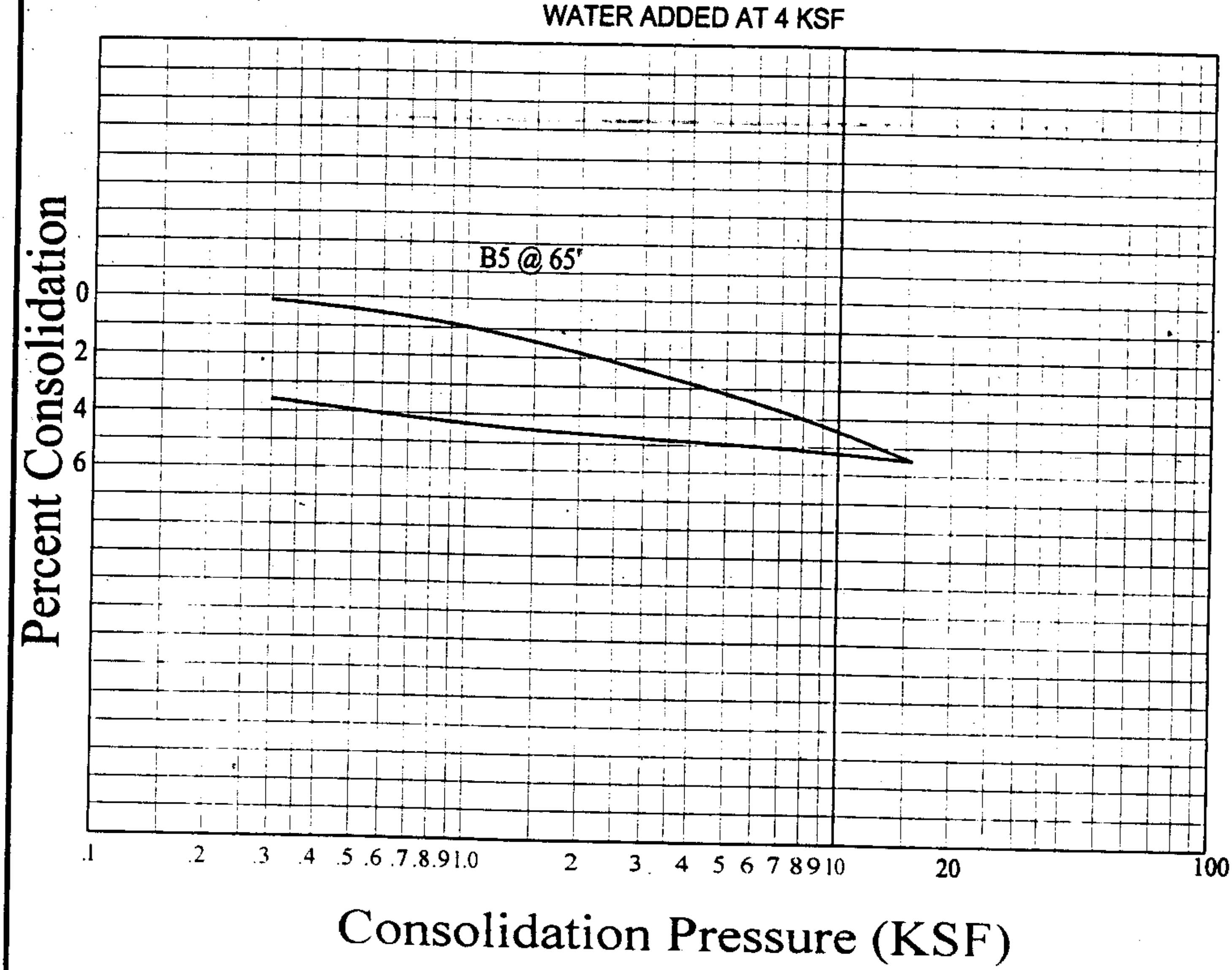
CONSOLIDATION TEST

JERRY KOVACS AND ASSOCIATES, INC.
CONSULTING GEOTECHNICAL ENGINEERS

PROJECT: THE CASDEN COMPANY

FILE NO. **17526-S**

PLATE: C-2



CONSOLIDATION TEST

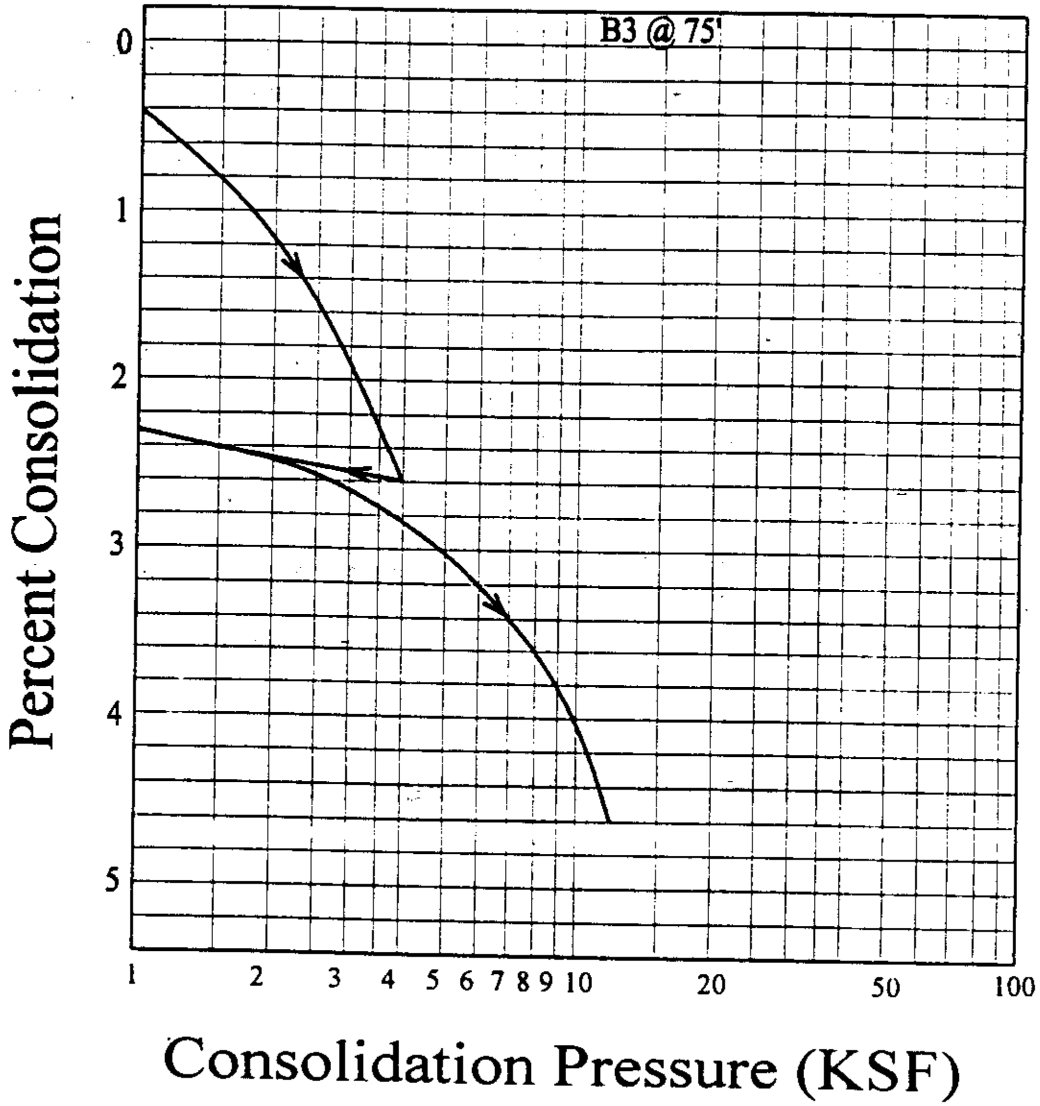
JERRY KOVACS AND ASSOCIATES, INC.
CONSULTING GEOTECHNICAL ENGINEERS

PROJECT: THE CASDEN COMPANY

FILE NO. 17526-S

PLATE: C-3

WATER ADDED PRIOR TO LOADING



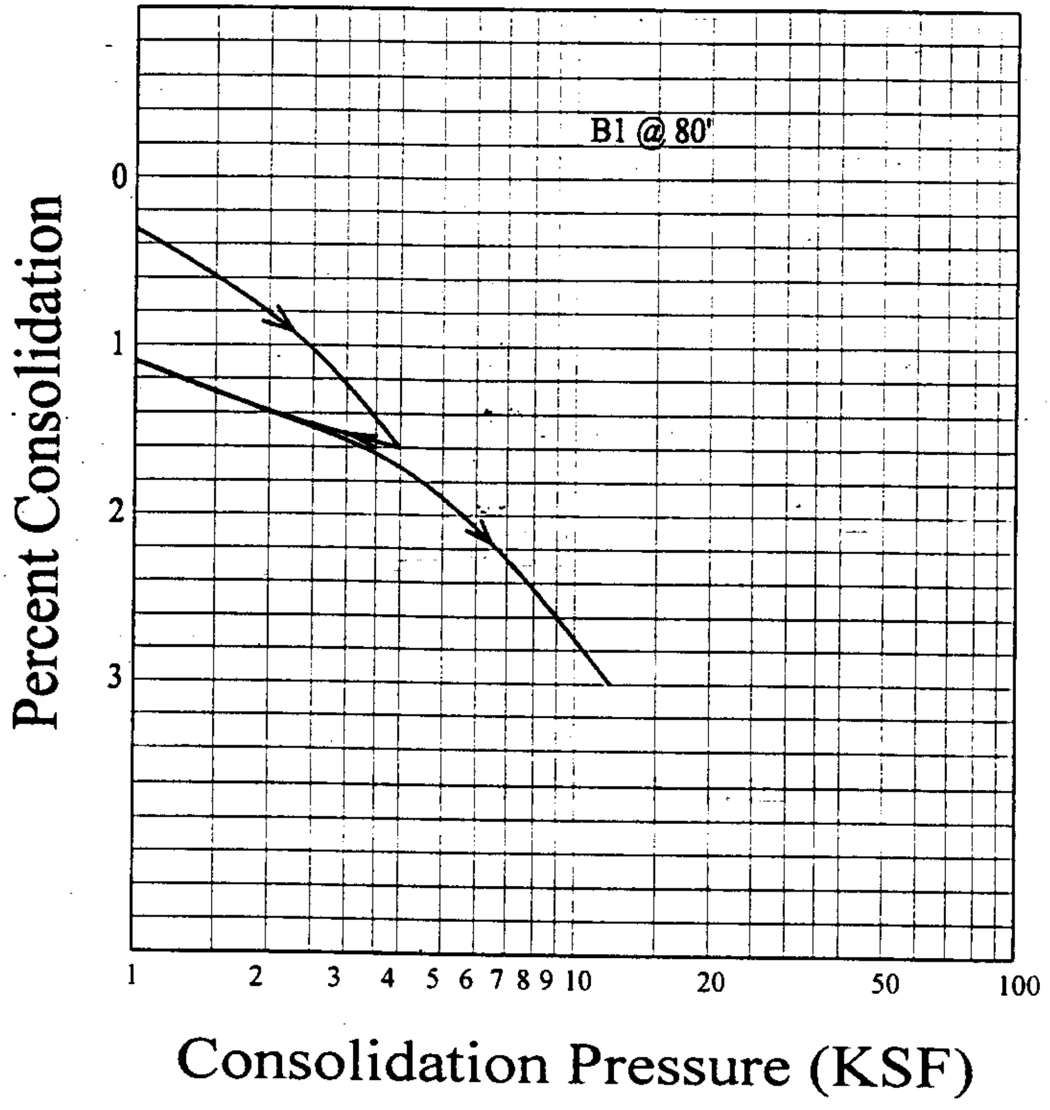
CONSOLIDATION TEST

JERRY KOVACS AND ASSOCIATES, INC.
CONSULTING GEOTECHNICAL ENGINEERS

PROJECT: THE CASDEN COMPANY

FILE NO. 17526-S | PLATE: C-4

WATER ADDED PRIOR TO LOADING



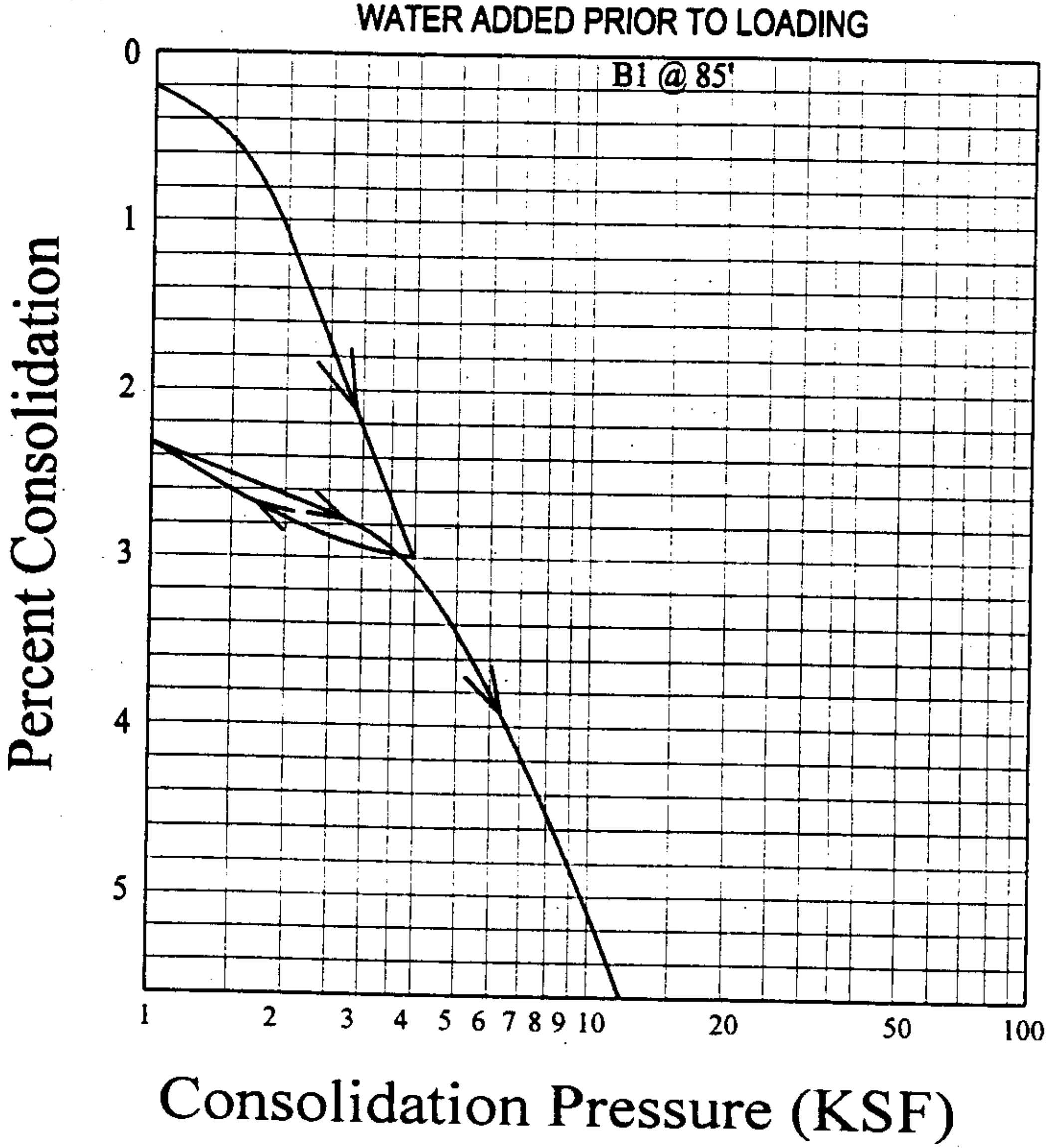
CONSOLIDATION TEST

JERRY KOVACS AND ASSOCIATES, INC.
CONSULTING GEOTECHNICAL ENGINEERS

PROJECT: THE CASDEN COMPANY

FILE NO. 17526-S

PLATE: C-5

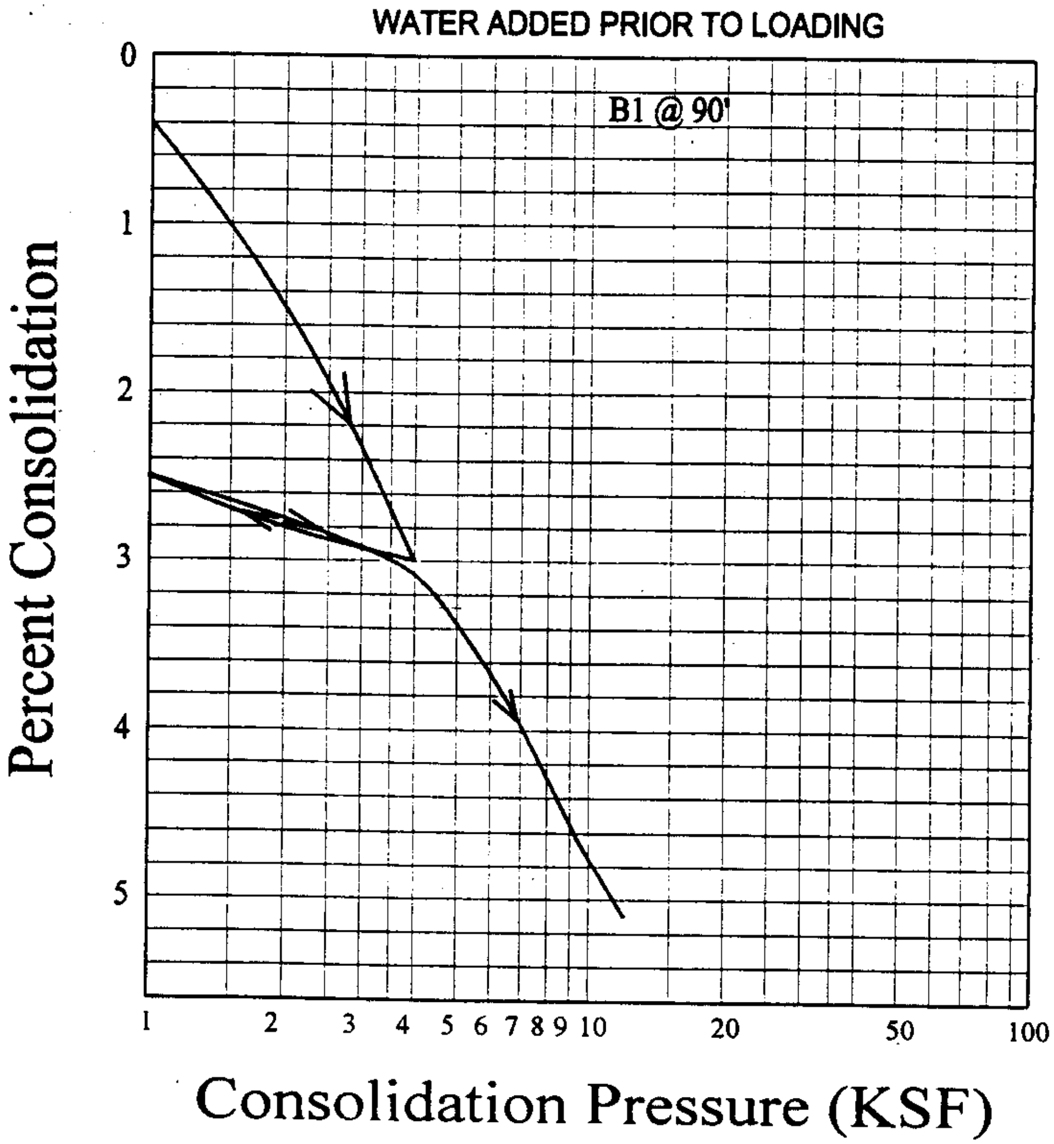


CONSOLIDATION TEST

JERRY KOVACS AND ASSOCIATES, INC.
CONSULTING GEOTECHNICAL ENGINEERS

PROJECT: THE CASDEN COMPANY

FILE NO. **17526-S** | PLATE: C-6



CONSOLIDATION TEST

JERRY KOVACS AND ASSOCIATES, INC.
CONSULTING GEOTECHNICAL ENGINEERS

PROJECT: THE CASDEN COMPANY

FILE NO. 17526-S

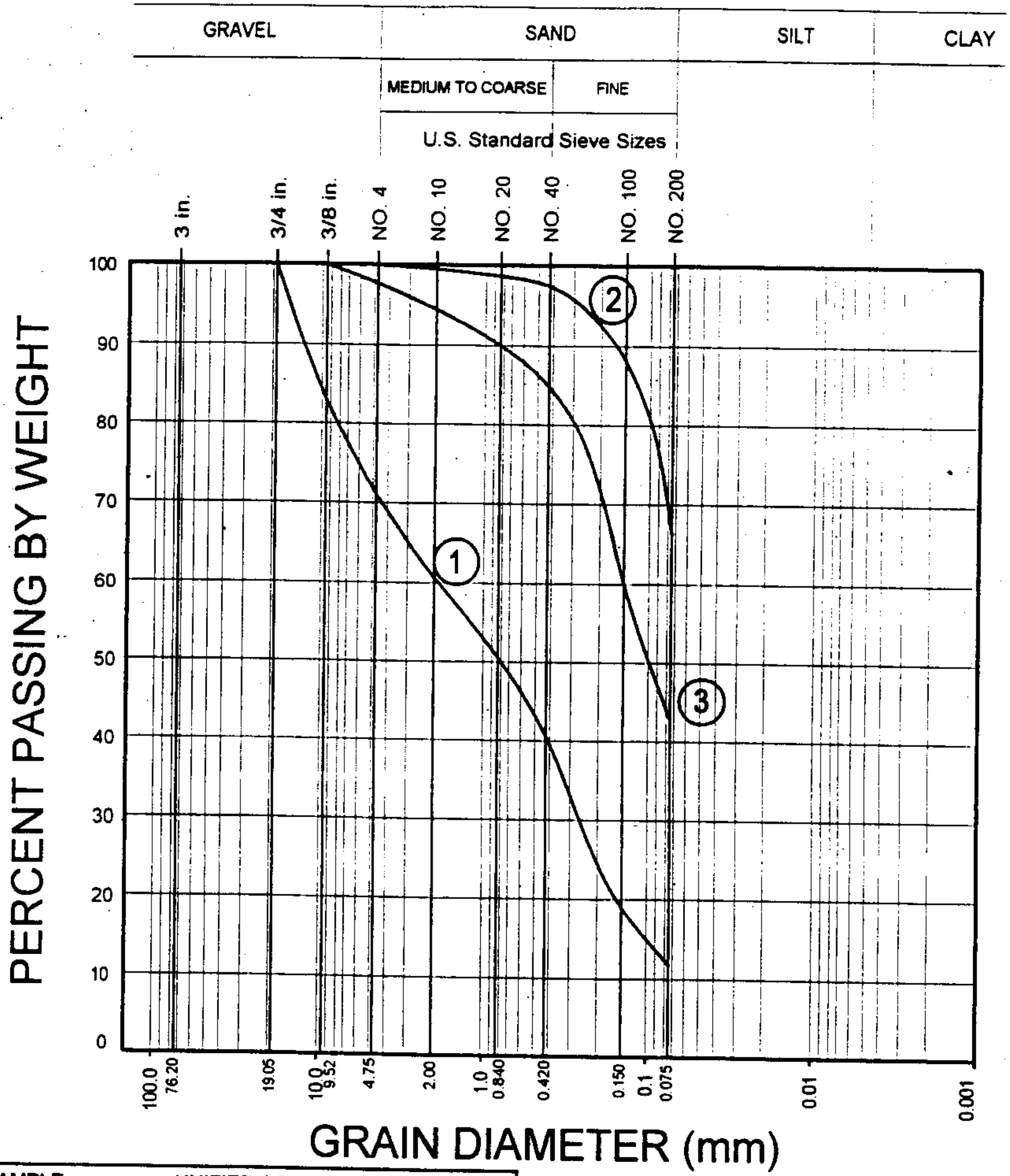
PLATE: C-7

EXPANSION DATA SHEET

EXPANSION INDEX

Sample	Boring 4 at 2 feet	Boring 2 at 3 feet	Boring 5 at 50 feet (Remolded)	Boring 6 at 52 feet (Remolded)
Swell - 60 Pounds Per Square Foot				
Air Dry (percent)	-3.6	-4.2	-0.5	-0.6
Saturation (percent)	0.6	0.9	5.1	7.5
Total (percent)	4.2	5.1	5.6	8.1
Expansion Index -- UBC Standard 18-2	22	31	36	61
Expansion Character	Low	Low	Low	Moderate

PLATE D



SAMPLE	UNIFIED SOIL CLASSIFICATION
1. B2 @ 55'	CLAYEY SAND
2. B2 @ 60'	CLAY
3. B3 @ 55'	CLAYEY SAND

GRAIN SIZE DISTRIBUTION

JERRY KOVACS AND ASSOCIATES, INC.
CONSULTING GEOTECHNICAL ENGINEERS

PROJECT: THE CASDEN COMPANY

FILE NO. **17526-S**

PLATE: E

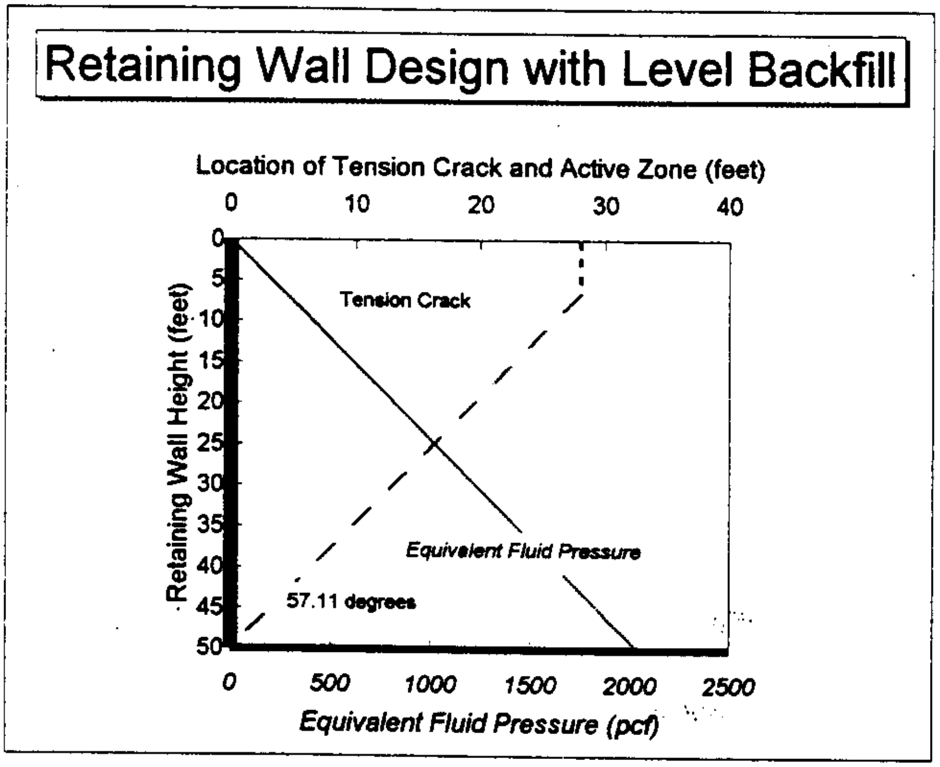
Jerry Kovacs & Associates, Inc.
Project: CASDEN
File #: 17526-S
Description: Retaining Wall Design

Retaining Wall Design with Level Backfill

Input:

Retaining Wall Height (feet)	(H)	50.00	feet
Unit Weight of Retained Soils	(W)	130.00	pcf
Friction Angle of Retained Soils	(P)	34.00	degrees
Cohesion of Retained Soils	(C)	420.00	psf
Factor of Safety	(FS)	1.50	
Factored Parameters	*(P/FS) =*	*24.21*	*degrees*
	(C/FS) =	*280.00*	*psf*

Tension Crack = 6.66 feet
Height of Active Zone = 43.34 feet
Pressure at Base of Wall = 2356.90 psf
Maximum Horizontal Thrust = 51073.81 lbs
Location of Thrust (measured from base of wall) = 16.67 feet
Critical Failure Angle from Horizontal = 57.11 degrees
Equivalent Fluid Pressure = 40.86 pcf
Design Equivalent Fluid Pressure = 40.86 pcf



Jerry Kovacs & Associates, Inc.
Project: CASDEN
File #: 17526-S
Description: Shoring Design

Shoring Design with Level Backfill

Input:

Height of Excavation (feet)	(H)	55.00	feet
Unit Weight of Retained Soils	(W)	130.00	pcf
Friction Angle of Retained Soils	(P)	34.00	degrees
Cohesion of Retained Soils	(C)	420.00	psf
Factor of Safety	(FS)	1.25	
Factored Parameters	*(P/FS) =*	*28.35*	*degrees*
	(C/FS) =	*336.00*	*psf*

Tension Crack = 8.66 feet
Height of Active Zone = 46.34 feet
Pressure at Base of Excavation = 2144.72 psf
Maximum Horizontal Thrust = 49689.80 lbs
Location of Thrust (measured from base of excavation) = 18.33 feet
Critical Failure Angle from Horizontal = 59.18 degrees
Equivalent Fluid Pressure = 32.85 pcf
Design Equivalent Fluid Pressure = 32.85 pcf

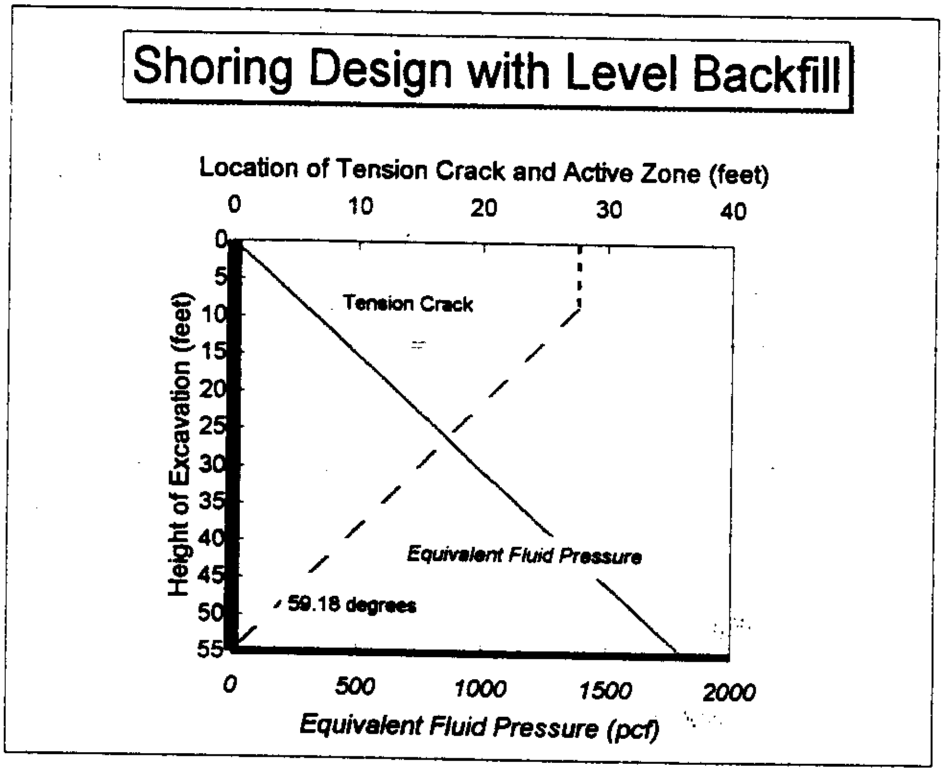


TABLE I - FAULT LOCATIONS AND DETERMINISTIC SITE PARAMETERS

DETERMINISTIC SITE PARAMETERS

Page 1

ABBREVIATED FAULT NAME	APPROX. DISTANCE mi (km)	MAX. CREDIBLE EVENT			MAX. PROBABLE EVENT		
		MAX. CRED. MAG.	PEAK SITE ACC. g	SITE INTENS MM	MAX. PROB. MAG.	PEAK SITE ACC. g	SITE INTENS MM
SAN ANDREAS - San Bernardi	55 (89)	7.30	0.108	VII	7.30	0.108	VII
SAN ANDREAS - Southern	55 (89)	7.40	0.114	VII	7.30	0.108	VII
SAN ANDREAS - Mojave	39 (62)	7.10	0.128	VIII	7.10	0.128	VIII
SAN ANDREAS - Carrizo	44 (71)	7.20	0.122	VII	7.20	0.122	VII
SAN ANDREAS - 1857 Rupture	39 (62)	7.80	0.186	VIII	7.50	0.158	VIII
SAN JACINTO-SAN BERNARDINO	55 (88)	6.70	0.079	VII	6.70	0.079	VII
ELSINORE-GLEN IVY	48 (78)	6.80	0.092	VII	6.30	0.071	VI
WHITTIER	25 (40)	6.80	0.153	VIII	5.90	0.095	VII
CHINO-CENTRAL AVE. (Elsino	40 (64)	6.70	0.123	VII	5.50	0.065	VI
GARLOCK (West)	59 (95)	7.10	0.092	VII	6.50	0.067	VI
NEWPORT-INGLEWOOD (Offshor	44 (71)	6.90	0.103	VII	5.80	0.058	VI
CLAMSHELL-SAWPIT	27 (43)	6.50	0.151	VIII	5.00	0.068	VI
CUCAMONGA	41 (66)	7.00	0.141	VIII	6.10	0.087	VII
HOLLYWOOD	2 (3)	6.40	0.637	X	5.30	0.356	IX
HOLSER	25 (41)	6.50	0.158	VIII	4.90	0.068	VI
MALIBU COAST	6 (9)	6.70	0.502	X	4.90	0.194	VIII
M.RIDGE-ARROYO PARIDA-SANT	50 (80)	6.70	0.103	VII	5.40	0.052	VI
NEWPORT-INGLEWOOD (L.A.Bas	4 (7)	6.90	0.537	X	5.60	0.270	IX
OAK RIDGE (Onshore)	28 (45)	6.90	0.179	VIII	6.20	0.124	VII
PALOS VERDES	9 (14)	7.10	0.380	X	6.20	0.237	IX
RAYMOND	13 (21)	6.50	0.254	IX	5.00	0.115	VII
RED MOUNTAIN	53 (86)	6.80	0.104	VII	5.90	0.065	VI
SAN CAYETANO	32 (51)	6.80	0.155	VIII	6.40	0.126	VIII
SAN GABRIEL	20 (32)	7.00	0.201	VIII	5.60	0.096	VII
SAN JOSE	32 (52)	6.50	0.130	VIII	5.00	0.059	VI

DETERMINISTIC SITE PARAMETERS

Page 2

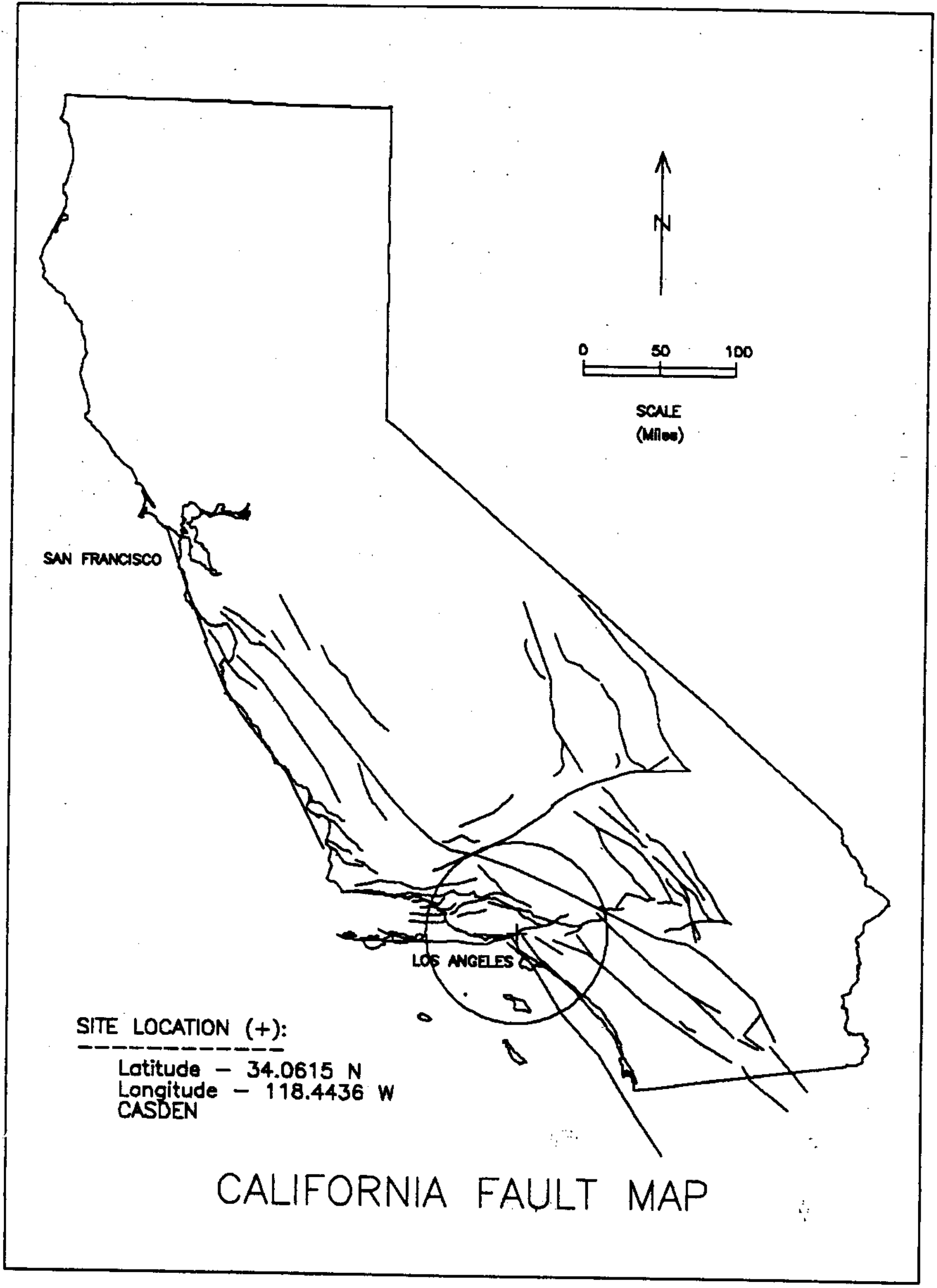
ABBREVIATED FAULT NAME	APPROX. DISTANCE mi (km)	MAX. CREDIBLE EVENT			MAX. PROBABLE EVENT		
		MAX. CRED. MAG.	PEAK SITE ACC. g	SITE INTENS MM	MAX. PROB. MAG.	PEAK SITE ACC. g	SITE INTENS MM
SANTA MONICA	1 (1)	6.60	0.768	XI	5.50	0.430	X
SANTA YNEZ (East)	45 (72)	7.00	0.108	VII	5.90	0.061	VI
SANTA SUSANA	17 (28)	6.60	0.220	IX	6.30	0.188	VIII
SIERRA MADRE (San Fernando	16 (26)	6.70	0.247	IX	5.60	0.138	VIII
SIERRA MADRE	16 (26)	7.00	0.288	IX	6.20	0.189	VIII
SIMI-SANTA ROSA	26 (41)	6.70	0.173	VIII	5.50	0.092	VII
VENTURA - PITAS POINT	44 (71)	6.80	0.119	VII	5.50	0.060	VI
VERDUGO	12 (19)	6.70	0.305	IX	5.20	0.138	VIII
COMPTON THRUST	12 (20)	6.80	0.257	IX	5.80	0.152	VIII
ELYSIAN PARK THRUST	13 (21)	6.70	0.240	IX	5.80	0.149	VIII
NORTHRIDGE (E. Oak Ridge)	15 (24)	6.90	0.236	IX	5.80	0.132	VIII
ANACAPA-DUME	15 (25)	7.30	0.350	IX	6.30	0.206	VIII
CHANNEL IS. THRUST (Easter	48 (76)	7.40	0.128	VIII	6.00	0.061	VI
MONTALVO-OAK RIDGE TREND	49 (79)	6.60	0.081	VII	5.50	0.046	VI
OAK RIDGE(Blind Thrust Off	47 (76)	6.90	0.099	VII	6.10	0.065	VI
CLEGHORN	59 (94)	6.50	0.068	VI	6.00	0.052	VI

-END OF SEARCH- 41 FAULTS FOUND WITHIN THE SPECIFIED SEARCH RADIUS.

THE SANTA MONICA FAULT IS CLOSEST TO THE SITE.
IT IS ABOUT 0.8 MILES AWAY.

LARGEST MAXIMUM-CREDIBLE SITE ACCELERATION: 0.768 g

LARGEST MAXIMUM-PROBABLE SITE ACCELERATION: 0.430 g



SITE LOCATION (+):

Latitude – 34.0615 N
Longitude – 118.4436 W
CASDEN

CALIFORNIA FAULT MAP

FIGURE I

TABLE II - HISTORICAL EARTHQUAKE EPICENTERS 1800 TO 2000

Page 1

FILE CODE	LAT. NORTH	LONG. WEST	DATE	TIME (GMT) H M Sec	DEPTH (km)	QUAKE MAG.	SITE ACC. g	SITE MM INT.	APPROX. DISTANCE mi [km]
DMG	34.370	117.650	12/ 8/1812	15 0 0.0	5.6	7.00	0.121	VII	50 [81]
T-A	34.000	118.250	9/23/1827	0 0 0.0	5.6	5.00	0.125	VII	12 [19]
DMG	34.000	119.000	9/24/1827	4 0 0.0	5.6	7.00	0.170	VIII	32 [52]
T-A	34.830	118.750	11/27/1852	0 0 0.0	5.6	7.00	0.111	VII	56 [90]
MGI	34.100	118.100	7/11/1855	415 0.0	5.6	6.30	0.170	VIII	20 [32]
T-A	34.000	118.250	1/10/1856	0 0 0.0	5.6	5.00	0.125	VII	12 [19]
MGI	34.000	117.500	12/16/1858	10 0 0.0	5.6	7.00	0.114	VII	54 [87]
T-A	34.000	118.250	3/26/1860	0 0 0.0	5.6	5.00	0.125	VII	12 [19]
DMG	34.200	117.900	8/28/1889	215 0.0	5.6	5.50	0.076	VII	33 [52]
DMG	34.300	118.600	4/ 4/1893	1940 0.0	5.6	6.00	0.152	VIII	19 [30]
DMG	34.100	119.400	5/19/1893	035 0.0	5.6	5.50	0.051	VI	55 [88]
DMG	34.300	117.600	7/30/1894	512 0.0	5.6	6.00	0.070	VI	51 [82]
DMG	34.300	117.500	7/22/1899	2032 0.0	5.6	6.50	0.085	VII	56 [91]
MGI	34.000	118.000	12/25/1903	1745 0.0	5.6	5.00	0.070	VI	26 [41]
MGI	34.000	118.300	9/ 3/1905	540 0.0	5.6	5.30	0.175	VIII	9 [15]
MGI	34.000	119.000	12/14/1912	0 0 0.0	5.6	5.70	0.086	VII	32 [52]
DMG	34.700	119.000	10/23/1916	254 0.0	5.6	5.50	0.051	VI	54 [87]
MGI	33.800	117.600	4/22/1918	2115 0.0	5.6	5.00	0.041	V	52 [83]
MGI	34.000	118.500	11/19/1918	2018 0.0	5.6	5.00	0.210	VIII	5 [9]
MGI	34.080	118.260	7/16/1920	18 8 0.0	5.6	5.00	0.136	VIII	11 [17]
DMG	34.000	118.500	8/ 4/1927	1224 0.0	5.6	5.00	0.210	VIII	5 [9]
DMG	33.950	118.632	8/31/1930	04036.0	5.6	5.20	0.128	VIII	13 [21]
DMG	33.617	117.967	3/11/1933	154 7.8	5.6	6.30	0.097	VII	41 [66]
DMG	33.750	118.083	3/11/1933	2 9 0.0	5.6	5.00	0.063	VI	30 [48]
DMG	33.750	118.083	3/11/1933	230 0.0	5.6	5.10	0.066	VI	30 [48]
DMG	33.750	118.083	3/11/1933	323 0.0	5.6	5.00	0.063	VI	30 [48]
DMG	33.700	118.067	3/11/1933	51022.0	5.6	5.10	0.061	VI	33 [53]
DMG	33.575	117.983	3/11/1933	518 4.0	5.6	5.20	0.053	VI	43 [69]
DMG	33.683	118.050	3/11/1933	658 3.0	5.6	5.50	0.073	VII	35 [56]
DMG	33.700	118.067	3/11/1933	85457.0	5.6	5.10	0.061	VI	33 [53]
DMG	33.750	118.083	3/11/1933	910 0.0	5.6	5.10	0.066	VI	30 [48]
DMG	33.850	118.267	3/11/1933	1425 0.0	5.6	5.00	0.093	VII	18 [29]
DMG	33.750	118.083	3/13/1933	131828.0	5.6	5.30	0.074	VII	30 [48]
DMG	33.617	118.017	3/14/1933	19 150.0	5.6	5.10	0.054	VI	39 [63]
DMG	33.783	118.133	10/ 2/1933	91017.6	5.6	5.40	0.086	VII	26 [42]
DMG	33.699	117.511	5/31/1938	83455.4	5.6	5.50	0.048	VI	59 [95]
DMG	33.783	118.250	11/14/1941	84136.3	5.6	5.40	0.097	VII	22 [36]
DMG	34.519	118.198	8/23/1952	10 9 7.1	5.6	5.00	0.056	VI	35 [56]
DMG	34.270	117.540	9/12/1970	143053.0	5.6	5.40	0.049	VI	54 [86]
DMG	34.411	118.401	2/ 9/1971	14 041.8	5.6	6.40	0.154	VIII	24 [39]
DMG	34.411	118.401	2/ 9/1971	14 1 8.0	5.6	5.80	0.112	VII	24 [39]
DMG	34.411	118.401	2/ 9/1971	14 244.0	5.6	5.80	0.112	VII	24 [39]
DMG	34.411	118.401	2/ 9/1971	141028.0	5.6	5.30	0.086	VII	24 [39]
DMG	34.308	118.454	2/ 9/1971	144346.7	5.6	5.20	0.107	VII	17 [27]
DMG	34.065	119.035	2/21/1973	144557.3	5.6	5.90	0.092	VII	34 [54]
DMG	33.986	119.475	8/ 6/1973	232917.0	5.6	5.00	0.037	V	59 [95]
PAS	33.944	118.681	1/ 1/1979	231438.9	5.6	5.00	0.101	VII	16 [25]
PAS	33.671	119.111	9/ 4/1981	155050.3	5.6	5.30	0.052	VI	47 [75]
PAS	34.061	118.079	10/ 1/1987	144220.0	5.6	5.90	0.133	VIII	21 [34]
PAS	34.073	118.098	10/ 4/1987	105938.2	5.6	5.30	0.101	VII	20 [32]
PAS	33.919	118.627	1/19/1989	65328.8	5.6	5.00	0.109	VII	14 [23]
GSP	34.140	117.700	2/28/1990	234336.6	5.6	5.20	0.053	VI	43 [69]
GSP	34.262	118.002	6/28/1991	144354.5	5.6	5.40	0.080	VII	29 [46]
GSP	34.213	118.537	1/17/1994	123055.4	5.6	6.70	0.309	IX	12 [19]
GSB	34.301	118.565	1/17/1994	204602.4	5.6	5.20	0.103	VII	18 [29]
GSP	34.326	118.698	1/17/1994	233330.7	5.6	5.60	0.104	VII	23 [38]
GSP	34.377	118.698	1/18/1994	004308.9	5.6	5.20	0.077	VII	26 [42]
GSB	34.379	118.711	1/19/1994	210928.6	5.6	5.50	0.089	VII	27 [43]

Dewatering Test Results

Dewatering testing was performed in February, 1996 by Hydroquip Pump and Dewatering Corporation, with the results detailed in a report entitled "Dewatering Testing Results, Westwood Village, Westwood, California," dated February 20, 1996. The results of the dewatering testing are discussed briefly here. The reader is referred to the report by Hydroquip for a more complete description of the testing and results.

The dewatering testing consisted of excavating three borings 30 to 36 inches in diameter, to depths between 70 and 80 feet below the existing site grade. Wells were then set to allow future monitoring of the groundwater levels. Upon initial drilling, it was repoirted that no free groundwater was encountered in any of the borings. It was also reported that no caving or sloughing occurred in any of the borings. After 5 to 6 days, water was measured in the borings as follows:

Date	Well No.	Ex. Grd. El.	Depth to Water	Water Elev.
2/15/96	TW1	+ - 335*	57'	278*
2/15/96	TW2	+ - 337*	69' 9"	267*
2/15/96	TW3	+ - 326*	Dry	Below 256*

* No formal survey was available for this program. References to many existing features were used for the purpose of this report. Data should be considered reliable to + - 1 foot.

```
Page   2

----------------------------------------------------------------------------------------
    |       |       |             | TIME     |      |       |  SITE   |SITE|  APPROX.
FILE| LAT.  | LONG. |    DATE     |  (GMT)   |DEPTH |QUAKE  |  ACC.   | MM |  DISTANCE
CODE|NORTH  | WEST  |             | H M Sec  | (km) | MAG.  |   g     |INT.|  mi  [km]
----|-------|-------|-------------|----------|------|-------|---------|----|-----------
GSP |34.378|118.618|  1/19/1994 |211144.9|   5.6| 5.10 | 0.078 | VII|  24 [  39]
GSP |34.305|118.579|  1/29/1994 |112036.0|   5.6| 5.10 | 0.095 | VII|  19 [  30]
GSP |34.231|118.475|  3/20/1994 |212012.3|   5.6| 5.30 | 0.147 |VIII|  12 [  19]
GSP |34.394|118.669|  6/26/1995 |084028.9|   5.6| 5.00 | 0.069 | VI |  26 [  42]
GSP |34.369|118.672|  4/26/1997 |103730.7|   5.6| 5.10 | 0.076 | VII|  25 [  40]
******************************************************************************
-END OF SEARCH-    63   RECORDS FOUND

COMPUTER TIME REQUIRED FOR EARTHQUAKE SEARCH:   0.2 minutes

MAXIMUM SITE ACCELERATION DURING TIME PERIOD 1800 TO 2000:  0.309g

MAXIMUM SITE INTENSITY (MM) DURING TIME PERIOD 1800 TO 2000:  IX

MAXIMUM MAGNITUDE ENCOUNTERED IN SEARCH: 7.00

NEAREST HISTORICAL EARTHQUAKE WAS ABOUT    5 MILES AWAY FROM SITE.

NUMBER OF YEARS REPRESENTED BY SEARCH:  201  years
```

Even at the deeper excavation levels of 65 to 70 feet below grade planned at the time the dewatering report was prepared. Hydroquip concluded that any required dewatering could be conducted on an 'as-encountered' basis with interior 'french drain' systems.

REGIONAL GEOLOGY

The subject site is located within the northwest portion of the Los Angeles Basin, on the Santa Monica Plain. The Santa Monica Plain is dissected by streams originating within the Santa Monica Mountains to the north. The Santa Monica Plain is underlain by late Pleistocene-age alluvial and marine deposits.

The site is within the extreme southern portion of the Transverse Ranges geomorphic province, just north of the Peninsular Ranges geomorphic province. The Transverse Ranges are dominated by east-west trending, reverse and thrust faults. The Peninsular Ranges are dominated by northwest-trending, strike-slip faults.

FAULTING AND SEISMICITY

Based on criteria established by the California Division of Mines and Geology (CDMG), faults may be categorized as active, potentially active or inactive (Jennings, 1994). Active faults are those which show evidence of surface displacement within the last 11,000 years (Holocene-age). Potentially-active faults are those that show evidence of last displacement within the last 1.6 million years (Quaternary-age). Faults showing no evidence of displacement within the last 1.6 million years may be considered inactive for most purposes, except for some critical structures.

Surface Fault Rupture

In 1972, the Alquist-Priolo Special Studies Zones Act (now known as the Alquist-Priolo Earthquake Fault Zoning Act) was passed into law. The Act defines "active" and "potentially active" faults utilizing the same aging criteria as that used by the CDMG, above. However, the established policy is to zone only those potentially active faults that have a relatively high potential for ground rupture. Therefore, not all faults termed "potentially active" by the CDMG are zoned under the Alquist-Priolo Act.

The subject site is not located in Alquist-Priolo Earthquake Fault Zone. Based on review of existing geologic information and site reconnaissance, no known fault traces cross the site. Therefore, the probability of surface rupture due to faulting occurring on the site is considered remote.

Fault Locations

The closest known fault to the site is the potentially active Santa Monica Fault. In the Safety Element of the Los Angeles County General Plan (Leighton, 1990), the Santa Monica Fault is shown as close as 0.8 miles south-southeast of the subject site. At about this point of closest approach to the subject site, the fault then splits into two main segments and continues to the west. Crook and Ward (1983) concluded that the latest surface faulting episode on the Santa Monica Fault is pre-Holocene.

The closest known active fault to the site is the Hollywood Fault, located about 1.9 miles to the northeast (Leighton, 1990). The next closest active fault is the Alquist-Priolo zoned portion of the Newport-Inglewood fault, located 4.1 miles to the southeast. In addition, Leighton (1990) shows a potentially active segment extending further to the north, as close as 2.3 miles southeast of the subject site.

The potentially active Charnock and Overland faults are also in relatively close proximity to the subject site. The closest traces of the Charnock and Overland Faults are 1.4 miles to the south and 0.9 mile to the southeast, respectively (Leighton, 1990).

Other faults within 10 miles of the subject site, as determined by the computer program EQFAULT (Blake, 1996) include the Malibu Coast Fault, located 6 miles from the site, and the Palos Verdes Fault, located 9 miles from the subject site. Leighton (1990) also shows the active North Hollywood Fault approximately 7.7 miles north-northeast of the subject site. A list of significant faults and fault segments within a 60-mile radius of the site and their distances to the site is presented in Table I in the Appendix, as compiled by the computer program EQFAULT, which uses the CDMG fault database.

HISTORIC SEISMICITY

As with all of Southern California, the site has experienced historic earthquakes from various regional faults. The epicenters of recorded earthquakes with magnitudes equal to or greater than 5.0 within a radius of 60 miles of the site are shown on Table II in the Appendix. The historic seismic record indicates that 63 earthquakes of magnitude 5.0 and greater have occurred within 60 miles of the site between the years 1800 and 2000. Larger, more distant earthquakes, such as the 1857 Fort Tejon earthquake on the San Andreas Fault may have also affected the site.

The strongest, most recent event near the site was the magnitude 6.7 Northridge earthquake of January 17, 1994. This earthquake occurred on a previously unknown buried thrust fault underlying the northern portion of the San Fernando Valley. The epicenter of this event was approximately 12 miles north of the site. According to contours of maximum horizontal ground acceleration provided in Chang (1994), the maximum horizontal ground acceleration at the subject site is estimated to have been on the order of 0.35 to 0.40 g.

Other recent earthquakes on relatively close faults to have impacted the site include the February 9, 1971, magnitude 6.4 San Fernando earthquake and the October 1, 1987, magnitude 5.9 Whittier Narrows earthquake. The epicenter of the San Fernando earthquake occurred on the San Fernando Fault, located approximately 24 miles north of the site. Unlike the fault responsible for the Northridge earthquake, the rupture on the San Fernando fault propagated to the surface in the Sylmar-San Fernando area of the San Fernando Valley. Based on the results of the EQSEARCH computer run and nearby strong-motion recordings, it is estimated that the peak ground accelerations at the subject site as a result of the San Fernando earthquake were on the order of 0.1g horizontal.

The Whittier earthquake occurred approximately 21 miles east of the subject site on a buried thrust fault located beneath the Elysian Park-Montebello Hills area of Los Angeles County. As with the Northridge earthquake, no surface fault ruptures were observed. Estimated peak ground accelerations at the site were on the order of 0.05 to 0.08 g horizontal.

GROUND MOTION PARAMETERS

The seismic exposure of the site may be investigated in two ways. The deterministic method assigns a maximum earthquake to a fault derived from formulas which correlate the length of the fault trace to the theoretical maximum magnitude earthquake. The probabalistic method considers the probability of exceedance of various levels of ground motion and is calculated by consideration of risk contributions from regional faults.

Deterministic Method

The deterministic approach recognizes two levels of earthquakes, maximum probable and maximum credible, postulated on regional faults. The maximum credible earthquake is the theoretical maximum event which could occur along a fault. The maximum credible earthquake assigned to a fault is derived from formulas which correlate the length of the fault trace to the theoretical maximum magnitude earthquake. The maximum probable earthquake is the maximum earthquake that may reasonably be expected within a 100-year period.

Table I in the Appendix presents a deterministic approach. The computer program EQFAULT (Blake, 1996) was utilized to make these calculations. Maximum credible and maximum probable earthquake magnitudes are assigned to all regional faults. The ground motion generated by such earthquakes is attenuated to the site using the attenuation equation by Boore et al (1993). site class B option. The resulting peak horizontal accelerations are shown on Table I. These values are the mean plus one standard deviation

Using this methodology, the maximum credible earthquake resulting in the highest peak horizontal accelerations at the site will be a magnitude 6.6 event on the Santa Monica Fault. Such an event would be expected to generate a peak horizontal acceleration on the order of 0.77g. The maximum probable earthquake resulting in the highest peak horizontal acceleration at the site will be a magnitude 5.5 event on the same fault. Such an event would be expected to generate a peak horizontal acceleration on the order of 0.43g. These values are the mean plus one standard deviation.

Probabilistic Method

The probabilistic method uses earthquake activity levels, earthquake magnitude distributions, fault lengths, and other parameters estimated for the regional faults. The probability of exceedance of various levels of ground motion is calculated by summing the risk contributions of all of the regional faults to obtain values for the site.

For this study, 41 regional faults and fault segments were used. These 41 are within the specified search radius of 60 miles from the site. Figure III in the Appendix indicates the return periods of various levels of peak horizontal acceleration. The typical ground motions used for design are often taken as those with 10 percent and 50 percent probability of exceedance in a 50 year period.

The computer program FRISKSP (Blake, 1998) was utilized to make these calculations. The ground motions were attenuated to the site utilizing the attenuation relation of Boore et al (1997), and modeling the soil underlying the site as a 1999 City of Los Angeles Building Code Soil Profile Type S_C. The ground motions with 10 percent and 50 percent probability of exceedance in a 50 year period are 0.46g and 0.23g, respectively. These values are one standard deviation above the mean.

Response Spectra

Response spectra are utilized to estimate the reaction of a structure to ground vibrations generated during earthquakes. Response spectra represent the linear response of a set of single degree-of-freedom systems to applied vibratory motion, in this case seismic ground motion. The response spectra will reflect the frequency and amplitude of the time history of ground motion which is a function of type of fault displacement, travel path and site conditions. Generally, these are unique for each earthquake recorded at each particular site.

Digitized spectral ordinates are provided in the Appendix as Tables 3 through 10. These spectral ordinates represent the maximum response amplitude of a linear elastic single degree-of-freedom system with equivalent viscous damping of 2, 5, 7 and 10 percent of critical damping, at return periods of 72 and 475 years. These spectra were developed utilizing the FRISKSP computer program by Thomas F. Blake (1998). These response spectra were developed by calculating pseudo-velocity for various different periods, using the attenuation relationship of Boore et al. (1997), and modeling the soil underlying the site as a 1999 City of Los Angeles Building Code Soil Profile Type S_c. The pseudo-velocity for the defined probability of exceedance are determined for each period calculated. Vertical spectral values should be considered equivalent to the horizontal spectra.

The response spectra provided in this report are "uniform" hazard response spectra. Uniform hazard spectra represent a multi-parameter description of ground motion. These spectra are created by independently calculating the ground motion at each spectral ordinant. In a uniform hazard spectrum, each of the ordinates has an equal likelihood of being exceeded and, as such, the spectral ordinates may be unrelated to each other. As long as no single aspect of the engineering design requires simultaneous consideration of both the short and long-period motions, the uniform hazard spectrum is well suited in providing design ground motion characteristics at many different periods. However, if both short and long-period motions are needed simultaneously, then the uniform hazard response spectrum may be a source of considerable conservatism, because it could imply the unlikely simultaneous occurrence of both a small, nearby earthquake and a large, distant one.

SECONDARY SEISMIC HAZARDS

Secondary effects of a strong nearby earthquake were also evaluated, including liquefaction, landsliding, flooding, tsunamis and seiches.

Liquefaction

Liquefaction involves a sudden loss in strength of a saturated, cohesionless soil which is caused by shock or strain and results in temporary transformation of the soil to a fluid mass. If the liquefying layer is near the surface, the effects are much like that of quicksand on any structure located on it. If the layer is in the subsurface, it may provide a sliding surface for the material above it.

Liquefaction typically occurs in areas where the groundwater is less than 50 feet from the surface and where the soils are composed of poorly consolidated fine to medium-grained sand. In addition to the necessary soil conditions, the ground acceleration and duration of the earthquake must also be of a sufficient level to initiate liquefaction.

Due to the depth of the groundwater on the subject site and the firm to dense nature of the soils, the potential for liquefaction occurring beneath the site is considered to be remote.

Landsliding

Due to the relatively flat nature of the site, probability of seismically-induced landslides occurring on the site is considered to be nil.

Earthquake-Induced Flooding

Earthquake-induced flooding is flooding caused by failure of dams or other water-retaining structures due to earthquakes. Review of the County of Los Angeles Flood and Inundation Hazards Map, (Leighton, 1990), indicates the site lies within the potential inundation boundary of the Stone Canyon Reservoir in Bel Air. The inundation areas are based on an assumed catastrophic failure of the reservoir during peak storage capacity. The inundation boundary shown on the map encompasses all probable routes that a flood might follow after exiting the reservoir, thus the map shows very large and conservative inundation areas.

Tsunamis and Seiches

Tsunamis are tidal waves generated by fault displacement or major ground movement below the ocean. The site is high enough and far enough from the ocean to preclude being prone to hazards of a tsunami.

Seiches are large waves generated in enclosed bodies of water in response to ground shaking. No major water-retaining structures are located immediately up gradient from the project site. Therefore, the risk of flooding from a seismically-induced seiche is considered to be remote.

CONCLUSIONS AND RECOMMENDATIONS

Based upon the exploration and laboratory testing, it is the finding of this firm that construction of the proposed mixed-use development is feasible from a geotechnical engineering standpoint, provided our advice and recommendations are followed and are implemented during construction.

Conventional column and wall footings founded in the dense native soils at the proposed subterranean garage level may be utilized for foundation support. Floor slabs should be cast upon undisturbed native soils or properly compacted fill.

Excavation of the proposed subterranean garage will require shoring measures to provide a stable excavation due to the depth of the excavation and the proximity of adjacent improvements. It is recommended that a soldier pile shoring system be utilized.

Pumping of the soils near the bottom of the excavation may occur during the operation of heavy equipment. If pumping does occur, a mat of gravel approximately 1 to 2 feet in thickness may be required to provide a stable working surface. Typically, such pumping is more severe with rubber-tire equipment.

Due to the potential for groundwater at or near the lowest subterranean garage level, an underslab drainage system is recommended to prevent hydrostatic pressures from developing below the slab.

If conditions encountered during construction appear to differ from those disclosed herein, notify this office immediately so the need for modifications may be considered in a timely manner. Compliance with the design concepts, specifications or recommendations during construction requires review by this firm during the course of construction.

SEISMIC DESIGN CONSIDERATIONS

The subject site is not underlain by the surface traces of any known faults, and the site is not located within an Alquist-Priolo Earthquake Fault Zone. In addition, the potential for secondary seismic hazards such as liquefaction, landsliding, dynamic settlements and flooding are considered very low to non-existent. Therefore, the major seismic hazard to the site, as with all of Southern California, is expected moderate to strong ground shaking from an earthquake on a local or regional fault.

Based on the 1999 City of Los Angeles Building Code, a Soil Profile Type S_C. and near-source factors $N_a = 1.3$ and $N_v = 1.6$ may be utilized in the design of the structure. The controlling fault is the Santa Monica Fault, which is a Seismic Source Type B, located less than 2 kilometers from the subject site.

SITE DRAINAGE

All site drainage should be collected and transferred to the street in non-erosive drainage devices. Drainage should not be allowed to pond anywhere on the site, and especially not against any foundation or retaining wall. During construction, drainage should not be allowed to flow into the excavation.

EXPANSIVE SOILS

The onsite soils in their undisturbed state are in the low expansion range. When disturbed and recompacted, however, as typically occurs with the soils at the subgrade level, the clayey soils on site become moderately expansive. Additional reinforcing is required, as recommended in the Floor Slabs section of this report.

CONVENTIONAL FOUNDATIONS

Allowable Bearing Values

Conventional footings embedded a minimum of 24 inches into the dense, native soils found at the subterranean garage level may be designed for an allowable bearing value of 7,000 pounds per square foot. This bearing value may be increased by 1,000 pounds per square foot for each additional foot of embedment. The maximum allowable bearing capacity is 9,000 pounds per square foot.

The bearing values indicated above are for the total of dead and frequently applied live loads, and may be increased by one third for short duration loading, which includes the effects of wind or seismic forces.

Since the recommended bearing value is a net value, the weight of concrete in the footings may be taken as 50 pounds per cubic foot and the weight of the soil backfill may be neglected when determining the downward load on the footings.

If depth increases are utilized for the exterior wall footings, this office should be provided a copy of the final construction plans so that the excavation recommendations presented herein could be properly reviewed and revised if necessary.

Lateral Design

Resistance to lateral loading may be provided by soil friction and by the passive resistance of the soils. A coefficient of friction of 0.4 may be used with the dead load forces between footings and the underlying supporting soils.

Passive earth pressure for the sides of footings poured against undisturbed soil may be computed as an equivalent fluid having a density of 300 pounds per cubic foot, with a maximum earth pressure of 3,000 pounds per square foot. When combining passive and friction for lateral resistance, the passive component should be reduced by one third. A one-third increase in the passive value may be used for wind or seismic loads.

Foundation Settlement

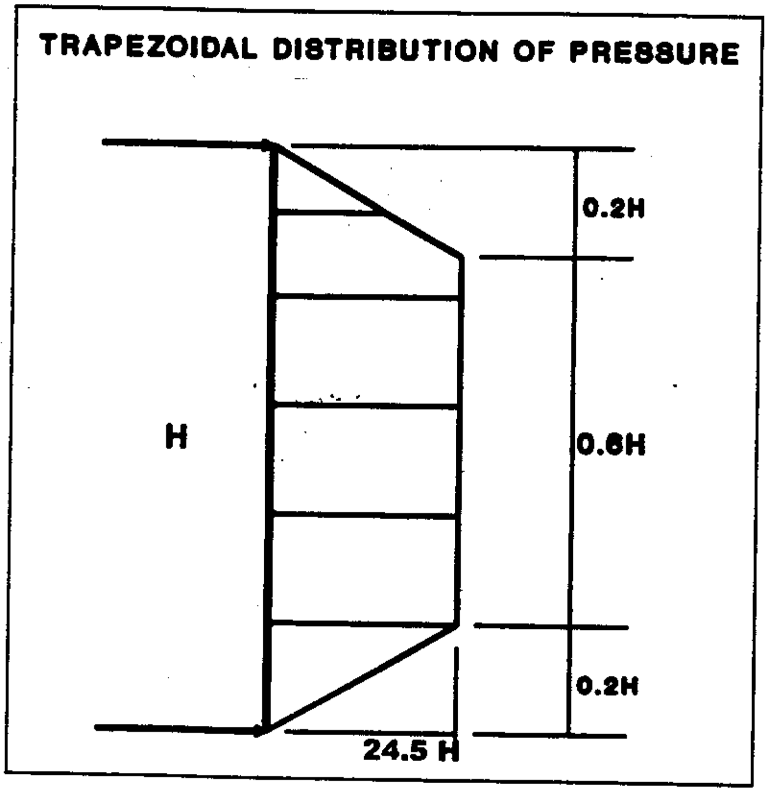
A majority of the settlement of the conventional foundations is expected to occur on initial application of loading. Estimated total settlements are not expected to exceed 1 inch. Differential settlements between adjacent columns is not expected to exceed 1/4 inch.

Foundation Installation

All footing excavations should be observed by personnel of this firm to verify penetration into the undisturbed native soils. Footings should be deepened if necessary to extend into satisfactory soils. Footing excavations should be dewatered and cleaned of loose soils prior to placing concrete. Any required backfill should be mechanically compacted; flooding is not permitted.

BASEMENT RETAINING WALLS

Subterranean garage walls may be designed to resist a trapezoidal pressure distribution of lateral earth pressure as indicated in the diagram below. The conditions for this loading diagram include the installation of retaining wall and underslab drainage systems, as described in this report. Additional active pressure should be added for a surcharge condition due to sloping ground, vehicular traffic or adjacent structures.



Design walls for 24.5 H
Where H is the height of the walls in feet

In addition to the recommended earth pressure, the upper 10 feet of the basement walls adjacent to the streets and alley should be designed to resist a uniform lateral pressure of 100 pounds per square foot, acting as a result of an assumed 300 pounds per square foot surcharge behind the walls due to normal street traffic. If the traffic is kept back at least 10 feet from the basement walls, the traffic surcharge may be neglected.

Also, the basement walls should be designed to accommodate any surcharge pressures that may be imposed by the existing buildings on the adjacent properties, unless those existing footings are underpinned and extended to the same depth as the new footings.

Wall Drainage

It is recommended that a wall drainage system be installed. Since it is expected that soldier beams with wood lagging will be utilized for temporary shoring of the parking excavation, a wall drainage system consisting of geotextile composite draped on the shored embankment would be recommended. Typically, such composites consist of impermeable plastic on the garage side with filter fabric bonded to the soil or lagging side of the composite. Water enters the composite through the filter fabric and is transported down the composite to a weep drain installed through the garage wall and underneath the garage slab. The unperforated weep hole pipe is then connected to an unperforated pipe located beneath the slab. Several manufacturers of these composites provide the proper connections and consultation for the installation of their products.

Free-standing retaining walls, such as loading dock walls, with backfill exposed to the atmosphere should be provided with a subdrain or weepholes covered with a minimum of 12 inches of gravel, and a compacted fill blanket or other seal at the surface. Retaining walls may be backfilled with a minimum of 12 inches of gravel adjacent to the wall to within 2 feet of the ground surface. The onsite earth materials, when used for retaining wall backfill, should be compacted to a minimum of 90 percent of the maximum density as determined by ASTM D 1557-91 or equivalent.

Sump Pump Design

The purpose of the recommended retaining wall backdrainage system and underslab drainage system, is to relieve potential hydrostatic pressure. In their 1996 report, Hydroquip Pump and Dewatering Company estimated a total volume of less than 50 gallons per minute to dewater the site, with a planned excavation level of 60 to 70 feet deep. Although the currently planned garage depth of 50 feet is shallower than what was anticipated in the 1996 Hydroquip report, the stabilized water level recently measured in wells at the site was significantly higher. For preliminary purposes, it is recommended that the permanent sump pump system be designed to accomodate a similar volume, although adjustments may be required after the excavation has been made, and the actual volume of water seeping into the excavation can be observed.

Wall Backfill

Retaining walls may be backfilled with the on-site soils or imported, free-draining soils. Where the voids to be filled behind walls are confined, the use of crushed rock, pea gravel, or lean mix slurry may be recommended. Any required backfill should be mechanically compacted in layers not more than 8 inches thick, to at least 95 percent of the maximum density obtainable by the ASTM Designation D 1557 method of compaction. Flooding should not be permitted. Proper compaction of the backfill will be necessary to reduce settlement of the backfill, and to reduce settlement of overlying walks and paving. Some settlement of required backfill should be anticipated, and any utilities supported therein should be designed to accept differential settlement, particularly at the points of entry to the structure.

TEMPORARY EXCAVATIONS

It is anticipated that excavations ranging up to 55 feet in vertical height will be required for the proposed subterranean parking levels. Due to the proximity to property lines and existing structures, it is anticipated that the entire excavation will require shoring. Where space permits, temporary unsurcharged embankments may be cut at 1:1 (horizontal to vertical) or flatter. Adjacent to the existing buildings, the bottom of any unshored excavation should be restricted so as not to extend below a plane drawn at 1-1/2:1 downward from the existing buildings.

Where sloped embankments are utilized, the tops of the slopes should be barricaded to prevent vehicles and storage loads within five feet of the tops of the slopes. If the temporary construction embankments are to be maintained during the rainy season, berms are suggested along the tops of the slopes where necessary to prevent runoff from entering the excavation and eroding the slope faces. The soils exposed in the cut slopes should be inspected during excavation by personnel from this office so that modifications of the slopes can be made if variations in the soil conditions occur. All excavations should be stabilized within 30 days of initial excavation.

SHORING

Soldier Pile Design and Installation

The following information on the design and installation of the shoring is as complete as possible at this time. It is suggested that a review of the final shoring plans and specifications be made by this office prior to bidding or negotiating with a shoring contractor.

The recommended method of shoring would consist of steel soldier piles, placed in drilled holes and backfilled with concrete, and/or slurry, depending on the design. The soldier piles should be designed as laterally braced utilizing either drilled tie-back anchors or raker braces.

Drilled cast-in-place soldier piles should be placed no closer than 2 diameters on center. The minimum diameter of the piles is 18 inches. Structural concrete should be used for the soldier piles below the excavation; lean-mix concrete may be employed above that level. As an alternative, lean-mix concrete may be used throughout the pile where the reinforcing consists of a wideflange section.

The slurry must be of sufficient strength to impart the lateral bearing pressure developed by the wideflange section to the soil. For design purposes, an allowable passive value for the soils below the bottom plane of excavation, may be assumed to be 700 pounds per square foot per foot of depth, up to a maximum of 7,000 pounds per square foot. To develop the full lateral value, provisions should be implemented to assure firm contact between the soldier piles and the undisturbed soils.

Encounter with water should be expected during the excavations for the proposed soldier piles, although volumes may be minor. Piles placed below the water level require the use of a tremie to place the concrete into the bottom of the hole. The tremie tube should be kept full of concrete. The flow should be continuous until the work is completed and the resulting concrete seal should be monolithic and homogeneous. The tip of the tremie tube should always be kept about 5 feet below the surface of the concrete and definite steps and safeguards should be taken to insure that the tip of the tremie tube is never raised above the surface of the concrete.

Casing may be required should caving be experienced in the saturated and/or granular soils. If casing is used, extreme care should be employed so that the pile is not pulled apart as the casing is withdrawn. At no time should the distance between the surface of the concrete and the bottom of the casing be less than 5 feet.

The frictional resistance between the soldier piles and retained soil may be used to resist the vertical component of the anchor load. The coefficient of friction may be taken as 0.4 based on uniform contact between the steel beam and lean-mix concrete and retained earth. The portion of soldier piles below the plane of excavation may also be employed to resist the downward loads. The downward capacity may be determined using a frictional resistance of 600 pounds per square foot.

Lagging

If the clear spacing between soldier piles does not exceed six feet, lagging between soldier piles could be omitted within the cohesive soils. In the less cohesive soils, such as the sands and gravels, lagging would be necessary. It is recommended that the exposed soils be observed by the soils engineer to verify the cohesive nature of the soils and the areas where lagging may be omitted.

If any water seepage zones are encountered during excavation, continuous lagging would be required. In any areas where water seepage is occurring from the excavation face, the excavations for lagging and lagging placement should be carefully coordinated. The time between lagging excavation and lagging placement should be as short as possible.

Soldier piles and anchors should be designed for the full anticipated pressures. Due to arching in the soils, the pressure on the lagging will be less. It is recommended that the lagging be designed for the full design pressure but be limited to a maximum of 400 pounds per square foot.

Tie-back Anchors

Tie-back anchors may be used to resist lateral loads. Friction anchors are recommended. For design purposes, it may be assumed that the active wedge adjacent to the shoring is defined by a plane drawn 35 degrees with the vertical through the bottom plane of the excavation. Friction anchors should extend a minimum of 20 feet beyond the potentially active wedge, and to greater lengths if necessary to develop the desired capacities.

The capacities of the anchors should be determined by testing of the initial anchors as outlined in a following section. Drilled friction anchors constructed without utilizing pressure-grouting techniques may be designed for a skin friction of 600 pounds per square foot. Depending on the techniques utilized, and the experience of the contractor performing the installation, it is anticipated that a skin friction of 2500 pounds per square foot could be utilized for pressure-grouted anchors. Only the frictional resistance developed beyond the active wedge should be utilized in resisting lateral loads. Anchors should be placed at least 6 feet on center to be considered isolated.

Anchor Installation

Tied-back anchors may be installed between 20 and 40 degrees below the horizontal. Caving of the anchor shafts, particularly within sand and gravel deposits and below water, should be anticipated and provisions should be implemented in order to minimize such caving. It is suggested that hollow-stem auger driling equipment be used to install the anchors. The anchor shafts should be filled with concrete by pumping from the tip out, and the concrete should extend from the tip of the anchor to the active wedge. In order to minimize the chances of caving, it is recommended that the portion of the anchor shaft within the active wedge be backfilled with sand before testing the anchor. This portion of the shaft should be filled tightly and flush with the with the face of the excavation. The sand backfill should be placed by pumping; the sand may contain a small amount of cement to facilitate pumping.

Anchor Testing

At least ten percent of the anchors should be selected for "quick", 200 percent tests and three additional anchors be selected for 24-hour, 200 percent tests. These tests should be performed prior to installation of additional tiebacks. The purpose of the 200 percent tests is to verify the friction value assumed in design. The anchors should be tested to develop twice the assumed friction value.

Where satisfactory test results are not achieved on the initial anchors, the anchor diameter and/or length should be increased until satisfactory test results are obtained.

The total deflection during the 24-hour, 200 percent test should not exceed 12 inches. During the 24-hour tests, the anchor deflection should not exceed 0.75 inches measured after the 200 percent test load is applied.

For the "quick" 200 percent tests, the 200 percent test load should be maintained for 30 minutes. The total deflection of the anchor during the 200 percent quick tests should not exceed 12 inches; the deflection after the 200 percent load has been applied should not exceed 0.25 inch during the 30-minute period.

All of the remaining anchors should be tested to at least 150 percent of design load. The total deflection during this test should not exceed 12 inches. The rate of creep under the 150 percent test load should not exceed 0.1 inch over a 15 minute period in order for the anchor to be approved for the design loading.

After a satisfactory test, each anchor should be locked-off at the design load. This should be verified by rechecking the load in the anchor. The load should be within 10 percent of the design load. The installation and testing of the anchors should be observed by a representative of this firm.

Internal Bracing

Rakers may be utilized to brace the soldier piles in lieu of tieback anchors. The raker bracing could be supported laterally by temporary concrete footings (deadmen) or by the permanent, interior footings. For design of such temporary footings or deadmen, poured with the bearing surface normal to rakers inclined at 45 degrees, a bearing value of 3,000 pounds per square foot may be used, provided the shallowest point of the footing is at least 2 feet below the lowest adjacent grade.

Lateral Pressures

Laterally-braced shoring should be designed for a trapezoidal distribution of lateral earth pressure, plus the surcharge loading resulting from adjacent traffic and structures. A lateral earth pressure of 20H should be utilized, where H is the height of the shoring in feet. The lateral pressure should be distributed in the same trapezoidal configuration shown in the "Basement Retaining Walls" section of this report. Where a combination of sloped embankment and shoring is used, the pressure will be greater, and must be determined for each combination.

Deflection

It is difficult to accurately predict the amount of deflection of a shored embankment. It should be realized that some deflection will occur. It is estimated that the deflection could be on the order of 1 inch at the top of the shored embankment. If greater deflection occurs during construction, additional bracing may be necessary to minimize settlement of adjacent buildings and utilities in adjacent streets. If desired to reduce the deflection, a greater active pressure could be used in the shoring design. Where internal bracing is used, the rakers should be tightly wedged to minimize deflection. The proper installation of the raker braces and the wedging will be critical to the performance of the shoring.

Monitoring

Because of the depth of the excavation, some means of monitoring the performance of the shoring system is suggested. The monitoring should consist of periodic surveying of the lateral and vertical locations of the tops of all soldier piles and the lateral movement along the entire lengths of selected soldier piles. Also, some means of periodically checking the load on selected anchors will be necessary, where applicable.

Some movement of the shored embankments should be anticipated as a result of the relatively deep excavation. It is recommended that photographs of the existing buildings on the adjacent properties be made during construction to record any movements for use in the event of a dispute.

UNDERSLAB DRAINAGE SYSTEM

Due to the recently measured water levels of approximately 42 to 45 feet below the ground surface, an underslab drainage system should be installed below the lower garage floor level. This drainage system should consist of a 1-foot thick layer of 3/4-inch gravel underlying the entire floor slab, which would be drained by subdrain pipes placed in gravel-filled drainage trenches leading to a sump pump. The drain lines should consist of 4-inch perforated pipe, perforations down, placed in trenches approximately 1 foot wide and 1 foot deep. The pipes would then be covered with 3/4-inch gravel, and the entire gravel and pipe system within the trenches would be wrapped in filter fabric. The gravel filled drainage trenches should be spaced approximately 40 feet apart, depending on the final foundation plan layout.

SLABS ON GRADE

The required underslab drainage system will provide adequate support for the proposed floor slab. Any soils loosened or over-excavated should be wasted from the site or properly compacted to 95 percent of the maximum dry density. The lower subterranean garage floor slab may be supported directly on the gravel drainage material, provided the lower floor would not be adversely affected by moisture. In any areas where dampness would be objectionable, it is recommended that the floor slab be supported on an impermeable moisture barrier, such as 6-mil visqueen. If the membrane is used, a low-slump concrete should be used to minimize possible curling of the slabs. The barrier should be covered with a thin layer of sand, approximately 2 inches, to prevent punctures and aid in the concrete cure. A modulus of subgrade reaction of 250 pounds per cubic inch may be utilized in the design of the slab.

The subterranean garage floor slab should be a minimum of 5 inches thick, and should be reinforced with a minimum of #4 steel bars at 16 inches on center each way.

SOIL CORROSIVITY

Soil corrosivity testing was performed on five representative samples of the onsite soils by the laboratory of M.J. Schiff and Associates. The reader is referred to the attached report by their office for complete results, discussion of results and recommended mitigating measures.

Briefly, the results of the corrosivity testing indicate that the electrical resistivities of the soils are in the mildly and moderately corrosive categories with as-received moisture, and moderately corrosive to corrosive when saturated. Soil pH values of the samples range from 6.4 to 7.3, indicating mildly acidic to neutral conditions, which does not particularly increase soil corrosivity. The chemical content was low in all of the samples. Ammonium and nitrate were detected but in low concentrations. In summary, the soil is classified as corrosive to ferrous metals.

The sulfate content of the samples tested by M.J. Schiff and Associates was found to be in the negligible range. Therefore, the use of special sulfate resistant cements in concrete which will be in contact with the site soils is not required.

CONSTRUCTION SITE MAINTENANCE

It is the responsibility of the contractor to maintain a safe construction site. When excavations exist onsite, the area should be fenced and warning signs posted. All pile excavations must be properly covered and secured. The excavated materials generated by foundation and subgrade excavations should be either removed from the site or properly placed as a certified compacted fill.

Compliance with the design concepts, specifications or recommendations during construction requires review by this firm during the course of construction. It is critical that all foundations be observed by a representative of this office prior to placing concrete or steel. Any fill which is placed

APPENDIX

REFERENCES

Applied Technology Council, 1978, Tentative Provisions for Development of Seismic Regulations for Buildings: ATC Publication ATC 3-06, NBS Special Publication 510, NSF Publication 78-8.

ASCE, 1994, Technical Engineering and Design Guides as Adapted from the US Army Corps of Engineers, No. 9, Settlement Analysis. ASCE Press, pp 46-52.

Blake, T.F., 1996, EQFAULT - A Computer Program for the Deterministic Prediction of Peak Horizonal Acceleration from Digitized California Faults, Version 2.20.

Blake, T.F., 1996, EQSEARCH - A Computer Program for the Estimation of Peak Horizontal Acceleration from California Historical Earthquake Catalogs, Version 2.20.

Blake, T.F., 1998, FRISKSP - A Computer Program for the Probabilistic Estimation of Uniform-Hazard Spectra Using 3-D Faults as Earthquake Sources.

Blake, T.F., 1994-1996, LIQ2_30.WQ1 - A QUATTRO-PRO Spreadsheet Computer Program for the determination of liquefaction potential using the NCEER (1996) semi-empirical method.

Boore, D.M., Joyner, W.B., and Fumal, T.E., 1993a, Estimation of Response Spectra and Peak Accelerations From Western North American Earthquakes: An interim Report, U.S. Geological Survey Open-File Report 93-509, 15 pp.

Boore, D.M., Joyner, W.B., and Fumal, T.E., 1997, "Equations for Estimating Horizontal Response Spectra and Peak Acceleration from Western North American Earthquakes: A Summary of Recent Work," Seismological Research Letters, Vol. 68, No. 1, pp. 128-153.

Chang, S.W., et. al., 1994, "Ground Motions and Local Site Effects". In University of California at Berkeley Earthquake Engineering Research Center, Report No. UCB/EERC-94/08, p.28.

Crook, R., Jr., and Ward, K., 1983, Seismicity of the Santa Monica and Hollywood Faults, Determined by Trenching: U.S.G.S. Final Technical Report, Contract No. 14-08-0001-20523.

REFERENCES (Cont'd)

Crook, R., Jr., Proctor, R.J., 1992, The Hollywood and Santa Monica Fault and the Southern Boundary of the Transverse Ranges Province: in Pipkin, B., and Proctor, R.J. (eds.)

Engineering Geology Practice in Southern California, Star Publishing Company, Belmont, California.

Dibblee. Thomas W. Junior. 1991, Geologic Map of the Beverly Hills and Van Nuys (South 1/2) Quadrangles, Dibblee Foundation Map #DF-31.

Greensfelder, Roger W., 1974, Maximum Credible Rock Acceleration from Earthquakes in California, California Division of Mines and Geology.

Hauksson, E., 1992, Seismicity, Faults, and Earthquake Potential in Los Angeles, Southern California: in Pipkin, B., and Proctor, R.J. (eds.) Engineering Geology Practice in Southern California, Star Publishing Company, Belmont, California.

Hydroquip Pump & Dewatering Corporation, February 20, 1996, Dewatering Testing Results, Westwood Village, Westwood, California.

Jennings, Charles W., 1994, Fault Activity Map of California and Adjacent Areas, California Division of Mines and Geology.

Jerry Kovacs and Associates, July 15, 1996, Geotechnical Engineering Investigation, Proposed Village Center Mixed-Use Development, South Side of Weyburn Avenue, Between Tiverton and Glendon Avenues, Westwood, California.

Leighton and Associates, Inc., 1990, Technical Appendix to the Safety Element of the Los Angeles County General Plan: Hazard Reduction in Los Angeles County.

LeRoy Crandall and Associates, January 25, 1990, Report of Geotechnical Investigation, Proposed Westwood Village Mixed Use Project.

Martin, G.R., and Lew, M., 1999, Co-chairs and Editors of the Implementation Committee, "Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Liquefaction Hazards in California," Organized through the Southern California Earthquake Center, University of Southern California.

REFERENCES (Cont'd)

Mendenhall, W.G., 1905, Development of Underground Waters in the Western Coastal Plain Region of Southern California: U.S.G.S. Water Supply Paper 139.

Newmark, N.M. and Hall, W.J., 1982, Earthquake Spectra and Design, Earthquake Engineering Research Institute.

Seed, H.B. , Idriss, I.M., and Arango, I., 1983, Evaluation of Liquefaction Potential Using Field Performance Data, Journal of the Geotechnical Engineering Division, American Society of Civil Engineers, vol. 109, no. 3, pp. 458-482.

Tokimatsu, K., and Yoshimi, Y., 1983, Empirical Correlation of Soil Liquefaction Based on SPT N-Value and Fines Content, Soils and Foundations, Japanese Society of Soil Mechanics and Foundation Engineering, vol. 23, no. 4, pp. 56-74.

should be observed, tested, and verified if used for engineered purposes. Please advise this office at least twenty-four hours prior to any required site visit.

The City of Los Angeles requires that the entire subdrain system behind retaining walls be observed by a representative of this firm. In addition, any subdrain pipe utilized must be checked by a representative of this firm to see that it is an approved grade. Any gravel backfill placed above the subdrain must be observed by a representative of this firm prior to placing a minimum of 2 feet of controlled fill as a cap. Where the area between the wall and the excavation exceeds 18 inches, the gravel must be vibrated or wheel-rolled.

If conditions encountered during construction appear to differ from those disclosed herein, notify this office immediately so the need for modifications may be considered in a timely manner.

Should you have any questions, please call.

Respectfully submitted,
JERRY KOVACS AND ASSOCIATES, INC.

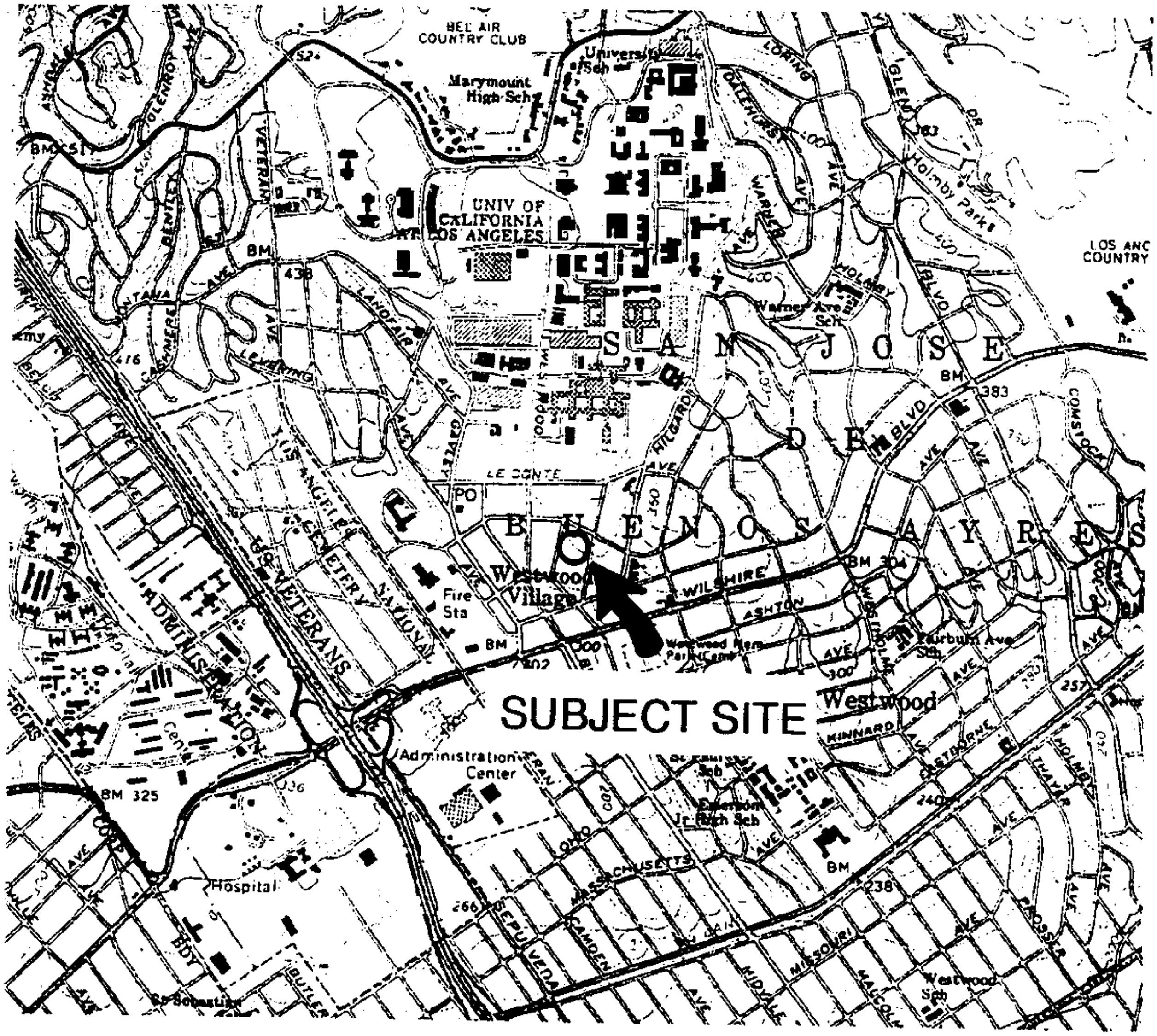
SCOTT W. MOORE
R.C.E. 55905

SWM:rh

xc: (5) Addressee
(2) Van Tilburg, Banvard & Soderberg

REGISTERED PROFESSIONAL ENGINEER
SCOTT W. MOORE
No. 55905
Exp. 12/31/00
CIVIL
STATE OF CALIFORNIA

REGISTERED PROFESSIONAL ENGINEER
SCOTT W. MOORE
No. 55905
Exp. 12/31/00
CIVIL
STATE OF CALIFORNIA



N

REFERENCE: U.S.G.S. TOPOGRAPHIC MAPS. 7.5 MINUTE SERIES.
BEVERLY HILLS. CA QUADRANGLE

SCALE: 1" = 2000'

VICINITY MAP

JERRY KOVACS AND ASSOCIATES, INC.
CONSULTING GEOTECHNICAL ENGINEERS

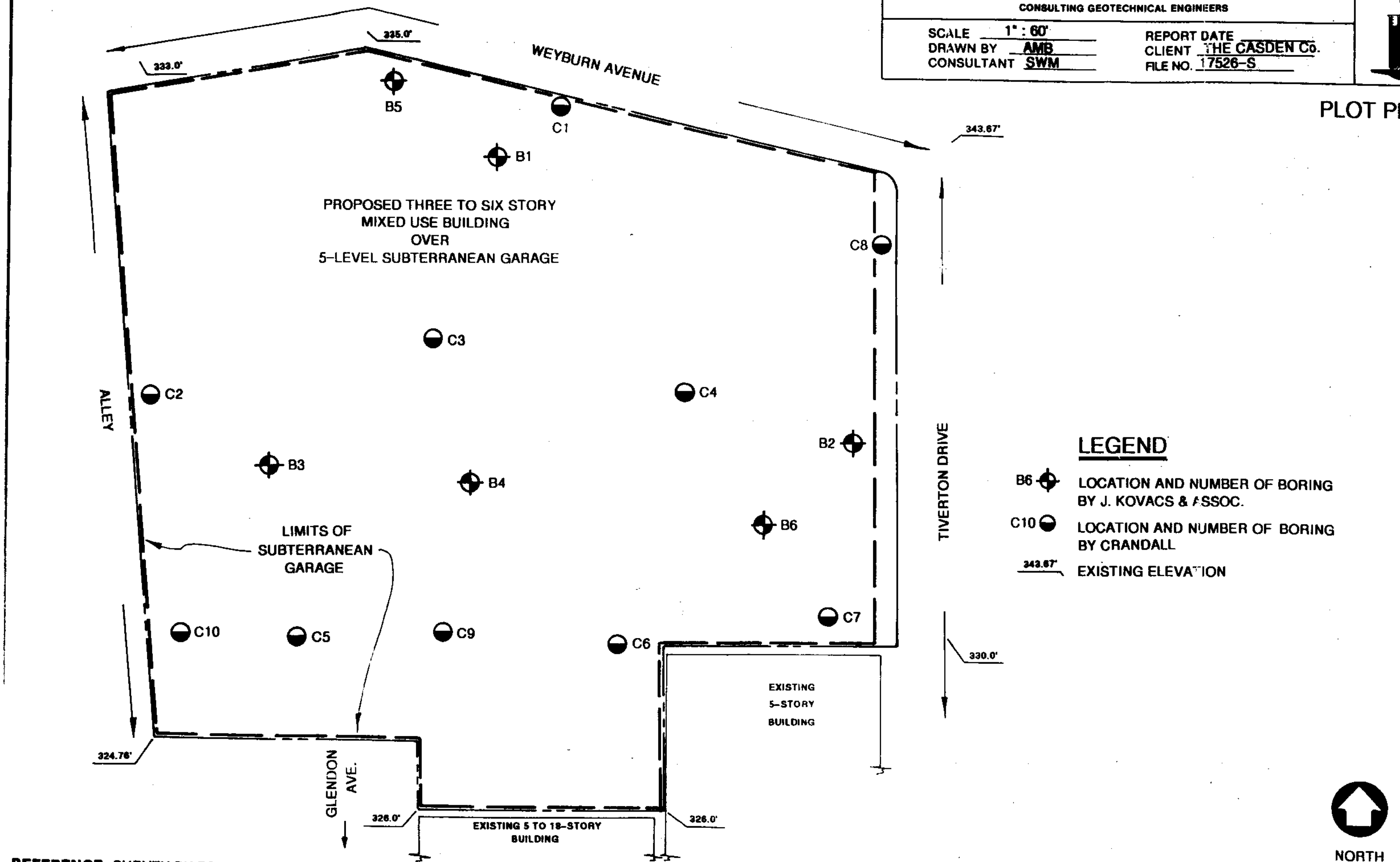
THE CASDEN COMPANY

FILE NO. 17526-S

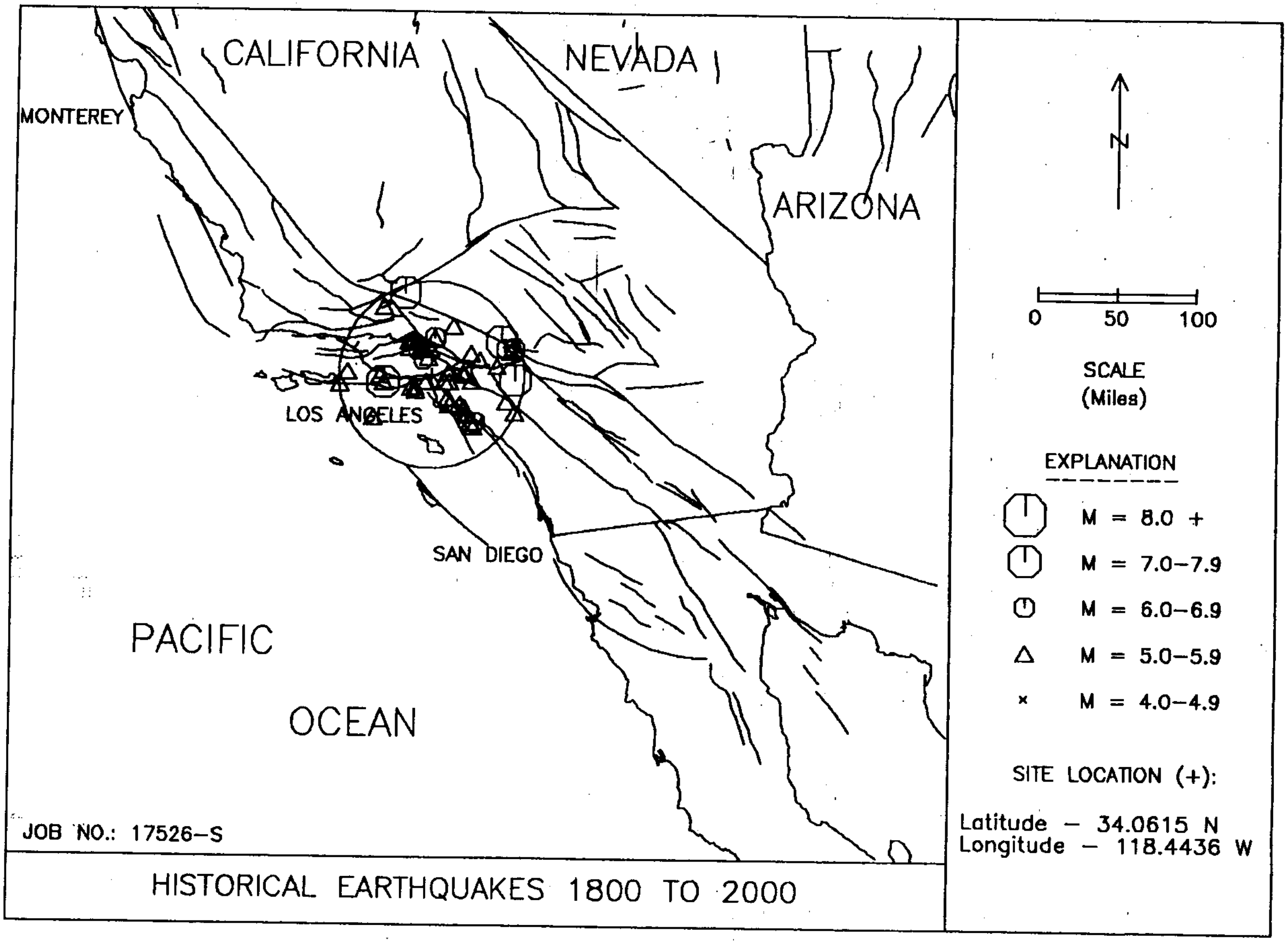
Jerry Kovacs and Associates Inc.
CONSULTING GEOTECHNICAL ENGINEERS

SCALE 1" : 60'
DRAWN BY AMB
CONSULTANT SWM
REPORT DATE
CLIENT THE CASDEN Co.
FILE NO. 17526-S

PLOT PLAN



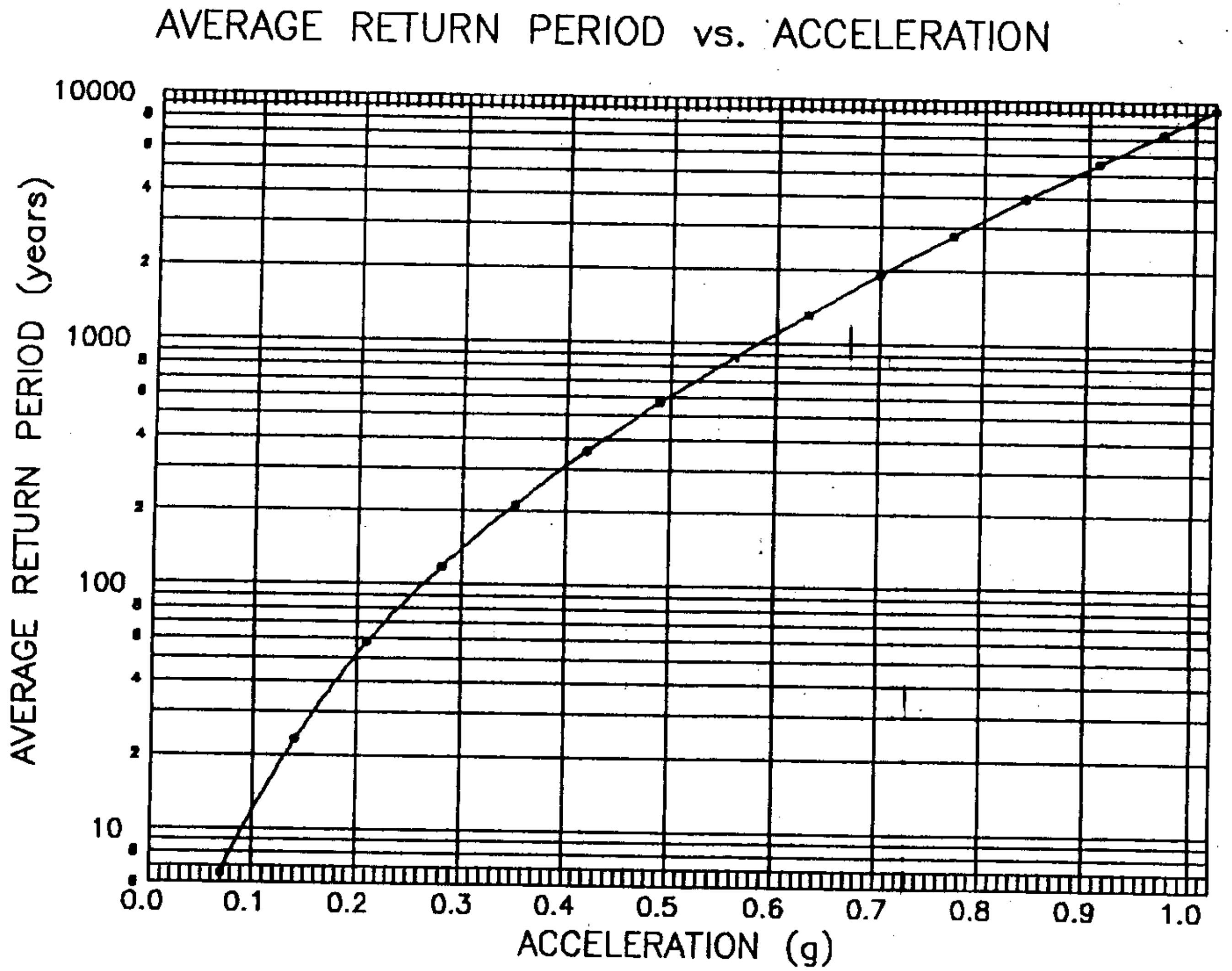
REFERENCE: SURVEY BY PSOMAS & ASSOC., DATED 1/25/89, AND VERBAL INFORMATION BY VAN TILBERG, BANVARD AND SOUERBERGH ARCHITECTS.



HISTORICAL EARTHQUAKES 1800 TO 2000

FIGURE II

FIGURE III

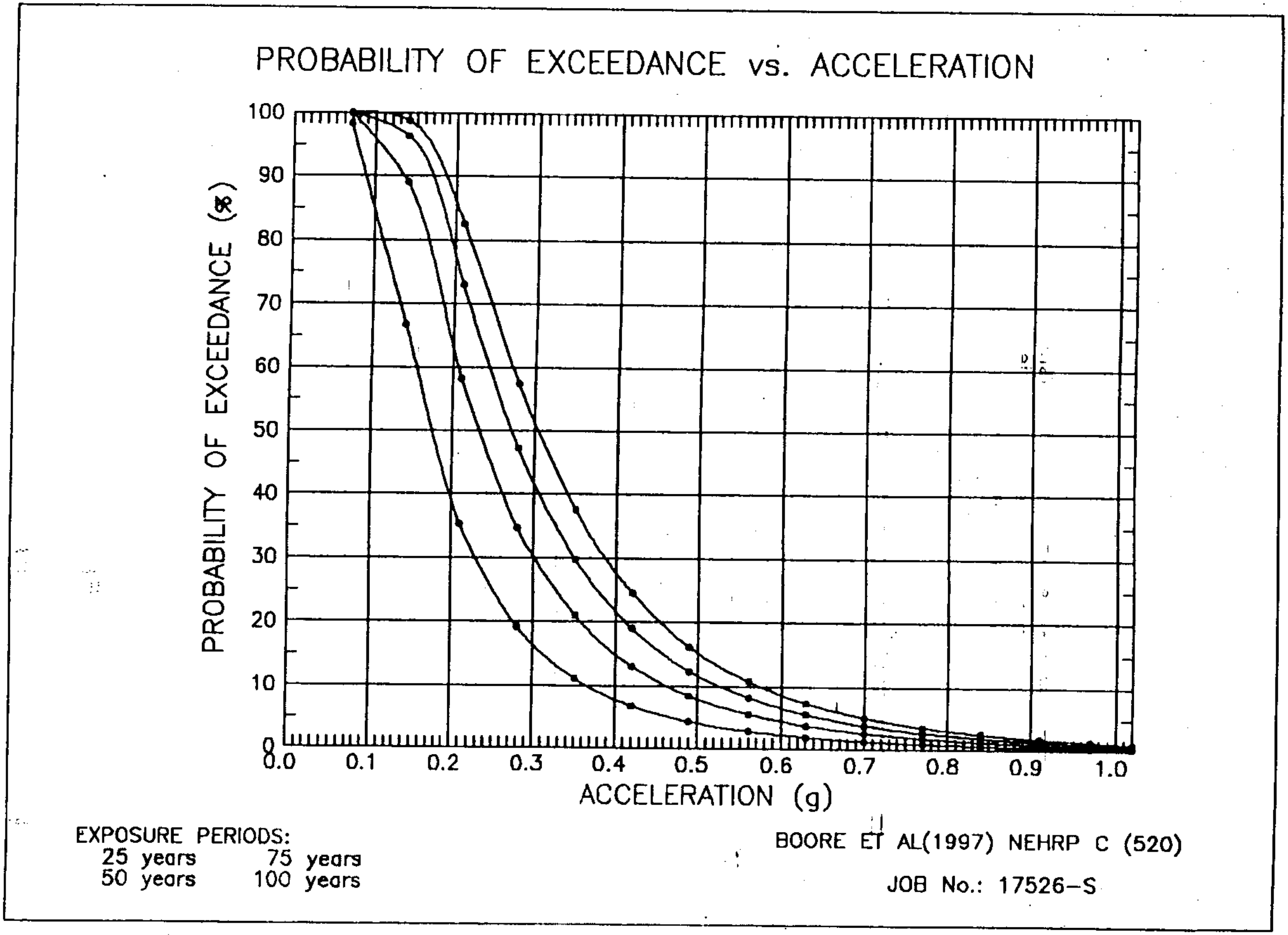


CASDEN

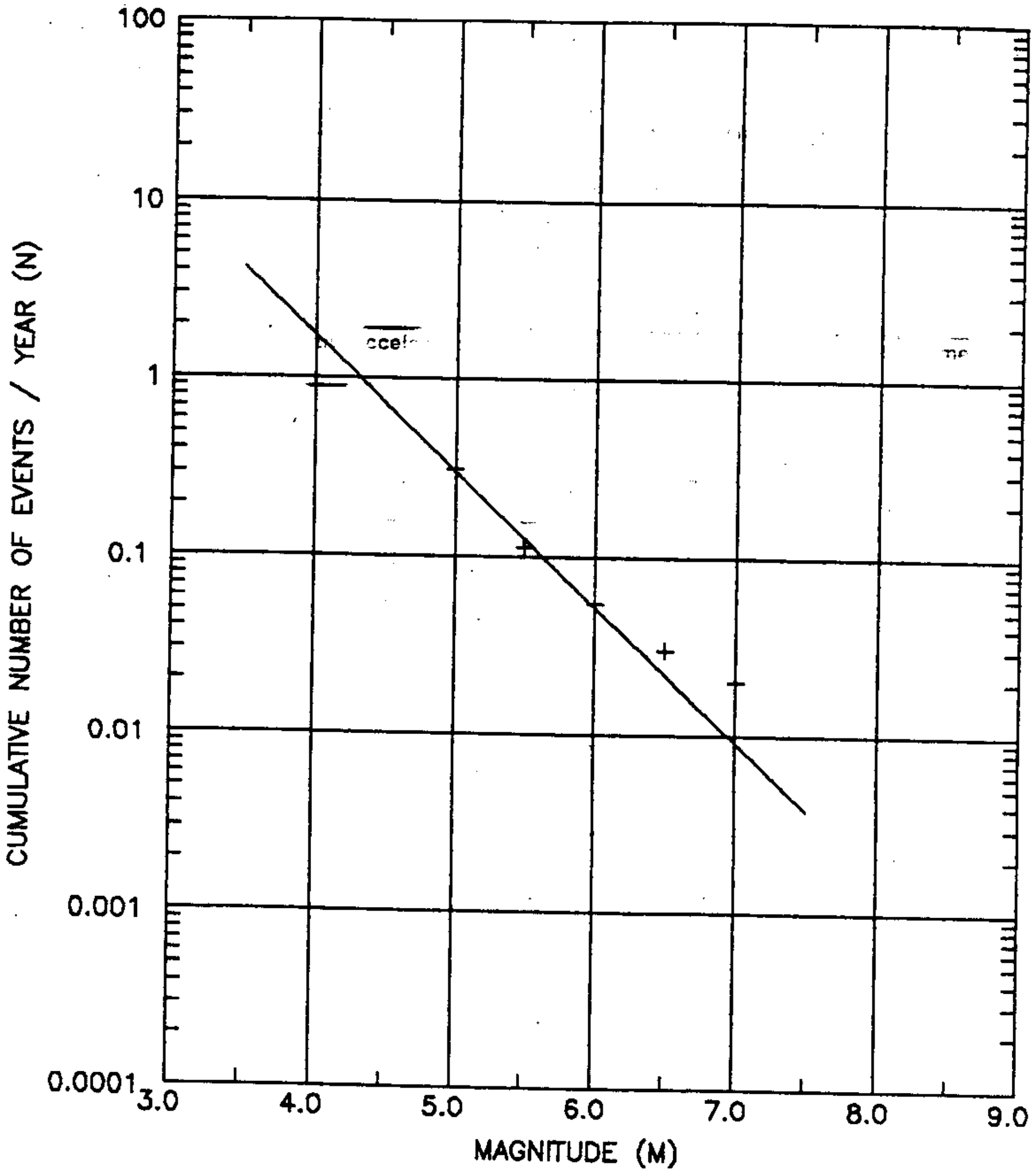
BOORE ET AL(1997) NEHRP C (520)

JOB No.: 17526-S

FIGURE IV.


PROBABILITY OF EXCEEDANCE vs. ACCELERATION
PROBABILITY OF EXCEEDANCE (%)
100
90
80
70
60
50
40
30
20
10
0
0.0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0
ACCELERATION (g)
EXPOSURE PERIODS:
25 years
50 years
75 years
100 years
BOORE ET AL(1997) NEHRP C (520)
JOB No.: 17526-S

LOG N = 3.266 − 0.758M



SEISMIC RECURRENCE CURVE

HISTORICAL EARTHQUAKES FROM 1800 TO 2000

CASDEN

FIGURE V

Table 3

Spectral Ordinates Having 50 Percent Probability of Being Exceeded in 50 Years (72 Year Return Period)

Spectral Damping = 2 Percent

Period, T (seconds)	Pseudo-Acceleration (g)	Pseudo-Relative Velocity (ft./sec.)	Relative Displacement (ft.)
0.01	0.229	0.0118	0.000019
0.03	0.230	0.0353	0.000169
0.10	0.485	0.249	0.00396
0.15	0.624	0.480	0.01145
0.20	0.672	0.689	0.02193
0.30	0.651	1.000	0.04776
0.40	0.570	1.168	0.07436
0.50	0.464	1.190	0.09467
0.75	0.319	1.225	0.14627
1.00	0.238	1.218	0.19393
1.50	0.164	1.261	0.30114
2.00	0.133	1.364	0.43420
2.50	0.107	1.375	0.54728
3.00	0.089	1.375	0.65674

Table 4

Spectral Ordinates Having 50 Percent Probability of Being Exceeded in 50 Years (72 Year Return Period)

Spectral Damping = 5 Percent

Period, T (seconds)	Pseudo-Acceleration (g)	Pseudo-Relative Velocity (ft./sec.)	Relative Displacement (ft.)
0.01	0.229	0.0118	0.000019
0.03	0.230	0.0353	0.000169
0.10	0.375	0.192	0.00306
0.15	0.482	0.371	0.00885
0.20	0.519	0.532	0.01694
0.30	0.503	0.773	0.03689
0.40	0.440	0.902	0.05745
0.50	0.378	0.969	0.07708
0.75	0.260	0.998	0.11911
1.00	0.194	0.992	0.15791
1.50	0.134	1.027	0.24520
2.00	0.108	1.111	0.35356
2.50	0.087	1.120	0.44563
3.00	0.073	1.120	0.53476

Table 5

Spectral Ordinates Having 50 Percent Probability of Being Exceeded in 50 Years (72 Year Return Period)

Spectral Damping = 7 Percent

Period, T (seconds)	Pseudo-Acceleration (g)	Pseudo-Relative Velocity (ft./sec.)	Relative Displacement (ft.)
0.01	0.229	0.0118	0.000019
0.03	0.230	0.0353	0.000169
0.10	0.334	0.171	0.00273
0.15	0.430	0.331	0.00789
0.20	0.463	0.475	0.01511
0.30	0.448	0.689	0.03291
0.40	0.393	0.805	0.05124
0.50	0.347	0.888	0.07069
0.75	0.238	0.915	0.10922
1.00	0.178	0.910	0.14480
1.50	0.123	0.942	0.22485
2.00	0.099	1.019	0.32421
2.50	0.080	1.027	0.40865
3.00	0.067	1.027	0.49038

Table 6

Spectral Ordinates Having 50 Percent Probability of Being Exceeded in 50 Years
(72 Year Return Period)

Spectral Damping = 10 Percent

Period, T (seconds)	Pseudo-Acceleration (g)	Pseudo-Relative Velocity (ft./sec.)	Relative Displacement (ft.)
0.01	0.229	0.0118	0.000019
0.03	0.230	0.0353	0.000169
0.10	0.292	0.149	0.00238
0.15	0.375	0.288	0.00688
0.20	0.404	0.414	0.01318
0.30	0.391	0.601	0.02870
0.40	0.342	0.702	0.04469
0.50	0.313	0.803	0.06390
0.75	0.215	0.827	0.09873
1.00	0.160	0.822	0.13090
1.50	0.111	0.851	0.20326
2.00	0.090	0.921	0.29308
2.50	0.072	0.928	0.36941
3.00	0.060	0.928	0.44330

Table 7

Spectral Ordinates Having 10 Percent Probability of Being Exceeded in 50 Years (475 Year Return Period)

Spectral Damping = 2 Percent

Period, T (seconds)	Pseudo-Acceleration (g)	Pseudo-Relative Velocity (ft./sec.)	Relative Displacement (ft.)
0.01	0.462	0.0237	0.000038
0.03	0.462	0.0710	0.000339
0.10	1.065	0.546	0.00869
0.15	1.325	1.019	0.02432
0.20	1.432	1.468	0.04672
0.30	1.420	2.183	0.10425
0.40	1.274	2.611	0.16622
0.50	1.054	2.700	0.21486
0.75	0.733	2.817	0.33627
1.00	0.538	2.755	0.43842
1.50	0.351	2.699	0.64441
2.00	0.275	2.819	0.89717
2.50	0.220	2.825	1.12388
3.00	0.184	2.825	1.34866

Table 8

Spectral Ordinates Having 10 Percent Probability of Being Exceeded in 50 Years (475 Year Return Period)

Spectral Damping = 5 Percent

Period, T (seconds)	Pseudo-Acceleration (g)	Pseudo-Relative Velocity (ft./sec.)	Relative Displacement (ft.)
0.01	0.462	0.0237	0.000038
0.03	0.462	0.0710	0.000339
0.10	0.823	0.422	0.00671
0.15	1.024	0.787	0.01879
0.20	1.106	1.134	0.03609
0.30	1.097	1.687	0.08053
0.40	0.984	2.017	0.12840
0.50	0.858	2.199	0.17495
0.75	0.597	2.294	0.27382
1.00	0.438	2.243	0.35699
1.50	0.286	2.198	0.52472
2.00	0.224	2.295	0.73054
2.50	0.180	2.300	0.91514
3.00	0.150	2.300	1.09817

Table 9

Spectral Ordinates Having 10 Percent Probability of Being Exceeded in 50 Years (475 Year Return Period)

Spectral Damping = 7 Percent

Period, T (seconds)	Pseudo-Acceleration (g)	Pseudo-Relative Velocity (ft./sec.)	Relative Displacement (ft.)
0.01	0.462	0.0237	0.000038
0.03	0.462	0.0710	0.000339
0.10	0.734	0.376	0.00599
0.15	0.913	0.702	0.01676
0.20	0.987	1.011	0.03219
0.30	0.979	1.504	0.07183
0.40	0.878	1.799	0.11453
0.50	0.787	2.016	0.16043
0.75	0.547	2.104	0.25109
1.00	0.401	2.057	0.32736
1.50	0.262	2.016	0.48117
2.00	0.205	2.105	0.66990
2.50	0.165	2.109	0.83918
3.00	0.137	2.109	1.00702

Table 10

Spectral Ordinates Having 10 Percent Probability of Being Exceeded in 50 Years (475 Year Return Period)

Spectral Damping = 10 Percent

Period, T (seconds)	Pseudo-Acceleration (g)	Pseudo-Relative Velocity (ft./sec.)	Relative Displacement (ft.)
0.01	0.462	0.0237	0.000038
0.03	0.462	0.0710	0.000339
0.10	0.640	0.328	0.00522
0.15	0.796	0.612	0.01462
0.20	0.860	0.882	0.02807
0.30	0.853	1.312	0.06264
0.40	0.765	1.569	0.09988
0.50	0.711	1.822	0.14503
0.75	0.495	1.902	0.22698
1.00	0.363	1.859	0.29593
1.50	0.237	1.822	0.43497
2.00	0.186	1.903	0.60559
2.50	0.149	1.907	0.75862
3.00	0.124	1.907	0.91034

M.J. SCHIFF & ASSOCIATES, INC.

Consulting Corrosion Engineers - Since 1959
1308 Monte Vista Avenue, Suite 6
Upland, CA 91786

Phone: (909)931-1360 / Fax: (909)931-1361
E-mail: mjsa@mjs-a.com
http://www.mjs-a.com

February 4, 2000

JERRY KOVACS & ASSOCIATES, INC.
439 Western Avenue
Glendale, California

Attention: Mr. Scott W. Moore

Re: Soil Corrosivity Study
Casden
Westwood, California
Your #17526-S, MJS&A #00-0042HQ

INTRODUCTION

Laboratory tests have been completed on five soil samples you sent for the referenced multi-story commercial building project with a 50-foot deep parking structure. Water table is 62 to 64 feet below grade. The purpose of these tests was to determine if the soils may have deleterious effects on underground utility piping, hydraulic elevator cylinders, and concrete foundations. We assume that the samples provided are representative of the most corrosive soils at the site.

The scope of this study is limited to a determination of soil corrosivity and general corrosion control recommendations for materials likely to be used for construction. If the architects and/or engineers desire more specific information, designs, specifications, or review of design, we will be happy to work with them as a separate phase of this project.

TEST PROCEDURES

The electrical resistivity of each sample was measured in a soil box per ASTM G57 in its as-received condition and again after saturation with distilled water. Resistivities are at about their lowest value when the soil is saturated. The pH of the saturated samples was measured. A 5:1 water:soil extract from each sample was chemically analyzed for the major chemicals commonly found in soils and for ammonium and nitrate in the two shallow samples. Test results are shown on Table 1.

SOIL CORROSIVITY

A major factor in determining soil corrosivity is electrical resistivity. The electrical resistivity of a soil is a measure of its resistance to the flow of electrical current. Corrosion of buried metal is an electrochemical process in which the amount of metal loss due to corrosion is directly proportional to the flow of electrical current (DC) from the metal into the soil. Corrosion currents, following Ohm's Law, are inversely proportional to soil resistivity. Lower electrical resistivities result from higher moisture and chemical contents and indicate corrosive soil.

A correlation between electrical resistivity and corrosivity toward ferrous metals is:

Soil Resistivity in ohm-centimeters			Corrosivity Category
over		10,000	mildly corrosive
2,000	to	10,000	moderately corrosive
1,000	to	2,000	corrosive
below		1,000	severely corrosive

Other soil characteristics that may influence corrosivity towards metals are pH, chemical content, soil types, aeration, anaerobic conditions, and site drainage.

Electrical resistivities were in mildly and moderately corrosive categories with as-received moisture. When saturated, the resistivities were in moderately corrosive and corrosive categories.

Soil pH values varied from 6.4 to 7.3. This range is mildly acidic to neutral and does not particularly increase soil corrosivity.

The chemical content of the samples was low.

Ammonium and nitrate were detected but in low concentrations.

Tests were not made for sulfide and negative oxidation-reduction (redox) potential because the samples did not exhibit characteristics typically associated with these conditions.

This soil is classified as corrosive to ferrous metals.

CORROSION CONTROL

The life of buried materials depends on thickness, strength, loads, construction details, soil moisture, etc., in addition to soil corrosivity, and is, therefore, difficult to predict. Of more practical value are corrosion control methods that will increase the life of materials that would be subject to significant corrosion.

Steel Pipe

Abrasive blast underground steel piping and apply a dielectric coating such as polyurethane, extruded polyethylene, a tape coating system, hot applied coal tar enamel, or fusion bonded epoxy intended for underground use.

Bond underground steel pipe with rubber gasketed, mechanical, grooved end, or other nonconductive type joints for electrical continuity. Electrical continuity is necessary for corrosion monitoring and cathodic protection.

Electrically insulate each buried steel pipeline from dissimilar metals, dissimilar coatings (cement-mortar vs. dielectric), and above ground steel pipe to prevent dissimilar metal corrosion cells and to facilitate the application of cathodic protection.

Apply cathodic protection to steel piping as per NACE International RP-0169-96.

As an alternative to dielectric coating and cathodic protection, apply a 3/4 inch cement mortar coating or encase in cement-slurry or concrete 3 inches thick, using any type of cement.

Hydraulic Elevator

Coat hydraulic elevator cylinders as described above. Electrically insulate each cylinder from building metals by installing dielectric material between the piston platen and car, insulating the bolts, and installing an insulated joint in the oil line. Apply cathodic protection to hydraulic cylinders as per NACE International RP-0169-96. As an alternative to electrical insulation and cathodic protection, place each cylinder in a plastic casing with a plastic watertight seal at the bottom.

The elevator oil line should be placed above ground if possible but, if underground, should be protected as described above for steel utilities.

Iron Pipe

Encase ductile iron water piping in 8 mil thick low-density polyethylene or 4 mil thick high-density, cross-laminated polyethylene plastic tubes or wraps per AWWA Standard C105 or coat with a polyurethane intended for underground use. Electrically insulate underground iron pipe from dissimilar metals and from above ground iron pipe with insulated joints.

Cast iron drain piping does not require special protective measures such as a plastic wrap. However, to avoid possibly creating corrosion problems, cast iron should not be placed partially in contact with concrete such as thrust blocks. Use a dielectric coating as described above for steel or use 8 mil thick low-density polyethylene or 4 mil thick high-density polyethylene plastic sheets per

AWWA C105 to prevent such contact. Electrically insulate underground iron pipe from dissimilar metals and above ground iron pipe with insulated joints.

Copper Tube

No special precautions are necessary for bare copper tubing for cold water. Hot water tubing may be subject to a higher corrosion rate. Hot copper can be protected by applying cathodic protection or preventing soil contact. Soil contact may be prevented by placing the tubing above ground or inside a plastic pipe. The amount of cathodic protection current needed can be minimized by coating the tubing.

Plastic and Vitrified Clay Pipe

No special precautions are required for plastic and vitrified clay piping placed underground from a corrosion viewpoint. Protect any iron fittings with a double polyethylene wrap per AWWA C105. Protect any iron valves with a dielectric coating such as epoxy, polyurethane, mastic, or wax tape intended for underground use.

All Pipe

On all pipe, coat bare steel appurtenances such as bolts, joint harnesses, or flexible couplings with a coal tar or elastomer based mastic, coal tar epoxy, moldable sealant, wax tape, or equivalent after assembly.

Where metallic pipelines penetrate concrete structures such as building floors, vault walls, and thrust blocks use plastic sleeves, rubber seals, or other dielectric material to prevent pipe contact with the concrete and reinforcing steel.

Concrete

Any type of cement may be used for concrete structures and pipe because the sulfate concentration is negligible, 0 to 0.1 percent, per 1997 Uniform Building Code (UBC) Table 19-A-4 and American Concrete Institute (ACI-318) Table 4.3.1.

Standard concrete cover over reinforcing steel may be used for concrete structures and pipe in contact with these soils. Use 2 inches minimum cover over embedded steel at the edges of concrete slabs and footings above grade as well as below.

Our services have been performed with the usual thoroughness and competence of the engineering profession. No other warranty or representation, either expressed or implied, is included or intended.

Please call if you have any questions.

Respectfully Submitted,
M.J. SCHIFF & ASSOCIATES, INC.

James T. Keegan

Paul R Smith
EXP. 6-30-01

Reviewed by,

Paul R Smith

Paul R. Smith, P.E.

Enc: Table 1

M. J. Schiff & Associates, Inc.

Consulting Corrosion Engineers - Since 1959

1308 Monte Vista Avenue, Suite 6
Upland, CA 91786-8224
Phone: 909/931-1360

Table 1 - Laboratory Tests on Soil Samples

Casden, Westwwod, CA
Your #17526-S, MJS&A #00-0042HQ
28-Jan-00

Sample ID			B1 @ 4' Clayey Sand	B1 @ 30' Clayey Sand	B1 @ 50' Sandy Clay Clayey Sand	B2 @ 2' Clayey Sand	B2 @ 60' Silty Clay
Resistivity		**Units**					
as-received		ohm-cm	18,000	5,000	6,600	2,900	2,900
saturated		ohm-cm	3,000	1,800	4,400	2,100	1,600
pH			7.3	6.4	6.6	7.2	6.4
Electrical Conductivity		mS/cm	0.14	0.08	0.04	0.10	0.08
Chemical Analyses							
Cations							
calcium	Ca^{2+}	mg/kg	12	12	16	16	16
magnesium	Mg^{2+}	mg/kg	15	15	10	12	15
sodium	Na^{1+}	mg/kg	63	3	ND	45	ND
Anions							
carbonate	CO_3^{2-}	mg/kg	ND	ND	ND	ND	ND
bicarbonate	HCO_3^{1-}	mg/kg	217	37	34	107	49
chloride	Cl^{1-}	mg/kg	14	25	18	18	43
sulfate	SO_4^{2-}	mg/kg	29	31	ND	72	ND
Other Tests							
ammonium	NH_4^{1+}	mg/kg	2.4	na	na	1.7	na
nitrate	NO_3^{1-}	mg/kg	2.4	na	na	3.5	na
sulfide	S^{2-}	qual	na	na	na	na	na
Redox		mv	na	na	na	na	na

Electrical conductivity in millisiemens/cm and chemical analysis were made on a 1:5 soil-to-water extract.
mg/kg = milligrams per kilogram (parts per million) of dry soil.
Redox = oxidation-reduction potential in millivolts
ND = not detected
na = not analyzed